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KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT

COMMUNICATION

FROM

THE ASSISTANT SECRETARY OF THE ARMY, CIVIL WORKS, DEPARTMENT OF DEFENSE

TRANSMITTING

THE KANSAS CITYS, MISSOURI AND KANSAS FLOOD RISK MANAGEMENT PROJECT REPORT FOR MAY 2014

PART 2 of 2

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EXHIBIT A-8.4

Gatewell Sample Calculations

Gatewell 106+49, Existing Condition Strength and Uplift Stability Analysis:

Major tasks assocaited with this analysis:

1. Obtain the geometry for the gatewell and levee which are taken from the Record drawings.
2. Obtain the geotechnical parameters which are obtained from the Geotechnical Engineer.
3. Perform uplift analysis assuming gate is empty.

Calculation Convention Notes:

1. All input values are highlighted in
2. Results are highlighted in green
3. Assumption notes are displayed in red

References:

Gatewell Sta. 106+49 elevations, location and reinforcing details from American Royal Arena Storm Sewer System for KC, MO Department of Public Works plans by Shafer, Kline and Warren 1973

Geotech Parameters:

Information Used to Calculate Structures HGL's - Blanket Information			
Station	Condition	Bottom of Landside Blanket Elevation	Top of Blanket Elevation
49+00 to 51+00	Existing	719	751
51+00 to 57+00	Existing	720	750
57+00 to 59+00	Existing	722	750
59+00 to 63+00	Existing	725	735
63+00 to 68+00	Existing	728	735
68+00 to 73+00	Existing	718	733
77+00 to 82+00	Existing	720	738
82+00 to 85+00	Existing	730	740
85+00 to 88+00	Existing	730	742
88+00 to 90+00	Existing	725	732
92+00 to 94+50	Existing	735	744
102+00 to 107+00	Existing	730	742
119+00 to 127+00	Existing	722	756
136+00 to 143+00	Existing	725	753
150+00 to 153+00	Existing	725	751
159+00 to 163+00	Existing	725	745

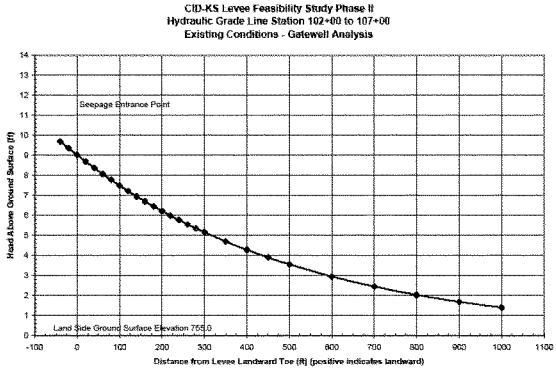
Geotech Parameters (con't):

Recommended Parameters for Phase II Feasibility Study

	Unit Weight (pcf)		Drained Shear Strength		Undrained Shear Strength*	
Material	Moist	Saturated	♦' (degrees)	c' (psf)	♦ (degrees)	c (psf)
Embankment	115	120	29	0	0	1000
Fill (cinders/rubble)	110	115	21	0	N/A	N/A
Foundation Blanket	110	115	24	0	0	500
Foundation Sand	115	120	31	0	N/A	N/A

*Estimated from average strength values for these materials

Hydraulic Gradeline:



HGL & blanket elevations provided by Glen Bellew 8/8/07

Soil Elevation Assumed by geotech: $El_{soil\_geotech} = 0ft$

Elevation of the HGL using geotech datum: $HGL_{geotech} = 764ft$

Elevation of bottom of blanket: $El_{bot\_blanket} = 730ft$

Input Values

From record drwg $El_{top\_gw} = 765.1ft$

Top of floodwall EL=xxxxx'

This is determined from the Record drawings

$El_{soil\_at\_gw} \equiv El_{top\_gw}$

$El_{soil\_at\_gw} = 765.1\ ft$

$H_{blanket} \equiv El_{soil\_at\_gw} - El_{bot\_blanket}$

The HGL from the geotech needs to be adjusted using the soil elevation at the gatewell as the datum:

$HGL := HGL_{geotech} \cdots + 0ft - (El_{soil\_at\_gw} - El_{soil\_geotech})$

$HGL = -1.1\ ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 0ft$

Buried Height of

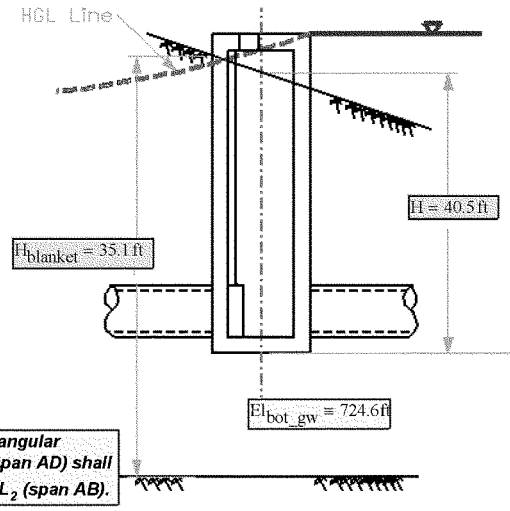
$H \equiv El_{soil\_at\_gw} - El_{bot\_gw}$

$H = 40.5\ ft$

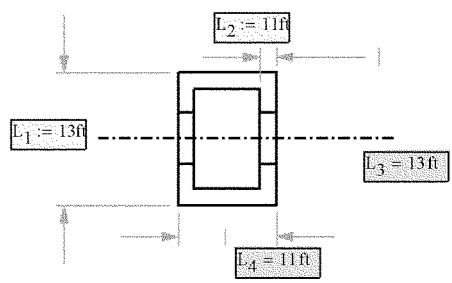
Projected Height of Gatewell above soil

$H' := El_{top\_gw} - El_{soil\_at\_gw}$

$H' = 0\ ft$



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).



Width analyzed: $b := 12in$

$L_3 := L_1$

Floor Thickness $D_f := 24in$

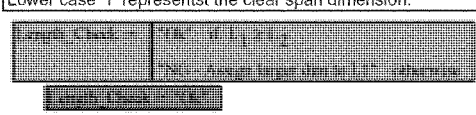
$L_4 := L_2$

Slab clear cover $C_{cf} := 3.5in$

Slab effective depth $d_f := D_f - C_{cf}$

$d_f = 20.5 \cdot in$

Note: Upper case "L" represents the out-to-out dimension. Lower case "l" representst the clear span dimension.



Wall Thickness

$D_1 := 18in$ Wall 1 (Span AD) wall thickness

$D_2 := 18in$ Wall 2 (Span AB) wall thickness

$D_3 := D_1$ Wall 3 (Span BC) must be of the same thickness as Wall 1 (Span AD)

$D_4 := D_2$ Wall 4 (Span CD) must be of the same thickness as Wall 2 (Span AB)

Clear Spans

$l_1 := L_1 - 2 \cdot D_2$ $l_1 = 10ft$ Clear span wall AD

$l_2 := L_2 - 2D_1$ $l_2 = 8ft$ Clear span of wall AB

$l_3 := l_1$ $l_3 = 10ft$ Clear span of wall BC

$l_4 := l_2$ $l_4 = 8ft$ Clear span of wall CD

Concrete Section Areas $b = 1ft$

$A_{g1} := D_1 \cdot b$ $A_{g1} = 1.5ft^2$

$A_{g2} := D_2 \cdot b$ $A_{g2} = 1.5ft^2$

$A_{g3} := A_{g1}$ $A_{g3} = 1.5ft^2$

$A_{g4} := A_{g2}$ $A_{g4} = 1.5ft^2$

$d_f = 20.5 \cdot in$

Base Slab Reinforcment

Reinforcement information provided by the record drawings $Bar\_Sp_{slb} := 12in$

$BarNo_{slb} :=$ $BarNo_{slb} = 5$

Base Slab Reinforcement

$$A_{slb} := \frac{A_{slb}}{\frac{Bar\_Sp_{slb}}{12in}}$$ $A_{slb} = 0.31 \cdot in^2$

Wall Reinforcement Areas At Elevation Bot of Gatewell:

Reinforcement information provided by the record drawings

I. WALL AD

$L_1 = 13ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar\_Spacing_{ADout} := 10in$ $Bar\_Spacing_{ADin} := 6in$

$Bar\_No_{ADout} :=$ $Bar\_No_{ADout} = 7$ $Bar\_No_{ADin} :=$ $Bar\_No_{ADin} = 8$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ADout} := \frac{A_{out}}{\frac{Bar\_Spacing_{ADout}}{12in}}$$ $$A_{ADin} := \frac{A_{in}}{\frac{Bar\_Spacing_{ADin}}{12in}}$$ $A_{out} = 0.6 \cdot in^2$ $A_{in} = 0.79 \cdot in^2$

$A_{ADout} = 0.72 \cdot in^2$ $A_{ADin} = 1.58 \cdot in^2$

Reinf at wall AB is based on the Half-Section thru gate chamber detail on sheet 4 of 5

II. WALL AB

$L_2 = 11ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar\_Spacing_{ABout} := 6in$ $Bar\_Spacing_{ABin} := 6in$

$Bar\_No_{ABout} :=$ $Bar\_No_{ABout} = 6$ $Bar\_No_{ABin} :=$ $Bar\_No_{ABin} = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ABout} := \frac{A_{out}}{\frac{Bar\_Spacing_{ABout}}{12in}} \qquad A_{ABin} := \frac{A_{in}}{\frac{Bar\_Spacing_{ABin}}{12in}} \qquad A_{out} = 0.44 \cdot in^2 \qquad A_{in} = 0.6 \cdot in^2$$

$$\boxed{A_{ABout} = 0.88 \cdot in^2} \qquad \boxed{A_{ABin} = 1.2 \cdot in^2}$$

Reinforcement Bar Diameters

Reinforcement Clear Cover

Wall AD

$\boxed{coverADout := 3in}$

$\boxed{coverADin := 3in}$

Wall AB

$\boxed{coverABout := 3in}$

$\boxed{coverABin := 3in}$

Effective Depth:

These effective depths are used for the AD end analysis

$D_1 = 18 \cdot in \qquad diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$

$$d_{AD\_in\_end} := coverADin + \frac{diaADin}{2} \qquad d_{AD\_out\_end} := D_1 - coverADout - \frac{diaADout}{2}$$

$$\boxed{d_{AD\_in\_end} = 3.5 \cdot in} \qquad \boxed{d_{AD\_out\_end} = 14.563 \cdot in}$$

These effective depths are used for the AB end analysis

$diaABin = 0.875 \cdot in \qquad diaABout = 0.75 \cdot in \qquad D_2 = 1.5 ft$

$$d_{AB\_in\_end} := coverABin + \frac{diaABin}{2} \qquad d_{AB\_out\_end} := D_2 - coverABout - \frac{diaABout}{2}$$

$$\boxed{d_{AB\_in\_end} = 3.437 \cdot in} \qquad \boxed{d_{AB\_out\_end} = 14.625 \cdot in}$$

These effective depths are used for the AD mid analysis

$diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$

$$d_{AD\_in\_mid} := D_1 - coverADin - \frac{diaADin}{2} \qquad d_{AD\_out\_mid} := coverADout + \frac{diaADout}{2}$$

$$\boxed{d_{AD\_in\_mid} = 14.5 \cdot in} \qquad \boxed{d_{AD\_out\_mid} = 3.437 \cdot in}$$

These effective depths are used for the AB mid analysis

$diaABin = 0.875 \cdot in$ $diaABout = 0.75 \cdot in$

$d_{AB\_in\_mid} := D_2 - coverABin - \frac{diaABin}{2}$

$d_{AB\_in\_mid} = 14.563 \cdot in$

$d_{AB\_out\_mid} := coverABout + \frac{diaABout}{2}$

$d_{AB\_out\_mid} = 3.375 \cdot in$

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, Engineered Concrete Structures, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi_B := 21deg$ $\gamma_{soil} := 115pcf$

At Rest Soil Pressure $K_o := 1 - \sin(\phi_B)$ $K_o = 0.642$

Active Soil Pressure: $K_a := \tan\left(45deg - \frac{\phi_B}{2}\right)^2$ $K_a = 0.472$

Concrete Unit Weight $\gamma_c := 150pcf$ Concrete Properties $f'_c := 3.0ksi$ Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40.0ksi$ Modulus of Elasticity: $Es := 29000ksi$

$\beta 1 := if\left(f'_c \le 4ksi, 0.85, if\left(f'_c > 8ksi, 0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4ksi}{ksi}\right)\right)$ $\beta 1 = 0.85$ $f'_c = 3000psi$

Load & Resistance Factor Design Resistance and Load Factor Values

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

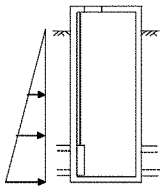
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) and Strength Reduction Factors (.85 for shear, 0.90 for bending) are not applied for in this initial evaluation. Instead a factor of safety is computed for the existing structure. If a strength concern is found, a design approach (applying these factors directly) will be taken for any recommended solutions.

Wall Loading

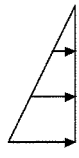
Wall Loading

WALLS

Special Case with Monolithically Placed exit and entrance:



Soil



$H = 40.5\,ft$

$H_2 := H$ OR

$H_2 = 40.5\,ft$ $H_3 := H_2 - 7ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H_2$

$W_1 = 2472 \cdot psf$

Soil & Water

$H_2 = 33.5\,ft$

&

$H_3 := \text{if}\left[H < H_{blanket}, \frac{H}{H_{blanket}} \cdot (HGL + H_{blanket}) - HEAD, H + HGL - HEAD\right]$

$H_3 = 39.4\,ft$

$H_3 := H_3 - 7ft$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[K_o \cdot (\gamma_{soil} - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_2 = 3152 \cdot psf$

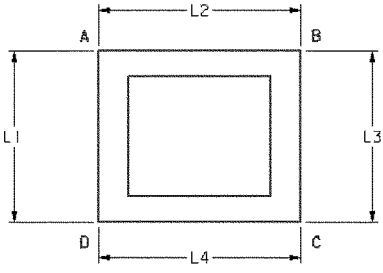
Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case $W := \text{if}(W_1 > W_2, W_1, W_2) \cdot b$ $W = 3152 \cdot plf$

Moment Distribution of Gatewell Walls

NOTE: Check continuity of reinforcement at corners. If continuity is present, use moment distribution to determine the demand moment values. Otherwise, analyze the gatewell wall sections as simply supported beams.

The following calculations determine the distributed moment based on relative stiffness. In order for the calculations to be valid, the following must be true:
1. The structure has four walls. The walls that are parallel to each other are of the same thickness.
2. The EI value is based on the wall length and the wall thickness.
3. The walls are orthagonal.
4. The walls are exposed to uniform loading W, and $W = 3152 \cdot plf$ is the same for each wall.

Determine Moment and Shear:



Design Section Length

$L_1 = 13\,ft$ $L_2 = 11\,ft$ $L_3 = 13\,ft$ $L_4 = 11\,ft$

Moment of Inertia:

$b = 1\,ft$ $D_1 = 1.5\,ft$ $D_2 = 1.5\,ft$

$D_4 = 1.5\,ft$ $D_3 = 1.5\,ft$

$I_{beam1} := \frac{b \cdot D_1^3}{12}$ $I_{beam1} = 5832 \cdot in^4$

$I_{beam2} := \frac{b \cdot D_2^3}{12}$ $I_{beam2} = 5832 \cdot in^4$

$I_{beam3} := I_{beam1}$ $I_{beam3} = 5832 \cdot in^4$

$I_{beam4} := I_{beam2}$ $I_{beam4} = 5832 \cdot in^4$

Enter End Conditions:

All carryover factors are equal to 0.5 (idealize as a continuous beam over supports A, B, C and D).

$COF_A := 0.5$ $COF_B := COF_A$ $COF_C := COF_A$ $COF_D := COF_A$

$A := COF_A$ $B := COF_B$ $C := COF_C$ $D' := COF_D$

Fixed End Moments For A Uniform Load Between Fixed Supports using center of support to center of support distance per ACI 318-02 8.7.2 :

$L_1 = 13\,ft$ $L_2 = 132 \cdot in$ $L_3 = 13\,ft$ $L_4 = 11\,ft$ $W = 3152.38 \cdot plf$

$M_{fix\_1} := \frac{W \cdot (L_1 - D_2)^2}{12}$ $M_{fix\_1} = 34.742 \cdot kip \cdot ft$ $M_{fix\_2} := \frac{W \cdot (L_2 - D_1)^2}{12}$ $M_{fix\_2} = 23.709 \cdot kip \cdot ft$

$M_{fix\_3} := \frac{W \cdot (L_3 - D_2)^2}{12}$ $M_{fix\_3} = 34.742 \cdot kip \cdot ft$ $M_{fix\_4} := \frac{W \cdot (L_4 - D_1)^2}{12}$ $M_{fix\_4} = 23.709 \cdot kip \cdot ft$

Distribution Factors

	M_{AD}	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
COF	0.50	0.50	0.50	0.50	0.50	0.50	0.50
DF	0.46	0.54	0.54	0.46	0.46	0.54	0.54
FEM	-34.74	23.71	-23.71	34.74	-34.74	23.71	-23.71
D#1	5.057	5.976	-5.976	-5.057	5.057	5.976	-5.976
CO#1	-2.528	-2.988	2.988	2.528	-2.528	-2.988	2.988
DF#2	2.528	2.988	-2.988	-2.528	2.528	2.988	-2.988
CO#2	-1.264	-1.494	1.494	1.264	-1.264	-1.494	1.494
DF#3	1.264	1.494	-1.494	-1.264	1.264	1.494	-1.494
CO#3	-0.632	-0.747	0.747	0.632	-0.632	-0.747	0.747
DF#4	0.632	0.747	-0.747	-0.632	0.632	0.747	-0.747
CO#3	-0.316	-0.374	0.374	0.316	-0.316	-0.374	0.374
DF#4	0.316	0.374	-0.374	-0.316	0.316	0.374	-0.374
	-29.68	29.68	-29.68	29.68	-29.68	29.68	-29.68

Attach proper units to distributed values

$M_{ADend} := M'_{AD} \cdot ft \cdot kip$ $M_{ADend} := M'_{AB} \cdot ft \cdot kip$

$M_{CBend} := M'_{CB} \cdot ft \cdot kip$ $M_{DCend} := M'_{DC} \cdot ft \cdot kip$

End-of-Span Moments After Distribution

$M_{ADend} = -29.685 \cdot kip \cdot ft$ $M_{ADend} = -29.685 \cdot kip \cdot ft$

$M_{CBend} = -29.685 \cdot kip \cdot ft$ $M_{DCend} = -29.685 \cdot kip \cdot ft$

Note that the end-of-span moment value is the same at each joint because of uniform and equal loading present on all spans.

$W = 3152.38 \cdot plf$

Note: L_1 represents span AD, L_2 represents span AB, L_3 represents span BC and L_4 represents span CD

Mid-Span Moment Values

Moment Envelope Values

$$M_{UADin} := \left[\frac{W\cdot(L_1 - D_2)^2}{8} - |M_{ADend}|\right] \qquad M_{UADin} = 22.428\cdot kip\cdot ft \qquad \frac{W\cdot(L_1 - D_2)^2}{8} = 52.113\cdot kip\cdot ft$$

$$M_{UABin} := \left[\frac{W\cdot(L_2 - D_1)^2}{8} - |M_{ADend}|\right] \qquad M_{UABin} = 5.878\cdot kip\cdot ft \qquad \frac{W\cdot(L_2 - D_1)^2}{8} = 35.563\cdot kip\cdot ft$$

$$M_{UCBin} := \left[\frac{W\cdot(L_3 - D_2)^2}{8} - |M_{CBend}|\right] \qquad M_{UCBin} = 22.428\cdot kip\cdot ft \qquad \frac{W\cdot(L_3 - D_2)^2}{8} = 52.113\cdot kip\cdot ft$$

$$M_{UDCin} := \left[\frac{W\cdot(L_4 - D_1)^2}{8} - |M_{DCend}|\right] \qquad M_{UDCin} = 5.878\cdot kip\cdot ft \qquad \frac{W\cdot(L_4 - D_1)^2}{8} = 35.563\cdot kip\cdot ft$$

The demand moment will be taken at the support face IAW ACI 318-02, 8.7.2 and 8.7.3.

$$M_{UADface} := \left[M_{ADend} + \frac{W\cdot D_2}{8}\cdot(2L_1 - 3D_2)\right]$$

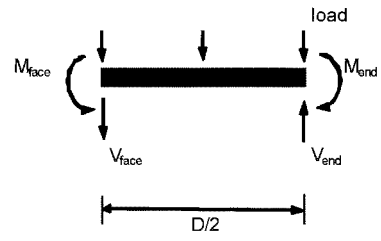
$$M_{UADface} = -16.977\cdot kip\cdot ft$$

$$M_{UABface} := \left[M_{ADend} + \frac{W\cdot D_1}{8}\cdot(2\cdot L_2 - 3D_1)\right]$$

$$M_{UABface} = -19.341\cdot kip\cdot ft$$

$$M_{UCBface} := M_{UADface} \qquad M_{UDCface} := M_{UABface}$$

Free Body Diagram

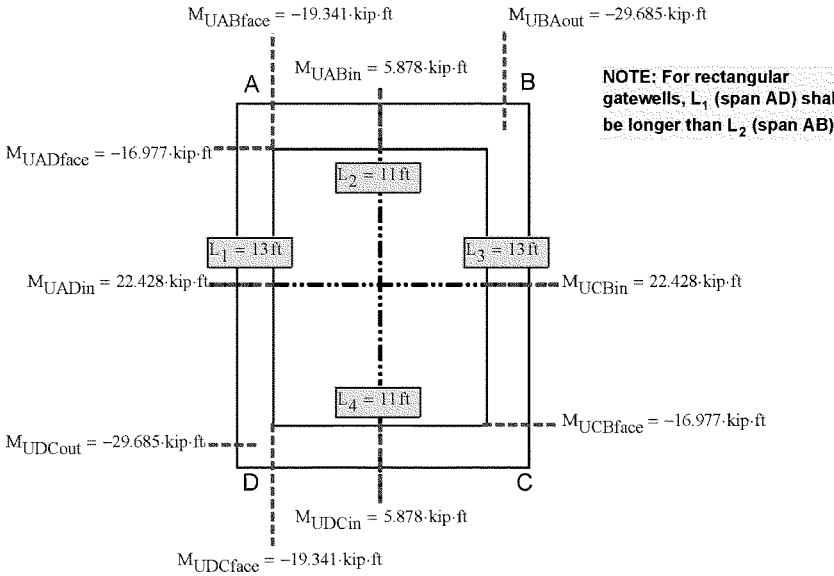


"D" represents the orthagonal wall thickness

$M_{UDCout} := M_{DCend}$ $\boxed{M_{UDCout} = -29.685 \cdot kip \cdot ft}$

$M_{UABout} := M_{ADend}$ $\boxed{M_{UABout} = -29.685 \cdot kip \cdot ft}$ $M_{UBAout} := M_{ADend}$

FACTORED MOMENT DISTRIBUTION VALUES



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

The "L" values are the "outside-to-outside" dimensions.

Ensure the reinforcement size entered earlier reflects the steel that will be in tension at the mid-span.
For example:
IF: The mid-span moment demand value obtained from moment distribution is negative.
THEN: The steel located near the wall's exterior face is in tension. Select the reinforcement size that is present near the wall exterior face.

Wall Thrust

Thrust was considered when evaluating the wall section moment and shear capacity. A higher value of thrust within the tension-controlled region of the interaction diagram results in a higher nominal strength value. The calculated thrust is based upon the active (lower) value of lateral loading. This is deemed conservative relative to the at-rest (higher) value of lateral loading. Note that for lateral loading on the walls, the at-rest loading condition was used.

$K_a = 0.472$ $\gamma_{soil} = 115 \cdot pcf$ $H_2 = 33.5\,ft$

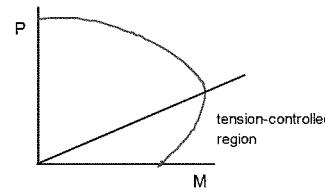
$W_{1active} := \gamma_L \cdot \gamma_H \cdot K_a \cdot \gamma_{soil} \cdot H_2$

$W_{2active} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_a \cdot (\gamma_{soil} - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_{1active} = 1819.748 \cdot psf$ $W_{2active} = 2854.097 \cdot psf$

$Wactive := if\left(W_{1active} > W_{2active}, W_{1active}, W_{2active}\right) \cdot b$

$\boxed{Wactive = 2854.097 \cdot plf}$ $W = 3152.38 \cdot plf$



Total Trust Acting On Wall AD:

$P_{ADthrust} := \frac{Wactive \cdot L_2}{2}$ $\boxed{P_{ADthrust} = 15.698 \cdot kip}$

Total Trust Acting On Wall AB:

$P_{ABthrust} := \frac{Wactive \cdot L_1}{2}$ $\boxed{P_{ABthrust} = 18.552 \cdot kip}$

Wall AD (Wall 1) Analysis

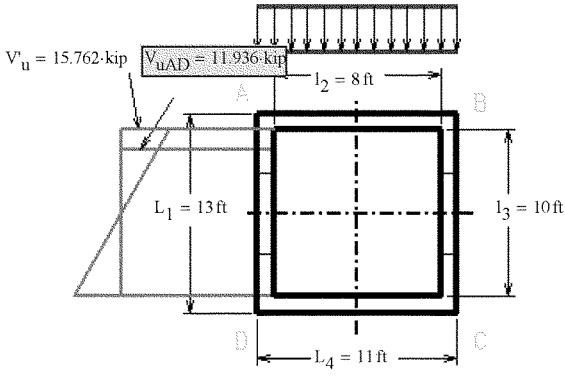
Shear Demand:

$W = 3152.38 \cdot plf \qquad l_1 = 10\,ft$

$V'_u := W \cdot \frac{l_1}{2} \qquad V'_u = 15.762 \cdot kip$

$d_{AD\_out\_end} = 14.563 \cdot in$

$V_{uAD} := V'_u \cdot \frac{\frac{l_1}{2} - d_{AD\_out\_end}}{\frac{l_1}{2}} \qquad V_{uAD} = 11.936 \cdot kip \qquad W = 3152.38 \cdot plf$



Shear Capacity and Factor of Safety:

$d_{AD\_out\_end} = 14.563 \cdot in \qquad \frac{P_{ADthrust}}{2000 \cdot A_{g1}} = 0.036\,psi \qquad f'_c = 3 \cdot ksi \quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$\phi V_{n\_AD} := \phi_V \left[2 \cdot \left(1 + \frac{P_{ADthrust}}{2000 \cdot A_{g1}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AD\_out\_end} \right) \right]$ ACI EQ (11-4)

$\phi V_{n\_AD} = 19.838 \cdot kip$

$V_{uAD} = 11.936 \cdot kip$

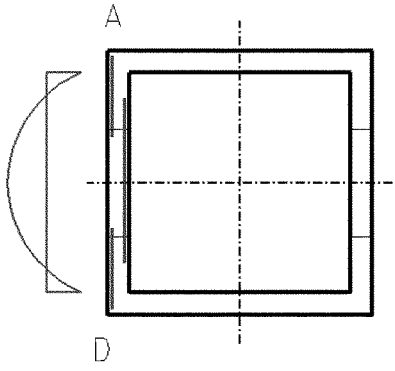
Factor of Safety $FS_{ADshear} := \frac{\phi V_{n\_AD}}{V_{uAD}}$

$FS_{ADshear} = 1.662$

DEPTH OF EQUIVALENT RECTANGULAR STRESS BLOCK

Wall AD Con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.152 \cdot klf$ $L_1 = 13\,ft$



$A_{ADout} = 0.72 \cdot in^2$ $a_{AD\_out} := \frac{A_{ADout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ADin} = 1.58 \cdot in^2$ $a_{AD\_in} := \frac{A_{ADin} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{AD\_out} = 0.941 \cdot in$ $a_{AD\_in} = 2.065 \cdot in$

Negative Moment Redistribution

Wall AD Con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

Beta one value is equal to 0.85 for f'c equal to or less than 4000 psi

$c_{AD\_out} := \frac{a_{AD\_out}}{\beta 1}$ $\boxed{c_{AD\_out} = 1.107\cdot in}$ $c_{AD\_in} := \frac{a_{AD\_in}}{\beta 1}$ $\boxed{c_{AD\_in} = 2.43\cdot in}$

The net tensile strain values

$\varepsilon_{tout} := 0.003\cdot\left(\frac{d_{AD\_out\_end}}{c_{AD\_out}} - 1\right) \qquad \varepsilon_{tout} = 0.036$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AD end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UADface}|}{\phi_B\cdot b\cdot d_{AD\_out\_end}^2} \qquad R_{nface} = 80.055\cdot psi \qquad M_{UADface} = -16.977\cdot kip\cdot ft$

$M_{UADin} = 22.428\cdot kip\cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003\cdot\left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17}\cdot\frac{R_{nface}}{f'_c}}} - 1\right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UADin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

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$\varepsilon'_{tface} = 0.077$

$$\varepsilon'\_ADfacedelta := \begin{cases} \varepsilon'_{tface}\cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases}$$

$\varepsilon'\_ADfacedelta = 0.200 \qquad \varepsilon'_{tface}\cdot 10 = 0.769$

$$RedistAD := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases}$$

$\boxed{RedistAD = \text{"Permitted"}}$

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UADface} := \begin{cases} M_{UADface}\cdot(1 - \varepsilon'\_ADfacedelta) & \text{if } \varepsilon'\_ADfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$\boxed{M'_{UADface} = -13.582\cdot kip\cdot ft}$ $M_{UADface} = -16.977\cdot kip\cdot ft$

Wall AD Con't

$$M'_{UADout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'\_ADfacedelta) & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -23.748 \cdot kip \cdot ft$ $M_{UADface} = -16.977 \cdot kip \cdot ft$ $M_{UDCout} = -29.685 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UADin} := \begin{cases} \frac{W \cdot (L_1 - D_2)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UADout} & \text{if } M'_{UADout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -23.748 \cdot kip \cdot ft$

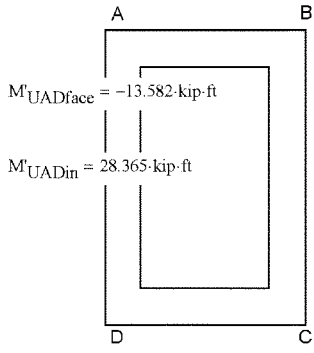
$M'_{UADin} = 28.365 \cdot kip \cdot ft$

$$\frac{W \cdot (L_1 - D_2)^2}{8} = 52.113 \cdot kip \cdot ft$$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAD = "Permitted"



Note: If redistribution is not allowed, "N/A" is shown.

Wall AD Magnified Moment Check

Wall AD Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, [illegible] are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{wl^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UADface}$ $\qquad$ $M_2 := M_{UADface}$

$M_1 = -16.977 \cdot kip \cdot ft$ $\qquad$ $M_2 = -16.977 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_1$ $\qquad$ $l_u = 13\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_1 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of [illegible] r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_1$ $\qquad$ $r = 0.45\,ft$

$Slender\_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender\_Ratio = 17.333$

$$ADSlenderness\_Check := if\left[Slender\_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], \text{"Ok"}, \text{"Consider Slenderness using ACI 10.10.1"}\right]$$

$ADSlenderness\_Check = \text{"Ok"}$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AD Mid-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ADin} = 8$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar\_No_{ADin} = 8$ $diaADin = 1 \cdot in$

$row := 2$

Iterations $i := 1 .. 2$ $i =$

1
2

Bar area at i $A_1 := A_{ADout}$ $A_2 := A_{ADin}$ $A_i =$

0.72
1.58

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$ $coverADin = 3 \cdot in$ $diaADin = 1 \cdot in$

$$dd_1 := \begin{cases} coverADout + \dfrac{diaADout}{2} & \text{if } M_{UADin} > 0ft \cdot kip \\ coverADin + \dfrac{diaADin}{2} & \text{otherwise} \end{cases}$$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

$dd_1 = 3.437 \cdot in$

$$dd_2 := \begin{cases} D_1 - \left(coverADin + \dfrac{diaADin}{2}\right) & \text{if } M_{UADin} > 0ft \cdot kip \\ D_1 - \left(coverADout + \dfrac{diaADout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.5 \cdot in$

Wall AD, Midspan Column Interaction Results

Wall AD Midspan

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \qquad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \geq F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -F_y \end{array} \right] \right] \cdot A_i \right] \ldots \cdot \phi_B$$

$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 3.207 \cdot in \qquad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \qquad a = 2.726 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0	-6.262 ·ks	0.72 ·in²	-4.509 ·kip	-25.079 ·kip·in
1.208	-0.011	-40	1.58	-63.2	347.6

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 323 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 637 \cdot kip \cdot in$

$\phi M_{nADin1} := (Mc + Mst) \qquad \phi M_{nADin1} = 79.96 \cdot kip \cdot ft$

$$\phi M_{nADin} := \begin{cases} \phi M_{nADin1} & \text{if } M_{UADin} > 0ft \cdot kip \\ \phi M_{nADin1} \cdot (-1) & \text{otherwise} \end{cases}$$

Wall AD Mid-Span Nomimal Moment Strength $\quad \phi M_{nADin} = 79.959 \cdot kip \cdot ft$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADmid}

Wall AD Mid-Span Con't

$\phi M_{nADin} = 79.959 \cdot kip \cdot ft$

$M_{UADin} = 22.428 \cdot kip \cdot ft$

$$FS_{ADin} := \left(\left| \frac{\phi M_{nADin}}{M_{UADin}} \right| \right)$$

$FS_{ADin} = 3.565$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADmid}$

$\phi M_{nADin} = 79.959 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UADin} = 28.365 \cdot kip \cdot ft$

$$FS'_{ADin} := \begin{cases} \left| \frac{\phi M_{nADin}}{M'_{UADin}} \right| & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADin} = 2.819$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADinspanCheck := if(ADSlenderness_Check ≠ "Ok" ,"ANALYSIS INVALID" ,"OKAY")

ADinspanCheck = "OKAY"

Wall AD End-of-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ADout} = 7$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADout = 3 \cdot in$ Cover -in

$Bar\_No_{ADout} = 7$ $diaADin = 1 \cdot in$

$A_1 := A_{ADin}$ $A_2 := A_{ADout}$ $A_i =$

$$\begin{bmatrix} 1.58 \\ 0.72 \end{bmatrix} \cdot in^2$$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$

$dd_1 := d_{AD\_in\_end}$ $dd_1 = 3.5 \cdot in$

$dd_2 := d_{AD\_out\_end}$ $dd_2 = 14.563 \cdot in$

Wall AD, End-of-Span Column Interaction Results

Wall AD End-of-Span

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \geq F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 2.87 \cdot in \quad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \quad a = 2.44 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.292 ft	-0.001	-19.087 ·l	1.58 ·in²	-30.158 ·kip	-165.87 ·kip·in
1.214	-0.012	-40	0.72	-28.8	160.2

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -6 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 581 \cdot kip \cdot in$

$\phi M_{nADface1} := (Mc + Mst) \qquad \phi M_{nADface1} = 47.93 \cdot kip \cdot ft$

$\phi M_{nADface} := \begin{cases} \phi M_{nADface1} & \text{if } M_{UADface} > 0ft \cdot kip \\ \phi M_{nADface1} \cdot (-1) & \text{otherwise} \end{cases} \qquad M_{UADface} = -16.977 \cdot kip \cdot ft$

Wall AD End-of-Span Nominal Moment Strength $\quad \phi M_{nADface} = -47.93 \cdot kip \cdot ft$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADface}

$\phi M_{nADface} = -47.93 \cdot kip \cdot ft$ $M_{UADface} = -16.977 \cdot kip \cdot ft$

$$FS_{ADface} := \left(\left| \frac{\phi M_{nADface}}{M_{UADface}} \right| \right)$$

$FS_{ADface} = 2.823$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, FS'_{ADface}

$$FS'_{ADface} := \begin{cases} \left| \frac{\phi M_{nADface}}{M'_{UADface}} \right| & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADface} = 3.529$ The end-of-span factor of safety always increases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADoutspanCheck := if(ADSlenderness_Check ≠ "Ok" ,"ANALYSIS INVALID" ,"OKAY")

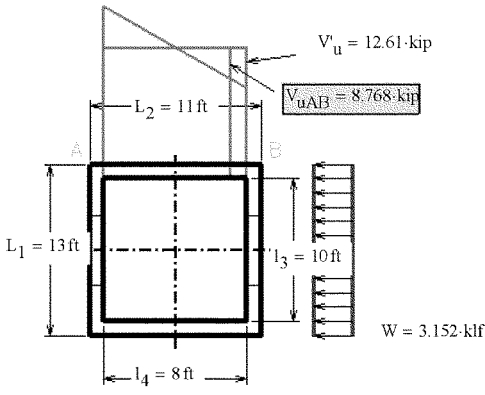
ADoutspanCheck = "OKAY"

Wall AB (Wall 2) Analysis

Shear Demand:

$W = 3152.38\,ft\cdot psf$ $\quad l_2 = 8\,ft$ $\quad d_{AB\_out\_end} = 14.625\cdot in$

$V'_u := W\cdot\frac{l_2}{2}$ $\quad V'_u = 12.61\cdot kip$ $\quad V_{uAB} := V'_u\cdot\frac{\frac{l_2}{2} - d_{AB\_out\_end}}{\frac{l_2}{2}}$ $\quad V_{uAB} = 8.768\cdot kip$



Total Thrust into Wall2:

$P_{ABthrust} = 18.552\cdot kip$

Note: Refer to wall AD calculations for derivation of the thrust values acting on wall AB.

Shear Capacity and Factor of Safety

$d_{AB\_out\_end} = 14.625\cdot in$ $\quad \frac{P_{ABthrust}}{2000\cdot A_{g2}} = 0.043\,psi$ $\quad f'_c = 3000\,psi$ $\quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$$\phi V_{n\_AB} := \phi_V\left[2\cdot\left(1 + \frac{P_{ABthrust}}{2000\cdot A_{g2}}\cdot\frac{in^2}{lbf}\right)\cdot\left(\sqrt{f'_c\cdot psi}\right)\cdot\left(b\cdot d_{AB\_out\_end}\right)\right]$$

ACI EQ (11-4) $\quad \phi V_{n\_AB} = 20.051\cdot kip$

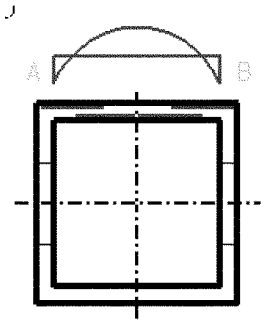
$V_{uAB} = 8.768\cdot kip$

Factor of Safety $\quad FS_{ABshear} := \frac{\phi V_{n\_AB}}{V_{uAB}}$ $\quad FS_{ABshear} = 2.287$

Depth of Equivalent Rectangular Stress Block

Wall AB con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.152 \cdot klf$ $L_1 = 13\,ft$



$A_{ABout} = 0.88 \cdot in^2$ $aABout := \frac{A_{ABout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ABin} = 1.2 \cdot in^2$ $aABin := \frac{A_{ABin} \cdot F_y}{0.85 f'_c \cdot b}$

$aABout = 1.15 \cdot in$ $aABin = 1.569 \cdot in$

Negative Moment Redistribution

Wall AB con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

$cABout := \frac{aABout}{\beta 1} \quad cABout = 1.353 \cdot in \qquad cABin := \frac{aABin}{\beta 1} \qquad c_{AD\_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AB\_out\_end}}{cABout} - 1 \right) \qquad \varepsilon_{tout} = 0.029$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AB end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UABface}|}{\phi_B \cdot b \cdot d_{AB\_out\_end}^2} \qquad R_{nface} = 90.425 \cdot psi \qquad M_{UABface} = -19.341 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UABin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

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$\varepsilon'_{tface} = 0.068$

$$\varepsilon'\_ABfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases} \qquad \varepsilon'\_ABfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 0.676$$

$$RedistAB := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases} \qquad \boxed{RedistAB = \text{"Permitted"}}$$

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UABface} := \begin{cases} M_{UABface} \cdot (1 - \varepsilon'\_ABfacedelta) & \text{if } \varepsilon'\_ABfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$\boxed{M'_{UABface} = -15.473 \cdot kip \cdot ft} \qquad M_{UABface} = -19.341 \cdot kip \cdot ft$

Wall AB con't

$$M'_{UABout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'\_ABfacedelta) & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABout} = -23.748 \cdot kip \cdot ft$ $M_{UABface} = -19.341 \cdot kip \cdot ft$ $M_{UDCout} = -29.685 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UABin} := \begin{cases} \frac{W \cdot (L_2 - D_1)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UABout} & \text{if } M'_{UABout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

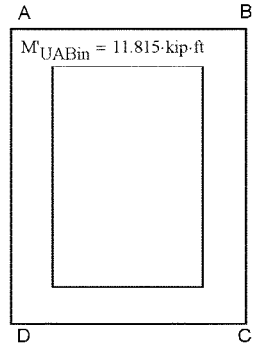
$M'_{UABin} = 11.815 \cdot kip \cdot ft$

$\frac{W \cdot (L_2 - D_1)^2}{8} = 35.563 \cdot kip \cdot f$ $M_{UABin} = 5.878 \cdot kip \cdot ft$

$M'_{UABin} = 11.815 \cdot kip \cdot ft$

Moments on Span AD After Redistribution RedistAB = "Permitted"

$M'_{UABface} = -15.473 \cdot kip \cdot ft$



Note: If redistribution is not allowed, "N/A" is shown.

Wall AB Magnified Moment Check

Wall AB Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, K values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UABface}$ $\qquad$ $M_2 := M_{UABface}$

$M_1 = -19.341 \cdot kip \cdot ft$ $\qquad$ $M_2 = -19.341 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_2$ $\qquad$ $l_u = 11\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_2 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_2$ $\qquad$ $r = 0.45\,ft$

$Slender\_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender\_Ratio = 14.667$

$$ABSlenderness\_Check := if\left[Slender\_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], "Ok", "Consider\ Slenderness\ using\ ACI\ 10.10.1"\right]$$

$ABSlenderness\_Check = "Ok"$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AB Mid-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ABin} = 7$ **Tension** bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar\_No_{ABin} = 7$ $diaABin = \blacksquare \cdot in$

$diaABout = 0.75 \cdot in$

Bar area at i $A_1 := A_{ABout}$ $A_2 := A_{ABin}$ $A_i =$

$$\begin{bmatrix} 0.88 \\ 1.2 \end{bmatrix} \cdot in^2$$

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$ $coverABin = 3 \cdot in$ $diaABin = 0.875 \cdot in$

$$dd_1 := \begin{cases} coverABout + \dfrac{diaABout}{2} & \text{if } M_{UABin} > 0ft \cdot kip \\ coverABin + \dfrac{diaABin}{2} & \text{otherwise} \end{cases}$$

$M_{UABin} = 5.878 \cdot kip \cdot ft$

$dd_1 = 3.375 \cdot in$

$$dd_2 := \begin{cases} D_2 - \left(coverABin + \dfrac{diaABin}{2}\right) & \text{if } M_{UABin} > 0ft \cdot kip \\ D_2 - \left(coverABout + \dfrac{diaABout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.563 \cdot in$

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB\_out\_mid}$ $dd_1 = 3.375 \cdot in$

$dd_2 := d_{AB\_in\_mid}$ $dd_2 = 14.563 \cdot in$

Wall AB, Mid-Span Column Interaction Results

Wall AB Mid-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \sum_{i=1}^{row} \left[\left[\begin{array}{l} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) \quad \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \cdot \phi_B \right]$$
$$+ 0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.965\cdot in \qquad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \qquad a = 2.521\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.281 ft	-0	-12.019 ·ksi	0.88 ·in²	-10.577 ·kip	-59.494 ·kip·in
1.214	-0.012	-40	1.2	-48	267

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 208\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 597\cdot kip\cdot in$

$\phi M_{nABin1} := (Mc + Mst) \qquad \phi M_{nABin1} = 67.04\cdot kip\cdot ft$

$$\phi M_{nABin} := \begin{cases} \phi M_{nABin1} & \text{if } M_{UABin} > 0ft\cdot kip \\ \phi M_{nABin1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB Mid-Span Nominal Moment Strength $\qquad \boxed{\phi M_{nABin} = 67.038\cdot kip\cdot ft}$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABmid}

Wall AB Mid-Span Con't

$\phi M_{nABin} = 67.038 \cdot kip \cdot ft$

$M_{UABin} = 5.878 \cdot kip \cdot ft$

$$FS_{ABin} := \left(\left| \frac{\phi M_{nABin}}{M_{UABin}} \right| \right)$$

$FS_{ABin} = 11.405$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ABmid}$

$\phi M_{nABin} = 67.038 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UABin} = 11.815 \cdot kip \cdot ft$

$$FS'_{ABin} := \begin{cases} \left| \frac{\phi M_{nABin}}{M'_{UABin}} \right| & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABin} = 5.674$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABinspanCheck := if(ABSlenderness_Check ≠ "Ok", "ANALYSIS INVALID", "OKAY")

ABinspanCheck = "OKAY"

Wall AB End-of-Span Nominal Moment Capacity $L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ABout} = 6$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABout = 3 \cdot in$ Cover -in

$Bar\_No_{ABout} = 6$ $diaABin = 0.875 \cdot in$

$A_1 := A_{ABin}$ $A_2 := A_{ABout}$ $A_i =$ $\begin{bmatrix} 1.2 \\ 0.88 \end{bmatrix} \cdot in^2$ <==The first value of A1 is the compression steel, A2 is the tension steel

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB\_in\_end}$ $dd_1 = 3.437 \cdot in$ Effective depth of the compression steel

$dd_2 := d_{AB\_out\_end}$ Effective depth of the tension steel

Wall AB, End-of-Span Column Interaction Results

Wall AB End-of-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row}\left[\left[\begin{array}{ll}\left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ +-\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{array}\right]\cdot A_i\right] \ldots \right]\cdot\phi_B$$
$$+0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.866\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.436\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0.001	-17.335 ·ksi	1.2 ·in²	-20.802 ·kip	-115.713 ·kip·in
1.219	-0.012	-40	0.88	-35.2	198

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 82\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 580\cdot kip\cdot in$

$\phi M_{nABface1} := (Mc + Mst) \qquad \phi M_{nABface1} = 55.2\cdot kip\cdot ft$

$$\phi M_{nABface} := \begin{cases}\phi M_{nABface1} & \text{if } M_{UABface} > 0ft\cdot kip \\ \phi M_{nABface1}\cdot(-1) & \text{otherwise}\end{cases}$$

Wall AB End-of-Span Nominal Moment Strength $\qquad \phi M_{nABface} = -55.204\cdot kip\cdot ft$

Wall AB End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABface}

$\phi M_{nABface} = -55.204 \cdot kip \cdot ft$ $M_{UABface} = -19.341 \cdot kip \cdot ft$

$$FS_{ABface} := \left(\left| \frac{\phi M_{nABface}}{M_{UABface}} \right| \right)$$

$FS_{ABface} = 2.854$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, FS'_{ABface}

$\varepsilon'\_ABfacedelta = 0.2$

$$FS'_{ABface} := \begin{cases} \left| \frac{\phi M_{nABface}}{M'_{UABface}} \right| & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABface} = 3.568$ The end-of-span factor of safety always increases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABoutspanCheck := if(ABSlenderness_Check ≠ "Ok" ,"ANALYSIS INVALID" ,"OKAY")

ABoutspanCheck = "OKAY"

Strength Analysis Results Summary

Span AD Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ADface} = 2.823$ $FS_{ADin} = 3.565$ $FS'_{ADface} = 3.529$ $FS'_{ADin} = 2.819$

$FS_{ADno\_redist} := \min(FS_{ADface}, FS_{ADin})$ $FS'_{ADredist} := \min(FS'_{ADface}, FS'_{ADin})$

$FS_{ADno\_redist} = 2.823$ $FS'_{ADredist} = 2.819$

$$FS_{ADmoment} := \begin{cases} \max(FS_{ADno\_redist}, FS'_{ADredist}) & \text{if } FS'_{ADredist} \neq \text{"N/A"} \\ FS_{ADno\_redist} & \text{otherwise} \end{cases}$$

$FS_{ADin} = 3.565$

$FS_{ADmoment} = 2.823$

$$AD\_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AD_Recommendation1 = "Perform Simple Beam Analysis"

$$AD\_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ADmoment} < 1.5 \\ \text{"Span AD OKAY for Moment Strength"} & \text{if } FS_{ADmoment} \geq 1.5 \end{cases}$$

AD_Recommendation2 = "Span AD OKAY for Moment Strength"

A listing of the span AD moment demand-capacity values, factors of safety and recommendations are provided in the following table

δ

Scalar Conversion Equations To Allow Text Entry Into Tables

Span AD Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-16.98	-47.93	2.82	22.43	79.96	3.57
Redistribution of Negative Moments	-13.58		3.53	28.36		2.82
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AD OKAY for Moment Strength						

N/A: Not Applicable

Span AB Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ABno\_redist} := \min(FS_{ABface}, FS_{ABin})$ $FS'_{ABredist} := \min(FS'_{ABface}, FS'_{ABin})$

$FS_{ABno\_redist} = 2.854$ $FS'_{ABredist} = 3.568$

$$FS_{ABmoment} := \begin{cases} \max(FS_{ABno\_redist}, FS'_{ABredist}) & \text{if } FS'_{ABredist} \neq \text{"N/A"} \\ FS_{ABno\_redist} & \text{otherwise} \end{cases}$$

$FS_{ABin} = 11.405$

$FS_{ABmoment} = 3.568$

$$AB\_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Required"} & \text{otherwise} \end{cases}$$

AB_Recommendation1 = "Perform Simple Beam Analysis"

$$AB\_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ABmoment} < 1.5 \\ \text{"Span AB OKAY for Moment Strength"} & \text{if } FS_{ABmoment} \geq 1.5 \end{cases}$$

AB_Recommendation2 = "Span AB OKAY for Moment Strength"

A listing of the span AB moment demand-capacity values, factors of safety and recommendations are provided in the following table

AB Scalar Conversion for Table

Span AB Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-19.34	-55.20	2.85	5.88	67.04	11.41
Redistribution of Negative Moments	-15.47		3.57	11.81		5.67
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AB OKAY for Moment Strength						

N/A: Not Applicable

Shear Factor of Safety Analysis Summary and Recommendations

Span AD

$$FS_{ADshear} = 1.662$$

Span AB

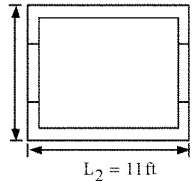
$$FS_{ABshear} = 2.287$$

NOTES TO STRENGTH SUMMARY RESULTS:

1. If simple beam analysis is performed on one wall, it must be performed on all four gatewell walls. This is required because fixity is lost when an adjacent wall is analyzed as a simple span. Stability should be considered when the gatewell is analyzed as having pinned supports.
2. Moment redistribution is not permitted when the mid-span and end-support moment values are negative.
3. Moment redistribution is not permitted when the reinforcement strain is less than 0.0075
4. Simple span analysis is recommended when the mid-span moment factor of safety is greater than 2.0. Final decision to use simple span analysis rests with the engineer.
5. Reliability analysis is recommended when the 1.0 < FS < 1.5
6. Caution is provided in the recommendations when the FS < 1.0

Properties of the Wall without an opening (Wall AB & CD)

Plan



$L_2 = 11\,ft$

$D_f = 2\,ft$

$H'' := H - D_f - y$

$H'' = 38.5\,ft$

Analyze wall at, $y := 0ft$ above base slab

Wall Thickness $D_2 = 18 \cdot in$ $l_2 = 8\,ft$

Clear Cover $C_c := coverABin$

MID-SPAN REINFORCEMENT

$Bar\_Spacing := 6in$

$Bar\_No :=$

$Bar\_No = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_s := \frac{A_{out}}{\frac{Bar\_Spacing}{12in}} \qquad \boxed{A_s = 1.2 \cdot in^2}$$

Reinforcement Bar Diameters

Effective Depth:

$$D_2 = 18 \cdot in \qquad dia = 0.875 \cdot in$$

$$d := D_2 - C_c - \frac{dia}{2}$$

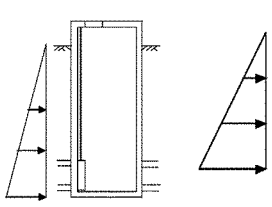
$$\boxed{d = 14.563 \cdot in}$$

(Wall without opening near base, cont'd)

Wall Loading

Wall Loading

WALLS



$H'' = 38.5\,ft$

$H'_2 := H''$

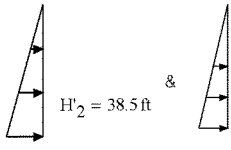
$H'_2 = 38.5\,ft$

Analysis of continuous wall perpendicular to opening
The walls above the opening were analyzed above.

$W'_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H'_2$

$W'_1 = 2841 \cdot psf$

Soil & Water



$H'_2 = 38.5\,ft$ &

$H'_3 := if\left[H'' < H_{blanket}, \frac{H''}{H_{blanket}} \cdot (HGL + H_{blanket}) - HEAD, H'' + HGL - HEAD\right]$

$H'_3 = 37.4\,ft$

$W'_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma_{soil} - \gamma_w) \cdot H'_2 + \gamma_w \cdot H'_3\right]$

$W'_2 = 3633 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

$W' := if(W'_1 > W'_2, W'_1, W'_2) \cdot b$ $\boxed{W' = 3633 \cdot plf}$

(Wall without opening near base, cont'd)

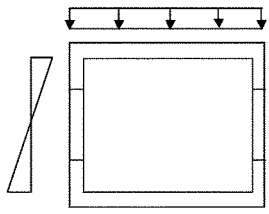
WALL without opening:

SHEAR

$f'_c = 3000\,psi$

$V'_u := W' \cdot \frac{l_2}{2}$

$V'_u = 14533\,lbf$



$V_u := V'_u \cdot \frac{\frac{l_2}{2} - d}{\frac{l_2}{2}}$

$b := 12in$

$V_u = 10 \cdot kip$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\phi V_n := \phi_V \left[2 \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot (d) \cdot b\right]$ ACI EQ (11-4) $\phi V_n = 19 \cdot kip$

Factor of Safety $FS_{opening\_shear} := \frac{\phi V_n}{V_u}$ $FS_{opening\_shear} = 1.9$

Bending

NOTE: Assumed to be simply supported with no thrust because of opening in adjacent walls.

Demand Moment:

$M_u := \frac{W' \cdot (L_2 - D_1)^2}{8}$ $M_u = 40.986 \cdot kip \cdot ft$

Moment Capacity:

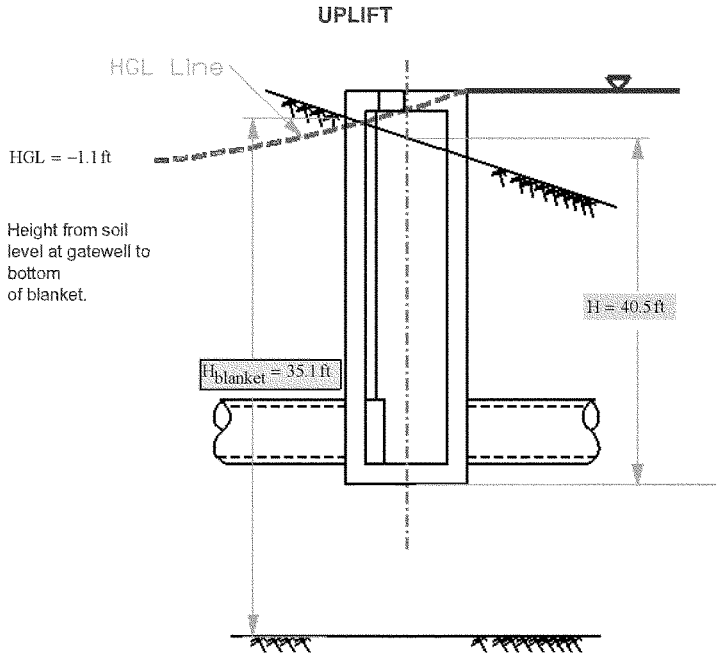
$a := \frac{A_s \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.569 \cdot in$

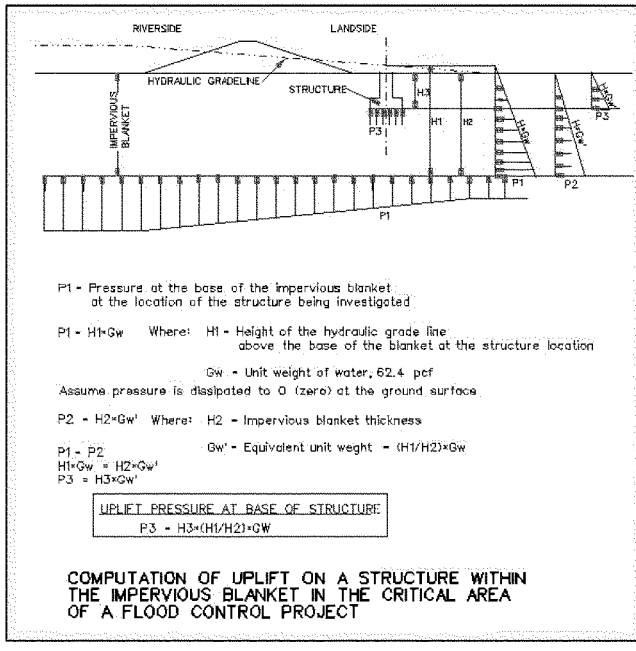
$\phi M_n := \phi_B \cdot A_s \cdot F_y \cdot \left(d - \frac{a}{2}\right)$ $\phi M_n = 55.1\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{opening\_mom} := \frac{\phi M_n}{M_u}$ $FS_{opening\_mom} = 1.3$

Uplift Analysis:

Uplift Using Local Protection Guidance





<< =from KC levees uplift guidance.

$A_{base} := L_1 \cdot L_2$ $\qquad A_{base} = 143\,ft^2$

Dissipated Head, if structural foundation in blanket.

$H_{blanket} = 35.1\,ft \quad HGL = -1.1\,ft \quad H = 40.5\,ft \qquad L_1 = 13\,ft \qquad L_2 = 11\,ft$

$$U_1 := \left(\frac{H_{blanket} + HGL}{H_{blanket}} \cdot H\right) \cdot L_1 \cdot L_2 \cdot \gamma_w \qquad U_1 = 350.1 \cdot kip$$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (L_1 \cdot L_2) \cdot \gamma_w\right] \qquad U_2 = 351.6 \cdot kip$$

Uplift

$H = 40.5\,ft$ $H_{blanket} = 35.1\,ft$ $H' = 0\,ft$

$U := if(H > H_{blanket}, U_2, U_1)$ $U = 351.6 \cdot kip$

$U_{uplift} := \frac{U}{L_1 \cdot L_2}$ $U_{uplift} = 2459 \cdot psf$

Weight of Structure

$Walls := (L_1 \cdot L_2 - l_1 \cdot l_2) \cdot (H + H') \cdot \gamma_c$ $Slabs := 2 \cdot (l_1 \cdot l_2) \cdot D_f \cdot \gamma_c$ $W_S := (Walls + Slabs)$ $W_S = 430.7 \cdot kip$

Surcharge Loads $Soil := 0lb$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

Weight of Surcharge Water $W_{water} := HEAD \cdot l_1 \cdot l_2 \cdot \gamma_w$ $W_{water} = 0 \cdot lb$ $W_g := 0lb$

Skin Friction

SOIL (Soil parameters entered previously)

$\phi = \blacksquare \cdot deg$	INTERNAL FRICTION ANGLE	
$\gamma_{soil} = 115 \cdot pcf$	DRY SOIL UNIT WEIGHT	
$\gamma_w = 62.4 \cdot pcf$	WATER UNIT WEIGHT	
$\gamma_{sub} := (\gamma_{soil} - \gamma_w)$	SUBMERGED SOIL UNIT WEIGHT	$\gamma_{sub} = 52.6 \cdot pcf$
$K_o = 0.642$	AT-REST SOIL COEFFICIENT. CALCULATED PREVIOUSLY	
$\mu := 0.25$	FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.	
$U_{uplift} = 2458.56 \cdot psf$	UPLIFT FORCE ACTING ON BASE OF STRUCTURE	
$c := 5ft$	ASSUMED CRACK DEPTH	

STRUCTURE

$H = 40.5\,ft$	DEPTH OF STRUCTURE BELOW GRADE	
$per := 2 \cdot L_1 + 2 \cdot L_2$	STRUCTURE PERIMETER	$per = 48\,ft$
$W_S = 430.725 \cdot kip$	STRUCTURE DEAD WEIGHT	

Skin Friction (Continued)

1. Effective Lateral Load that Contributes to Side Friction

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma_{soil}$ $R_1 = 92876.875\cdot\frac{lbf}{ft}$

$R_2 := (H - c)\cdot U_{uplift}\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $R_2 = 49027.025\cdot\frac{lbf}{ft}$

$R_1 - R_2 = 43849.85\cdot\frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.642$ $\mu = 0.25$ $per = 48ft$

$P_{v1} := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per$ $P_{v1} = 337.626\cdot kip$

Ignore_Skin_Friction :=

$P_v := \begin{cases} 0kip & \text{if Ignore\_Skin\_Friction = "Yes"} \\ P_{v1} & \text{otherwise} \end{cases}$ $P_v = 338\cdot kip$

Final Uplift Factor of Safety:

$FS_{Uplift} := \frac{W_S + W_{water} + Soil + P_v}{U}$ $FS_{Uplift} = 2.185$

NOTE: EM 1110-2-2100 defines 1.1 Factor of Safety acceptable for extreme loading case. The gatewells have minimal overhangs to produce weight of water above structure, W_G, therefore W_G ignored (conservative).

Base Slab Analysis

NOTE:
1. The following calculation assumes two opposite edges fixed and the other opposite edges simply supported. This occurs when a large pipe takes up most of the wall so that wall should be simply supported. Coef came from "Theory of Plates and Shells" second edition, by Timoshenko. This is conservative for gatewells without large pipe openings. So if FS is critical than may want to reanalize using the table of coeff for all sides fixed from "EM 27 - Moments and Reactions for Rect. Plates".
2. Only the end-of-span moment is checked, but in cases where the base slab top and bottom reinf is not the same, it may be necessary to check the mid-span.

Slab Loading:

Weight of structure: $W_S = 430.725 \cdot kip$ Uplift: $U = 351.574 \cdot kip$ Skin Friction: $P_{v1} = 337.626 \cdot kip$

Load acting up on baseslab:

$$W := if(U > W_S, U + P_{v1}, W_S) = 430.725 \cdot kip \qquad W_{UNIFORM} := \frac{\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot W}{L_1 \cdot L_2} = 3.012 \cdot ksf$$

END-OF-SPAN MOMENT IS CHECKED HERE:
Plate Coefficient and Demand Moment:

$k := 1..5$

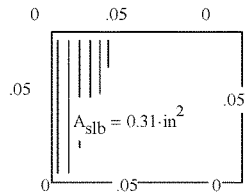
$ab_k :=$ $Coeff_k :=$

0.5	.1191
.625	.109
.769	.0938
0.9091	.0787
1	.0697

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

From plates tables:

a/b	Coefficient
1	.0697
.9091	.0787
.769	.0938
.625	.109
.5	.1191



$$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.09$$

$l_2 = 8\,ft \qquad b = 1\,ft$

$$M_{uFe} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b \qquad M_{uFe} = 17.438 \cdot kip \cdot ft$$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 4.9 \cdot \frac{in^2}{ft} \qquad d_f = 1.708\,ft$$

$$\phi M_{nFe} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFe} = 21\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomente} := \frac{\phi M_{nFe}}{M_{uFe}}$ $FS_{slabmomente} = 1.203$

Base Slab Analysis, con't

MID-SPAN MOMENT (My) IS CHECKED HERE:
Plate Coefficient and Demand Moment:

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

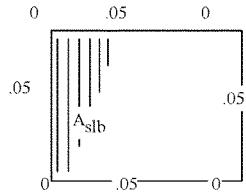
From plates tables:

a/b	Coefficient
1	0.0332
.9091	0.0371
.769	0.0426
.625	0.0469
.5	0.0474

$k := 1..5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0474
.625	0.0469
.769	0.0426
0.9091	0.0371
1	0.0332



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.041$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm1} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm1} = 7.977 \cdot kip \cdot ft$

$A_{slb} := 0.44in^2$ $C_{cf} := 3.0625in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 6.9 \cdot \frac{in^2}{ft}$ $d_f = 1.745\,ft$

$\phi M_{nFm1} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm1} = 30.3\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm1} := \frac{\phi M_{nFm1}}{M_{uFm1}}$ $FS_{slabmomentm1} = 3.797$

Base Slab Analysis, con't

MID-SPAN MOMENT (Mx) IS CHECKED HERE:
Plate Coefficient and Demand Moment:

$a/b = L_2/L_1$ $\quad \frac{l_2}{l_1} = 0.8$

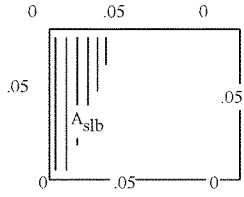
From plates tables:

a/b	Coefficient
1	0.0244
.9091	0.0307
.769	0.0446
.625	0.065
.5	0.0869

$k := 1..5$

$ab_k :=$ $\quad Coeff_k :=$

0.5	0.0869
.625	0.065
.769	0.0446
0.9091	0.0307
1	0.0244



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.042$

$l_2 = 8\,ft \quad b = 1\,ft$

$M_{uFm2} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $\quad M_{uFm2} = 8.005 \cdot kip \cdot ft$

$A_{slb} := 1.06in^2$ $\quad C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft}$ $\quad d_f = 1.802\,ft$

$\phi M_{nFm2} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\quad \phi M_{nFm2} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $\quad FS_{slabmomentm2} := \frac{\phi M_{nFm2}}{M_{uFm2}}$ $\quad \boxed{FS_{slabmomentm2} = 9.24}$

Base Slab Analysis, con't

Note that there is a concentration of reinforcing in one direction in the base slab. Assume that the base slab acts as a plate until reinforcing in the weak direction begins to yield and then the moments will redistribute to reinforcing in the other direction (a plastic hinge will form in the base slab at the wall). When this happens the base slab should be analyzed as if it were bending as if it were a simple span:

$L := 10ft$ $b = 1\,ft$

$$M_{ss} := \frac{W_{UNIFORM} \cdot L^2 \cdot b}{8} = 37.651\,ft \cdot \frac{kip \cdot ft}{ft}$$

$A_{slb} := 1.06in^2$ $C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft} \qquad d_f = 1.802\,ft$$

$$\phi M_{nFm3} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFm3} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomentm3} := \dfrac{\phi M_{nFm3}}{M_{ss}}$ $\boxed{FS_{slabmomentm3} = 1.964}$

**********Overall Factors of Safety**********

Uplift Stability

$FS_{Uplift} = 2.2$

Strength

Wall AD $L_1 = 13\,ft$

Wall AB $L_2 = 11\,ft$

WALL MOMENT

The overall wall moment factor of safety, $FS_{wallmoment}$

$FS_{ADmoment} = 2.823$ $FS_{ABmoment} = 3.568$ $FS_{opening\_mom} = 1.345$

$FS_{wallmoment} := \min(FS_{ADmoment}, FS_{ABmoment}, FS_{opening\_mom})$ $FS_{wallmoment} = 1.34$

WALL SHEAR

$FS_{ADshear} = 1.662$ $FS_{ABshear} = 2.287$ $FS_{opening\_shear} = 1.891$

$FS_{wallshear} := \min(FS_{ADshear}, FS_{ABshear}, FS_{opening\_shear})$ $FS_{wallshear} = 1.662$

BASE SLAB MOMENT FACTOR OF SAFETY

$FS_{slabmoment} := \max(\min(FS_{slabmomente}, FS_{slabmomentm1}, FS_{slabmomentm2}), FS_{slabmomentm3})$

$FS_{slabmoment} = 1.964$

Summary :

$FS_{strength} := \min(FS_{wallmoment}, FS_{wallshear}, FS_{slabmoment})$ $FS_{strength} = 1.3$

$$Mechanism := \begin{cases} \text{"Wall Bending"} & \text{if } FS_{strength} = FS_{wallmoment} \\ \text{"Wall Shear"} & \text{if } FS_{strength} = FS_{wallshear} \\ \text{"Base Slab Moment"} & \text{otherwise} \end{cases}$$

Mechanism = "Wall Bending"

$FS_{Uplift} = 2.2$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. The reasoning being the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because this is an existing structure in the field that is routinely inspected with no visible history of problems, a factor of safety of 1.5 is deemed acceptable. If modifications are required for strength, the modified structure must be evaluated from a standard design approach (include factors - no FS calculation)

END OF ANALYSIS

US Army Corps of Engineers

Probability of Failure

CIDKS GATEWELL 106+49
Bending in wall AB/CD

Comp by:KSM 7-3-08
Chkd by:

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. This structure does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 45 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Calculating Factors of Safety*

WATER AT TOP OF WALL

Condition under consideration from strength check: Mu for wall AB/CD (from mathcad strength analysis). $M_u := 40.986 \frac{kip \cdot ft}{ft}$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$A_s := 1.2 \frac{in^2}{ft}$ Area of Steel

$dist_{CL} := 3.4375in$ Distance from inside wall to centerline of reinforcement

$T_{wall} := 1.5ft$ Thickness of wall

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$d := T_{wall} - dist_{CL}$ $d = 14.563 \cdot in$

Mean Concrete Strength and Steel Yield Strength

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cM} \cdot b}$ $a = 1.412 \cdot in$

$\phi M_M := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\phi M_M = 62.35 \cdot \frac{kip \cdot ft}{ft}$

$FS_D := \frac{\phi M_M}{M_u}$ $FS_D = 1.521$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cU} \cdot b}$ $a = 1.238 \cdot in$

$\phi M_{cU} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\phi M_{cU} = 62.74 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcu} := \frac{\phi M_{cU}}{M_u}$ $FS_{fcu} = 1.531$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$a := \frac{A_s \cdot F_{yM}}{0.85 f'_{cL} \cdot b}$ $\quad a = 1.642 \cdot in$

$\phi M_{cL} := \phi_B A_s \cdot F_{yM} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{cL} = 61.84 \cdot \frac{kip \cdot ft}{ft}$

$FS_{fcl} := \frac{\phi M_{cL}}{M_u}$ $\quad FS_{fcl} = 1.509$

Upper Steel Yield Strength

$F_{yU} := F_{yM} + F_{yM} \cdot 0.14$ $\quad F_{yU} = 51.3 \cdot ksi$

$a := \frac{A_s \cdot F_{yU}}{0.85 f'_{cM} \cdot b}$ $\quad a = 1.609 \cdot in$

$\phi M_{sU} := \phi_B A_s \cdot F_{yU} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sU} = 70.58 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyu} := \frac{\phi M_{sU}}{M_u}$ $\quad FS_{Fyu} = 1.722$

Lower Steel Yield Strength

$F_{yL} := F_{yM} - F_{yM} \cdot 0.14$ $\quad F_{yL} = 38.7 \cdot ksi$

$a := \frac{A_s \cdot F_{yL}}{0.85 f'_{cM} \cdot b}$ $\quad a = 1.214 \cdot in$

$\phi M_{sL} := \phi_B A_s \cdot F_{yL} \cdot \left(d - \frac{a}{2}\right)$ $\quad \phi M_{sL} = 54.01 \cdot \frac{kip \cdot ft}{ft}$

$FS_{Fyl} := \frac{\phi M_{sL}}{M_u}$ $\quad FS_{Fyl} = 1.318$

$FS_D = 1.521$ $\quad FS_{fcu} = 1.531$ $\quad FS_{Fyu} = 1.722$

$FS_{fcl} = 1.509$ $\quad FS_{Fyl} = 1.318$

VI. Probability of Failure Calculation

$\Delta F_{Fy} := FS_{Fyu} - FS_{Fyl}$ $\Delta F_{Fy} = 0.404$

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\Delta F_{fc} = 0.022$

ACI EQ (11-4)

$$\sigma_F := \sqrt{\left(\frac{\Delta F_{Fy}}{2}\right)^2 + \left(\frac{\Delta F_{fc}}{2}\right)^2}$$ $\sigma_F = 0.202$

$$V_F := \frac{\sigma_F}{FS_D}$$ $V_F = 0.133$

$$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$$ $\beta_{LN} = 3.101$

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 99.9 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 0.1 \cdot \%$

Gatewell 106+49, Future Condition Strength and Uplift Stability Analysis:

Major tasks assocaited with this analysis:

1. Obtain the geometry for the gatewell and levee which are taken from the Record drawings.
2. Obtain the geotechnical parameters which are obtained from the Geotechnical Engineer.
3. Perform uplift analysis assuming gate is empty.

Calculation Convention Notes:

1. All input values are highlighted in
2. Results are highlighted in green
3. Assumption notes are displayed in red

References:

Gatewell Sta. 106+49 elevations, location and reinforcing details from American Royal Arena Storm Sewer System for KC, MO Department of Public Works plans by Shafer, Kline and Warren 1973

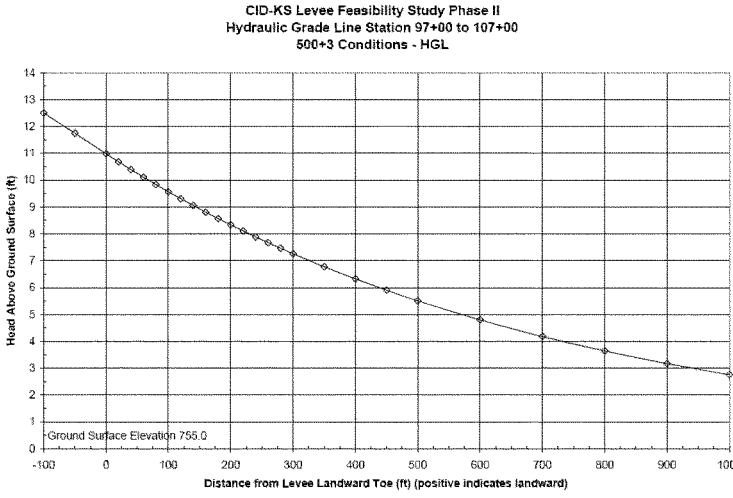
Geotech Parameters:

Information Used to Calculate Structures HGL's - Blanket Information			
Station	Condition	Bottom of Landside Blanket Elevation	Top of Blanket Elevation
49+00 to 51+00	Existing	719	751
51+00 to 57+00	Existing	720	750
57+00 to 59+00	Existing	722	750
59+00 to 63+00	Existing	725	735
63+00 to 68+00	Existing	728	735
68+00 to 73+00	Existing	718	733
77+00 to 82+00	Existing	720	738
82+00 to 85+00	Existing	730	740
85+00 to 88+00	Existing	730	742
88+00 to 90+00	Existing	725	732
92+00 to 94+50	Existing	735	744
102+00 to 107+00	Existing	730	742
119+00 to 127+00	Existing	722	756
136+00 to 143+00	Existing	725	753
150+00 to 153+00	Existing	725	751
159+00 to 163+00	Existing	725	745

Geotech Parameters (con't):

Recommended Parameters for Phase II Feasibility Study						
	Unit Weight (pcf)		Drained Shear Strength		Undrained Shear Strength*	
Material	Moist	Saturated	♦' (degrees)	c' (psf)	♦ (degrees)	c (psf)
Embankment	115	120	29	0	0	1000
Fill (cinders/rubble)	110	115	21	0	N/A	N/A
Foundation Blanket	110	115	24	0	0	500
Foundation Sand	115	120	31	0	N/A	N/A
*Estimated from average strength values for these materials						

Hydraulic Gradeline:



HGL & blanket elevations provided by Glen Bellew 6/2/08

Soil Elevation Assumed by geotech: $El_{soil\_geotech} = 0ft$

Elevation of the HGL using geotech datum: $HGL_{geotech} = 767ft$

Elevation of bottom of blanket: $El_{bot\_blanket} = 730ft$

Input Values

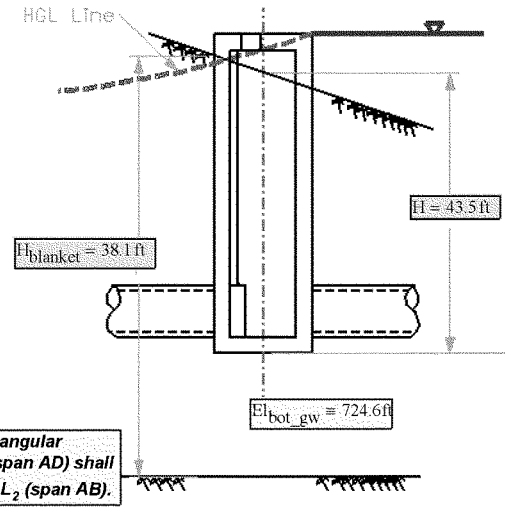
From record drwg $El_{top\_gw} \equiv 768.1ft$

RAISED GATEWELL 3 FEET FOR FC

This is determined from the Record drawings

$El_{soil\_at\_gw} \equiv El_{top\_gw}$

$El_{soil\_at\_gw} = 768.1\ ft$



$H_{blanket} \equiv El_{soil\_at\_gw} - El_{bot\_blanket}$

The HGL from the geotech needs to be adjusted using the soil elevation at the gatewell as the datum:

$HGL := HGL_{geotech} \cdots + 0ft - (El_{soil\_at\_gw} - El_{soil\_geotech})$

$HGL = -1.1\ ft$

Water head pressure in Wet Well under operating conditions.

$HEAD := 0ft$

Buried Height of

$H \equiv El_{soil\_at\_gw} - El_{bot\_gw}$

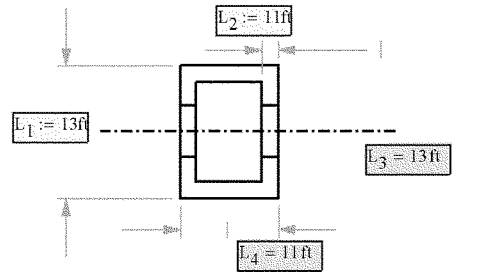
$H = 43.5\ ft$

NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

Projected Height of Gatewell above soil

$H' := El_{top\_gw} - El_{soil\_at\_gw}$

$H' = 0\ ft$



Width analyzed: $b := 12in$

$L_3 := L_1$

Floor Thickness $D_f := 24in$

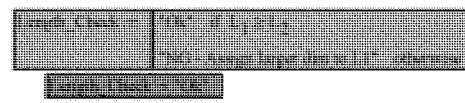
$L_4 := L_2$

Slab clear cover $C_{cf} := 3.5in$

Slab effective depth $d_f := D_f - C_{cf}$

$d_f = 20.5 \cdot in$

Note: Upper case "L" represents the out-to-out dimension.
Lower case "l" representst the clear span dimension.



Wall Thickness

$D_1 := 18in$ Wall 1 (Span AD) wall thickness

$D_2 := 18in$ Wall 2 (Span AB) wall thickness

$D_3 := D_1$ Wall 3 (Span BC) must be of the same thickness as Wall 1 (Span AD)

$D_4 := D_2$ Wall 4 (Span CD) must be of the same thickness as Wall 2 (Span AB)

Clear Spans

$l_1 := L_1 - 2 \cdot D_2$ $l_1 = 10ft$ Clear span wall AD

$l_2 := L_2 - 2D_1$ $l_2 = 8ft$ Clear span of wall AB

$l_3 := l_1$ $l_3 = 10ft$ Clear span of wall BC

$l_4 := l_2$ $l_4 = 8ft$ Clear span of wall CD

Concrete Section Areas $b = 1ft$

$A_{g1} := D_1 \cdot b$ $A_{g1} = 1.5ft^2$

$A_{g2} := D_2 \cdot b$ $A_{g2} = 1.5ft^2$

$A_{g3} := A_{g1}$ $A_{g3} = 1.5ft^2$

$A_{g4} := A_{g2}$ $A_{g4} = 1.5ft^2$

$d_f = 20.5 \cdot in$

Base Slab Reinforcment

Reinforcement information provided by the record drawings $Bar\_Sp_{slb} := 12in$

$BarNo_{slb} :=$

$BarNo_{slb} = 5$

Base Slab Reinforcement

$A_{slb} := \dfrac{A_{slb}}{\dfrac{Bar\_Sp_{slb}}{12in}}$ $A_{slb} = 0.31 \cdot in^2$

Wall Reinforcement Areas At Elevation Bot of Gatewell:

Reinforcement information provided by the record drawings

I. WALL AD $L_1 = 13ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar\_Spacing_{ADout} := 10in$ $Bar\_Spacing_{ADin} := 6in$

$Bar\_No_{ADout} :=$ $Bar\_No_{ADout} = 7$ $Bar\_No_{ADin} :=$ $Bar\_No_{ADin} = 8$

Wall Reinforcement

The area of steel will be calculated /ft:

$A_{ADout} := \dfrac{A_{out}}{\dfrac{Bar\_Spacing_{ADout}}{12in}}$ $A_{ADin} := \dfrac{A_{in}}{\dfrac{Bar\_Spacing_{ADin}}{12in}}$ $A_{out} = 0.6 \cdot in^2$ $A_{in} = 0.79 \cdot in^2$

$A_{ADout} = 0.72 \cdot in^2$ $A_{ADin} = 1.58 \cdot in^2$

Reinf at wall AB is based on the Half-Section thru gate chamber detail on sheet 4 of 5.

II. WALL AB $L_2 = 11ft$

END SUPPORT AND MID-SPAN REINFORCEMENT

$Bar\_Spacing_{ABout} := 6in$ $Bar\_Spacing_{ABin} := 6in$

$Bar\_No_{ABout} :=$ $Bar\_No_{ABout} = 6$ $Bar\_No_{ABin} :=$ $Bar\_No_{ABin} = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_{ABout} := \frac{A_{out}}{\frac{Bar\_Spacing_{ABout}}{12in}} \qquad A_{ABin} := \frac{A_{in}}{\frac{Bar\_Spacing_{ABin}}{12in}} \qquad A_{out} = 0.44 \cdot in^2 \qquad A_{in} = 0.6 \cdot in^2$$

$$\boxed{A_{ABout} = 0.88 \cdot in^2} \qquad \boxed{A_{ABin} = 1.2 \cdot in^2}$$

Reinforcement Bar Diameters

Reinforcement Clear Cover

Wall AD

$\boxed{coverADout := 3in}$

$\boxed{coverADin := 3in}$

Wall AB

$\boxed{coverABout := 3in}$

$\boxed{coverABin := 3in}$

Effective Depth:

These effective depths are used for the AD end analysis

$D_1 = 18 \cdot in \qquad diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$

$$d_{AD\_in\_end} := coverADin + \frac{diaADin}{2} \qquad d_{AD\_out\_end} := D_1 - coverADout - \frac{diaADout}{2}$$

$$\boxed{d_{AD\_in\_end} = 3.5 \cdot in} \qquad \boxed{d_{AD\_out\_end} = 14.563 \cdot in}$$

These effective depths are used for the AB end analysis

$diaABin = 0.875 \cdot in \qquad diaABout = 0.75 \cdot in \qquad D_2 = 1.5 ft$

$$d_{AB\_in\_end} := coverABin + \frac{diaABin}{2} \qquad d_{AB\_out\_end} := D_2 - coverABout - \frac{diaABout}{2}$$

$$\boxed{d_{AB\_in\_end} = 3.437 \cdot in} \qquad \boxed{d_{AB\_out\_end} = 14.625 \cdot in}$$

These effective depths are used for the AD mid analysis

$diaADin = 1 \cdot in \qquad diaADout = 0.875 \cdot in$

$$d_{AD\_in\_mid} := D_1 - coverADin - \frac{diaADin}{2} \qquad d_{AD\_out\_mid} := coverADout + \frac{diaADout}{2}$$

$$\boxed{d_{AD\_in\_mid} = 14.5 \cdot in} \qquad \boxed{d_{AD\_out\_mid} = 3.437 \cdot in}$$

These effective depths are used for the AB mid analysis

$diaABin = 0.875 \cdot in$ $diaABout = 0.75 \cdot in$

$d_{AB\_in\_mid} := D_2 - coverABin - \frac{diaABin}{2}$

$d_{AB\_in\_mid} = 14.563 \cdot in$

$d_{AB\_out\_mid} := coverABout + \frac{diaABout}{2}$

$d_{AB\_out\_mid} = 3.375 \cdot in$

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, Engineered Concrete Structures, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Soil Properties $\phi_B := 21deg$ $\gamma_{soil} := 115pcf$

At Rest Soil Pressure $K_o := 1 - \sin(\phi_B)$ $K_o = 0.642$

Active Soil Pressure: $K_a := \tan\left(45deg - \frac{\phi_B}{2}\right)^2$ $K_a = 0.472$

Concrete Unit Weight $\gamma_c := 150pcf$ Concrete Properties $f'_c := 3ksi$ Water Unit Weight $\gamma_w := 62.4pcf$

Steel Properties $F_y := 40ksi$ Modulus of Elasticity: $Es := 29000ksi$

$\beta 1 := if\left(f'_c \le 4ksi, 0.85, if\left(f'_c > 8ksi, 0.65, 0.85 - 0.05 \cdot \frac{f'_c - 4ksi}{ksi}\right)\right)$ $\beta 1 = 0.85$ $f'_c = 3000psi$

Load & Resistance Factor Design Resistance and Load Factor Values

Strength Reduction Factors

Shear Strength $\phi_V := 1.0$

Flexural Strength $\phi_B := 1.0$

Load Factors

Dead and Live Load Factor $\gamma_L := 1.0$ Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Load Factor $\gamma_H := 1.0$ Hydraulic Factor EM 1110-2-2104 (3-2)

Extreme Case Factor $\gamma_X := 1.0$ Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

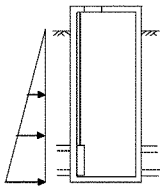
Note: Load Factors (1.6 for live load and 1.3 for hydraulic structure) and Strength Reduction Factors (.85 for shear, 0.90 for bending) are not applied for in this initial evaluation. Instead a factor of safety is computed for the existing structure. If a strength concern is found, a design approach (applying these factors directly) will be taken for any recommended solutions.

Wall Loading

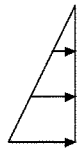
Wall Loading

WALLS

Special Case with Monolithically Placed exit and entrance:



Soil



$H = 43.5\,ft$

$H_2 := H$ OR

$H_2 = 43.5\,ft$ $H_2 := H_2 - 7ft$

$W_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H_2$

$W_1 = 2693 \cdot psf$

Soil & Water

$H_2 = 36.5\,ft$

&

$H_3 := \text{if}\left[H < H_{blanket}, \frac{H}{H_{blanket}} \cdot (HGL + H_{blanket}) - HEAD, H + HGL - HEAD\right]$

$H_3 = 42.4\,ft$

$H_2 := H_3 - 7ft$

$W_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[K_o \cdot (\gamma_{soil} - \gamma_w) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_2 = 3441 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

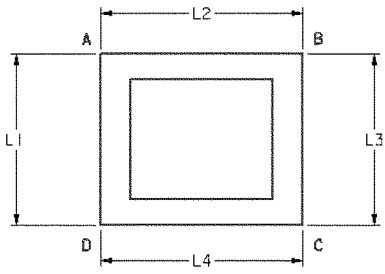
$W := \text{if}(W_1 > W_2, W_1, W_2) \cdot b$ $W = 3441 \cdot plf$

Moment Distribution of Gatewell Walls

NOTE: Check continuity of reinforcement at corners. If continuity is present, use moment distribution to determine the demand moment values. Otherwise, analyze the gatewell wall sections as simply supported beams.

The following calculations determine the distributed moment based on relative stiffness. In order for the calculations to be valid, the following must be true:
1. The structure has four walls. The walls that are parallel to each other are of the same thickness.
2. The EI value is based on the wall length and the wall thickness.
3. The walls are orthagonal.
4. The walls are exposed to uniform loading W, and $W = 3441 \cdot plf$ is the same for each wall.

Determine Moment and Shear:



Design Section Length

$L_1 = 13ft$ $L_2 = 11ft$ $L_3 = 13ft$ $L_4 = 11ft$

Moment of Inertia:

$b = 1ft$ $D_1 = 1.5ft$ $D_2 = 1.5ft$

$D_4 = 1.5ft$ $D_3 = 1.5ft$

$I_{beam1} := \frac{b \cdot D_1^3}{12}$ $I_{beam1} = 5832 \cdot in^4$

$I_{beam2} := \frac{b \cdot D_2^3}{12}$ $I_{beam2} = 5832 \cdot in^4$

$I_{beam3} := I_{beam1}$ $I_{beam3} = 5832 \cdot in^4$

$I_{beam4} := I_{beam2}$ $I_{beam4} = 5832 \cdot in^4$

Enter End Conditions:

All carryover factors are equal to 0.5 (idealize as a continuous beam over supports A, B, C and D).

$COF_A := 0.5$ $COF_B := COF_A$ $COF_C := COF_A$ $COF_D := COF_A$

$A := COF_A$ $B := COF_B$ $C := COF_C$ $D' := COF_D$

Fixed End Moments For A Uniform Load Between Fixed Supports using center of support to center of support distance per ACI 318-02 8.7.2 :

$L_1 = 13ft$ $L_2 = 132 \cdot in$ $L_3 = 13ft$ $L_4 = 11ft$ $W = 3440.829 \cdot plf$

$M_{fix\_1} := \frac{W \cdot (L_1 - D_2)^2}{12}$ $M_{fix\_1} = 37.921 \cdot kip \cdot ft$ $M_{fix\_2} := \frac{W \cdot (L_2 - D_1)^2}{12}$ $M_{fix\_2} = 25.878 \cdot kip \cdot ft$

$M_{fix\_3} := \frac{W \cdot (L_3 - D_2)^2}{12}$ $M_{fix\_3} = 37.921 \cdot kip \cdot ft$ $M_{fix\_4} := \frac{W \cdot (L_4 - D_1)^2}{12}$ $M_{fix\_4} = 25.878 \cdot kip \cdot ft$

Distribution Factors

	M_{AD}	M_{AB}	M_{BA}	M_{BC}	M_{CB}	M_{CD}	M_{DC}
COF	0.50	0.50	0.50	0.50	0.50	0.50	0.50
DF	0.46	0.54	0.54	0.46	0.46	0.54	0.54
FEM	-37.92	25.88	-25.88	37.92	-37.92	25.88	-25.88
D#1	5.520	6.523	-6.523	-5.520	5.520	6.523	-6.523
CO#1	-2.760	-3.262	3.262	2.760	-2.760	-3.262	3.262
DF#2	2.760	3.262	-3.262	-2.760	2.760	3.262	-3.262
CO#2	-1.380	-1.631	1.631	1.380	-1.380	-1.631	1.631
DF#3	1.380	1.631	-1.631	-1.380	1.380	1.631	-1.631
CO#3	-0.690	-0.815	0.815	0.690	-0.690	-0.815	0.815
DF#4	0.690	0.815	-0.815	-0.690	0.690	0.815	-0.815
CO#3	-0.345	-0.408	0.408	0.345	-0.345	-0.408	0.408
DF#4	0.345	0.408	-0.408	-0.345	0.345	0.408	-0.408
	-32.40	32.40	-32.40	32.40	-32.40	32.40	-32.40

Attach proper units to distributed values

$M_{ADend} := M'_{AD} \cdot ft \cdot kip$ $M_{ADend} := M'_{AB} \cdot ft \cdot kip$

$M_{CBend} := M'_{CB} \cdot ft \cdot kip$ $M_{DCend} := M'_{DC} \cdot ft \cdot kip$

End-of-Span Moments After Distribution

$M_{ADend} = -32.401 \cdot kip \cdot ft$ $M_{ADend} = -32.401 \cdot kip \cdot ft$

$M_{CBend} = -32.401 \cdot kip \cdot ft$ $M_{DCend} = -32.401 \cdot kip \cdot ft$

Note that the end-of-span moment value is the same at each joint because of uniform and equal loading present on all spans.

$W = 3440.829 \cdot plf$

Note: L_1 represents span AD, L_2 represents span AB, L_3 represents span BC and L_4 represents span CD

Mid-Span Moment Values

Moment Envelope Values

$$M_{UADin} := \left[\frac{W\cdot(L_1 - D_2)^2}{8} - |M_{ADend}|\right] \qquad M_{UADin} = 24.48\cdot kip\cdot ft \qquad \frac{W\cdot(L_1 - D_2)^2}{8} = 56.881\cdot kip\cdot ft$$

$$M_{UABin} := \left[\frac{W\cdot(L_2 - D_1)^2}{8} - |M_{ADend}|\right] \qquad M_{UABin} = 6.416\cdot kip\cdot ft \qquad \frac{W\cdot(L_2 - D_1)^2}{8} = 38.817\cdot kip\cdot ft$$

$$M_{UCBin} := \left[\frac{W\cdot(L_3 - D_2)^2}{8} - |M_{CBend}|\right] \qquad M_{UCBin} = 24.48\cdot kip\cdot ft \qquad \frac{W\cdot(L_3 - D_2)^2}{8} = 56.881\cdot kip\cdot ft$$

$$M_{UDCin} := \left[\frac{W\cdot(L_4 - D_1)^2}{8} - |M_{DCend}|\right] \qquad M_{UDCin} = 6.416\cdot kip\cdot ft \qquad \frac{W\cdot(L_4 - D_1)^2}{8} = 38.817\cdot kip\cdot ft$$

The demand moment will be taken at the support face IAW ACI 318-02, 8.7.2 and 8.7.3.

$$M_{UADface} := \left[M_{ADend} + \frac{W\cdot D_2}{8}\cdot(2L_1 - 3D_2)\right]$$

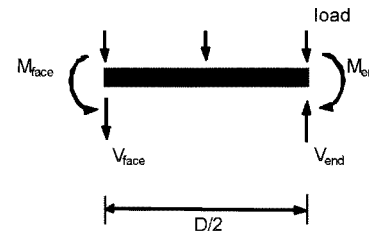
$$M_{UADface} = -18.53\cdot kip\cdot ft$$

$$M_{UABface} := \left[M_{ADend} + \frac{W\cdot D_1}{8}\cdot(2\cdot L_2 - 3D_1)\right]$$

$$M_{UABface} = -21.111\cdot kip\cdot ft$$

$$M_{UCBface} := M_{UADface} \qquad M_{UDCface} := M_{UABface}$$

Free Body Diagram

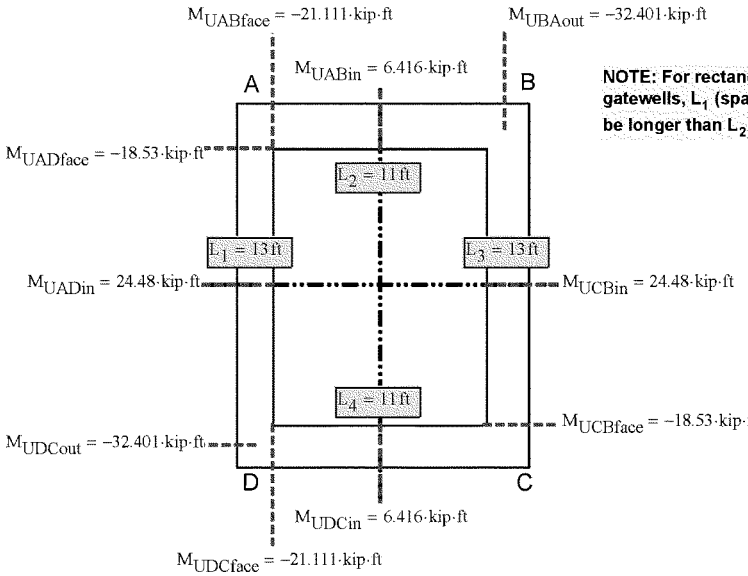


"D" represents the orthagonal wall thickness

$M_{UDCout} := M_{DCend}$ $\boxed{M_{UDCout} = -32.401 \cdot kip \cdot ft}$

$M_{UABout} := M_{ADend}$ $\boxed{M_{UABout} = -32.401 \cdot kip \cdot ft}$ $M_{UBAout} := M_{ADend}$

FACTORED MOMENT DISTRIBUTION VALUES



NOTE: For rectangular gatewells, L_1 (span AD) shall be longer than L_2 (span AB).

The "L" values are the "outside-to-outside" dimensions.

⚠ Ensure the reinforcement size entered earlier reflects the steel that will be in tension at the mid-span.
For example:
IF: The mid-span moment demand value obtained from moment distribution is negative.
THEN: The steel located near the wall's exterior face is in tension. Select the reinforcement size that is present near the wall exterior face.

Wall Thrust

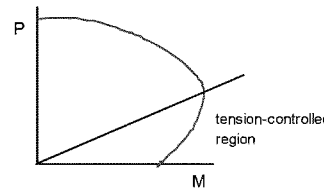
Thrust was considered when evaluating the wall section moment and shear capacity. A higher value of thrust within the tension-controlled region of the interaction diagram results in a higher nominal strength value. The calculated thrust is based upon the active (lower) value of lateral loading. This is deemed conservative relative to the at-rest (higher) value of lateral loading. Note that for lateral loading on the walls, the at-rest loading condition was used.

$K_a = 0.472$ $\gamma_{soil} = 115 \cdot pcf$ $H_2 = 36.5\,ft$

$W_{1active} := \gamma_L \cdot \gamma_H \cdot K_a \cdot \gamma_{soil} \cdot H_2$

$W_{2active} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_a \cdot \left(\gamma_{soil} - \gamma_w\right) \cdot H_2 + \gamma_w \cdot H_3\right]$

$W_{1active} = 1982.711 \cdot psf$ $W_{2active} = 3115.835 \cdot psf$



$Wactive := if\left(W_{1active} > W_{2active}, W_{1active}, W_{2active}\right) \cdot b$

$\boxed{Wactive = 3115.835 \cdot plf}$ $W = 3440.829 \cdot plf$

Total Trust Acting On Wall AD:

$P_{ADthrust} := \frac{Wactive \cdot L_2}{2}$ $\boxed{P_{ADthrust} = 17.137 \cdot kip}$

Total Trust Acting On Wall AB:

$P_{ABthrust} := \frac{Wactive \cdot L_1}{2}$ $\boxed{P_{ABthrust} = 20.253 \cdot kip}$

Wall AD (Wall 1) Analysis

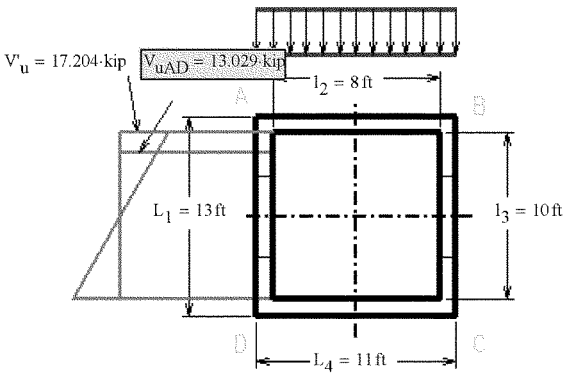
Shear Demand:

$W = 3440.829 \cdot plf \qquad l_1 = 10\,ft$

$V'_u := W \cdot \frac{l_1}{2} \qquad V'_u = 17.204 \cdot kip$

$d_{AD\_out\_end} = 14.563 \cdot in$

$V_{uAD} := V'_u \cdot \frac{\frac{l_1}{2} - d_{AD\_out\_end}}{\frac{l_1}{2}} \qquad V_{uAD} = 13.029 \cdot kip \qquad W = 3440.829 \cdot plf$



Shear Capacity and Factor of Safety:

$d_{AD\_out\_end} = 14.563 \cdot in \qquad \frac{P_{ADthrust}}{2000 \cdot A_{g1}} = 0.04\,psi \qquad f'_c = 3 \cdot ksi \quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support ACI 318R-11.1.3.1

$\phi V_{n\_AD} := \phi V \left[2 \cdot \left(1 + \frac{P_{ADthrust}}{2000 \cdot A_{g1}} \cdot \frac{in^2}{lbf} \right) \cdot \left(\sqrt{f'_c \cdot psi} \right) \cdot \left(b \cdot d_{AD\_out\_end} \right) \right]$ ACI EQ (11-4)

$\phi V_{n\_AD} = 19.902 \cdot kip$

$V_{uAD} = 13.029 \cdot kip$

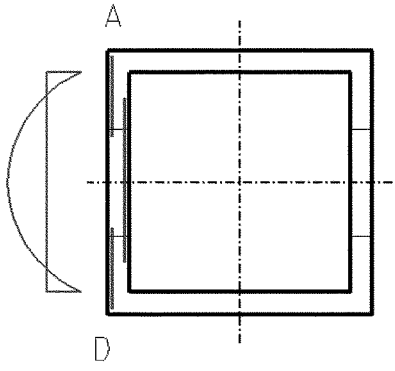
Factor of Safety $FS_{ADshear} := \frac{\phi V_{n\_AD}}{V_{uAD}}$

$FS_{ADshear} = 1.528$

DEPTH OF EQUIVALENT RECTANGULAR STRESS BLOCK

Wall AD Con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.441 \cdot klf$ $L_1 = 13\,ft$



$A_{ADout} = 0.72 \cdot in^2$ $a_{AD\_out} := \frac{A_{ADout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ADin} = 1.58 \cdot in^2$ $a_{AD\_in} := \frac{A_{ADin} \cdot F_y}{0.85 f'_c \cdot b}$

$a_{AD\_out} = 0.941 \cdot in$ $a_{AD\_in} = 2.065 \cdot in$

Negative Moment Redistribution

Wall AD Con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

Beta one value is equal to 0.85 for f'c equal to or less than 4000 psi

$c_{AD\_out} := \frac{a_{AD\_out}}{\beta 1}$ $\quad c_{AD\_out} = 1.107 \cdot in$ $\quad c_{AD\_in} := \frac{a_{AD\_in}}{\beta 1}$ $\quad c_{AD\_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AD\_out\_end}}{c_{AD\_out}} - 1 \right) \qquad \varepsilon_{tout} = 0.036$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AD end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UADface}|}{\phi_B \cdot b \cdot d_{AD\_out\_end}^2} \qquad R_{nface} = 87.38 \cdot psi \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UADin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

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$\varepsilon'_{tface} = 0.07$

$$\varepsilon'\_ADfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases}$$

$\varepsilon'\_ADfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 0.701$

$$RedistAD := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases}$$

RedistAD = "Permitted"

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UADface} := \begin{cases} M_{UADface} \cdot (1 - \varepsilon'\_ADfacedelta) & \text{if } \varepsilon'\_ADfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADface} = -14.824 \cdot kip \cdot ft \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$

Wall AD Con't

$$M'_{UADout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'\_ADfacedelta) & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -25.921 \cdot kip \cdot ft$ $M_{UADface} = -18.53 \cdot kip \cdot ft$ $M_{UDCout} = -32.401 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UADin} := \begin{cases} \frac{W \cdot (L_1 - D_2)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UADout} & \text{if } M'_{UADout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UADout} = -25.921 \cdot kip \cdot ft$

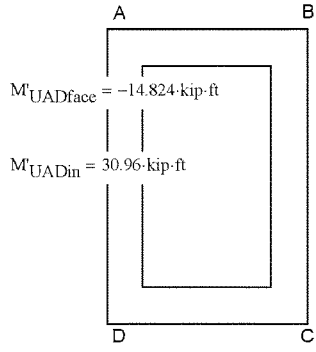
$M'_{UADin} = 30.96 \cdot kip \cdot ft$

$$\frac{W \cdot (L_1 - D_2)^2}{8} = 56.881 \cdot kip \cdot ft$$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

Moments on Span AD After Redistribution

RedistAD = "Permitted"



Note: If redistribution is not allowed, "N/A" is shown.

Wall AD Magnified Moment Check

Wall AD Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k\ell_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_0/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, [illegible] are given in Ref. 8.2.

M_0 = Simple span moment at midspan $\left(\frac{w\ell^2}{8}\right)$

M_a = Net midspan moment.

$M_1 := M_{UADface}$ $\qquad$ $M_2 := M_{UADface}$

$M_1 = -18.53 \cdot kip \cdot ft$ $\qquad$ $M_2 = -18.53 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k := 0.6$

Ubraced length: $l_u := L_1$ $\qquad$ $l_u = 13\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_1 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of [illegible] r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r := 0.3 \cdot D_1$ $\qquad$ $r = 0.45\,ft$

$Slender\_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender\_Ratio = 17.333$

$$ADSlenderness\_Check := if\left[Slender\_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], "Ok", "Consider\ Slenderness\ using\ ACI\ 10.10.1"\right]$$

$ADSlenderness\_Check = "Ok"$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AD Mid-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ADin} = 8$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar\_No_{ADin} = 8$ $diaADin = 1 \cdot in$

$row := 2$

Iterations $i := 1 .. 2$ $i =$

1
2

Bar area at i $A_1 := A_{ADout}$ $A_2 := A_{ADin}$ $A_i =$

0.72
1.58

$\cdot in^2$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$ $coverADin = 3 \cdot in$ $diaADin = 1 \cdot in$

$$dd_1 := \begin{cases} coverADout + \dfrac{diaADout}{2} & \text{if } M_{UADin} > 0ft \cdot kip \\ coverADin + \dfrac{diaADin}{2} & \text{otherwise} \end{cases}$$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$dd_1 = 3.437 \cdot in$

$$dd_2 := \begin{cases} D_1 - \left(coverADin + \dfrac{diaADin}{2}\right) & \text{if } M_{UADin} > 0ft \cdot kip \\ D_1 - \left(coverADout + \dfrac{diaADout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.5 \cdot in$

Wall AD, Midspan Column Interaction Results

Wall AD Midspan

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \qquad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \geq F_y \\ -F_y - \text{if}\left(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$
$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \qquad c = 3.237 \cdot in \qquad a := \text{if}\left(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c\right) \qquad a = 2.752 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}\left(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}\left(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - \text{if}\left(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi\right) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0	-5.375 ·ks	0.72 ·in²	-3.87 ·kip	-21.526 ·kip·in
1.208	-0.01	-40	1.58	-63.2	347.6

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 326 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2}\right) \qquad Mc = 642 \cdot kip \cdot in$

$\phi M_{nADin1} := (Mc + Mst) \qquad \phi M_{nADin1} = 80.67 \cdot kip \cdot ft$

$\phi M_{nADin} := \begin{cases} \phi M_{nADin1} & \text{if } M_{UADin} > 0ft \cdot kip \\ \phi M_{nADin1} \cdot (-1) & \text{otherwise} \end{cases}$

Wall AD Mid-Span Nomimal Moment Strength $\quad \phi M_{nADin} = 80.673 \cdot kip \cdot ft$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADmid}

Wall AD Mid-Span Con't

$\phi M_{nADin} = 80.673 \cdot kip \cdot ft$

$M_{UADin} = 24.48 \cdot kip \cdot ft$

$$FS_{ADin} := \left(\left| \frac{\phi M_{nADin}}{M_{UADin}} \right| \right)$$

$FS_{ADin} = 3.295$

Wall AD Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, FS'_{ADmid}

$\phi M_{nADin} = 80.673 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UADin} = 30.96 \cdot kip \cdot ft$

$$FS'_{ADin} := \begin{cases} \left| \frac{\phi M_{nADin}}{M'_{UADin}} \right| & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADin} = 2.606$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADinspanCheck := if(ADSlenderness_Check ≠ "Ok", "ANALYSIS INVALID", "OKAY")

ADinspanCheck = "OKAY"

Wall AD End-of-Span Nominal Moment Capacity

$L_1 = 13\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_1 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ADout} = 7$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverADout = 3 \cdot in$ Cover -in

$Bar\_No_{ADout} = 7$ $diaADin = 1 \cdot in$

$A_1 := A_{ADin}$ $A_2 := A_{ADout}$ $A_i =$

$$\begin{array}{|r|}\hline 1.58 \\ \hline 0.72 \\ \hline \end{array} \cdot in^2$$

$coverADout = 3 \cdot in$ $diaADout = 0.875 \cdot in$

$dd_1 := d_{AD\_in\_end}$ $dd_1 = 3.5 \cdot in$

$dd_2 := d_{AD\_out\_end}$ $dd_2 = 14.563 \cdot in$

Wall AD, End-of-Span Column Interaction Results

Wall AD End-of-Span

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_1 = 1.5\,ft \quad b = 1\,ft \quad \beta 1 = 0.85$

Given $\quad c := .2 \cdot D_1 \quad c = 3.6 \cdot in$

$$P_{ADthrust} = \left[\sum_{i=1}^{row} \left[\left[\begin{array}{ll} (c - dd_i) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } (c - dd_i) \cdot \frac{0.003}{c} \cdot Es < F_y \\ + -\text{if}(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0) & \\ F_y - \text{if}(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0) & \text{if } (c - dd_i) \cdot \frac{0.003}{c} \cdot Es \ge F_y \\ -F_y - \text{if}(dd_i < \beta 1 \cdot c, 0.85 \cdot f'_c, 0) & \text{if } (c - dd_i) \cdot \frac{0.003}{c} \cdot Es \le -F_y \end{array} \right] \cdot A_i \right] \ldots \right] \cdot \phi_B$$

$$+ 0.85 \cdot f'_c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c) \quad c = 2.887 \cdot in \quad a := \text{if}(\beta 1 \cdot c > D_1, D_1, \beta 1 \cdot c) \quad a = 2.454 \cdot in$

Find Moment:

$\varepsilon s_i := (c - dd_i) \cdot \frac{0.003}{c} \qquad fs_i := \text{if}(Es \cdot \varepsilon s_i > F_y, F_y, \text{if}(Es \cdot \varepsilon s_i < 0 - F_y, 0 - F_y, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - \text{if}(\beta 1 \cdot c > dd_i, 0.85 \cdot f'_c, 0ksi) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{D_1}{2} - dd_i \right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.292 ft	-0.001	-18.458 ·k	1.58 ·in²	-29.164 ·kip	-160.403 ·kip·in
1.214	-0.012	-40	0.72	-28.8	160.2

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = -0 \cdot kip \cdot in$

$Mc := 0.85 \cdot f'_c \cdot a \cdot b \cdot \left(\frac{D_1}{2} - \frac{a}{2} \right) \qquad Mc = 584 \cdot kip \cdot in$

$\phi M_{nADface1} := (Mc + Mst) \qquad \phi M_{nADface1} = 48.63 \cdot kip \cdot ft$

$\phi M_{nADface} := \begin{cases} \phi M_{nADface1} & \text{if } M_{UADface} > 0ft \cdot kip \\ \phi M_{nADface1} \cdot (-1) & \text{otherwise} \end{cases} \qquad M_{UADface} = -18.53 \cdot kip \cdot ft$

Wall AD End-of-Span Nominal Moment Strength $\quad \phi M_{nADface} = -48.629 \cdot kip \cdot ft$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ADface}

$\phi M_{nADface} = -48.629 \cdot kip \cdot ft$ $M_{UADface} = -18.53 \cdot kip \cdot ft$

$$FS_{ADface} := \left(\left| \frac{\phi M_{nADface}}{M_{UADface}} \right| \right)$$

$FS_{ADface} = 2.624$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, $FS`_{ADface}$

$$FS'_{ADface} := \begin{cases} \left| \frac{\phi M_{nADface}}{M'_{UADface}} \right| & \text{if } \varepsilon'\_ADfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ADface} = 3.28$

The end-of-span factor of safety always increases when negative moment redistribution is considered.

ADSlenderness_Check = "Ok"

RedistAD = "Permitted"

ADoutspanCheck := if(ADSlenderness_Check ≠ "Ok" , "ANALYSIS INVALID" , "OKAY")

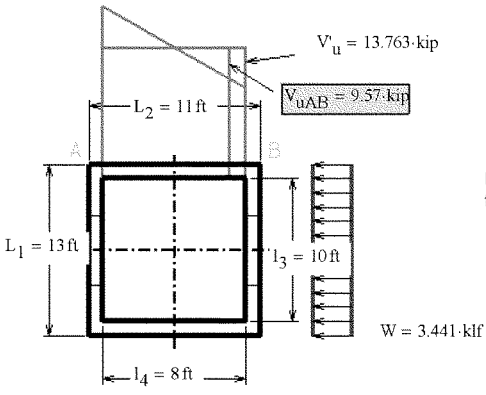
ADoutspanCheck = "OKAY"

Wall AB (Wall 2) Analysis

Shear Demand:

$W = 3440.829\,ft\cdot psf$ $\quad l_2 = 8\,ft$ $\quad d_{AB\_out\_end} = 14.625\cdot in$

$V'_u := W\cdot\frac{l_2}{2}$ $\quad V'_u = 13.763\cdot kip$ $\quad V_{uAB} := V'_u\cdot\frac{\frac{l_2}{2} - d_{AB\_out\_end}}{\frac{l_2}{2}}$ $\quad V_{uAB} = 9.57\cdot kip$



Total Thrust into Wall2:

$P_{ABthrust} = 20.253\cdot kip$

Note: Refer to wall AD calculations for derivation of the thrust values acting on wall AB.

Shear Capacity and Factor of Safety

$d_{AB\_out\_end} = 14.625\cdot in$ $\quad \frac{P_{ABthrust}}{2000\cdot A_{g2}} = 0.047\,psi$ $\quad f'_c = 3000\,psi$ $\quad b = 1\,ft$

Shear Concrete Resistance value at Distance d from Support
ACI 318R-11.1.3.1

$$\phi V_{n\_AB} := \phi_V\left[2\cdot\left(1 + \frac{P_{ABthrust}}{2000\cdot A_{g2}}\cdot\frac{in^2}{lbf}\right)\cdot\left(\sqrt{f'_c\cdot psi}\right)\cdot\left(b\cdot d_{AB\_out\_end}\right)\right]$$

ACI EQ (11-4) $\quad \phi V_{n\_AB} = 20.126\cdot kip$

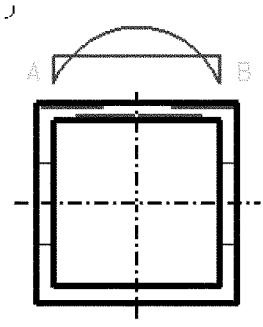
$V_{uAB} = 9.57\cdot kip$

Factor of Safety $\quad FS_{ABshear} := \frac{\phi V_{n\_AB}}{V_{uAB}}$ $\quad FS_{ABshear} = 2.103$

Depth of Equivalent Rectangular Stress Block

Wall AB con't

$F_y = 40 \cdot ksi$ $f'_c = 3 \cdot ksi$ $b = 1\,ft$ $W = 3.441 \cdot klf$ $L_1 = 13\,ft$



$A_{ABout} = 0.88 \cdot in^2$ $aABout := \frac{A_{ABout} \cdot F_y}{0.85 f'_c \cdot b}$

$A_{ABin} = 1.2 \cdot in^2$ $aABin := \frac{A_{ABin} \cdot F_y}{0.85 f'_c \cdot b}$

$aABout = 1.15 \cdot in$ $aABin = 1.569 \cdot in$

Negative Moment Redistribution

Wall AB con't

ACI 318-02, 8.4 allows moment redistribution (plastic hinge formation). Refer to 8.4 for moment redistribution limitations.

Determine if redistribution is possible. To occur, the steel must yield prior to concrete crushing.

$\beta 1 = 0.85 \qquad f'_c = 3000\,psi$

$cABout := \frac{aABout}{\beta 1} \quad cABout = 1.353 \cdot in \qquad cABin := \frac{aABin}{\beta 1} \qquad c_{AD\_in} = 2.43 \cdot in$

The net tensile strain values

$\varepsilon_{tout} := 0.003 \cdot \left(\frac{d_{AB\_out\_end}}{cABout} - 1 \right) \quad \varepsilon_{tout} = 0.029$

Negative moment distribution is permissible when the tensile strain is greater than 0.0075, per ACI 318-02, 8.4

Reduce negative moment at face of wall AB end support and increase the positive mid-span moment (if applicable)

$R_{nface} := \frac{|M_{UABface}|}{\phi_B \cdot b \cdot d_{AB\_out\_end}^2} \qquad R_{nface} = 98.7 \cdot psi \qquad M_{UABface} = -21.111 \cdot kip \cdot ft$

$$\varepsilon'_{tface} := \begin{cases} 0.003 \cdot \left(\frac{\beta 1}{1 - \sqrt{1 - \frac{40}{17} \cdot \frac{R_{nface}}{f'_c}}} - 1 \right) & \text{if } (\varepsilon_{tout} \ge 0.0075) \wedge (M_{UABin} > 0) \\ 0 & \text{otherwise} \end{cases}$$

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$\varepsilon'_{tface} = 0.062$

$$\varepsilon'\_ABfacedelta := \begin{cases} \varepsilon'_{tface} \cdot 10 & \text{if } 0 < \varepsilon'_{tface} \le 0.020 \\ 0 & \text{if } \varepsilon'_{tface} = 0 \\ 0.20 & \text{otherwise} \end{cases} \qquad \varepsilon'\_ABfacedelta = 0.200 \qquad \varepsilon'_{tface} \cdot 10 = 0.616$$

$$RedistAB := \begin{cases} \text{"Permitted"} & \text{if } \varepsilon'_{tface} \ne 0 \\ \text{"Not Permitted"} & \text{otherwise} \end{cases} \qquad \boxed{RedistAB = \text{"Permitted"}}$$

IF: The strain is greater than or equal to 0.0075 and the mid span moment is positive
THEN: Negative moment redistribution is permitted. OTHERWSE: Negative moment redistribution is not permitted and "N/A" is printed.

Redistributed (reduced) negative moment at the support face

$$M'_{UABface} := \begin{cases} M_{UABface} \cdot (1 - \varepsilon'\_ABfacedelta) & \text{if } \varepsilon'\_ABfacedelta \ne 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$\boxed{M'_{UABface} = -16.889 \cdot kip \cdot ft} \qquad M_{UABface} = -21.111 \cdot kip \cdot ft$

Wall AB con't

$$M'_{UABout} := \begin{cases} M_{UDCout} \cdot (1 - \varepsilon'\_ABfacedelta) & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$M'_{UABout} = -25.921 \cdot kip \cdot ft$ $M_{UABface} = -21.111 \cdot kip \cdot ft$ $M_{UDCout} = -32.401 \cdot kip \cdot ft$

Corresponding redistributed (increased) mid-span moment

$$M'_{UABin} := \begin{cases} \frac{W \cdot (L_2 - D_1)^2}{8} \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) + M'_{UABout} & \text{if } M'_{UABout} \neq \text{"N/A"} \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

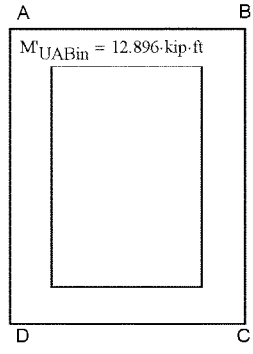
$M'_{UABin} = 12.896 \cdot kip \cdot ft$

$\frac{W \cdot (L_2 - D_1)^2}{8} = 38.817 \cdot kip \cdot f$ $M_{UABin} = 6.416 \cdot kip \cdot ft$

$M'_{UABin} = 12.896 \cdot kip \cdot ft$

Moments on Span AD After Redistribution RedistAB = "Permitted"

$M'_{UABface} = -16.889 \cdot kip \cdot ft$



Note: If redistribution is not allowed, "N/A" is shown.

Wall AB Magnified Moment Check

Wall AB Con't

The effects of wall slenderness shall be checked.

10.12.2 — In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\frac{k l_u}{r} \le 34 - 12(M_1/M_2) \qquad (10\text{-}8)$$

where the term $[34 - 12M_1/M_2]$ shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

Table 8-3 Deflection Coefficient K

		K
1.	Cantilevers (deflection due to rotation at supports not included)	2.40
2.	Simple beams	1.0
3.	Continuous beams	1.2-0.2 M_o/M_a
4.	Fixed-hinged beams (midspan deflection)	0.80
5.	Fixed-hinged beams (maximum deflection using maximum moment)	0.74
6.	Fixed-fixed beams	0.60

For other types of loading, values are given in Ref. 8.2.

M_o = Simple span moment at midspan $\left(\frac{wl^2}{8}\right)$

M_a = Net midspan moment.

$M_{1A} := M_{UABface}$ $\qquad$ $M_{2A} := M_{UABface}$

$M_1 = -21.111 \cdot kip \cdot ft$ $\qquad$ $M_2 = -21.111 \cdot kip \cdot ft$

From the table from the right, the k value for fixed fixed condition: $k_A := 0.6$

Ubraced length: $l_{uA} := L_2$ $\qquad$ $l_u = 11\,ft$

Radius of Gyration, r:

Thickness of Wall: $D_2 = 1.5\,ft$

10.11.2 — It shall be permitted to take the radius of gyration *r* equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute the radius of gyration for the gross concrete section.

$r_W := 0.3 \cdot D_2$ $\qquad$ $r = 0.45\,ft$

$Slender\_Ratio := k \cdot \frac{l_u}{r}$ $\qquad$ $Slender\_Ratio = 14.667$

$$ABSlenderness\_Check := if\left[Slender\_Ratio \le \left[34 - 12 \cdot \left(\frac{M_1}{M_2}\right)\right], "Ok", "Consider\ Slenderness\ using\ ACI\ 10.10.1"\right]$$

$ABSlenderness\_Check = "Ok"$

IF: ADSlenderness_Check is OK
THEN: Moment magnification is not required

IF: ADSlenderness_Check is NOT OK
THEN: Wall is slender. Moment magnification per ACI 318-02, 10.12 shall be followed

Wall AB Mid-Span Nominal Moment Capacity

$L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ABin} = 7$ **Tension** bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABin = 3 \cdot in$ Cover -in

Axial Compression with ACI Reduction value: $\phi_B = 1$ $Bar\_No_{ABin} = 7$ $diaABin = 0.875 \cdot in$

$diaABout = 0.75 \cdot in$

Bar area at i $A_1 := A_{ABout}$ $A_2 := A_{ABin}$ $A_i =$

$\blacksquare \cdot in^2$

$coverABont = 3 \cdot in$ $diaABout = 0.75 \cdot in$ $coverABin = 3 \cdot in$ $diaABin = 0.875 \cdot in$

$$dd_1 := \begin{cases} coverABout + \dfrac{diaABout}{2} & \text{if } M_{UABin} > 0ft \cdot kip \\ coverABin + \dfrac{diaABin}{2} & \text{otherwise} \end{cases}$$

$M_{UABin} = 6.416 \cdot kip \cdot ft$

$dd_1 = 3.375 \cdot in$

$$dd_2 := \begin{cases} D_2 - \left(coverABin + \dfrac{diaABin}{2}\right) & \text{if } M_{UABin} > 0ft \cdot kip \\ D_2 - \left(coverABout + \dfrac{diaABout}{2}\right) & \text{otherwise} \end{cases}$$

$dd_2 = 14.563 \cdot in$

$coverABont = 3 \cdot in$ $diaABout = \blacksquare \cdot in$

$dd_1 := d_{AB\_out\_mid}$ $dd_1 = 3.375 \cdot in$

$dd_2 := d_{AB\_in\_mid}$ $dd_2 = 14.563 \cdot in$

Wall AB, Mid-Span Column Interaction Results

Wall AB Mid-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \sum_{i=1}^{row} \left[\left[\begin{cases} \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ +-\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y \end{cases} \right]\cdot A_i \right] \ldots \cdot\phi_B$$

$+0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$

$c := \text{Find}(c) \quad c = 2.996\cdot in \qquad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \qquad a = 2.547\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.281 ft	-0	-10.999 ·ksi	0.88 ·in²	-9.679 ·kip	-54.443 ·kip·in
1.214	-0.012	-40	1.2	-48	267

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 213\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 602\cdot kip\cdot in$

$\phi M_{nABin1} := (Mc + Mst) \qquad \phi M_{nABin1} = 67.89\cdot kip\cdot ft$

$$\phi M_{nABin} := \begin{cases} \phi M_{nABin1} & \text{if } M_{UABin} > 0ft\cdot kip \\ \phi M_{nABin1}\cdot(-1) & \text{otherwise} \end{cases}$$

Wall AB Mid-Span Nominal Moment Strength $\qquad \phi M_{nABin} = 67.892\cdot kip\cdot ft$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABmid}

Wall AB Mid-Span Con't

$\phi M_{nABin} = 67.892 \cdot kip \cdot ft$

$M_{UABin} = 6.416 \cdot kip \cdot ft$

$$FS_{ABin} := \left(\left| \frac{\phi M_{nABin}}{M_{UABin}} \right| \right)$$

$FS_{ABin} = 10.582$

Wall AB Mid-Span Factor of Safety When Negative Moment Redistribution is Considered, FS'_{ABmid}

$\phi M_{nABin} = 67.892 \cdot kip \cdot ft$

Note: An apostrophe signifies a negative moment redistribution-related value.

$M'_{UABin} = 12.896 \cdot kip \cdot ft$

$$FS'_{ABin} := \begin{cases} \left| \frac{\phi M_{nABin}}{M'_{UABin}} \right| & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABin} = 5.265$

The mid-span factor of safety always decreases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABinspanCheck := if(ABSlenderness_Check ≠ "Ok" ,"ANALYSIS INVALID" ,"OKAY")

ABinspanCheck = "OKAY"

Wall AB End-of-Span Nominal Moment Capacity $L_2 = 11\,ft$

Note: The following routine calculates the moment capacity of a concrete column with the axial load set to the thrust introduced from the adjacent wall This assumes tension controlled interaction behavior (beam behavior). This has been verified for this type of structure.

$D_2 = 18 \cdot in$ Column Depth

$b = 12 \cdot in$ Column Width

$Bar\_No_{ABout} = 6$ **Tension** Bar Size

$f'_c = 3 \cdot ksi$ Conc. Str. - ksi

$F_y = 40 \cdot ksi$ Bar Yield - ksi

$coverABout = 3 \cdot in$ Cover -in

$Bar\_No_{ABout} = 6$ $diaABin = 0.875 \cdot in$

$A_1 := A_{ABin}$ $A_2 := A_{ABout}$ $A_i =$ $\begin{array}{|r|}\hline 1.2 \\ \hline 0.88 \\ \hline\end{array} \cdot in^2$ <==The first value of A1 is the compression steel, A2 is the tension steel

$coverABout = 3 \cdot in$ $diaABout = 0.75 \cdot in$

$dd_1 := d_{AB\_in\_end}$ $dd_1 = 3.437 \cdot in$ Effective depth of the compression steel

$dd_2 := d_{AB\_out\_end}$ Effective depth of the tension steel

Wall AB, End-of-Span Column Interaction Results

Wall AB End-of-Span con't

Find the Moment Capacity of a one foot strip under an axial load (thrust calculated previously), this equation is based on the tension region of the interaction diagram:

Assume c=0.2*D $\quad i := 1..2 \quad D_2 = 18\cdot in \quad b = 1\,ft$ $\qquad \beta 1 = 0.85$

Given $\quad c := .2\cdot D_2 \quad c = 3.6\cdot in$

$$P_{ABthrust} = \left[\sum_{i=1}^{row}\left[\left[\begin{array}{ll}\left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ldots & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es < F_y \\ +-\text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \\ F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \ge F_y \\ -F_y - \text{if}\left(dd_i < \beta 1\cdot c, 0.85\cdot f'_c, 0\right) & \text{if } \left(c - dd_i\right)\cdot\frac{0.003}{c}\cdot Es \le -F_y\end{array}\right]\cdot A_i\right] \ldots \cdot\phi_B\right]$$
$$+0.85\cdot f'_c\cdot\beta 1\cdot c\cdot b$$

$c := \text{Find}(c) \quad c = 2.891\cdot in \quad a := \text{if}\left(\beta 1\cdot c > D_2, D_2, \beta 1\cdot c\right) \quad a = 2.457\cdot in$

Find Moment:

$\varepsilon s_i := \left(c - dd_i\right)\cdot\frac{0.003}{c} \qquad fs_i := \text{if}\left(Es\cdot\varepsilon s_i > F_y, F_y, \text{if}\left(Es\cdot\varepsilon s_i < 0 - F_y, 0 - F_y, Es\cdot\varepsilon s_i\right)\right)$

$Fs_i := fs_i\cdot A_i - \text{if}\left(\beta 1\cdot c > dd_i, 0.85\cdot f'_c, 0ksi\right)\cdot A_i \qquad Ms_i := Fs_i\cdot\left(\frac{D_2}{2} - dd_i\right)$

$dd_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.286 ft	-0.001	-16.45 ·ks	1.2 ·in²	-19.739 ·kip	-109.801 ·kip·in
1.219	-0.012	-40	0.88	-35.2	198

$Mst := \sum_{i=1}^{2} Ms_i \qquad Mst = 88\cdot kip\cdot in$

$Mc := 0.85\cdot f'_c\cdot a\cdot b\cdot\left(\frac{D_2}{2} - \frac{a}{2}\right) \qquad Mc = 584\cdot kip\cdot in$

$\phi M_{nABface1} := (Mc + Mst) \qquad \phi M_{nABface1} = 56.05\cdot kip\cdot ft$

$$\phi M_{nABface} := \begin{cases}\phi M_{nABface1} & \text{if } M_{UABface} > 0ft\cdot kip \\ \phi M_{nABface1}\cdot(-1) & \text{otherwise}\end{cases}$$

Wall AB End-of-Span Nominal Moment Strength $\qquad \boxed{\phi M_{nABface} = -56.046\cdot kip\cdot ft}$

Wall AB End-of-Span Factor of Safety When Negative Moment Redistribution is NOT Considered, FS_{ABface}

$\phi M_{nABface} = -56.046 \cdot kip \cdot ft$ $M_{UABface} = -21.111 \cdot kip \cdot ft$

$$FS_{ABface} := \left(\left| \frac{\phi M_{nABface}}{M_{UABface}} \right| \right)$$

$FS_{ABface} = 2.655$

Wall AD End-of-Span Factor of Safety When Negative Moment Redistribution is Considered, FS'_{ABface}

$\varepsilon'\_ABfacedelta = 0.2$

$$FS'_{ABface} := \begin{cases} \left| \frac{\phi M_{nABface}}{M'_{UABface}} \right| & \text{if } \varepsilon'\_ABfacedelta \neq 0 \\ \text{"N/A"} & \text{otherwise} \end{cases}$$

$FS'_{ABface} = 3.319$ The end-of-span factor of safety always increases when negative moment redistribution is considered.

ABSlenderness_Check = "Ok"

RedistAB = "Permitted"

ABoutspanCheck := if(ABSlenderness_Check ≠ "Ok", "ANALYSIS INVALID", "OKAY")

ABoutspanCheck = "OKAY"

Strength Analysis Results Summary

Span AD Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ADface} = 2.624$ $FS_{ADin} = 3.295$ $FS'_{ADface} = 3.28$ $FS'_{ADin} = 2.606$

$FS_{ADno\_redist} := \min(FS_{ADface}, FS_{ADin})$ $FS'_{ADredist} := \min(FS'_{ADface}, FS'_{ADin})$

$FS_{ADno\_redist} = 2.624$ $FS'_{ADredist} = 2.606$

$$FS_{ADmoment} := \begin{cases} \max(FS_{ADno\_redist}, FS'_{ADredist}) & \text{if } FS'_{ADredist} \neq \text{"N/A"} \\ FS_{ADno\_redist} & \text{otherwise} \end{cases}$$

$FS_{ADin} = 3.295$

$FS_{ADmoment} = 2.624$

$$AD\_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Recommended"} & \text{otherwise} \end{cases}$$

AD_Recommendation1 = "Perform Simple Beam Analysis"

$$AD\_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ADmoment} < 1.5 \\ \text{"Span AD OKAY for Moment Strength"} & \text{if } FS_{ADmoment} \geq 1.5 \end{cases}$$

AD_Recommendation2 = "Span AD OKAY for Moment Strength"

A listing of the span AD moment demand-capacity values, factors of safety and recommendations are provided in the following table

δ

Scalar Conversion Equations To Allow Text Entry Into Tables

Span AD Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-18.53	-48.63	2.62	24.48	80.67	3.30
Redistribution of Negative Moments	-14.82		3.28	30.96		2.61
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AD OKAY for Moment Strength						

N/A: Not Applicable

Span AB Moment Factor of Safety Analysis Summary and Recommendations

$FS_{ABno\_redist} := \min(FS_{ABface}, FS_{ABin})$ $FS'_{ABredist} := \min(FS'_{ABface}, FS'_{ABin})$

$FS_{ABno\_redist} = 2.655$ $FS'_{ABredist} = 3.319$

$$FS_{ABmoment} := \begin{cases} \max(FS_{ABno\_redist}, FS'_{ABredist}) & \text{if } FS'_{ABredist} \neq \text{"N/A"} \\ FS_{ABno\_redist} & \text{otherwise} \end{cases}$$

$FS_{ABin} = 10.582$

$FS_{ABmoment} = 3.319$

$$AB\_Recommendation1 := \begin{cases} \text{"Perform Simple Beam Analysis"} & \text{if } FS_{ADin} \geq 2.0 \wedge FS_{ABin} \geq 2.0 \\ \text{"Simple Beam Analysis Not Required"} & \text{otherwise} \end{cases}$$

AB_Recommendation1 = "Perform Simple Beam Analysis"

$$AB\_Recommendation2 := \begin{cases} \text{"CAUTION - Reliability Questionable"} & \text{if } FS_{ABmoment} < 1.5 \\ \text{"Span AB OKAY for Moment Strength"} & \text{if } FS_{ABmoment} \geq 1.5 \end{cases}$$

AB_Recommendation2 = "Span AB OKAY for Moment Strength"

A listing of the span AB moment demand-capacity values, factors of safety and recommendations are provided in the following table

AB Scalar Conversion for Table

Span AB Moment Strength Analysis Summary						
	End-Support			Mid-Span		
Analysis	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS	Moment Demand, ft*kip	Moment Capacity, ft*kip	FS
Moment Distribution	-21.11	-56.05	2.65	6.42	67.89	10.58
Redistribution of Negative Moments	-16.89		3.32	12.90		5.26
Slenderness Check	Ok					
Redistribution of Negative Moments	Permitted					
Recommendations: Perform Simple Beam Analysis Span AB OKAY for Moment Strength						

N/A: Not Applicable

Shear Factor of Safety Analysis Summary and Recommendations

Span AD

$FS_{ADshear} = 1.528$

Span AB

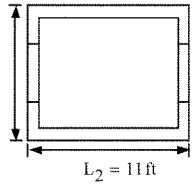
$FS_{ABshear} = 2.103$

NOTES TO STRENGTH SUMMARY RESULTS:

1. If simple beam analysis is performed on one wall, it must be performed on all four gatewell walls. This is required because fixity is lost when an adjacent wall is analyzed as a simple span. Stability should be considered when the gatewell is analyzed as having pinned supports.
2. Moment redistribution is not permitted when the mid-span and end-support moment values are negative.
3. Moment redistribution is not permitted when the reinforcement strain is less than 0.0075
4. Simple span analysis is recommended when the mid-span moment factor of safety is greater than 2.0. Final decision to use simple span analysis rests with the engineer.
5. Reliability analysis is recommended when the 1.0 < FS < 1.5
6. Caution is provided in the recommendations when the FS < 1.0

Properties of the Wall without an opening (Wall AB & CD)

Plan



$L_2 = 11\,ft$

$D_f = 2\,ft$

$H'' := H - D_f - y$

$H'' = 41.5\,ft$

Analyze wall at, $y := 0ft$ above base slab

Wall Thickness $D_2 = 18 \cdot in$ $l_2 = 8\,ft$

Clear Cover $C_c := coverABin$

MID-SPAN REINFORCEMENT

$Bar\_Spacing := 6in$

$Bar\_No :=$

$Bar\_No = 7$

Wall Reinforcement

The area of steel will be calculated /ft:

$$A_s := \frac{A_{out}}{\frac{Bar\_Spacing}{12in}} \qquad \boxed{A_s = 1.2 \cdot in^2}$$

Reinforcement Bar Diameters

Effective Depth:

$$D_2 = 18 \cdot in \qquad dia = 0.875 \cdot in$$

$$d := D_2 - C_c - \frac{dia}{2}$$

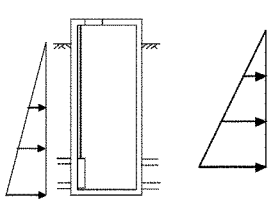
$$\boxed{d = 14.563 \cdot in}$$

(Wall without opening near base, cont'd)

Wall Loading

Wall Loading

WALLS



$H'' = 41.5\,ft$

$H'_2 := H''$

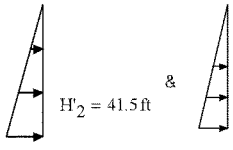
$H'_2 = 41.5\,ft$

Analysis of continuous wall perpendicular to opening
The walls above the opening were analyzed above

$W'_1 := \gamma_L \cdot \gamma_H \cdot K_o \cdot \gamma_{soil} \cdot H'_2$

$W'_1 = 3062 \cdot psf$

Soil & Water



$H'_2 = 41.5\,ft$ &

$H'_3 := if\left[H'' < H_{blanket}, \frac{H''}{H_{blanket}} \cdot (HGL + H_{blanket}) - HEAD, H'' + HGL - HEAD\right]$

$H'_3 = 40.4\,ft$

$W'_2 := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[K_o \cdot (\gamma_{soil} - \gamma_w) \cdot H'_2 + \gamma_w \cdot H'_3\right]$

$W'_2 = 3922 \cdot psf$

Check to determine if water to top of wet well with reduction factor, γ_X, or soil loading is worst case

$W' := if(W'_1 > W'_2, W'_1, W'_2) \cdot b$ $\boxed{W' = 3922 \cdot plf}$

(Wall without opening near base, cont'd)

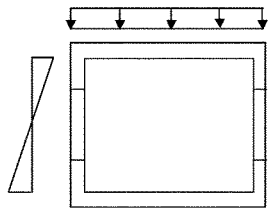
WALL without opening:

SHEAR

$f'_c = 3000\,psi$

$V'_u := W' \cdot \frac{l_2}{2}$

$V'_u = 15686\,lbf$



$V_u := V'_u \cdot \frac{\frac{l_2}{2} - d}{\frac{l_2}{2}}$

$b := 12in$

$V_u = 11 \cdot kip$

Shear at Distance d from Support
ACI 318R-11.1.3.1

$\phi V_n := \phi_V \cdot \left[2 \cdot \left(\sqrt{f'_c \cdot psi}\right) \cdot (d) \cdot b\right]$ ACI EQ (11-4) $\phi V_n = 19 \cdot kip$

Factor of Safety $FS_{opening\_shear} := \frac{\phi V_n}{V_u}$ $FS_{opening\_shear} = 1.8$

Bending

NOTE: Assumed to be simply supported with no thrust because of opening in adjacent walls.

Demand Moment:

$M_u := \frac{W' \cdot (L_2 - D_1)^2}{8}$ $M_u = 44.24 \cdot kip \cdot ft$

Moment Capacity:

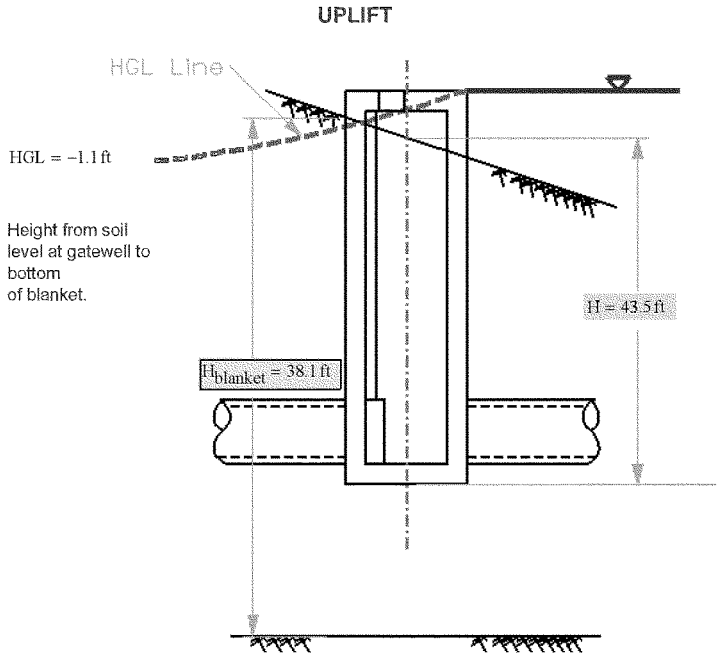
$a := \frac{A_s \cdot F_y}{0.85 f'_c \cdot b}$ $a = 1.569 \cdot in$

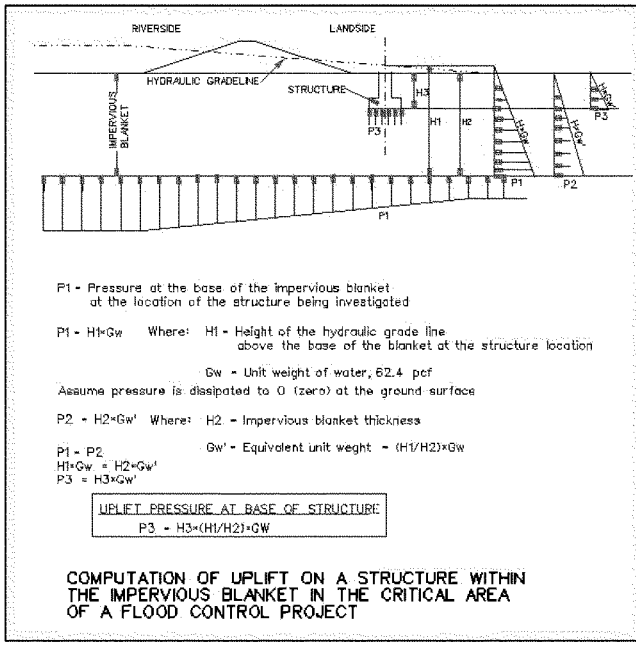
$\phi M_n := \phi_B \cdot A_s \cdot F_y \cdot \left(d - \frac{a}{2}\right)$ $\phi M_n = 55.1\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{opening\_mom} := \frac{\phi M_n}{M_u}$ $FS_{opening\_mom} = 1.2$

Uplift Analysis:

Uplift Using Local Protection Guidance





<< =from KC levees uplift guidance.

$A_{base} := L_1 \cdot L_2$ $A_{base} = 143\,ft^2$

Dissipated Head, if structural foundation in blanket.

$H_{blanket} = 38.1\,ft$ $HGL = -1.1\,ft$ $H = 43.5\,ft$ $L_1 = 13\,ft$ $L_2 = 11\,ft$

$$U_1 := \left(\frac{H_{blanket} + HGL}{H_{blanket}} \cdot H\right) \cdot L_1 \cdot L_2 \cdot \gamma_w \qquad U_1 = 377.0 \cdot kip$$

Full Head, if structural foundation extends through blanket.

$$U_2 := \left[(H + HGL) \cdot (L_1 \cdot L_2) \cdot \gamma_w\right] \qquad U_2 = 378.3 \cdot kip$$

Uplift

$H = 43.5\,ft$ $H_{blanket} = 38.1\,ft$ $H' = 0\,ft$

$U := if(H > H_{blanket}, U_2, U_1)$ $U = 378.3 \cdot kip$

$U_{uplift} := \frac{U}{L_1 \cdot L_2}$ $U_{uplift} = 2646 \cdot psf$

Weight of Structure

$Walls := (L_1 \cdot L_2 - l_1 \cdot l_2) \cdot (H + H') \cdot \gamma_c$ $Slabs := 2 \cdot (l_1 \cdot l_2) \cdot D_f \cdot \gamma_c$ $W_S := (Walls + Slabs)$ $W_S = 459.1 \cdot kip$

Surcharge Loads $Soil := 0lb$ WEIGHT OF SOIL LOCATED ON TOP OF THE BASE SLAB HEEL OR WALL STEP. EQUAL TO ZERO IF NO HEEL IS PRESENT.

Weight of Surcharge Water $W_{water} := HEAD \cdot l_1 \cdot l_2 \cdot \gamma_w$ $W_{water} = 0 \cdot lb$ $W_g := 0lb$

Skin Friction

SOIL (Soil parameters entered previously)

$\phi = \blacksquare \cdot deg$	INTERNAL FRICTION ANGLE	
$\gamma_{soil} = 115 \cdot pcf$	DRY SOIL UNIT WEIGHT	
$\gamma_w = 62.4 \cdot pcf$	WATER UNIT WEIGHT	
$\gamma_{sub} := (\gamma_{soil} - \gamma_w)$	SUBMERGED SOIL UNIT WEIGHT	$\gamma_{sub} = 52.6 \cdot pcf$
$K_o = 0.642$	AT-REST SOIL COEFFICIENT. CALCULATED PREVIOUSLY	
$\mu := 0.25$	FRICTION COEFICIENT BETWEEN STRUCTURE AND SOIL.	
$U_{uplift} = 2645.76 \cdot psf$	UPLIFT FORCE ACTING ON BASE OF STRUCTURE	
$c := 5ft$	ASSUMED CRACK DEPTH	

STRUCTURE

$H = 43.5\,ft$	DEPTH OF STRUCTURE BELOW GRADE	
$per := 2 \cdot L_1 + 2 \cdot L_2$	STRUCTURE PERIMETER	$per = 48\,ft$
$W_S = 459.075 \cdot kip$	STRUCTURE DEAD WEIGHT	

Skin Friction (Continued)

1. Effective Lateral Load that Contributes to Side Friction

$R_1 := (H - c)\cdot\left(\frac{c + H}{2}\right)\cdot\gamma_{soil}$ $R_1 = 107366.875\cdot\frac{lbf}{ft}$

$R_2 := (H - c)\cdot U_{uplift}\cdot\left(\frac{c}{2\cdot H} + \frac{1}{2}\right)$ $R_2 = 56785.004\cdot\frac{lbf}{ft}$

$R_1 - R_2 = 50581.871\cdot\frac{lbf}{ft}$

2. Force Created By Side Friction, Pv

$K_o = 0.642$ $\mu = 0.25$ $per = 48ft$

$P_{v1} := (R_1 - R_2)\cdot K_o\cdot\mu\cdot per$ $P_{v1} = 389.459\cdot kip$

Ignore_Skin_Friction :=

$P_v := \begin{cases} 0kip & \text{if Ignore\_Skin\_Friction = "Yes"} \\ P_{v1} & \text{otherwise} \end{cases}$ $P_v = 389\cdot kip$

Final Uplift Factor of Safety:

$FS_{Uplift} := \frac{W_S + W_{water} + Soil + P_v}{U}$ $FS_{Uplift} = 2.243$

NOTE: EM 1110-2-2100 defines 1.1 Factor of Safety acceptable for extreme loading case. The gatewells have minimal overhangs to produce weight of water above structure, W_G, therefore W_G ignored (conservative).

Base Slab Analysis

NOTE:
1. The following calculation assumes two opposite edges fixed and the other opposite edges simply supported. This occurs when a large pipe takes up most of the wall so that wall should be simply supported. Coef came from "Theory of Plates and Shells" second edition, by Timoshenko. This is conservative for gatewells without large pipe openings. So if FS is critical than may want to reanalize using the table of coeff for all sides fixed from "EM 27 - Moments and Reactions for Rect. Plates".
2. Only the end-of-span moment is checked, but in cases where the base slab top and bottom reinf is not the same, it may be necessary to check the mid-span.

Slab Loading:

Weight of structure: $W_S = 459.075 \cdot kip$ Uplift: $U = 378.344 \cdot kip$ Skin Friction: $P_{v1} = 389.459 \cdot kip$

Load acting up on baseslab:

$$W := if(U > W_S, U + P_{v1}, W_S) = 459.075 \cdot kip \qquad W_{UNIFORM} := \frac{\gamma_L \cdot \gamma_H \cdot \gamma_X \cdot W}{L_1 \cdot L_2} = 3.21 \cdot ksf$$

END-OF-SPAN MOMENT IS CHECKED HERE:
Plate Coefficient and Demand Moment:

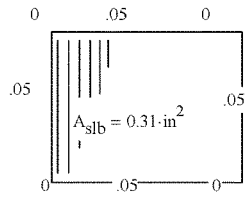
$k := 1..5$

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

From plates tables:

a/b	Coefficient
1	.0697
.9091	.0787
.769	.0938
.625	.109
.5	.1191

$ab_k :=$	$Coeff_k :=$
0.5	.1191
.625	.109
.769	.0938
0.9091	.0787
1	.0697



$$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.09$$

$l_2 = 8\,ft \qquad b = 1\,ft$

$$M_{uFe} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b \qquad M_{uFe} = 18.586 \cdot kip \cdot ft$$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 4.9 \cdot \frac{in^2}{ft} \qquad d_f = 1.708\,ft$$

$$\phi M_{nFe} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFe} = 21\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomente} := \frac{\phi M_{nFe}}{M_{uFe}}$ $FS_{slabmomente} = 1.129$

Base Slab Analysis, con't

MID-SPAN MOMENT (My) IS CHECKED HERE:
Plate Coefficient and Demand Moment:

a/b=L_2/L_1 $\frac{l_2}{l_1} = 0.8$

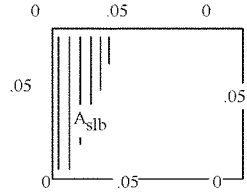
From plates tables:

a/b	Coefficient
1	0.0332
.9091	0.0371
.769	0.0426
.625	0.0469
.5	0.0474

$k := 1..5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0474
.625	0.0469
.769	0.0426
0.9091	0.0371
1	0.0332



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.041$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm1} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm1} = 8.503\,kip \cdot ft$

$A_{slb} := 0.44in^2$ $C_{cf} := 3.0625in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 6.9 \cdot \frac{in^2}{ft}$ $d_f = 1.745\,ft$

$\phi M_{nFm1} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm1} = 30.3\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm1} := \frac{\phi M_{nFm1}}{M_{uFm1}}$ $FS_{slabmomentm1} = 3.562$

Base Slab Analysis, con't

MID-SPAN MOMENT (Mx) IS CHECKED HERE:
Plate Coefficient and Demand Moment:

$a/b = L_2/L_1$ $\frac{l_2}{l_1} = 0.8$

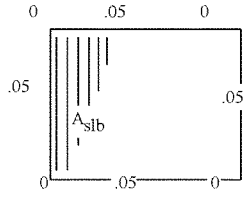
From plates tables:

a/b	Coefficient
1	0.0244
.9091	0.0307
.769	0.0446
.625	0.065
.5	0.0869

$k := 1..5$

$ab_k :=$ $Coeff_k :=$

0.5	0.0869
.625	0.065
.769	0.0446
0.9091	0.0307
1	0.0244



$Coefficient := linterp\left(ab, Coeff, \frac{l_2}{l_1}\right) = 0.042$

$l_2 = 8\,ft$ $b = 1\,ft$

$M_{uFm2} := Coefficient \cdot W_{UNIFORM} \cdot l_2^2 \cdot b$ $M_{uFm2} = 8.532 \cdot kip \cdot ft$

$A_{slb} := 1.06in^2$ $C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft}$ $d_f = 1.802\,ft$

$\phi M_{nFm2} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right)$ $\phi M_{nFm2} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$

Factor of Safety $FS_{slabmomentm2} := \frac{\phi M_{nFm2}}{M_{uFm2}}$ $FS_{slabmomentm2} = 8.669$

Base Slab Analysis, con't

Note that there is a concentration of reinforcing in one direction in the base slab. Assume that the base slab acts as a plate until reinforcing in the weak direction begins to yield and then the moments will redistribute to reinforcing in the other direction (a plastic hinge will form in the base slab at the wall). When this happens the base slab should be analyzed as if it were bending as if it were a simple span:

$L := 10ft$ $b = 1\,ft$

$$M_{ss} := \frac{W_{UNIFORM} \cdot L^2 \cdot b}{8} = 40.129\,ft \cdot \frac{kip \cdot ft}{ft}$$

$A_{slb} := 1.06in^2$ $C_{cf} := 2.375in$

$d_f := D_f - C_{cf}$

Moment Capacity:

$$a := \frac{A_{slb} \cdot F_y}{0.85 f'_c \cdot b} = 16.6 \cdot \frac{in^2}{ft} \qquad d_f = 1.802\,ft$$

$$\phi M_{nFm3} := \phi_B \cdot A_{slb} \cdot F_y \cdot \left(d_f - \frac{a}{2}\right) \qquad \phi M_{nFm3} = 74\,ft \cdot \frac{kip \cdot ft}{ft}$$

Factor of Safety $FS_{slabmomentm3} := \frac{\phi M_{nFm3}}{M_{ss}}$ $FS_{slabmomentm3} = 1.843$

**********Overall Factors of Safety**********

Uplift Stability

$FS_{Uplift} = 2.2$

Strength

Wall AD $L_1 = 13\,ft$

Wall AB $L_2 = 11\,ft$

WALL MOMENT

The overall wall moment factor of safety, $FS_{wallmoment}$

$FS_{ADmoment} = 2.624$ $FS_{ABmoment} = 3.319$ $FS_{opening\_mom} = 1.246$

$FS_{wallmoment} := \min(FS_{ADmoment}, FS_{ABmoment}, FS_{opening\_mom})$ $FS_{wallmoment} = 1.25$

WALL SHEAR

$FS_{ADshear} = 1.528$ $FS_{ABshear} = 2.103$ $FS_{opening\_shear} = 1.752$

$FS_{wallshear} := \min(FS_{ADshear}, FS_{ABshear}, FS_{opening\_shear})$ $FS_{wallshear} = 1.528$

BASE SLAB MOMENT FACTOR OF SAFETY

$FS_{slabmoment} := \max(\min(FS_{slabmomente}, FS_{slabmomentm1}, FS_{slabmomentm2}), FS_{slabmomentm3})$

$FS_{slabmoment} = 1.843$

Summary :

$FS_{strength} := \min(FS_{wallmoment}, FS_{wallshear}, FS_{slabmoment})$ $FS_{strength} = 1.2$

$$Mechanism := \begin{cases} \text{"Wall Bending"} & \text{if } FS_{strength} = FS_{wallmoment} \\ \text{"Wall Shear"} & \text{if } FS_{strength} = FS_{wallshear} \\ \text{"Base Slab Moment"} & \text{otherwise} \end{cases}$$

Mechanism = "Wall Bending"

$FS_{Uplift} = 2.2$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis. The reasoning being the load factor (live load neglecting hydraulic structure) divided by the strength reduction factor is approximately 1.75. Because this is an existing structure in the field that is routinely inspected with no visible history of problems, a factor of safety of 1.5 is deemed acceptable. If modifications are required for strength, the modified structure must be evaluated from a standard design approach (include factors - no FS calculation)

END OF ANALYSIS

EXHIBIT A-8.5

Stoplog Gap Sample Calculations

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By [illegible] Date FEB 03 Project CID MO Sheet 18 of 437

Chkd. By RM Date Nov 11 Subject PILE CAP TORSION Sheet ____ of ____

FOR A FEASIBILITY STUDY BASED UPON RISK ANALYSIS, A DETAILED & PRECISE ANALYSIS IS BEYOND BUDGET AND SCOPE. AN ESTIMATE FOR TORSION WILL BE BASED UPON THE MOMENT TRANSFERRED INTO THE PILE CAP FROM THE ABUTMENT. SOIL RESISTANCE TO PILE CAP IS IGNORED.

THIS ESTIMATE FOR TORSION WAS TESTED ON L385'S R1A CLOSURE. MAXIMUM TORSION, OCCURING NEAR ABUTMENT FACE, RANGED FROM 1300 TO 1500 K-ft, DEPENDING UPON PILE CONFIGURATION. THE MORE PILES, THE STIFFER THE PILE CAP AND THUS HIGHER TORSION. R1A WAS FINALLY DESIGNED WITH THE FEWER PILE CONFIGURATION; HOWEVER, AN EARLIER TRIAL RUN WITH 3 ROWS OF PILES AT 5'-6" SPACING YIELDED THE 1500 K-ft MAX TORSION. CID MO'S PILE CAPS ARE NOT VERY UNIFORM. PILES VARY FROM 6' TO 7' IN THE CLOSURE TO APPROXIMATELY 4' AT THE ABUTMENTS.

R1A'S MOMENT AT ABUTMENT:

68' CLOSURE WIDTH ; ABUTMENT WIDTH IS 2'

$w = 68'/2 = 34'$ $h = 16' - 2\frac{3}{4}'$

$M_{R1A} = \frac{1}{2} h^2 \gamma w (h/3)$ (IGNORING ABUTMENT WIDTH)

$M_{R1A} = 1512$ K-ft. > RPGF OUTPUTS (1300 - 1500 K-ft)

THIS IS A CRUDE ESTIMATE FOR TORSION, BUT A CONSERVATIVE ONE. THIS METHOD WILL BE USED TO DETERMINE THE TORSION OF CID MO'S PILE CAPS AND THEN COMPARE TO THE CRACKING TORSION. A TORSION WITH FS ≥ 1.5 WHEN COMPARED TO $\frac{1}{4} T_{CR}$ (ACI) WILL BE CONSIDERED RELIABLE.

NOTE: NONE OF THE PILE CAPS MEET A FS OF 1.5, THUS A PROBABILITY OF FAILURE WAS ESTABLISHED. REALIZE THAT MARGINALLY EXCEEDING THE $\frac{1}{4} T_L$ LIMIT BY ACI DOES NOT CONSTITUTE FAILURE. IT MAY MEAN SOME CRACKING WILL OCCUR OR THAT MOMENT AND SHEAR FS WILL BE REDUCED. IF THE FS FOR BOTH MOMENT AND SHEAR EXCEED, SAY 3.0, MEANING THERE'S ADEQUATE REINFORCING FOR SHEAR, FLEXURE, AND TORSION, THEN THE PILE CAP WOULD NOT BE INADEQUATELY DESIGNED. IT IS RECOMMENDED THAT A THOROUGH ANALYSIS BE COMPLETED BEFORE RECOMMENDING MODIFICATIONS TO THE WALL.

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Chkd. By FDM Date NOV 11 Subject PILE CAP TORSION Sheet ____ of ____

MONOLITH 145: ABUTMENT WIDTH = 2'-6"

OPENING = 17'-6" $H = 11'-0"$ $h_{SIDE} = 2'$

$W = 17.5'/2 = 8.75'$

$T_{145} = M_{145} = 1/2\, \gamma_{H_2O} \left[(H^2 w)(H/3 + h_{SIDE}) \right] = 187$ K-ft.

(241 K-ft w/ ABUTMENT WIDTH)

LOAD FACTORS = 1 FOR RELIABILITY.

$T_{145} = 187$ K-ft (241 w/ ABUTMENT WIDTH)

TORSIONAL THRESHOLD (τ_{cr}):

SEE ACI 11.6.1, WITH $\phi = 1$ FOR RELIABILITY.

$w = 13'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = w * h = 3744$ in^2

$p_{cp} = 2(w+h) = 360$ in

$\tau_{cr} = \phi \sqrt{f'_c} \left(A_{cp}^2 / p_{cp} \right)$ $f'_c = 3{,}000$ psi

$\tau_{cr} = 178$ K-ft $< T_{145}$ N.G.

RELIABILITY ANALYSIS REQUIRED

AGAIN, THE $1/4 T_{cr}$ CRITERION WAS JUDGED BY ACI TO REPRESENT THE THRESHOLD FOR TORSION THAT DOES NOT CAUSE SIGNIFICANT REDUCTION TO MOMENT AND SHEAR. A LARGE FS IN MOMENT AND SHEAR WOULD PERMIT SOME OF THE EXCESS TORSIONAL STRESS TO BE ABSORBED BY THE REINFORCED SECTION WITHOUT CRACKING.

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Comp. By [illegible] Date [illegible] Project CID MO Sheet 20 of 437

Chkd. By PJM Date NOV 11 Subject PILE CAP TORSION Sheet ____ of ____

MONOLITH 158:

OPENING = 17'-6" $H = 9.79'$ $h_{sill} = 3.21'$

$W = 17.5'/2 = 8.75'$

$T_{158} = M_{158} = \frac{1}{2}\gamma_{H_2O}(H^2 W)(H/3 + h_{sill}) = 169$ K-ft

LOAD FACTOR = 1 FOR RELIABILITY

$T_{158} = 169$ K-ft

$\tau_c = 178$ K-ft (SAME AS M 145)

$FS = \frac{\tau_c}{T_{158}} = 1.05 < 1.5$ N.G.

MONOLITH 162: THE CENTER POST IS ASSUMED TO CONTROL:

TRIBUTARY WIDTH = $2(15'/2) - 4.5' = 10.5'$ POSTWELL LENGTH = 4.5'

$H = 9'-1\frac{1}{2}''$ $h_{sill} = 3'-10\frac{1}{2}''$

$T_{162} = \frac{1}{2}\gamma_{H_2O}(H^2 W)(H/3 + h_{sill}) = 189$ K-ft

LOAD FACTOR = 1 FOR RELIABILITY

$T_{162} = 189$ K-ft

TORSIONAL THRESHOLD (τ_c):

$W = 13'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = W * h = 3744 \text{ in}^2$

$P_{cp} = 2(W+h) = 360 \text{ in}$

$\tau_c = \phi\sqrt{f'_c}\left(A_{cp}^2/P_{cp}\right)$ $f'_c = 3000$ psi

$\tau_c = 178$ K-ft $< T_{162}$ N.G.

RELIABILITY ANALYSIS REQUIRED.

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Comp. By JH Date FEB 08 Project CID MO Sheet 21 of 437

Chkd. By PDM Date NOV 11 Subject PILE CAP TORSION Sheet ____ of ____

LEVEE MONOLITH: OPENING = 24'

$W = 24'/2 = 12'$ $H = 6'-10\frac{1}{2}''$ $h_{silt} = 3'-1\frac{1}{2}''$

$T_{Lev} = \frac{1}{2}\gamma_{H_2O}(H^2 w)(H/3 + h_{silt}) = 95.9$ K-ft

LOAD FACTOR = 1 FOR RELIABILITY

$T_{Lev} = 95.9$ K-ft.

TORSIONAL THRESHOLD (τ_{th}):

$W = 10'$ $h = 2'$ (PILE CAP DIMENSIONS)

$A_{cp} = w * h = 2880$ in^2

$P_{cp} = 2(w+h) = 288$ in

$\tau_{th} = \phi\sqrt{f'_c}\left(A_{cp}^2/P_{cp}\right)$ $f'_c = 3$ ksi

$FS = \tau_{th}/T_{Lev} = 1.37 < 1.5$ N.G.

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US Army Corps of Engineers.

Probability of Failure
CID MO Monolith 145
Torsion in Pile Cap

Comp by: FRS
Chkd by: TDM
NOV 11

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. CID MO ~~M32~~ M145 sill does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength.
3. Material Properties used:

   Mean Concrete Strength, $f_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 50 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

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V. *Caclulating Factors of Safety*

Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis - Water to TOW). $T_u := 187kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 13ft$ Width of Pile Cap

$T_{toe} := 2.0ft$ Thickness of concrete section

$b := 12\frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 3.744 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 360 \cdot in$

Mean Concrete Strength

$\phi\tau_{IM} := \phi_B \cdot \sqrt{f'_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{IM} = 198.703 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{IM}}{T_u}$ $FS_D = 1.063$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$\phi\tau_{IU} := \phi_B \cdot \sqrt{f'_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{IU} = 212.16 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{IU}}{T_u}$ $FS_{fcu} = 1.135$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$\phi\tau_{IL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\quad \phi\tau_{IL} = 184.27 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi\tau_{IL}}{T_u}$ $\quad FS_{fcl} = 0.985$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\quad \Delta F_{fc} = 0.149$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\quad \sigma_F = 0.075$

$V_F := \frac{\sigma_F}{FS_D}$ $\quad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\quad \beta_{LN} = 0.831$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\quad R = 79.7 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\quad P_F = 20.3 \cdot \%$

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Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis - Water 1ft below TOW). $T_{u1} := 145.6 kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 13ft$ Width of Pile Cap

$T_{toe} := 2.0ft$ Thickness of concrete section

$b := 12 \frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 3.744 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 360 \cdot in$

Mean Concrete Strength

$\phi\tau_{IM} := \phi_B \cdot \sqrt{f'_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{IM} = 198.703 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{IM}}{T_{u1}}$ $FS_D = 1.365$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f'_{cU} := f'_{cM} + f'_{cM} \cdot 0.14$ $f'_{cU} = 4275 \cdot psi$

$\phi\tau_{IU} := \phi_B \cdot \sqrt{f'_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}} \right)$ $\phi\tau_{IU} = 212.16 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{IU}}{T_{u1}}$ $FS_{fcu} = 1.457$

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Lower Concrete Strength

$f_{cLw} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$\phi T_{IL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\quad \phi T_{IL} = 184.27 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi T_{IL}}{T_{u1}}$ $\quad FS_{fcl} = 1.266$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\quad \Delta F_{fc} = 0.192$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\quad \sigma_F = 0.096$

$V_F := \frac{\sigma_F}{FS_D}$ $\quad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\quad \beta_{LN} = 4.402$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\quad R = 100 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\quad P_F = 5.37 \times 10^{-4} \cdot \%$

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By [illegible] Date Aug 08 Project CID-MO Sheet 42 of 437

Chkd. By KSM Date 9/28/11 Subject POF FOR PILE-FOUNDED CLOSURES Sheet ____ of ____

TO ASSIST WITH UNDERSTANDING THE POF & CPGA ANALYSES FOR TENSION PULLOUT OF SHEETPILE.

1. IN CPGA INPUT, THE ALLOWABLES WERE USED SO THAT THE ALF AND CBF OUTPUT OF 1.0 WAS THE THRESHOLD. A FS=1.5 WAS USED FOR STRUCTURAL STRENGTH AND A FS = 1.7 WAS USED FOR GEOTECHNICAL STRENGTHS. THESE FS ARE USED AS THRESHOLDS FOR THE FEASIBILITY ANALYSIS; THEY DO NOT CONSTITUTE DESIGN PARAMETERS.
2. THE POF FOR PULLOUT USED ULTIMATE GEOTECHNICAL STRENGTHS FOR THE MEAN.

2011 REVISION

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Chkd: KSM 9/20/11

M145R.dat

```
10 CID-MO M145 GEOTECH REV STRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -60.56 31.44 -170.63 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145R.txt
```

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M145R.TXT

```
*********************************
* CASE PROGRAM   #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29   *  RUN DATE 06-SEP-2011   RUN TIME 13.57.54
*********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                        X          Y          Z
                                      -----      -----      -----
WITH DIAGONAL COORDINATES = (        -1.50 ,     2.00 ,      .00 )
                            (         1.50 ,    11.50 ,      .00 )

*******************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6

*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

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M145R.TXT

3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-60.6	31.4	-170.6	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.1039E+00	.1541E-01	-.1372E-03	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

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M145R.TXT

# INDICATES CBF BASED ON MOMENTS DUE TO (F3*EMIN) FOR CONCRETE PILES

B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-16.3	61.9	807.7	.0	.0	.76	1.40	.00	.00*
2	.0	-16.3	28.2	807.7	.0	.0	.35	1.57	.00	.00*
3	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
4	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
5	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*
6	.0	-7.0	-14.7	229.5	.0	.0	1.78	3.88	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-16.3	61.9	807.7	.0	.0
2	.0	-16.3	28.2	807.7	.0	.0
3	.0	-7.0	-14.7	229.5	.0	.0
4	.0	-7.0	-14.7	229.5	.0	.0
5	.0	-7.0	-14.7	229.5	.0	.0
6	.0	-7.0	-14.7	229.5	.0	.0

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2011 REVISION

US Army Corps of Engineers

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Muller 11/9/11

WATER EL @ T.O.W.

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top\_ftg} := 746.9ft$

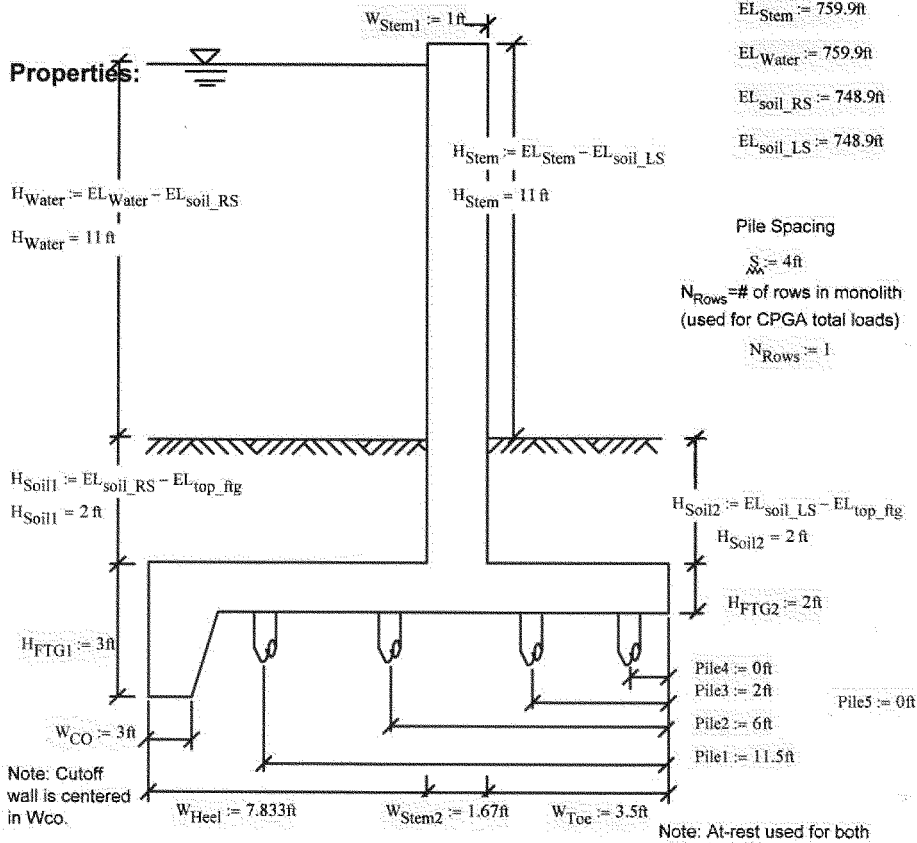
$EL_{Stem} := 759.9ft$

$EL_{Water} := 759.9ft$

$EL_{soil\_RS} := 748.9ft$

$EL_{soil\_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

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Lengths:

$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

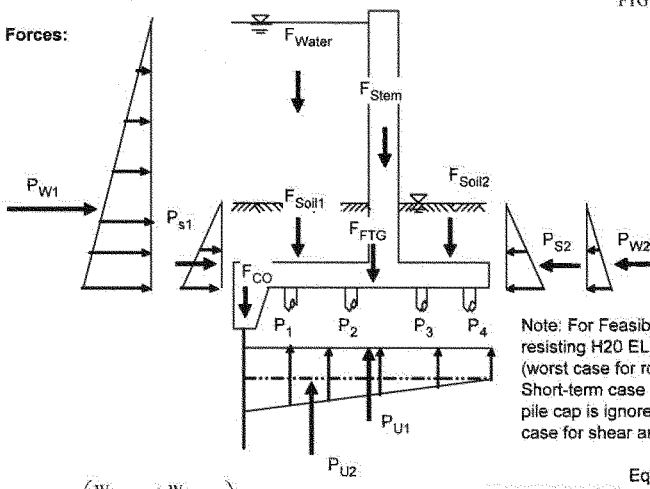
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$

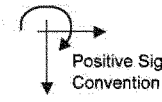
$L_{FTG} = 13\,ft$

$L_{W1} = 16\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 8.26 \frac{kip}{ft}$ Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*11²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 6354.13 \cdot plf$

$F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 7987.2 \cdot plf$

$P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 592.8 \cdot psf$

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1}) \gamma_w\right] + L_{PU1} \cdot \gamma_w = 624 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 346.2 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -608.4 \cdot plf$

$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3982.35 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1597.76 \cdot plf$

$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1497.6 \cdot plf$

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Inputs for CPGA

Assumes perfectly rigid pile cap.

Positive Sign Convention

Sum moments around bottom corner of Toe:

Moment Arm (from bottom corner of Toe)

$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$

$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$

$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$

$Arm_{closure} := 7.67\cdot ft$

$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$

$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$

$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$

$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$

$Arm_{W1} := \frac{1}{3}\cdot L_{W1} - L_{CO} = 4.33\,ft$

$Arm_{W2a} := 0.333\cdot\left(L_{PU1}\right) = 1.33\,ft$

$Arm_{W2b} := L_{CO} - \frac{L_{CO}\cdot\left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$

$Arm_{S1} := \frac{1}{3}\cdot\left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$

$Arm_{S2} := Arm_{W2a} = 1.33\,ft$

$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$

$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$

$Arm_{U3} := L_{FTG} - 0.25\cdot W_{CO} = 12.25\,ft$

Moment (about bottom corner of toe)

$M_{Stem} := -F_{Stem}\cdot Arm_{Stem} = -11.29\cdot\frac{kip\cdot ft}{ft}$

$M_{FTG} := -F_{FTG}\cdot Arm_{FTG} = -25.36\cdot\frac{kip\cdot ft}{ft}$

$M_{CO} := -F_{CO}\cdot Arm_{CO} = -5.18\cdot\frac{kip\cdot ft}{ft}$

$M_{closure} := F_{closure}\cdot Arm_{closure} = 63.35\cdot\frac{kip\cdot ft}{ft}$

$M_{Soil1} := -F_{Soil1}\cdot Arm_{Soil1} = -7.49\cdot\frac{kip\cdot ft}{ft}$

$M_{Soil2} := -F_{Soil2}\cdot Arm_{Soil2} = -0.64\cdot\frac{kip\cdot ft}{ft}$

$M_{Water1} := -F_{Water1}\cdot Arm_{Water1} = -57.74\cdot\frac{kip\cdot ft}{ft}$

$M_{Water2} := -F_{Water2}\cdot Arm_{Water2} = -0.76\cdot\frac{kip\cdot ft}{ft}$

$M_{W1} := P_{W1}\cdot Arm_{W1} = 34.61\cdot\frac{kip\cdot ft}{ft}$

$M_{W2a} := P_{W2a}\cdot Arm_{W2a} = -0.66\cdot\frac{kip\cdot ft}{ft}$

$M_{W2b} := -P_{W2b}\cdot Arm_{W2b} = 0.31\cdot\frac{kip\cdot ft}{ft}$

$M_{S1} := P_{S1}\cdot Arm_{S1} = 0.35\cdot\frac{kip\cdot ft}{ft}$

$M_{S2} := P_{S2}\cdot Arm_{S2} = -0.35\cdot\frac{kip\cdot ft}{ft}$

$M_{U1} := -P_{U1}\cdot Arm_{U1} = 22.9\cdot\frac{kip\cdot ft}{ft}$

$M_{U2} := -P_{U2}\cdot Arm_{U2} = 12.25\cdot\frac{kip\cdot ft}{ft}$

$M_{U3} := -P_{U3}\cdot Arm_{U3} = 18.35\cdot\frac{kip\cdot ft}{ft}$

$$\Sigma M := \left(M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3}\right)$$

$$\Sigma Vertical := \left(F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots + P_{U1} + P_{U2} + P_{U3}\right)$$

$$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$$

$\Sigma M = 42.66\cdot\frac{ft\cdot kip}{ft}$

$\Sigma Vertical = 7.86\cdot\frac{kip}{ft}$

$\Sigma Horizontal = 15.14\cdot\frac{kip}{ft}$

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Find Pile Loads Using CPGA:

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

Total number of piles:

Number = 3

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137,963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73,264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in3 (per Geotech)

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Concrete Sheet Pile Properties

Dimension of rectangular pile $D_{sp} := 10in$ $W_{sp} := 12in$

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{(W_{sp} \cdot D_{sp}^3)}{12}$ $I_{1sp} = 1000 \cdot in^4$ $I_{2sp} := \frac{D_{sp} \cdot W_{sp}^3}{12}$ $I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20,730klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in³ (per Geotech)

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$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

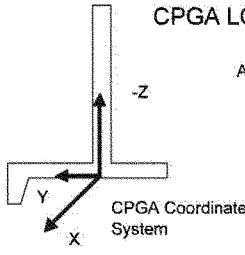
$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

$\Sigma M_T = 170.63 \cdot ft \cdot kip$

$\Sigma Vertical_T = 31.44 \cdot kip$

$\Sigma Horizontal_T = 60.56 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$

$PZ := \Sigma Vertical_T$

$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -60.56 \cdot kip$

$PZ = 31.44 \cdot kip$

$MX = -170.63 \cdot ft \cdot kip$

Loads input into CPGA filename: M145R.dat

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Pile Loading (CPGA OUTPUT)

$PileLoad1 := \frac{-14.7kip}{ft} \cdot S$ $PileLoad1 = -58.8 \cdot kip$ $PileM1 := 229.5 \frac{kip \cdot in}{ft} \cdot S$ $PileM1 = 918 \cdot kip \cdot in$

PileLoad 1 and Msht, These numbers are the output from CPGA for sheet piling (continuous) multiplied by the bearing pile spacing to get loading comparable to sheet piles for pile cap calcs

$PileLoad2 := 28.2kip$ $PileM2 := 807.7kip \cdot in$

$PileLoad3 := 61.9kip$ $PileM3 := 807.7kip \cdot in$

$PileLoad4 := 0kip$ $PileM4 := 0kip \cdot in$

$PileLoad5 := 0kip$ $PileM5 := 0kip \cdot in$

Check Vertical Loading

$$\Sigma Vertical = \frac{PileLoad1 + PileLoad2 + PileLoad3 + PileLoad4 + PileLoad5}{S} = 0.03 \cdot \frac{kip}{ft}$$

Checks if vertical loading calculated above and input into CPGA matches the vertical load output from CPGA. Checks if the equation above is approx. = 0.

Input Maximum Horizontal Pile Reaction: $Pile_{Hormax} := 16.3 \cdot kip$

Sheetpile Horizontal Reaction: $Sht_{Hormax} := 7 \cdot kip$

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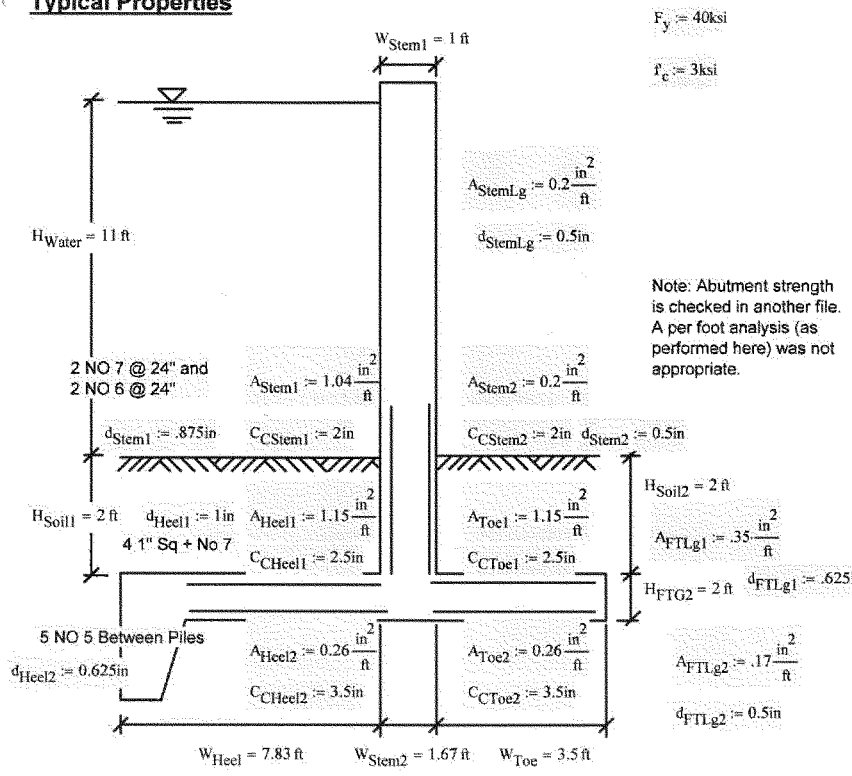
Typical Properties



Assumptions:

1. Pile cap is assumed rigid; load is distributed uniformly. Since abutment is checked in seperate file, the stem check in this file is for wall portion of stoplog gap closure only.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side water EL is the EL of the top of resisting grade.

4. Rebar Cutoff lengths assumed sufficient for feasibility study.

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Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1$	$\phi_T := 1.0$	Tensile Strength
Flexural Strength	$\phi_B := 1$		

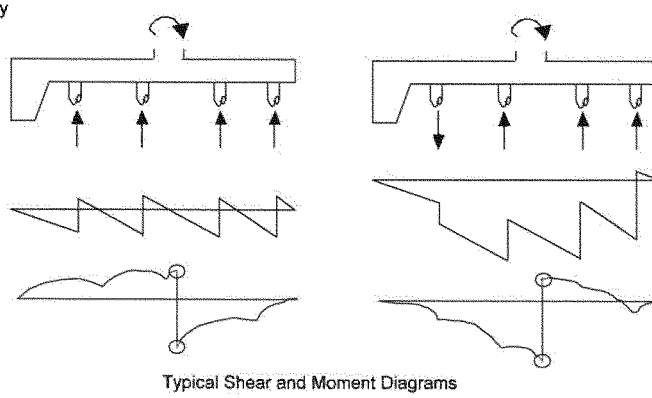
Load Factors

Dead and Live Load Factor	$\gamma_L := 1$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

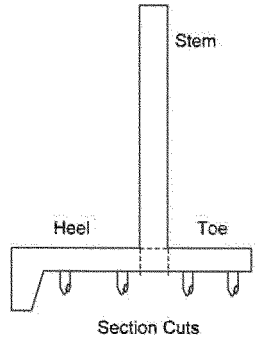
Note: Strength reduction and load factors not used in analysis of existing conditions.

Assumptions for Analysis

It has been assumed that in general the maximum transverse bending moment from loading will occur at or near the connection of the pile cap and stem. As a result section cuts at the intersection of the pile cap and stem wall have been made to evaluate the floodwalls flexural capacity



Typical Shear and Moment Diagrams



Section Cuts

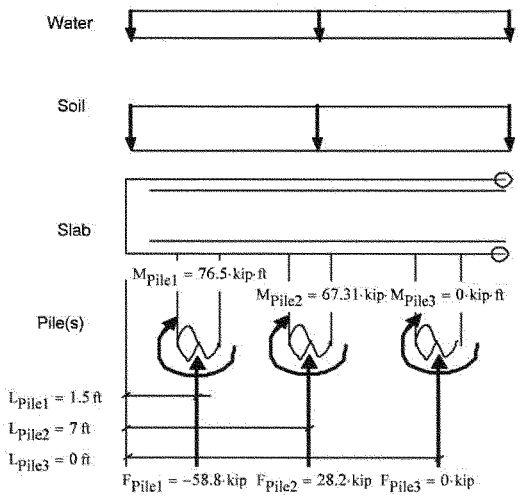
56/437 - **HEEL Strength Check:**

$F_{Pile1} := if(L_{FTG} - Pile1 < W_{Heel}, PileLoad1, 0kip)$ $L_{Pile1} := if(L_{FTG} - Pile1 < W_{Heel}, L_{FTG} - Pile1, 0ft)$

$F_{Pile2} := if(L_{FTG} - Pile2 < W_{Heel}, PileLoad2, 0kip)$ $L_{Pile2} := if(L_{FTG} - Pile2 < W_{Heel}, L_{FTG} - Pile2, 0ft)$

$F_{Pile3} := if(L_{FTG} - Pile3 < W_{Heel}, PileLoad3, 0kip)$ $L_{Pile3} := if(L_{FTG} - Pile3 < W_{Heel}, L_{FTG} - Pile3, 0ft)$

$M_{Pile1} := if(F_{Pile1} = 0, 0, PileM1)$ $M_{Pile2} := if(F_{Pile2} = 0, 0, PileM2)$ $M_{Pile3} := if(F_{Pile3} = 0, 0, PileM3)$



$W_w := \gamma_w \cdot (H_{Water} + H_{Soil1}) \cdot W_{Heel}$

$W_w = 6354 \frac{lb}{ft}$

$W_{sH} := (\gamma_{sat} - \gamma_w) \cdot H_{Soil1} \cdot W_{Heel}$

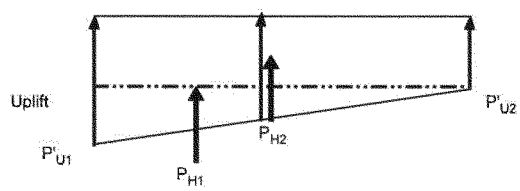
$W_{sH} = 824.03 \frac{lb}{ft}$

$d_{H1} := H_{FTG2} - C_{CHeel1} - d_{FTLg1} - \frac{1}{2} d_{Heel1}$

$d_{H2} := H_{FTG2} - C_{CHeel2} - d_{FTLg2} - \frac{1}{2} d_{Heel2}$

Pile Spacing (O.C.)

$S = 4\,ft$



Note: Assumes Sheet Pile reduces uplift press. by half.

$P'_{U1} := q_{umax} = 624 \cdot psf$

$P'_{U2} := \frac{(q_{umax} - q_{utoe})(W_{Toe} + W_{Stem2})}{(L_{FTG} - .5W_{CO})} + q_{utoe} = 471.06 \cdot psf$

$P'_{U2p} := \frac{(q_{umax} - q_{utoe})(L_{FTG} - L_{Pile2})}{(L_{FTG} - .5W_{CO})} + q_{utoe} = 491.1 \cdot psf$

$P_{H1} := \frac{1}{2}(P'_{U1} - P'_{U2}) \cdot (W_{Heel} - 0.5W_{CO}) = 484.29 \cdot plf$

$P_{H2} := P'_{U2} \cdot (W_{Heel} - .5W_{CO}) = 2983.2 \cdot plf$

$P_{H1p} := \frac{1}{2}(P'_{U1} - P'_{U2p}) \cdot (L_{Pile2} - 0.5W_{CO}) = 365.67 \cdot plf$

$P_{H2p} := P'_{U2} \cdot (L_{Pile2} - .5W_{CO}) = 2592.23 \cdot plf$

$P_{H3} := -P_{U3} = 1497.6 \cdot plf$

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- HEEL Strength Check (cont'd):

Loading: $F_{CO} = 450 \cdot plf$

Slab Weight $W_H := (W_{Heel} \cdot H_{FTG2}) \cdot \gamma_c$

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{F_{Pile1} \cdot (W_{Heel} - L_{Pile1}) + F_{Pile2} \cdot (W_{Heel} - L_{Pile2}) + F_{Pile3} \cdot (W_{Heel} - L_{Pile3})}{S} + P_{H1} \cdot \frac{2(W_{Heel} - .5W_{CO})}{3} \ldots \right.$$
$$+ P_{H2} \cdot \frac{(W_{Heel} - .5W_{CO})}{2} + P_{H3} \cdot (W_{Heel} - .25W_{CO}) - (W_H + W_w + W_{sH}) \cdot \frac{W_{Heel}}{2} \ldots$$
$$\left. + -F_{CO} \cdot (W_{Heel} - .5W_{CO}) + \frac{M_{Pile1} + M_{Pile2} + M_{Pile3}}{S} \right]$$

Positive Sign Convention

$M_H = -69.32 \cdot \frac{kip \cdot ft}{ft}$ $M_{uH} := |M_H|$ $M_{uH} = 69.32 \cdot \frac{kip \cdot ft}{ft}$

Shear

Shear at Stem

$$V_{Ha} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(F_{CO} + W_H + W_w + W_{sH}) - \frac{F_{Pile1} + F_{Pile2}}{S} - P_{H2} - P_{H3} - P_{H1} \right] = 12662.96 \cdot plf$$

Shear at Pile 2

$$V_{Hb} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left[F_{CO} + (W_H + W_w + W_{sH}) \cdot \frac{L_{Pile2}}{W_{Heel}} \right] - \frac{F_{Pile1}}{S} - P_{H1p} - P_{H2p} - P_{H3} \right] = 19212.95 \frac{lb}{ft}$$

$V_H := \max(V_{Ha}, V_{Hb})$ $V_{uH} := |V_H|$ $V_{uH} = 19.21 \frac{kip}{ft}$

Capacity:

Flexural Capacity

Top Steel: $A_{s1} := A_{Heel1}$

$b := 12 \frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f_c \cdot b}$ $a1 = 1.5 \cdot in$

$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2}\right)$

$\phi M_{H1} = 75.22 \cdot \frac{kip \cdot ft}{ft}$

Bottom Steel: $A_{s2} := A_{Heel2}$

$b := 12 \frac{in}{ft}$

$a2 := \frac{A_{s2} \cdot F_y}{0.85 f_c \cdot b}$ $a2 = 0.34 \cdot in$

$\phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2}\right)$

$\phi M_{H2} = 16.92 \cdot \frac{kip \cdot ft}{ft}$

$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1})$ $\phi M_H = 75.22 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi}$ $\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$

$\phi V_{c1} = 26784 \frac{lb}{ft}$ $\phi V_{c2} = 25880 \frac{lb}{ft}$

$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1})$ $\phi V_H = 26.78 \cdot \frac{kip}{ft}$

58/437 **- HEEL Strength Check (cont'd):**

Factors of Safety

Bending $\phi M_H = 75.22 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$

$M_{uH} = 69.32 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 = 1.09$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "NO GOOD"$

Shear $\phi V_H = 26.78 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 19.21 \cdot \frac{kip}{ft}$ $\quad FS_2 = 1.39$

$Check2a := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2a = "NO GOOD"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 1.09$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Check Longitudinal Flexure Across Piles:

Note: For Feasibility and due to rigid body assumptions, an average of the total load is used to span a middle pile, minus P_{u3} and F_{CO}. Calculations at present assume no My Reaction at pile.

Length of span (pile spacing): $S = 4\ ft$

Number of piles: $Number = 3$

Tributary Pile Width: $W_{trib} := \frac{L_{FTG}}{Number} = 4.33\ ft$

Uniform Load:

$$\omega := \frac{\Sigma Vertical - (P_{U3} + F_{CO})}{L_{FTG}} \cdot W_{trib} = 2969.07 \cdot plf$$

$M_{\omega neg} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot 0.106 \cdot \omega \cdot S^2 = 5.04 \cdot kip \cdot ft$ $\quad M_{\omega pos} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot 0.078 \omega \cdot S^2 = 3.71 \cdot kip \cdot ft$

$roe_b := 0.0371$ $\quad roe_{max} := \frac{1}{2} \cdot roe_b = 0.02$ $\quad roe_{min} := 0.0050$

$roe_{neg} := \frac{A_{FTLg1}}{b \cdot d_{H1}} = 0.0014$ $\quad roe_{pos} := \frac{A_{FTLg2}}{b \cdot d_{H2}} = 0.0007$

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Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe_{neg} > roe_{min} = 0$ Not Good; a 3/4 Flexure Reduction is incurred.

$roe_{neg} \le roe_{max} = 1$ OK

$roe_{pos} > roe_{min} = 0$ Not Good; a 3/4 Flexure Reduction is incurred.

$roe_{pos} \le roe_{max} = 1$ OK

$$a_{neg} := \frac{A_{FTLg1} \cdot F_y}{0.85 \cdot f'_c \cdot b} = 0.46 \cdot in \qquad a_{pos} := \frac{A_{FTLg2} \cdot F_y}{0.85 \cdot f'_c \cdot b} = 0.22 \cdot in$$

$$\phi M_{yneg} := \phi_B \cdot \frac{3}{4} \cdot A_{FTLg1} \cdot ft \cdot F_y \cdot \left(d_{H1} - \frac{a_{neg}}{2}\right) = 17.63 \cdot kip \cdot ft \qquad \phi M_{ypos} := \phi_B \cdot \frac{3}{4} \cdot A_{FTLg2} \cdot ft \cdot F_y \cdot \left(d_{H2} - \frac{a_{pos}}{2}\right) = 8.32\, ft \cdot kip \cdot \frac{ft}{ft}$$

Factor of Safety:

$$FS_{2b} := \frac{\phi M_{yneg}}{M_{\omega neg}} \qquad FS_{2b} = 3.5$$

$$Check2b := if\left(\phi M_{yneg} > 1.5 M_{\omega neg}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check2b = \text{"OKAY"}$$

$$FS_{2c} := \frac{\phi M_{ypos}}{M_{\omega pos}} \qquad FS_{2c} = 2.25$$

$$Check2c := if\left(\phi M_{ypos} > 1.5 M_{\omega pos}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check2c = \text{"OKAY"}$$

$$FS_{LF} := \min\left(FS_{2b}, FS_{2c}\right) = 2.25$$

Shear is sufficient by inspection since it is adequate, from concrete capacity only, in the primary loading direction.

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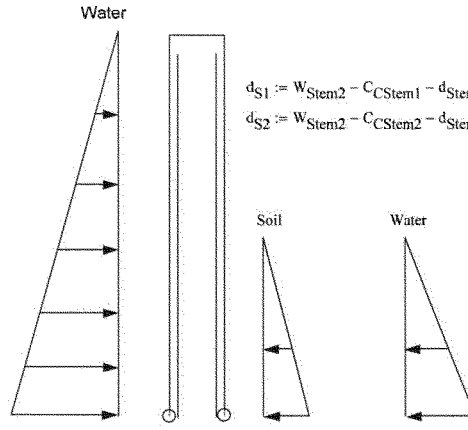
STEM Strength Check

For stoplog gap sections other than abutment. Abutment was checked in another file.



$d_{S1} := W_{Stem2} - C_{CStem1} - d_{StemLg} - 0.5d_{Stem1}$

$d_{S2} := W_{Stem2} - C_{CStem2} - d_{StemLg} - 0.5d_{Stem2}$

$P_{ws1} := (H_{Water} + H_{Soil1}) \cdot \gamma_w$

$P_{ws1} = 811 \frac{lb}{ft^2}$

$P_{ss1} := K_o \cdot (H_{Soil1}) \cdot (\gamma_{sat} - \gamma_w)$

$P_{ss1} = 65.79 \frac{lb}{ft^2}$

$P_{ss} := H_{Soil2} \cdot K_o \cdot (\gamma_{sat} - \gamma_w)$ $\quad P_{ws2} := H_{Soil2} \cdot \gamma_w$

$P_{ss} = 65.79 \frac{lb}{ft^2}$ $\quad P_{ws2} = 124.8 \frac{lb}{ft^2}$

Loading:

Bending

$$M_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} \cdot \frac{H_{Water} + H_{Soil1}}{3} + \frac{P_{ss1} \cdot (H_{Soil1})}{2} \cdot \frac{H_{Soil1}}{3} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \cdot \frac{H_{Soil2}}{3} \right]$$

$M_S = 23 \cdot \frac{kip \cdot ft}{ft}$ $\quad M_{uS} := |M_S|$

$M_{uS} = 23 \cdot \frac{kip \cdot ft}{ft}$

Positive Sign Convention

Shear

$$V_S := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{P_{ws1} \cdot (H_{Water} + H_{Soil1})}{2} + \frac{P_{ss1} \cdot H_{Soil1}}{2} - \frac{(P_{ss} + P_{ws2}) \cdot (H_{Soil2})}{2} \right]$$

$V_S = 5.15 \cdot \frac{kip}{ft}$ $\quad V_{uS} := |V_S|$

$V_{uS} = 5.15 \cdot \frac{kip}{ft}$

$A_{ts} := \frac{1}{2} \cdot \left(\frac{W_{Stem1} + W_{Stem2}}{2} \right) \cdot 0.0028 = 0.27 \frac{1}{ft} \cdot in^2$ Per face.

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- STEM Strength Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 1.36 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 56.93 \frac{kip \cdot ft}{ft}$

Shear Capacity

$\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_{S1} := \phi_V \cdot A_{Stem2} \cdot F_y \cdot \mu = 8000\frac{lb}{ft}$ Does not rely on any flexural reinforcing.

$\phi V_{S2} := \phi_V \cdot \left(2 \cdot b \cdot d_{S1} \cdot \sqrt{f'_c \cdot psi}\right) = 22481.82\frac{lb}{ft}$

$\phi V_S := \min\left(\phi V_{S1}, \phi V_{S2}\right) = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8000\frac{lb}{ft}$

Temperature and Shrinkage

$A_{StemLg} = 0.2\frac{1}{ft} \cdot in^2$ $\quad A_{ts} = 0.27\frac{1}{ft} \cdot in^2$ $\quad FS_{3a} := \frac{A_{StemLg}}{A_{ts}}$ $\quad FS_{3a} = 0.74$

Temperature and Shrinkage is not a failure mechanism for Risk and Reliability; however, it is an indication of structural condition.

Factors of Safety

Bending

$\phi M_S = 56.93 \frac{kip \cdot ft}{ft}$ $\quad FS_{3b} := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 22.77 \frac{kip \cdot ft}{ft}$ $\quad FS_{3b} = 2.5$

$Check3b := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check3b = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 5.15 \cdot \frac{kip}{ft}$ $\quad FS_4 = 1.55$

$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_{3b}, FS_4\right)$ $\quad FS_S = 1.55$

Controlling Mechanism

$Comment_S := if\left(FS_{3b} > FS_4, "Shear\ in\ Stem", if\left(M_S > 0, "Stem\ Flexural\ Riverside\ Steel", "Stem\ Flexural\ Landside\ Steel"\right)\right)$

$Comment_S = "Shear\ in\ Stem"$

Note: No Cut-off bar checks were made for Feasibility Study.

TOE Strength Check

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$F_{Pile2} := if(Pile2 < W_{Toe}, PileLoad2, 0kip)$ $F_{Pile3} := if(Pile3 < W_{Toe}, PileLoad3, 0kip)$ $F_{Pile4} := if(Pile4 < W_{Toe}, PileLoad4, 0kip)$

$L_{Pile2} := if(Pile2 < W_{Toe}, Pile2, 0ft)$ $L_{Pile3} := if(Pile3 < W_{Toe}, Pile3, 0ft)$ $L_{Pile4} := if(Pile4 < W_{Toe}, Pile4, 0ft)$

$M_{Pile2} := if(Pile2 < W_{Toe} + 6in, PileM2, 0kip)$ $M_{Pile3} := if(F_{Pile3} = 0, 0, PileM3)$ $M_{Pile4} := if(F_{Pile4} = 0, 0, PileM4)$

Soil

$W_{swT} := (\gamma_{sat} \cdot H_{Soil2}) \cdot W_{Toe}$ $W_{swT} = 805 \frac{lb}{ft}$

Slab

$d_{T1} := H_{FTG2} - C_{CToe1} - d_{FTLg1} - 0.5d_{Heel1}$

$d_{T2} := H_{FTG2} - C_{CToe2} - d_{FTLg2} - 0.5d_{Heel2}$

$M_{Pile2} = 0 \cdot kip \cdot ft$ $M_{Pile3} = 67.31 \cdot kip \cdot ft$ $M_{Pile4} = 0 \cdot kip \cdot ft$

Pile Spacing (O.C.)

$S = 4\,ft$

$L_{Pile3} = 2\,ft$ $L_{Pile4} = 0$

Pile(s)

$L_{Pile2} = 0\,ft$

$F_{Pile2} = 0 \cdot kip$ $F_{Pile3} = 61900\,lb$

Note: Assumes Sheet Piling reduces uplift pressures by half.

Uplift

$q_{utoe} = 346.2 \cdot psf$

$P_{H4} := q_{utoe} \cdot W_{Toe} = 1211.7 \frac{lb}{ft}$

P_{U3} P_{H4}

$$P'_{U3} := \frac{(q_{umax} - q_{utoe})(W_{Toe})}{(L_{FTG} - 0.5W_{CO})} + q_{utoe} = 430.73 \cdot psf \qquad P_{U3} := \frac{1}{2} \cdot (P'_{U3} - q_{utoe}) \cdot W_{Toe} = 147.92 \frac{lb}{ft}$$

Loading:

Slab Weight $W_T := (W_{Toe} \cdot H_{FTG2}) \cdot \gamma_c$ $W_T = 1050 \frac{lb}{ft}$

Pile Centroid

$$L_{PileT} := \frac{L_{Pile2} \cdot F_{Pile2} + L_{Pile3} \cdot F_{Pile3} + L_{Pile4} \cdot F_{Pile4}}{F_{Pile2} + F_{Pile3} + F_{Pile4}} \qquad L_{PileT} = 2\,ft$$

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[(W_T + W_{swT}) \cdot \frac{W_{Toe}}{2} - \left(\frac{F_{Pile2} + F_{Pile3} + F_{Pile4}}{S} \right) \cdot (W_{Toe} - L_{PileT}) - P_{H3} \cdot \frac{W_{Toe}}{3} \cdots + -P_{H4} \cdot \frac{W_{Toe}}{2} + \frac{M_{Pile2} + M_{Pile3} + M_{Pile4}}{S} \right]$$

Positive Sign Convention

$M_T = -5.43 \cdot \frac{kip \cdot ft}{ft}$ $M_{uT} := |M_T|$ $M_{uT} = 5.43 \cdot \frac{kip \cdot ft}{ft}$

TOE Strength Check (Cont'd) 63/437

Shear

Shear at Stem.

$$V_{Ta} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left(W_T + W_{swT}\right) - \left(\frac{F_{Pile2} + F_{Pile3} + F_{Pile4}}{S} \right) - P_{H3} - P_{H4} \right] = -14979.62 \frac{lb}{ft}$$

$$V_{Ta} := \left| V_{Ta} \right|$$

Shear at Pile

$$V_{Tb} := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\left(W_T + W_{swT}\right) \cdot \frac{L_{Pile3}}{W_{Toe}} - \left[q_{utoe} \cdot L_{Pile3} + .5 \cdot \frac{\left(q_{umax} - q_{utoe}\right) \cdot L_{Pile3}}{\left(L_{FTG} - .5 W_{CO}\right)} \cdot L_{Pile3} \right] - \frac{F_{Pile3}}{S} \right] = -15155.7 \frac{lb}{ft}$$

$$V_{Tb} := \left| V_{Tb} \right| \qquad V_{uT} := \max\left(V_{Ta}, V_{Tb}\right) \qquad V_{uT} = 15.16 \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Top Steel: $A_{s1} := A_{Toe1}$

$$b := 12 \frac{in}{ft}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 1.5 \cdot in$$

$$\phi M_{T1} := \phi_B \cdot A_{s1} \cdot F_y \cdot \left(d_{T1} - \frac{a1}{2}\right)$$

$$\phi M_{T1} = 75.22 \frac{kip \cdot ft}{ft}$$

Bottom Steel: $A_{s2} := A_{Toe2}$

$$b := 12 \frac{in}{ft}$$

$$a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b} \qquad a2 = 0.34 \cdot in$$

$$\phi M_{T2} := \phi_B \cdot A_{s2} \cdot F_y \cdot \left(d_{T2} - \frac{a2}{2}\right)$$

$$\phi M_{T2} = 16.92 \frac{kip \cdot ft}{ft}$$

$$\phi M_T := if\left(M_T > 0, \phi M_{T1}, \phi M_{T2}\right) \qquad \phi M_T = 16.92 \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{T1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c1} = 26784 \frac{lb}{ft}$$

$$\phi V_{c2} := \phi_V \cdot 2 \cdot b \cdot d_{T2} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} = 25880 \frac{lb}{ft}$$

$$\phi V_T := if\left(M_{uT} > 0, \phi V_{c1}, \phi V_{c2}\right) \qquad \phi V_T = 26.78 \frac{kip}{ft}$$

Factors of Safety

Bending

$$\phi M_T = 16.92 \frac{kip \cdot ft}{ft} \qquad M_{uT} = 5.43 \frac{kip \cdot ft}{ft} \qquad FS_5 := \frac{\phi M_T}{M_{uT}} \qquad FS_5 = 3.11$$

$$Check5 := if\left(\phi M_T > 1.5 M_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check5 = \text{"OKAY"}$$

Shear

$$\phi V_T = 26.78 \frac{kip}{ft} \qquad V_{uT} = 15.16 \frac{kip}{ft} \qquad FS_6 := \frac{\phi V_T}{V_{uT}} \qquad FS_6 = 1.77$$

$$Check6 := if\left(\phi V_T > 1.5 V_{uT}, \text{"OKAY"}, \text{"NO GOOD"}\right) \qquad Check6 = \text{"OKAY"}$$

Controlling Factor of Safety

$$FS_T := \min\left(FS_5, FS_6\right) \qquad FS_T = 1.77$$

Controlling Mechanism

$$Comment_T := if\left(FS_5 > FS_6, \text{"Shear in Heel"}, if\left(M_T > 0, \text{"Flexural Top Steel in Toe"}, \text{"Flexural Bottom Steel in Toe"}\right)\right)$$

$$Comment_T = \text{"Shear in Heel"}$$

Pile Checks:

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Tension Pile Embedment: Pullout Force per foot: $T_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \frac{-PileLoad1 \cdot 1ft}{S} = 14.7 \cdot kip$

Equations are established for straight bars.

$\psi_t := 1.0$ $\psi_e := 1.0$ $\psi_s := 0.8$ $\lambda := 1.0$

Embedment Length $l_{em} := 18in$

Bar diameter: $d_b := .75in$ $c_b := 2in$ K_{tr} is assumed zero.

Number of bars per foot: $N_b := 4.8$ Area of 1 bar: $A_{bt} := 0.44in^2$

$$c_{mod} := if\left(\frac{c_b}{d_b} > 2.5, 2.5, \frac{c_b}{d_b}\right) = 2.5 \qquad l_d := \frac{3}{40} \cdot \frac{F_y}{\sqrt{f'_c \cdot psi}} \cdot \frac{\psi_t \cdot \psi_s \cdot \psi_e \cdot \lambda}{c_{mod}} \cdot d_b = 13.15 \cdot in$$

$Check7 := if(l_{em} > l_d, "OKAY", "NO GOOD")$ Check7 = "OKAY"

$T_n := \phi_T \cdot N_b \cdot A_{bt} \cdot F_y = 84.48 \cdot kip$

$Check8 := if(T_n > 1.5 \cdot T_u, "OKAY", "NO GOOD")$ $FS_8 := \frac{T_n}{T_u} = 5.75$ Check8 = "OKAY"

Punching Shear Check (assuming 3rd pile in max compression and distance to edge of pile cap is > d/2):

Side of a square pile: $D = 18 \cdot in$ Compression Force: $F_c := PileLoad3 \cdot (\gamma_L \cdot \gamma_H \cdot \gamma_X) = 61.9 \cdot kip$

Pile Embedment into pile cap: $Pile_{em} := 12in$ Height of concrete cone: $h_{cone} := H_{FTG2} - Pile_{em} = 1\ ft$

$\alpha_s := 20$ $d := d_{H1} - h_{cone} = 8.37 \cdot in$ $b_o := 4 \cdot (D + d) = 105.5 \cdot in$ $\alpha_{mod} := if\left(\frac{\alpha_s \cdot d}{b_o} + 2 > 4, 4, \frac{\alpha_s \cdot d}{b_o} + 2\right) = 3.59$

$V_c := \phi_V \cdot \alpha_{mod} \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 173.62 \cdot kip$

$Check9 := if(V_c > 1.5 \cdot F_c, "OKAY", "NO GOOD")$ $FS_9 := \frac{V_c}{F_c} = 2.8$ Check9 = "OKAY"

Vertical Bearing Check (assuming 3rd pile in max compression):

$B_{pile} := \phi_V \cdot D^2 \cdot 0.85 \cdot f'_c = 826.2 \cdot kip$

$Check10 := if(B_{pile} > 1.5 \cdot F_c, "OKAY", "NO GOOD")$ $FS_{10} := \frac{B_{pile}}{F_c} = 13.35$ Check10 = "OKAY"

Horzontal Shear and Bearing Checks.

$B_{pile2} := \phi_V \cdot Pile_{em} \cdot D \cdot 0.85 \cdot f'_c = 550.8 \cdot kip$ $FS_{11} := \frac{B_{pile2}}{Pile_{Hormax}} = 33.79$

$Check11 := if(B_{pile2} > 1.5 \cdot Pile_{Hormax}, "OKAY", "NO GOOD")$ Check11 = "OKAY"

$B_{sht} := \phi_V \cdot Pile_{em} \cdot W_{sp} \cdot 0.85 \cdot f'_c = 367.2 \cdot kip$ $FS_{12} := \frac{B_{sht}}{Pile_{Hormax}} = 22.46$

$Check12 := if(B_{pile2} > 1.5 \cdot Pile_{Hormax}, "OKAY", "NO GOOD")$ Check12 = "OKAY"

$FS_B := min(FS_{10}, FS_{11}, FS_{12}) = 13.35$

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Shear. Due to the continuous construction of the sheet pile, b = 12".

$Sht_{Hormax} = 7 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $\quad b_{ev} := 12in$

$V_{ev} := \phi_V \cdot 2 \cdot \sqrt{f_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if\left(V_c > 1.5 \cdot Sht_{Hormax}, \text{"OKAY"}, \text{"NO GOOD"}\right)$ $\quad FS_{13} := \frac{V_c}{Sht_{Hormax}} = 1.57$ $\quad$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.09$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.25$
Stem Factor of Safety:	$FS_S = 1.55$
Toe Factor of Safety:	$FS_T = 1.77$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 5.75$
Punching Shear Factor of Safety:	$FS_9 = 2.8$
Bearing Factor of Safety:	$FS_B = 13.35$
Pile Cap Shear Factor of Safety:	$FS_{13} = 1.57$

$FoS := \min\left(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13}\right)$

$FoS = 1.09$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

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Comp by: Sheffield
Chkd by: Marx 9/28/11

Units:

CID MO 10"x12" (16') Sheetpile

$kips := 1000lb$

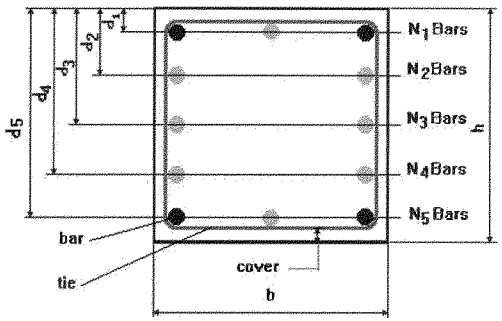
$psi := \frac{lb}{in^2}$ $\quad psf := \frac{lb}{ft^2}$

$ton := 2000lb$ $\quad tsf := \frac{ton}{ft^2}$

Note: The file has been used by many Engineers on previous projects who have also verified the results with independent programs. It was not closely re-examined for a Reliability Analysis.

$klf := \frac{kips}{ft}$ $\quad ksi := \frac{kips}{in^2}$ $\quad pcf := \frac{lb}{ft^3}$ $\quad plf := \frac{lb}{ft}$

This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8



Total Rows of Bars- NOTE: This can be changed and more rows added if required.

$h := 10in$	Column Depth	$cover := 2.5in$	Cover -in	
$w := 12in$	Column Width	$N_1 := 2.4$	Bars in Row 1	
$bar := 6$	Bar Size	$N_2 := 2.4$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 0$	Bars in Row 3	
$f'c := 3ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row = 2$

$A_v := \left[\pi \cdot \left(\frac{tie \cdot in}{8 \cdot 2}\right)^2\right] = 0.049 \cdot in^2$ $\quad s_v := 8 \cdot in$

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 3000psi$

FEMA 310 Allowed for concrete aging: $FEMA\_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA\_Factor$ $\quad f'c = 3000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

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$areabar = (0in^2\ \ 0in^2\ \ .05in^2\ \ 0.11in^2\ \ 0.2in^2\ \ 0.31in^2\ \ 0.44in^2\ \ 0.6in^2\ \ 0.79in^2\ \ 1in^2\ \ 1.27in^2\ \ 1.56in^2\ \ 0in^2\ \ 0in^2\ \ 2.25in^2)$

$diabar = (0in\ \ 0in\ \ .25in\ \ 0.38in\ \ 0.5in\ \ 0.63in\ \ 0.75in\ \ 0.88in\ \ 1in\ \ 1.13in\ \ 1.27in\ \ 1.41in\ \ 0in\ \ 0in\ \ 1.69in)$

Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1..row$

Bar Diameter $db := diabar_{0,bar}$ $db = 0.75\cdot in$

Bar area at i $A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $dt := diabar_{0,tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 3.125\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 3.75\cdot in$ $d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	2.4	1.056 $\cdot in^2$	3.125 $\cdot in$
2	2.4	1.056	6.875

Gross Area: $Ag := w\cdot h$ $Ag = 120\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 2.11\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0176$ $b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$ Effective Width $b = 12\cdot in$

$Ag := b\cdot h$ $Ag = 120\cdot in^2$ $\frac{Ast}{Ag} = 0.0176$

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Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $\quad Pn_1 = 308 \cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 308.076 \cdot kips \quad Mn_0 := 0kips \cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi \cdot Pn_1 = 308 \cdot kips$

Assume c=h $\quad c := h \quad c = 10 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[\left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right)\right] \cdot A_i\right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b \qquad Pn_1 = 308.076 \cdot kips$$

$c := Find(c) \quad c = 8.94 \cdot in$

Calculate Width of Compression block: $\quad a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 7.598 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c} \quad \varepsilon s_i =$

0.002
0.001

$fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right) \quad fs_i =$

40	·ksi
20.086	

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$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Fs_i =$ $\begin{bmatrix} 39.547 \\ 18.518 \end{bmatrix} \cdot kips$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $\quad Ms_i =$ $\begin{bmatrix} 74.151 \\ -34.722 \end{bmatrix} \cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 39.429 \cdot kips \cdot in$ $\quad Fst := \sum_{i=1}^{row} Fs_i$ $\quad Fst = 58.065 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 23.27 \cdot kips \cdot ft$ $\quad Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $\quad Fc = 232.497 \cdot kips$

$Mn_1 := Mc + Mst$ $\quad Mn_1 = 318.666 \cdot kips \cdot in$ $\quad e_1 := \frac{Mn_1}{Pn_1}$ $\quad e_1 = 1.03 \cdot in$

$d_{row} = 6.875 \cdot in$

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Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 6.875 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 5.844 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$	$Fs_i =$	$Ms_i =$	$A_i =$	$d_i =$
1	0.002	40 ·ksi	39.547 ·kips	74.151 ·kips·in	1.056 ·in²	3.125 ·in
2	0	20.086	21.211	-39.771	1.056	6.875

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 34.38 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 371.608 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 405.988 \cdot kips \cdot in$ $\quad Mn_2 = 406 \cdot kips \cdot in$

$Mn_1 = 318.666 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 318.666 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 240 \cdot kips$ $\quad Pn_1 = 308.076 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 239.577 \cdot kips$

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Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 6.875 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c} \qquad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$$

$$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.26 ft	0.001	40 ·ksi	1.056 $\cdot in^2$	39.547 ·kips	74.151 ·kips·in
0.573	-0	-10	1.056	-10.56	19.8

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 94 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 381.609 \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 476 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 189 \cdot kips$$

Find Pn and Mn at point of 0.50 *fy tension in bars:

$d_{row} = 6.875 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 5.59 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.26 ft	0.001	38.364 ·ksi	1.056 $\cdot in^2$	37.819 ·kips	70.911 ·kips·in
0.573	-0.001	-20	1.056	-21.12	39.6

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 111 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 381.555 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 492 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 162 \cdot kips$

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Find Pn and Mn at Balanced:

$d_{row} = 6.875 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 4.71 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.26 ft	0.001	29.273 ·ksi	1.056 $\cdot in^2$	28.219 ·kips	52.911 ·kips·in
0.573	-0.001	-40	1.056	-42.24	79.2

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 132 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 367.298 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 499 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 108 \cdot kips$

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Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 36 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 36 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5$ $\quad c = 5 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $\quad c = 3.07 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\quad a = 2.61 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c}$ $\quad fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
3.125 $\cdot in$	-0	-1540.645 $\cdot psi$	1.056 $\cdot in^2$	-1.627 $\cdot kips$	-3.05 $\cdot in \cdot kips$
6.875	-0.004	-40000	1.056	-42.24	79.2

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$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 76 \cdot kips \cdot in$$

$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 295.107 \cdot kips \cdot in$$

$$Mn_6 := Mc + Mst \qquad Mn_6 = 371 \cdot kips \cdot in$$

$$Pn_6 = 36 \cdot kips$$

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Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\begin{cases} (c - d_i) \cdot \frac{0.003}{c} \cdot Es \ldots + -if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es < fy \\ fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0) & \text{if } (c - d_i) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{cases} \cdot A_i \right] \ldots + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $\qquad$ $c = 2.503 \cdot in$ $\qquad$ $a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 2.127 \cdot in$

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Find Moment:

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$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\qquad$ $fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\qquad$ $Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.26	ft
0.573	

$\varepsilon s_i =$

-0.001
-0.005

$fs_i =$

-21.639	·ksi
-40	

$A_i =$

1.056	·in
1.056	

$Fs_i =$

-22.851	·kips
-42.24	

$Ms_i =$

-42.846	·kips·in
79.2	

$Mst := \sum_{i=1}^{row} Ms_i$ $\qquad$ $Mst = 36 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\qquad$ $Mc = 256 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$ $\qquad$ $Mn_7 = 293 \cdot kips \cdot in$

$Pn_8 := -fy \cdot Ast = -84.48 \cdot kips$ $\qquad$ $Mn_8 := 0$

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$j := 0..8$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$		$\phi_j \cdot Mn_j =$	
308.076	·kip	0	·kips·in
308.076		318.666	
239.577		405.988	
189.371		475.56	
162.094		492.066	
108.477		499.409	
36		371.257	
0		292.58	
-84.48		0	

Criteria:

Depth	$h = 10 \cdot in$	$bar = 6$
Width	$w = 12 \cdot in$	$tie = 2$
Eff. Width	$b = 12 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.5 \cdot in$ $f'c = 3000 \cdot psi$

$Ag = 120 \cdot in^2$ $Ast = 2.112 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0176$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0176$ Eff. ratio ρ

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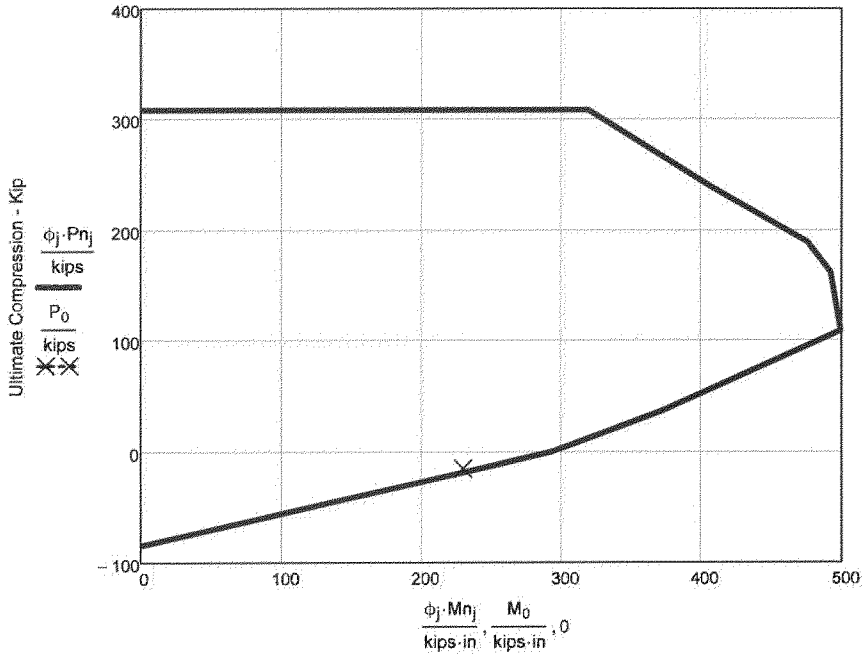
Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := -14.7kips$ Moment from CPGA output: $M_0 := 229.5kips \cdot in$

$V_{max} := 7kips$



Interaction Diagram

Nominal Curve Used to Find Factor of Safety:

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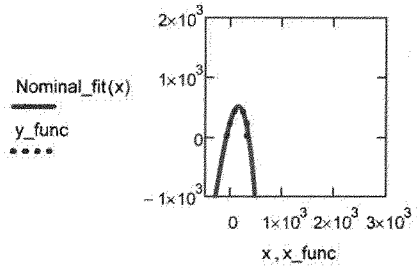
Data Points set to function: $y\_func := \frac{Mn}{kips \cdot in}$ $x\_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

$Nominal := regress(x\_func, y\_func, k)$ $Nominal\_fit(x) := interp(Nominal, x\_func, y\_func, x)$



Allowable Axial Capacity (geotech):
(assuming minimum of two 2.5' long sheet piles engaged for pullout).

Allowable Axial Compression Load for 15ft sheet pile (To be calculated by Geotechs)

$$P_c := \frac{20.73kips}{1ft}\left(\frac{5 \cdot ft}{4}\right) = 25.912 \cdot kips$$

Allowable Axial Tension Load for 15ft sheet pile (To be calculated by Geotechs)

$$P_t := \frac{-11.231kips}{1ft}\left(\frac{5ft}{4}\right) = -14.039 \cdot kips$$

Tension Pile Geotech FS:

$$FS_{GT} := \frac{P_t}{P_0} = 0.955$$

FS_{GT} Must be > 1.7 to meet Feasibility Criteria without necessitating R&R analysis.

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Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = -14.7 \cdot kips$)

$M_{capacity} := Nominal\_fit\left(\frac{P_0}{kips}\right)$ $\quad M_{capacity} = 244.675$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $\quad M_{demand} = 229.5$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $\quad FS_{strength} = 1.07$

Check Shear Through Pile:

d for pile: $d_{sht} := 6.875 \cdot in$

$$V_c := 2\left(1 + \frac{\frac{P_0}{lb}}{500 \cdot \frac{h \cdot w}{in^2}}\right) \cdot \sqrt{f'c \cdot psi} \cdot w \cdot d_{sht} = 6.823 \cdot kips \qquad V_s := \frac{A_v \cdot fy \cdot d_{sht}}{s_v} = 1.687 \cdot kips$$

$V_n := V_c + V_s = 8.511 \cdot kips$ $\quad FS_{shear} := \frac{V_n}{V_{max}} = 1.216$

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FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

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CID MO 18"x18" (34') Piles

Comp by: Sheffield
Chkd by: Marx 9/28/11

Units:

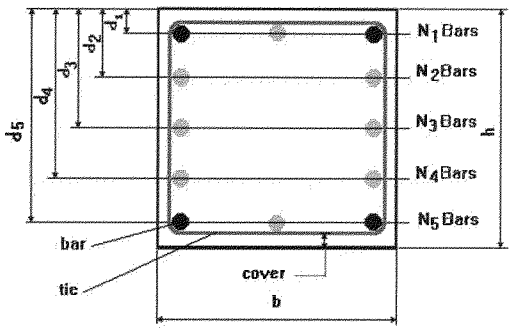
$kips := 1000lb$ $psf := \frac{lb}{ft^2}$

$psi := \frac{lb}{in^2}$

$ton := 2000lb$ $tsf := \frac{ton}{ft^2}$ $klf := \frac{kips}{ft}$ $plf := \frac{lb}{ft}$ $pcf := \frac{lb}{ft^3}$ $ksi := \frac{kips}{in^2}$

Note: The file has been used by many Engineers on previous projects who have also verified the results with independent programs. It was not closely re-examined for a Reliability Analysis.

This program calulates the column interaction curve for a rectangular concrete column using the methods of the Concrete Reinforcing Steel Institute. Note this program also accounts for less than 1 percent steel by reducing the width. Procedure based on CRSI Manual Pg 3-5 to Pg 3-8



$h := 18in$	Column Depth	$cover := 2.25in$	Cover -in	
$w := 18in$	Column Width	$N_1 := 3$	Bars in Row 1	
$bar := 7$	Bar Size	$N_2 := 2$	Bars in Row 2	
$tie := 2$	Tie Size	$N_3 := 3$	Bars in Row 3	
$f'c := 3ksi$	Conc. Str. - ksi	$N_4 := 0$	Bars in Row 4	
$fy := 40ksi$	Bar Yield - ksi	$N_5 := 0$	Bars in Row 5	$row := 3$

Total Rows of Bars- NOTE: This can be changed and more rows added if required.

Modulus of Elasticty: $Es := 29000ksi$

Concrete Strength: $f'c := 3000psi$

FEMA 310 Allowed for concrete aging: $FEMA\_Factor := 1.00$

Nominal Concrete Strength assumed orginally used: $f'c := f'c \cdot FEMA\_Factor$ $f'c = 3000 \cdot psi$

NOTE - if FEMA factor is 1.00 then concrete strength is the nominal concrete strength. If trying to get the expected concrete strength the FEMA factor should be 1.25. See FEMA 310 guidance.

$areabar := (0in^2\ \ 0in^2\ \ .05in^2\ \ 0.11in^2\ \ 0.2in^2\ \ 0.31in^2\ \ 0.44in^2\ \ 0.6in^2\ \ 0.79in^2\ \ 1in^2\ \ 1.27in^2\ \ 1.56in^2\ \ 0in^2\ \ 0in^2\ \ 2.25in^2)$

$diabar := (0in\ \ 0in\ \ .25in\ \ 0.38in\ \ 0.5in\ \ 0.63in\ \ 0.75in\ \ 0.88in\ \ 1in\ \ 1.13in\ \ 1.27in\ \ 1.41in\ \ 0in\ \ 0in\ \ 1.69in)$

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Calculate effective stress block per ACI 318 sect. 10.2.7.3 $\beta 1 := if\left[f'c \le 4ksi, 0.85, if\left[f'c > 8ksi, 0.65, 0.85 - 0.05\cdot\left(\frac{f'c}{ksi} - 4\right)\right]\right]$ $\beta 1 = 0.85$

Iterations $i := 1..row$

Bar Diameter $db := diabar_{0,bar}$ $db = 0.88\cdot in$

Bar area at i $A_i := N_i\cdot areabar_{0,bar}$

Tie Diameter $dt := diabar_{0,tie}$ $dt = 0.250\cdot in$

Distances from compression face $d_1 := cover + \frac{db}{2} + dt$ $d_1 = 2.94\cdot in$

Spacing between rows of bars $space := \frac{h - 2\cdot d_1}{row - 1}$ $space = 6.06\cdot in$ $d_i := d_1 + (i - 1)\cdot space$

$i =$	$N_i =$	$A_i =$	$d_i =$
1	3	1.8 $\cdot in^2$	2.94 $\cdot in$
2	2	1.2	9
3	3	1.8	15.06

Gross Area $Ag := w\cdot h$ $Ag = 324\cdot in^2$

Steel Area $Ast := \sum_{i=1}^{row} A_i$ $Ast = 4.8\cdot in^2$

Reduce width if $\rho < 0.01$: $\frac{Ast}{Ag} = 0.0148$ $b := if\left(\frac{Ast}{Ag} > 0.01, w, \frac{Ast}{0.01\cdot h}\right)$

Effective Width $b = 18\cdot in$

$Ag := b\cdot h$ $Ag = 324\cdot in^2$ $\frac{Ast}{Ag} = 0.0148$

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Calculate Axial Load at no eccentricity, e (min) at ϕPn(max):

$Pn_1 := 0.8 \cdot [0.85 \cdot f'c \cdot (Ag - Ast) + fy \cdot Ast]$ $\quad Pn_1 = 805 \cdot kips$

Equate as the first set of points on the interaction diagram: $\quad Pn_0 := Pn_1 \quad Pn_0 = 804.768 \cdot kips \quad Mn_0 := 0kips \cdot in$

Axial Compression with Aci Reduction value: $\quad \phi := 1 \quad \phi \cdot Pn_1 = 805 \cdot kips$

Assume c=h $\quad c := h \quad c = 18 \cdot in$

Given

$$Pn_1 = \sum_{i=1}^{row} \left[\left[\left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right)\right] \cdot A_i\right] + 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b \qquad Pn_1 = 804.768 \cdot kips$$

$c := Find(c) \quad c = 16.1 \cdot in$

Calculate Width of Compression block: $\quad a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c) \quad a = 13.687 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c}$ $\quad \varepsilon s_i =$

0.002
0.001
0

$fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$ $\quad fs_i =$

40
38.373
5.631

$\cdot ksi$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $Fs_i =$

67.41
42.988
10.136

$\cdot kips$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$ $Ms_i =$

408.505
0
-61.423

$\cdot kips \cdot in$

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$Mst := \sum_{i=1}^{row} Ms_i$ $Mst = 347.082 \cdot kips \cdot in$ $Fst := \sum_{i=1}^{row} Fs_i$ $Fst = 120.533 \cdot kips$

$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $Mc = 112.901 \cdot kips \cdot ft$ $Fc := 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$ $Fc = 628.227 \cdot kips$

$Mn_1 := Mc + Mst$ $Mn_1 = 1701.896 \cdot kips \cdot in$ $e_1 := \frac{Mn_1}{Pn_1}$ $e_1 = 2.11 \cdot in$

$d_{row} = 15.06 \cdot in$

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Find Pn and Mn at point of zero tension in bars:

$c := d_{row}$ $\quad c = 15.06 \cdot in$ $\quad a := \beta 1 \cdot c$ $\quad a = 12.801 \cdot in$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$i =$	$\varepsilon s_i =$	$fs_i =$		$Fs_i =$		$Ms_i =$		$A_i =$		$d_i =$	
1	0.002	40	$\cdot ksi$	67.41	$\cdot kips$	408.505	$\cdot kips \cdot in$	1.8	$\cdot in^2$	2.94	$\cdot in$
2	0.001	38.373		42.988		0		1.2		9	
3	0	5.631		10.136		-61.423		1.8		15.06	

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 347.082 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$ $\quad Mc = 1527.378 \cdot kips \cdot in$

$Mn_2 := Mc + Mst$ $\quad Mn_2 = 1874.46 \cdot kips \cdot in$ $\quad Mn_2 = 1874 \cdot kips \cdot in$

$Mn_1 = 1701.896 \cdot kips \cdot in$ $\quad Mn_1 := if(Mn_1 < Mn_2, Mn_1, Mn_2)$ $\quad Mn_1 = 1701.896 \cdot kips \cdot in$

$Pn_2 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_2 = 708 \cdot kips$ $\quad Pn_1 = 804.768 \cdot kips$ $\quad Pn_2 := if(Pn_2 < Pn_1, Pn_2, Pn_1)$ $\quad Pn_2 = 708.099 \cdot kips$

Find Pn and Mn at point of 0.25 *fy tension in bars:

$$c := \frac{d_{row}}{1 + \frac{0.25 \cdot fy}{Es \cdot 0.003}} \qquad d_{row} = 15.06 \cdot in \qquad a := \beta 1 \cdot c$$

Find Moment:

$$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c} \qquad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$$

$$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i \qquad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$$

$d_i =$

0.245	ft
0.75	
1.255	

$\varepsilon s_i =$

0.002
0.001
-0

$fs_i =$

40	$\cdot ksi$
29.032	
-10	

$A_i =$

1.8	$\cdot in^2$
1.2	
1.8	

$Fs_i =$

67.41	$\cdot kips$
31.778	
-18	

$Ms_i =$

408.505	$\cdot kips \cdot in$
0	
109.08	

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 518 \cdot kips \cdot in \qquad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1717.649 \cdot kips \cdot in \qquad Mn_3 := Mc + Mst \qquad Mn_3 = 2235 \cdot kips \cdot in$$

$$Pn_3 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b \qquad Pn_3 = 608 \cdot kips$$

Find Pn and Mn at point of 0.50 *fy tension in bars:

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$d_{row} = 15.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{0.50 \cdot fy}{Es \cdot 0.003}}$ $\quad c = 12.245 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.245 ft	0.002	40 ·ksi	1.8 $\cdot in^2$	67.41 ·kips	408.505 ·kips·in
0.75	0.001	23.056	1.2	24.607	0
1.255	-0.001	-20	1.8	-36	218.16

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 627 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1813.434 \cdot kips \cdot in$ $\quad Mn_4 := Mc + Mst$ $\quad Mn_4 = 2440 \cdot kips \cdot in$

$Pn_4 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_4 = 534 \cdot kips$

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Find Pn and Mn at Balanced:

$d_{row} = 15.06 \cdot in$ $\quad c := \dfrac{d_{row}}{1 + \dfrac{fy}{Es \cdot 0.003}}$ $\quad c = 10.317 \cdot in$ $\quad a := \beta 1 \cdot c$

Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \dfrac{0.003}{c}$ $\quad fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\dfrac{h}{2} - d_i\right)$

$d_i =$	$\varepsilon s_i =$	$fs_i =$	$A_i =$	$Fs_i =$	$Ms_i =$
0.245 ft	0.002	40 ·ksi	1.8 $\cdot in^2$	67.41 ·kips	408.505 ·kips·in
0.75	0	11.104	1.2	13.324	0
1.255	-0.001	-40	1.8	-72	436.32

$Mst := \sum_{i=1}^{row} Ms_i$ $\quad Mst = 845 \cdot kips \cdot in$ $\quad Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\dfrac{h}{2} - \dfrac{a}{2}\right)$ $\quad Mc = 1857.727 \cdot kips \cdot in$ $\quad Mn_5 := Mc + Mst$ $\quad Mn_5 = 2703 \cdot kips \cdot in$

$Pn_5 := \sum_{i=1}^{row} Fs_i + 0.85 \cdot f'c \cdot a \cdot b$ $\quad Pn_5 = 411 \cdot kips$

Find Mn at ϕ Pn = 0.1*f'c*Ag:

$Pn_6 := \frac{0.1 \cdot f'c \cdot w \cdot h}{\phi}$ $\quad Pn_6 = 97.2 \cdot kips$ $\quad Pn_6 := if(Pn_6 > Pn_5, Pn_5, Pn_6)$ $\quad Pn_6 = 97.2 \cdot kips$

Assume c=h/2 $\quad c := h \cdot 0.5$ $\quad c = 9 \cdot in$

Given

$$Pn_6 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - if\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := Find(c)$ $\quad c = 4.37 \cdot in$

$a := if(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\quad a = 3.72 \cdot in$

Find Moment:

$\varepsilon s_i := \left(c - d_i\right) \cdot \frac{0.003}{c}$ $\quad fs_i := if\left(Es \cdot \varepsilon s_i > fy, fy, if\left(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i\right)\right)$

$Fs_i := fs_i \cdot A_i - if\left(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi\right) \cdot A_i$ $\quad Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

2.94
9
15.06

$\cdot in$

$\varepsilon s_i =$

0.001
-0.003
-0.007

$fs_i =$

28479.727
-40000
-40000

$\cdot psi$

$A_i =$

1.8
1.2
1.8

$\cdot in^2$

$Fs_i =$

46.674
-48
-72

$\cdot kips$

$Ms_i =$

282.841
0
436.32

$\cdot in \cdot kips$

$$Mst := \sum_{i=1}^{row} Ms_i \qquad Mst = 719 \cdot kips \cdot in$$

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$$Mc := 0.85 \cdot f'c \cdot a \cdot b \cdot \left(\frac{h}{2} - \frac{a}{2}\right) \qquad Mc = 1217.971 \cdot kips \cdot in$$

$$Mn_6 := Mc + Mst \qquad Mn_6 = 1937 \cdot kips \cdot in$$

$$Pn_6 = 97 \cdot kips$$

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Find Mn at ϕ Pn = 0.0:

$Pn_7 := 0lb$ $\qquad$ $Pn_7 = 0lb$

Assume c=0.2h $\qquad$ $c := 0.2 \cdot h$

Given

$$Pn_7 = \sum_{i=1}^{row} \left[\left[\begin{array}{ll} \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \ldots & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es < fy \\ + -\text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \\ fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \geq fy \\ -fy - \text{if}\left(d_i < \beta 1 \cdot c, 0.85 \cdot f'c, 0\right) & \text{if } \left(c - d_i\right) \cdot \frac{0.003}{c} \cdot Es \leq -fy \end{array} \right] \cdot A_i \right] \ldots$$

$$+ 0.85 \cdot f'c \cdot \beta 1 \cdot c \cdot b$$

$c := \text{Find}(c)$ $\qquad$ $c = 2.998 \cdot in$ $\qquad$ $a := \text{if}(\beta 1 \cdot c > h, h, \beta 1 \cdot c)$ $\qquad$ $a = 2.548 \cdot in$

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Find Moment:

$\varepsilon s_i := (c - d_i) \cdot \frac{0.003}{c}$

$fs_i := if(Es \cdot \varepsilon s_i > fy, fy, if(Es \cdot \varepsilon s_i < 0 - fy, 0 - fy, Es \cdot \varepsilon s_i))$

$Fs_i := fs_i \cdot A_i - if(\beta 1 \cdot c > d_i, 0.85 \cdot f'c, 0ksi) \cdot A_i$

$Ms_i := Fs_i \cdot \left(\frac{h}{2} - d_i\right)$

$d_i =$

0.245
0.75
1.255

ft

$\varepsilon s_i =$

0
-0.006
-0.012

$fs_i =$

1.664
-40
-40

$\cdot ksi$

$A_i =$

1.8
1.2
1.8

$\cdot in^2$

$Fs_i =$

3.032
-48
-72

$\cdot kips$

$Ms_i =$

18.371
0
436.32

$\cdot kips \cdot in$

$Mst := \sum_{i=1}^{row} Ms_i$

$Mst = 455 \cdot kips \cdot in$

$Mc := 0.85 \cdot f'c \cdot a \cdot w \cdot \left(\frac{h}{2} - \frac{a}{2}\right)$

$Mc = 904 \cdot kips \cdot in$

$Mn_7 := Mc + Mst$

$Mn_7 = 1358 \cdot kips \cdot in$

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$j := 0..7$ $\phi_j := 1$

Results

$\phi_j \cdot Pn_j =$		$\phi_j \cdot Mn_j =$	
804.768	·kip	0	·kips·in
804.768		1701.896	
708.099		1874.46	
608.18		2235.234	
533.757		2440.098	
411.24		2702.552	
97.2		1937.132	
0		1358.37	

Criteria:

Depth	$h = 18 \cdot in$	bar = 7
Width	$w = 18 \cdot in$	tie = 2
Eff. Width	$b = 18 \cdot in$	$fy = 40 \cdot ksi$

$cover = 2.25 \cdot in$ $f'c = 3000 \cdot psi$

$Ag = 324 \cdot in^2$ $Ast = 4.8 \cdot in^2$

$\frac{Ast}{w \cdot h} = 0.0148$ Actual ratio ρ

$\frac{Ast}{Ag} = 0.0148$ Eff. ratio ρ

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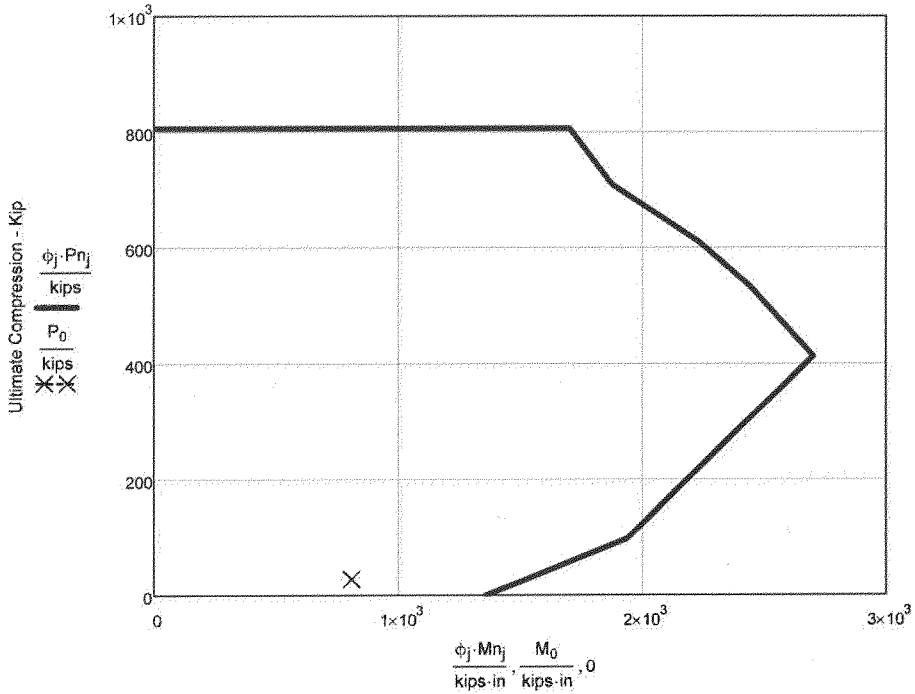
Demand Loads Water to Top of Wall:

Axial Load from CPGA output: $P_0 := 28.2kips$ Moment from CPGA output: $M_0 := 807.7kips \cdot in$

$P_{0GT} := 61.9kips$ $V_{max} := 16.3kips$



Interaction Diagram

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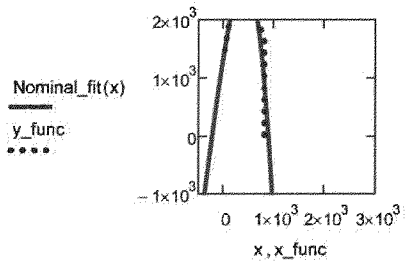
Nominal Curve Used to Find Factor of Safety:

Data Points set to function: $y\_func := \frac{Mn}{kips \cdot in}$ $x\_func := \frac{Pn}{kips}$

Degree of Polynominal Fit: $k := 3$

Returns a vector which interp uses to find the kth order polynomial that best fits the x and y data values

Nominal := regress(x_func, y_func, k) Nominal_fit(x) := interp(Nominal, x_func, y_func, x)



Allowable Axial Capacity (geotech):

Allowable Axial Compression Load for 33ft pile (To be calculated by Geotechs)

$P_c := 137.963kips$

Allowable Axial Tension Load for 339ft pile (To be calculated by Geotechs)

$P_t := 73.264kips$

Compression Pile Geotech FS:

$FS_{GT} := \frac{P_c}{P_{0GT}} = 2.229$

FS_{GT} Must be > 1.7 to meet Feasibility Criteria without necessitating R&R analysis.

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Strength Factor of Safety:

Read the Corresponding Moment for the demand axial load ($P_0 = 28.2 \cdot kips$)

$M_{capacity} := Nominal\_fit\left(\frac{P_0}{kips}\right)$ $\quad M_{capacity} = 1542.921$

Demand Moment: $M_{demand} := \frac{M_0}{kips \cdot in}$ $\quad M_{demand} = 807.7$

Factor of Safety: $FS_{strength} := \frac{M_{capacity}}{M_{demand}}$ $\quad FS_{strength} = 1.9$

Check Shear through pile (conservatively ignoring shear reinforcing):

d for pile: $d_{pile} := 14.8in$

$V_{Pile} := 2 \cdot \sqrt{f'c \cdot psi} \cdot w \cdot d_{pile} = 29.183 \cdot kips$

$FS_{shear} := \frac{V_{Pile}}{V_{max}} = 1.79$

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FEMA 310 GUIDANCE

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make

2011 REVISION

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US Army Corps of Engineers.

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Marx 9/28/11

Water EL: TOW - 1'

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top\_ftg} := 746.9ft$

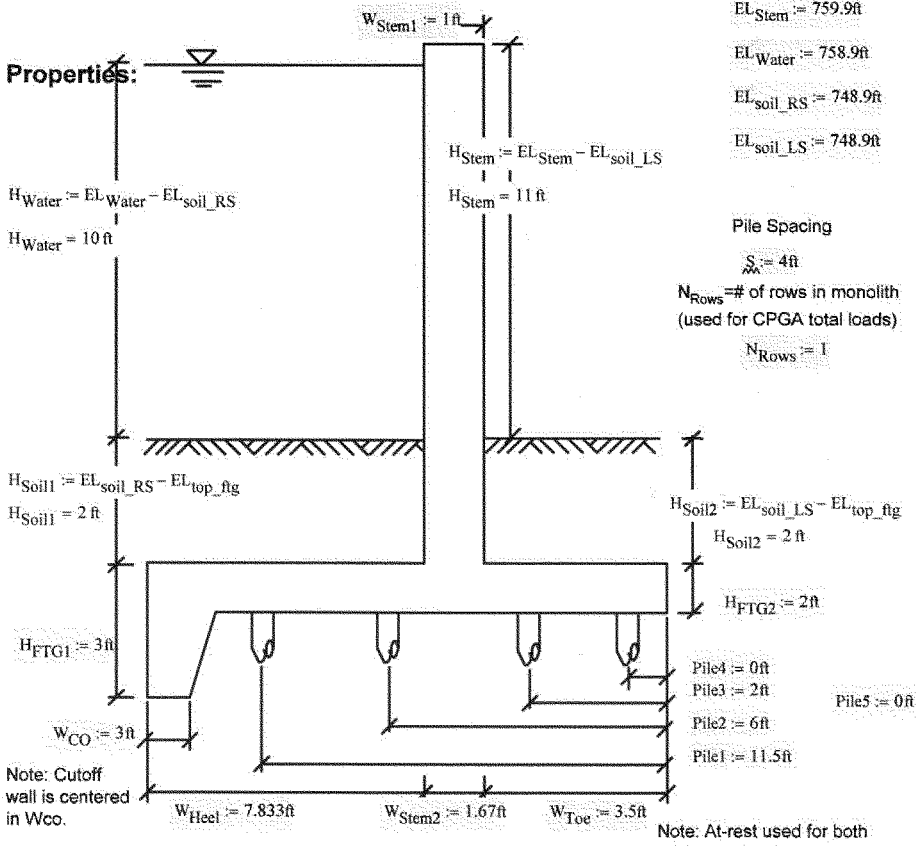
$EL_{Stem} := 759.9ft$

$EL_{Water} := 758.9ft$

$EL_{soil\_RS} := 748.9ft$

$EL_{soil\_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

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Lengths:

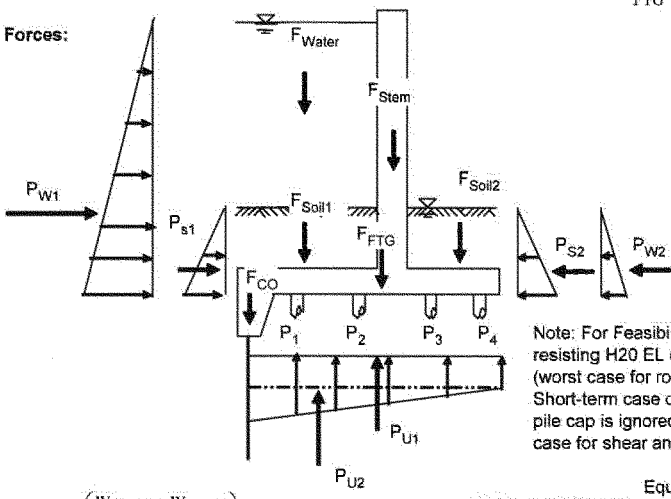
$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

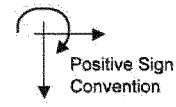
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$ $\quad L_{FTG} = 13\,ft$

$L_{W1} = 15\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 6.825 \frac{kip}{ft}$ Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*10²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$

$F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 5865.35 \cdot plf$

$F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 7020 \cdot plf$

$P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 561.6 \cdot psf$

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1})\gamma_w\right] + L_{PU1} \cdot \gamma_w = 592.8 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 338.15 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -577.2 \cdot plf$

$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3889.75 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1464.62 \cdot plf$

$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1404 \cdot plf$

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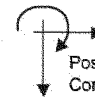
Inputs for CPGA

Assumes perfectly rigid pile cap.

Sum moments around bottom corner of Toe:



Moment Arm (from bottom corner of Toe)

$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$

$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$

$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$

$Arm_{closure} := 4 \cdot ft + \frac{H_{Water}}{3} = 7.33\,ft$

$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$

$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$

$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$

$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$

$Arm_{W1} := \frac{1}{3} \cdot L_{W1} - L_{CO} = 4\,ft$

$Arm_{W2a} := 0.333 \cdot \left(L_{PU1}\right) = 1.33\,ft$

$Arm_{W2b} := L_{CO} - \frac{L_{CO} \cdot \left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$

$Arm_{S1} := \frac{1}{3} \cdot \left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$

$Arm_{S2} := Arm_{W2a} = 1.33\,ft$

$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$

$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$

$Arm_{U3} := L_{FTG} - 0.25 \cdot W_{CO} = 12.25\,ft$

Moment (about bottom corner of toe)

$M_{Stem} := -F_{Stem} \cdot Arm_{Stem} = -11.29 \cdot \frac{kip \cdot ft}{ft}$

$M_{FTG} := -F_{FTG} \cdot Arm_{FTG} = -25.36 \cdot \frac{kip \cdot ft}{ft}$

$M_{CO} := -F_{CO} \cdot Arm_{CO} = -5.18 \cdot \frac{kip \cdot ft}{ft}$

$M_{closure} := F_{closure} \cdot Arm_{closure} = 50.05 \cdot \frac{kip \cdot ft}{ft}$

$M_{Soil1} := -F_{Soil1} \cdot Arm_{Soil1} = -7.49 \cdot \frac{kip \cdot ft}{ft}$

$M_{Soil2} := -F_{Soil2} \cdot Arm_{Soil2} = -0.64 \cdot \frac{kip \cdot ft}{ft}$

$M_{Water1} := -F_{Water1} \cdot Arm_{Water1} = -53.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{Water2} := -F_{Water2} \cdot Arm_{Water2} = -0.76 \cdot \frac{kip \cdot ft}{ft}$

$M_{W1} := P_{W1} \cdot Arm_{W1} = 28.08 \cdot \frac{kip \cdot ft}{ft}$

$M_{W2a} := P_{W2a} \cdot Arm_{W2a} = -0.66 \cdot \frac{kip \cdot ft}{ft}$

$M_{W2b} := -P_{W2b} \cdot Arm_{W2b} = 0.29 \cdot \frac{kip \cdot ft}{ft}$

$M_{S1} := P_{S1} \cdot Arm_{S1} = 0.35 \cdot \frac{kip \cdot ft}{ft}$

$M_{S2} := P_{S2} \cdot Arm_{S2} = -0.35 \cdot \frac{kip \cdot ft}{ft}$

$M_{U1} := -P_{U1} \cdot Arm_{U1} = 22.37 \cdot \frac{kip \cdot ft}{ft}$

$M_{U2} := -P_{U2} \cdot Arm_{U2} = 11.23 \cdot \frac{kip \cdot ft}{ft}$

$M_{U3} := -P_{U3} \cdot Arm_{U3} = 17.2 \cdot \frac{kip \cdot ft}{ft}$

$$\Sigma M := \begin{pmatrix} M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots \\ + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3} \end{pmatrix}$$

$$\Sigma Vertical := \begin{pmatrix} F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots \\ + P_{U1} + P_{U2} + P_{U3} \end{pmatrix}$$

$$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$$

$\Sigma M = 24.55 \cdot \frac{ft \cdot kip}{ft}$

$\Sigma Vertical = 7.69 \cdot \frac{kip}{ft}$

$\Sigma Horizontal = 12.77 \cdot \frac{kip}{ft}$

Find Pile Loads Using CPGA:

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

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Total number of piles:

Number = 3

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137.963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73.264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in^3 (per Geotech)

Concrete Sheet Pile Properties

Dimension of rectangular pile $D_{sp} := 10in$ $W_{sp} := 12in$

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Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{(W_{sp} \cdot D_{sp}^3)}{12}$ $I_{1sp} = 1000 \cdot in^4$ $I_{2sp} := \frac{D_{sp} \cdot W_{sp}^3}{12}$ $I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20.73klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in³ (per Geotech)

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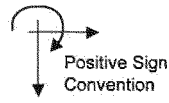
$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

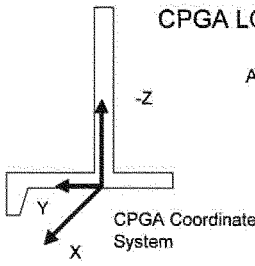


$\Sigma M_T = 98.19 \cdot ft \cdot kip$

$\Sigma Vertical_T = 30.76 \cdot kip$

$\Sigma Horizontal_T = 51.07 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$

$PZ := \Sigma Vertical_T$

$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -51.07 \cdot kip$

$PZ = 30.76 \cdot kip$

$MX = -98.19 \cdot ft \cdot kip$

Loads input into CPGA filename: M145RL1.dat

Shear. Due to the continuous construction of the sheet pile, b = 12". 106/437

$Sht_{Hormax} = 5.9 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $b_{ev} := 12in$

$V_{ex} := \phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if(V_c > 1.5 \cdot Sht_{Hormax}, "OKAY", "NO GOOD")$ $FS_{13} := \frac{V_c}{Sht_{Hormax}} = 1.87$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.37$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.31$
Stem Factor of Safety:	$FS_S = 1.83$
Toe Factor of Safety:	$FS_T = 2.11$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 7.35$
Punching Shear Factor of Safety:	$FS_9 = 3.33$
Bearing Factor of Safety:	$FS_B = 15.83$
Pile Cap Shear Factor of Safety:	$FS_{13} = 1.87$

$FoS := \min(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13})$

$FoS = 1.37$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

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M145RL1.dat

2011 REVISION

Chkd: Marx 9/20/11

```
10 CID-MO M145 GEOTECH REV STRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-1ft-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -51.07 30.76 -98.19 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145RL1.txt
```

Page 1

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M145RL1.TXT

```
**********************************
* CASE PROGRAM   #  X0080        *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29    *  RUN DATE 06-SEP-2011   RUN TIME 14.38.26
**********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW-1FT
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                        X          Y          Z
                                      -----      -----      -----
WITH DIAGONAL COORDINATES = (       -1.50 ,     2.00 ,      .00 )
                            (        1.50 ,    11.50 ,      .00 )

*******************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6

*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01     L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01     L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

Page 1

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3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-51.1	30.8	-98.2	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.8757E-01	.1296E-01	-.1138E-03	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

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M145RL1.TXT

# INDICATES CBF BASED ON MOMENTS DUE TO (F3*EMIN) FOR CONCRETE PILES

B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-13.7	52.2	681.8	.0	.0	.64	1.26	.00	.00*
2	.0	-13.7	24.3	681.8	.0	.0	.30	1.40	.00	.00*
3	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
4	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
5	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*
6	.0	-5.9	-11.5	193.6	.0	.0	1.39	3.23	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-13.7	52.2	681.8	.0	.0
2	.0	-13.7	24.3	681.8	.0	.0
3	.0	-5.9	-11.5	193.6	.0	.0
4	.0	-5.9	-11.5	193.6	.0	.0
5	.0	-5.9	-11.5	193.6	.0	.0
6	.0	-5.9	-11.5	193.6	.0	.0

2011 REVISION

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US Army Corps of Engineers.

CID-MO Floodwall
Floodwall Analysis - Existing, M145
4 foot Stop Log Abutment Monolith
Kansas City Levees

Comp by: Sheffield
Chkd by: Marx 9/28/11

Water EL: TOW - 2'

Assumptions: For Feasibility Analysis, it is assumed that the sheetpile cutoff is adequately strength designed for a 50% head differential, the pile cap under the closure is adequately designed, and that the floodwall sections of the monolith are accounted for by analysis of adjacent floodwall monoliths. Soil Surfaces assumed horizontal.

Variables

$kip := 1000lb$

$psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$ $plf := \frac{lb}{ft}$

$ksf \equiv \frac{1000lb}{ft^2}$ $psf \equiv \frac{lb}{ft^2}$

$EL_{top\_ftg} := 746.9ft$

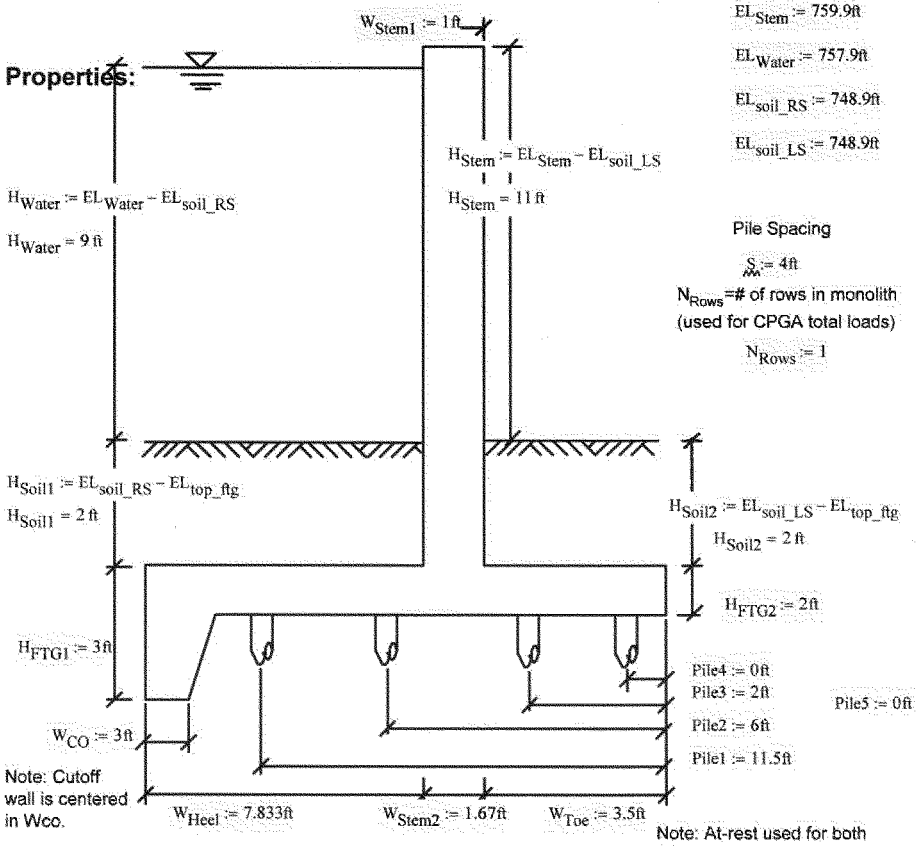
$EL_{Stem} := 759.9ft$

$EL_{Water} := 757.9ft$

$EL_{soil\_RS} := 748.9ft$

$EL_{soil\_LS} := 748.9ft$

Properties:



Pile Spacing

$S := 4ft$

N_{Rows}=# of rows in monolith (used for CPGA total loads)

$N_{Rows} := 1$

Note: Cutoff wall is centered in Wco.

Note: At-rest used for both driving and resisting for Feasibility and to require piles to fully resist stability.

Concrete Properties $\gamma_c := 150pcf$

Water Weight $\gamma_w := 62.4pcf$

Soil Properties $\gamma_{sat} := 115pcf$ $\phi := 22deg$

$K_o := 1 - \sin(\phi)$ $K_o = 0.63$

Lengths: 112/457

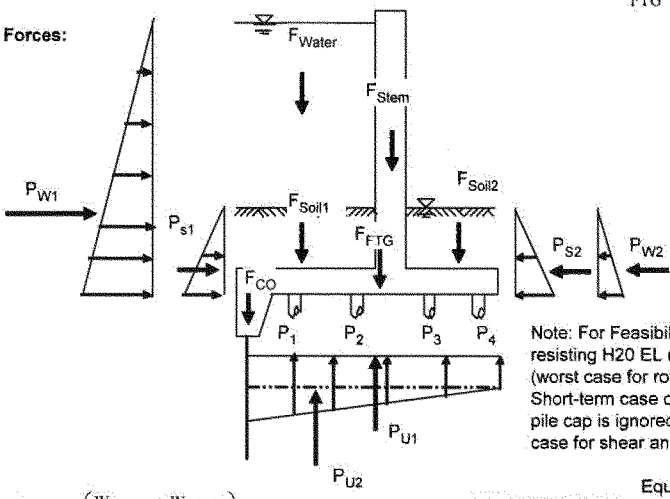
$L_{CO} := H_{FTG1} - H_{FTG2}$ $\quad L_{W1} := H_{Water} + H_{Soil1} + H_{FTG1}$ $\quad L_{STM} := H_{Stem} + H_{Soil2}$

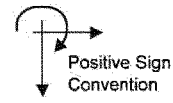
$L_{FTG} := W_{Heel} + W_{Stem2} + W_{Toe}$ $\quad L_{PU1} := H_{Soil2} + H_{FTG2}$ $\quad L_{FTG} = 13\,ft$

$L_{W1} = 14\,ft$

$L_{PU1} = 4\,ft$

Forces:





Note: For Feasibility, only long-term case of resisting H20 EL (top of soil) is considered (worst case for rotation and pile pull-out). Short-term case of little or no resisting H20 on pile cap is ignored, though this is the worst case for shear and sliding.

$F_{Stem} := \left(\frac{W_{Stem1} + W_{Stem2}}{2}\right) \cdot L_{STM} \cdot \gamma_c = 2603.25 \cdot plf$

$F_{closure} := 5.53 \frac{kip}{ft}$

Equivalent Point Load acting on abutment from Stop Logs per tributary width of pile cap.

$F_{closure}$ = 1/2*.0624*10²*17.5/2*(1/4ft)

$F_{FTG} := H_{FTG2} \cdot L_{FTG} \cdot \gamma_c = 3900.9 \cdot plf$

$F_{CO} := L_{CO} \cdot W_{CO} \cdot \gamma_c = 450 \cdot plf$ (Treated Rectangular)

$F_{Soil1} := H_{Soil1} \cdot W_{Heel} \cdot (\gamma_{sat} - \gamma_w) = 824.03 \cdot plf$ $\quad F_{Soil2} := H_{Soil2} \cdot W_{Toe} \cdot (\gamma_{sat} - \gamma_w) = 368.2 \cdot plf$

$F_{Water1} := (H_{Water} + H_{Soil1}) \cdot W_{Heel} \cdot \gamma_w = 5376.57 \cdot plf$ $\quad F_{Water2} := H_{Soil2} \cdot W_{Toe} \cdot \gamma_w = 436.8 \cdot plf$

$P_{W1} := \frac{1}{2} \cdot L_{W1}^2 \cdot \gamma_w = 6115.2 \cdot plf$ $\quad P_{W2a} := -\left[\frac{1}{2} \cdot (L_{PU1})^2 \cdot \gamma_w\right] = -499.2 \cdot plf$

$q_{uCO} := (1 - 0.5) \cdot \gamma_w \left[(H_{Water} + H_{Soil1} + H_{FTG2}) - L_{PU1}\right] + L_{PU1} \cdot \gamma_w = 530.4 \cdot psf$

Cutoff wall assumed to be 50% effective. See Figure C-3 of EM 1110-2-2100

$q_{umax} := \left[(1 - 0.5)(L_{W1} - L_{PU1})\gamma_w\right] + L_{PU1} \cdot \gamma_w = 561.6 \cdot psf$

$q_{utoe} := L_{PU1} \cdot \gamma_w + \frac{L_{PU1}}{L_{FTG} - .5W_{CO} + L_{PU1}} (q_{umax} - L_{PU1} \cdot \gamma_w) = 330.1 \cdot psf$

Note: Soil below base of pile cap acting on key is essentially equal and opposite since all resistance is by piles.

$P_{W2b} := -\left[\frac{1}{2} \cdot (q_{uCO} + q_{umax}) \cdot L_{CO}\right] = -546 \cdot plf$

$P_{S1} := K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil1} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = 263.17 \cdot plf$ $\quad P_{S2} := -K_o \cdot \left[\frac{1}{2} \cdot (H_{Soil2} + H_{FTG2})^2 \cdot (\gamma_{sat} - \gamma_w)\right] = -263.17 \cdot plf$

$P_{U1} := -(L_{FTG} - .5W_{CO}) \cdot (q_{utoe}) = -3797.15 \cdot plf$ $\quad P_{U2} := -0.5(L_{FTG} - .5 \cdot W_{CO}) \cdot (q_{umax} - q_{utoe}) = -1331.47 \cdot plf$

$P_{U3} := -.5W_{CO} \cdot (L_{W1} \cdot \gamma_w) = -1310.4 \cdot plf$

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Inputs for CPGA

Assumes perfectly rigid pile cap.

Positive Sign Convention

Sum moments around bottom corner of Toe:

Moment Arm (from bottom corner of Toe)

$Arm_{Stem} := \left(W_{Toe} + \frac{W_{Stem2}}{2}\right) = 4.34\,ft$

$Arm_{FTG} := \frac{L_{FTG}}{2} = 6.5\,ft$

$Arm_{CO} := L_{FTG} - \frac{W_{CO}}{2} = 11.5\,ft$

$Arm_{closure} := 4 \cdot ft + \frac{H_{Water}}{3} = 7\,ft$

$Arm_{Soil1} := \left(W_{Toe} + W_{Stem2} + 0.5W_{Heel}\right) = 9.09\,ft$

$Arm_{Soil2} := 0.5W_{Toe} = 1.75\,ft$

$Arm_{Water1} := Arm_{Soil1} = 9.09\,ft$

$Arm_{Water2} := Arm_{Soil2} = 1.75\,ft$

$Arm_{W1} := \frac{1}{3} \cdot L_{W1} - L_{CO} = 3.67\,ft$

$Arm_{W2a} := 0.333 \cdot \left(L_{PU1}\right) = 1.33\,ft$

$Arm_{W2b} := L_{CO} - \frac{L_{CO} \cdot \left(2q_{uCO} + q_{umax}\right)}{3\left(q_{uCO} + q_{umax}\right)} = 0.5\,ft$

$Arm_{S1} := \frac{1}{3} \cdot \left(H_{Soil1} + H_{FTG2}\right) = 1.33\,ft$

$Arm_{S2} := Arm_{W2a} = 1.33\,ft$

$Arm_{U1} := \frac{L_{FTG} - 0.5W_{CO}}{2} = 5.75\,ft$

$Arm_{U2} := \frac{2\left(L_{FTG} - 0.5W_{CO}\right)}{3} = 7.67\,ft$

$Arm_{U3} := L_{FTG} - 0.25 \cdot W_{CO} = 12.25\,ft$

Moment (about bottom corner of toe)

$M_{Stem} := -F_{Stem} \cdot Arm_{Stem} = -11.29 \cdot \frac{kip \cdot ft}{ft}$

$M_{FTG} := -F_{FTG} \cdot Arm_{FTG} = -25.36 \cdot \frac{kip \cdot ft}{ft}$

$M_{CO} := -F_{CO} \cdot Arm_{CO} = -5.18 \cdot \frac{kip \cdot ft}{ft}$

$M_{closure} := F_{closure} \cdot Arm_{closure} = 38.71 \cdot \frac{kip \cdot ft}{ft}$

$M_{Soil1} := -F_{Soil1} \cdot Arm_{Soil1} = -7.49 \cdot \frac{kip \cdot ft}{ft}$

$M_{Soil2} := -F_{Soil2} \cdot Arm_{Soil2} = -0.64 \cdot \frac{kip \cdot ft}{ft}$

$M_{Water1} := -F_{Water1} \cdot Arm_{Water1} = -48.85 \cdot \frac{kip \cdot ft}{ft}$

$M_{Water2} := -F_{Water2} \cdot Arm_{Water2} = -0.76 \cdot \frac{kip \cdot ft}{ft}$

$M_{W1} := P_{W1} \cdot Arm_{W1} = 22.42 \cdot \frac{kip \cdot ft}{ft}$

$M_{W2a} := P_{W2a} \cdot Arm_{W2a} = -0.66 \cdot \frac{kip \cdot ft}{ft}$

$M_{W2b} := -P_{W2b} \cdot Arm_{W2b} = 0.28 \cdot \frac{kip \cdot ft}{ft}$

$M_{S1} := P_{S1} \cdot Arm_{S1} = 0.35 \cdot \frac{kip \cdot ft}{ft}$

$M_{S2} := P_{S2} \cdot Arm_{S2} = -0.35 \cdot \frac{kip \cdot ft}{ft}$

$M_{U1} := -P_{U1} \cdot Arm_{U1} = 21.84 \cdot \frac{kip \cdot ft}{ft}$

$M_{U2} := -P_{U2} \cdot Arm_{U2} = 10.21 \cdot \frac{kip \cdot ft}{ft}$

$M_{U3} := -P_{U3} \cdot Arm_{U3} = 16.06 \cdot \frac{kip \cdot ft}{ft}$

$\Sigma M := \left(M_{Stem} + M_{FTG} + M_{CO} + M_{closure} + M_{Soil1} + M_{Soil2} + M_{Water1} + M_{Water2} + M_{W1} + M_{W2a} \cdots + M_{W2b} + M_{S1} + M_{S2} + M_{U1} + M_{U2} + M_{U3}\right)$

$\Sigma M = 9.28 \cdot \frac{ft \cdot kip}{ft}$

$\Sigma Vertical := \left(F_{Stem} + F_{FTG} + F_{CO} + F_{Soil1} + F_{Soil2} + F_{Water1} + F_{Water2} \cdots + P_{U1} + P_{U2} + P_{U3}\right)$

$\Sigma Vertical = 7.52 \cdot \frac{kip}{ft}$

$\Sigma Horizontal := \left(F_{closure} + P_{W1} + P_{W2a} + P_{W2b} + P_{S1} + P_{S2}\right)$

$\Sigma Horizontal = 10.6 \cdot \frac{kip}{ft}$

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Find Pile Loads Using CPGA:

Number := if(Pile5 = 0, 4, 5)
Number := if(Pile4 = 0, 3, Number)
Number := if(Pile3 = 0, 2, Number)
Number := if(Pile2 = 0, 1, Number)
Number := if(Pile1 = 0, 0, Number)

Total number of piles:

Number = 3

Concrete Pile Properties

Dimension of square pile $D := 18in$

Assume all piles to pile cap connections are fixed (details show rebar and pile extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $E_C = 3122.02 \cdot ksi$

$I := \frac{D^4}{12}$ $I = 8748 \cdot in^4$

C33=2.0 for full skin friction
B66 = 0 torsional stiffness
Length = 34ft

$Area := D^2$ $Area = 324 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):
AC = Allowable Pile Compression Load, = 137.963/1.7 = 81.1 k
AT = Allowable Pile Tension Load, = 73.264/1.7 '= 43.1 k

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):
PO = Axial Compression Design Load Strength, 536 k (P_{n0} in Mathcad)
PT = Axial Tension Design Load Strength, Fy*As= 128 k
PB = Axial Design Load Strength at Balanced Condition, 274 k (P_{n5} in Mathcad)
MB = Design Moment Strength at Balanced Condition, 1802 k-in (M_{n5} in Mathcad)
MO = Design Moment Strength Under Pure Flexure, 905 k-in (M_{n7} in Mathcad)

Soil:
NH
Soil Modulus, 0.05 k/in³ (per Geotech)

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Concrete Sheet Pile Properties

Dimension of rectangular pile $D_{sp} := 10in$ $W_{sp} := 12in$

Pile Spacing, S=1ft

Assume all sheet pile to pile cap connections are fixed (details show rebar extending into base).

Pile Properties:

$fc := 3000 \cdot psi$

$I_{1sp} := \frac{\left(W_{sp} \cdot D_{sp}^{3}\right)}{12}$ $I_{1sp} = 1000 \cdot in^4$ $I_{2sp} := \frac{D_{sp} \cdot W_{sp}^{3}}{12}$ $I_{2sp} = 1440 \cdot in^4$

C33=2.0 for full skin friction

B66 = 0 torsional stiffness

Length = 15ft

$E_C := 57000 \cdot \sqrt{fc \cdot psi}$ $Area := D_{sp} \cdot W_{sp}$

$E_C = 3122.02 \cdot ksi$ $Area = 120 \cdot in^2$

Allowable Design Load Strengths from Geotechs (FS = 1.7 for extreme):

AC = Allowable Pile Compression Load, = 5'/4'*20.73klf/1.7 = 15.2 k

AT = Allowable Pile Tension Load, = 5'/4'*11.231klf/1.7 = 8.26 k

Assumes two 2.5ft wide sheetpiles engaged; strength is equally divided among four 1' wide piles.

The following allowable design strengths are from the existing pile capacity MathCAD file (FS = 1.5):

PO = Axial Compression Design Load Strength, 205 k (P_{n0} in Mathcad)

PT = Axial Tension Design Load Strength, Fy*As= 56.3 k

PB = Axial Design Load Strength at Balanced Condition, 72 k (P_{n5} in Mathcad)

MB = Design Moment Strength at Balanced Condition, 333 k-in (M_{n5} in Mathcad)

MO = Design Moment Strength Under Pure Flexure, 195 k-in (M_{n7} in Mathcad)

Soil:

NH

Soil Modulus, 0.05 k/in^3 (per Geotech)

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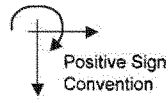
$\Sigma M_T := \Sigma M \cdot S \cdot N_{Rows}$

$\Sigma Vertical_T := \Sigma Vertical \cdot S \cdot N_{Rows}$

$\Sigma Horizontal_T := \Sigma Horizontal \cdot S \cdot N_{Rows}$

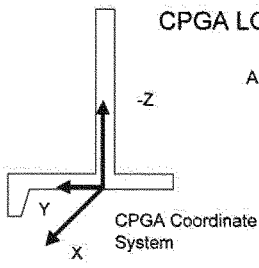


$\Sigma M_T = 37.1 \cdot ft \cdot kip$

$\Sigma Vertical_T = 30.08 \cdot kip$

$\Sigma Horizontal_T = 42.4 \cdot kip$

CPGA LOADING FOR WATER AT TOP OF WALL



Applied Loads

$PY := -\Sigma Horizontal_T$

$PZ := \Sigma Vertical_T$

$MX := -\Sigma M_T$

- Signs added to agree with CPGA sign convention.

$PY = -42.4 \cdot kip$

$PZ = 30.08 \cdot kip$

$MX = -37.1 \cdot ft \cdot kip$

Loads input into CPGA filename: M145RL2.dat

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Shear. Due to the continuous construction of the sheet pile, b = 12".

$Sht_{Hormax} = 4.9 \cdot kip$

For 1 ft wide pile.

$d_f := 18in - \frac{D_{sp}}{2} - 2.5in - .5in - .5 \cdot \frac{7}{8}in = 9.56 \cdot in$ $b_{av} := 12in$

$V_c := \phi_V \cdot 2 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d = 11.01 \cdot kip$

$Check13 := if(V_c > 1.5 \cdot Sht_{Hormax}, "OKAY", "NO GOOD")$ $FS_{13} := \frac{V_c}{Sht_{Hormax}} = 2.25$ Check13 = "OKAY"

Overall Factor of Safety

Factor of Safety

Heel Factor of Saftey:	$FS_H = 1.78$
Longitudinal Flexure of Pile Cap:	$FS_{LF} = 2.39$
Stem Factor of Safety:	$FS_S = 2.19$
Toe Factor of Safety:	$FS_T = 2.53$
Tension Pile Embedment:	Check7 = "OKAY"
Tension Pile Factor of Safety:	$FS_8 = 9.71$
Punching Shear Factor of Safety:	$FS_9 = 3.96$
Bearing Factor of Safety:	$FS_B = 18.86$
Pile Cap Shear Factor of Safety:	$FS_{13} = 2.25$

$FoS := \min(FS_H, FS_{LF}, FS_S, FS_T, FS_8, FS_9, FS_B, FS_{13})$

$FoS = 1.78$

It has been decided that a Factor of Safety of 1.5 or greater for existing structures will be acceptable when using unfactored loads and unreduced strengths for analysis.

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2011 REVISION

Chkd: Marx 9/20/11

M145RL2.dat

```
10 CID-MO M145 GEOTECH REV sTRENGTH 4ft abutment-
20 SEP 2011 Existing conditions h20 TOW-2ft-
30 GT FS is 1.7 and Structural FS is 1.5
40 LOAD 1 0 -42.4 30.08 -37.1 0 0
51 PILE 1 0 2 0
52 PILE 2 0 6 0
53 PILE 3 -1.5 11.5 0
54 PILE 4 -.5 11.5 0
55 PILE 5 .5 11.5 0
56 PILE 6 1.5 11.5 0
110 SOIL  NH 0.05 LEN 33  0  1 to 2
111 PROP 3122 8748 8748 324 2 0 1 to 2
112 DLS S 81.1 43.1 536 128 274 1802 905 H 18 1 TO 2
120 SOIL  NH 0.05 LEN 15  0  3 TO 6
121 PROP 3122 1000 1440 120 2 0 3 to 6
121 DLS S 15.2 8.26 205 56.3 72 333 195 H 10 3 TO 6
140 FIX 1 TO 6
150 PFO 1 TO 6
500 FOUT 1 2 3 4 5 6 7 M145RL2.txt
```

M145RL2.TXT 119/437

```
*********************************
* CASE PROGRAM   #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
* VERSION NUMBER # 1993/03/29   *  RUN DATE 06-SEP-2011   RUN TIME 15.08.53
*********************************


CID-MO M145 GEOTECH REV STRENGTH 4FT ABUTMENT
SEP 2011 EXISTING CONDITIONS H20 TOW-2FT
GT FS IS 1.7 AND STRUCTURAL FS IS 1.5


THERE ARE     6 PILES AND
              1 LOAD CASES IN THIS RUN.

ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                        X          Y          Z
                                      -----      -----      -----
WITH DIAGONAL COORDINATES = (        -1.50 ,     2.00 ,      .00 )
                            (         1.50 ,    11.50 ,      .00 )

*******************************************************************************

         PILE PROPERTIES AS INPUT


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .87480E+04   .87480E+04   .32400E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   1    2


      E           I1           I2           A           C33          B66
     KSI         IN**4        IN**4        IN**2
  .31220E+04   .10000E+04   .14400E+04   .12000E+03   .20000E+01   .00000E+00

THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

   3    4    5    6

*******************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .33000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2


   NH      ESOIL       LENGTH        L            LU
          K/IN**3                   FT           FT
          .50000E-01    L        .15000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -
```

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3 4 5 6

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
1	.00	2.00	.00	V	.00	33.00	F
2	.00	6.00	.00	V	.00	33.00	F
3	-1.50	11.50	.00	V	.00	15.00	F
4	-.50	11.50	.00	V	.00	15.00	F
5	.50	11.50	.00	V	.00	15.00	F
6	1.50	11.50	.00	V	.00	15.00	F

						126.00	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	.0	-42.4	30.1	-37.1	.0	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.66258E+03	.00000E+00	.00000E+00	.00000E+00	.29315E+05	-.61195E+05
.00000E+00	.61826E+03	.00000E+00	-.26988E+05	.00000E+00	.00000E+00
.00000E+00	.00000E+00	.26868E+05	.27882E+07	.00000E+00	.00000E+00
.00000E+00	-.26988E+05	.27882E+07	.34850E+09	.00000E+00	.00000E+00
.29315E+05	.00000E+00	.00000E+00	.00000E+00	.51547E+07	-.24726E+07
-.61195E+05	.00000E+00	.00000E+00	.00000E+00	-.24726E+07	.72376E+07

LOAD CASE 1. NUMBER OF FAILURES = 6. NUMBER OF PILES IN TENSION = 4.

****

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DY IN	DZ IN	RX RAD	RY RAD	RZ RAD
1	.0000E+00	-.7266E-01	.1082E-01	-.9344E-04	.0000E+00	.0000E+00

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE

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M145RL2.TXT

# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF		
1	.0	-11.4	43.8	566.4	.0	.0	.54	1.13	.00	.00*
2	.0	-11.4	20.9	566.4	.0	.0	.26	1.24	.00	.00*
3	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
4	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
5	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*
6	.0	-4.9	-8.7	160.7	.0	.0	1.05	2.64	.00	.00*

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	.0	-11.4	43.8	566.4	.0	.0
2	.0	-11.4	20.9	566.4	.0	.0
3	.0	-4.9	-8.7	160.7	.0	.0
4	.0	-4.9	-8.7	160.7	.0	.0
5	.0	-4.9	-8.7	160.7	.0	.0
6	.0	-4.9	-8.7	160.7	.0	.0

US ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT

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US Army Corps of Engineers

Probability of Failure
CID MO Monolith 145
Pullout of Sheetpile: H2O to TOW

Comp by:FRS
Chkd by: Marx 9/28/11

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996

III. Situation

1. CID MO M145 did not meet the 1.7 geotech factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad and CPGA analyses of a 4' wide pile cap for existing condition.
2. Material Properties used:

Mean (Ultimate) Pullout, $A_T := 11.231 \cdot kip \cdot \frac{5}{4} = 14.039 \cdot kip$ for 12" of pile length

(assuming minimum of two 2.5' long sheet piles engaged for pullout - strength is equally divided among four 1' wide piles).

4. From *Geotechnical Data Supplied by Glenn Bellew*, the amount of variation anticipated in one standard deviation for pullout is 18%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

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V. Caclulating Factors of Safety

Tension Load on Sheetpile (from M145R.dat CPGA file): $P_u := 14.7 \cdot kip$

Mean Sheetpile Pullout:

$\phi := 1.0$ For Risk and Reliability

$FS_D := \frac{\phi \cdot A_T}{P_u}$ $FS_D = 0.955$

Upper Sheetpile Pullout

$A_{TU} := A_T + A_T \cdot 0.18$ $A_{TU} = 16.566 \cdot kip$

$FS_{ATu} := \frac{\phi \cdot A_{TU}}{P_u}$ $FS_{ATu} = 1.127$

Lower Sheetpile Pullout

$A_{TL} := A_T - A_T \cdot 0.18$ $A_{TL} = 11.512 \cdot kip$

$FS_{ATl} := \frac{\phi \cdot A_{TL}}{P_u}$ $FS_{ATl} = 0.783$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATu} - FS_{ATl}$ $\Delta F_{fc} = 0.344$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.172$

$V_F := \frac{\sigma_F}{FS_D}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = -0.347$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 36.43 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 63.57 \cdot \%$

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POF For Water 1 ft Below TOW:

Tension Load on Sheetpile (see M145RL1.dat CPGA file). $P_{u1} := 11.5kip$

Ultimate Pullout at Water EL 759ft: $A_{T1} := A_T = 14.039 \cdot kip$

Mean Concrete Strength

$$FS_{D1} := \frac{\phi \cdot A_{T1}}{P_{u1}} \qquad FS_{D1} = 1.221$$

Upper Sheetpile Pullout

$$A_{TU1} := A_{T1} + A_{T1} \cdot 0.18 \qquad A_{TU1} = 16.566 \cdot kip$$

$$FS_{ATu1} := \frac{\phi \cdot A_{TU1}}{P_{u1}} \qquad FS_{ATu1} = 1.44$$

Lower Sheetpile Pullout

$$A_{TL1} := A_{T1} - A_{T1} \cdot 0.18 \qquad A_{TL1} = 11.512 \cdot kip$$

$$FS_{ATl1} := \frac{\phi \cdot A_{TL1}}{P_{u1}} \qquad FS_{ATl1} = 1.001$$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATul} - FS_{ATll}$ $\Delta F_{fc} = 0.439$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.22$

$V_F := \frac{\sigma_F}{FS_{D1}}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_{D1}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = 1.028$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 84.8 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 15.2 \cdot \%$

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POF For Water 2 ft Below TOW:

Tension Load on Sheetpile (see M145L2.dat CPGA file). $P_{u2} := 8.7kip$

Ultimate Pullout at Water EL 758ft: $A_{T2} := A_T = 14.039\,kip$

Mean Concrete Strength

$$FS_{D2} := \frac{\phi \cdot A_{T2}}{P_{u2}} \qquad FS_{D2} = 1.614$$

Upper Sheetpile Pullout

$$A_{TU2} := A_{T2} + A_{T2} \cdot 0.18 \qquad A_{TU2} = 16.566 \cdot kip$$

$$FS_{ATu2} := \frac{\phi \cdot A_{TU2}}{P_{u2}} \qquad FS_{ATu2} = 1.904$$

Lower Sheetpile Pullout

$$A_{TL2} := A_{T2} - A_{T2} \cdot 0.18 \qquad A_{TL2} = 11.512 \cdot kip$$

$$FS_{ATl2} := \frac{\phi \cdot A_{TL2}}{P_{u2}} \qquad FS_{ATl2} = 1.323$$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{ATu2} - FS_{ATl2}$ $\Delta F_{fc} = 0.581$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\sigma_F = 0.29$

$V_F := \frac{\sigma_F}{FS_{D2}}$ $V_F = 0.18$

$\beta_{LN} := \frac{\ln\left(\frac{FS_{D2}}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\beta_{LN} = 2.59$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := \mathrm{cnorm}(\beta_{LN})$ $R = 99.52 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $P_F = 0.48 \cdot \%$

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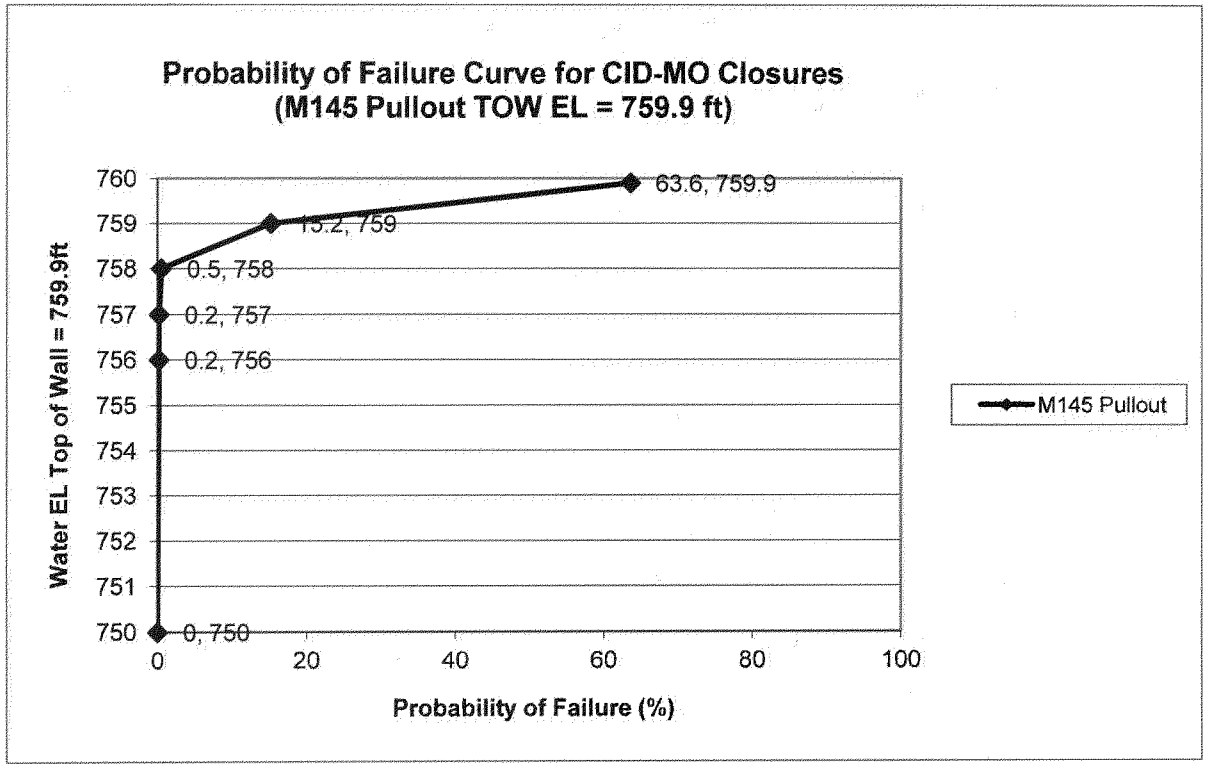

VKSM 11/16/11

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Comp. By [illegible] Date JAN 08 Project CID MO Sheet 177 of 437

Chkd. By KSM Date 9/28/11 Subject STABILITY NOTES FOR CLOSURES Sheet ___ of ___

PER THE 2005 STRUCTURAL SUMMIT, A LOAD TO TOP OF WALL MAY BE CHECKED, AND IF $FS_{SLIDE} > 1.3$, % BASE IN COMPRESSION > 25%, AND $q_a = q_u/2 >$ BEARING THEN A RELIABILITY STUDY MAY BE FOREGONE WITH RELIABILITY ASSUMED 99.8%.

FOR FEASIBILITY, FOOTERS ARE ANALYZED RIGID. IF THE FOOTERS ARE SIZED SUCH THAT THE TORSIONAL SHEAR STRESS IS LESS THAN 1/4 OF THE CRACKING SHEAR STRESS, AND HORIZONTAL FOOTER STEEL (PARALLEL TO WALL ALIGNMENT) IS PRESENT, IT SEEMS REASONABLE TO ANALYZE THE CLOSURES AS RIGID BODIES; HOWEVER THIS NEEDS ESTABLISHED IN DESIGN.

ALL SOIL PROFILES, LIKE STRUCTURE GEOMETRIES, ARE TAKEN FROM ORIGINAL DRAWINGS. FOR A (FEASIBILITY) RELIABILITY ANALYSIS THIS 50-YEAR OLD DATA SHOULD BE SUFFICIENT.

CREEP WILL BE ESTABLISHED BY LINE OF CREEP METHOD & DETERMINED BY CTWALL.

FILE NOMENCLATURE:

MXXX DRR OR MXXX UNR

MXXX ≡ MONOLITH NUMBER

DR ≡ DRAINED SOIL STRENGTHS

UN ≡ UNDRAINED SOIL STRENGTHS

LAST "R" ≡ RIGID BODY ASSUMPTION (i.e. ALL LOADS ARE UNIFORM).

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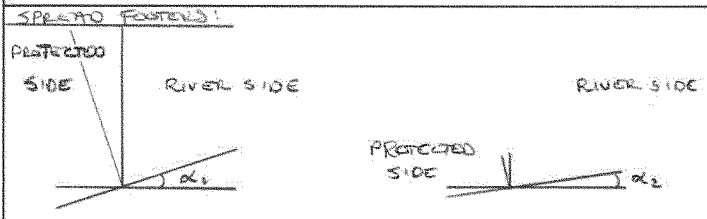
Comp. By [illegible] Date JAN 08 Project CID MO Sheet 178 of 437

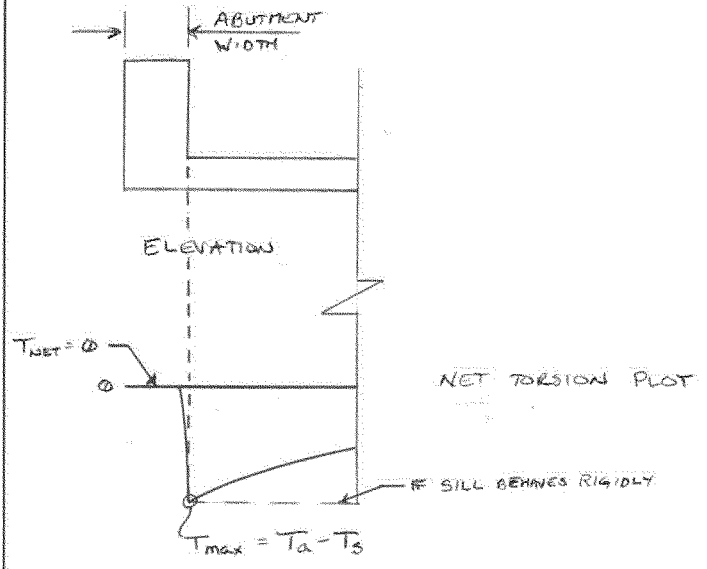
Chkd. By TM Date NOV 11 Subject TORSION Sheet ____ of ____

SPREAD FOOTERS:



ROTATION AT ABUTMENT DUE TO T_a

ROTATION AT SILL DUE TO T_s $(< T_a)$



→ NOTE: THE BENEFIT OF RESISTANCE FROM SOIL UNDER TOE IS IGNORED.

→ NOTE: THE STABILITY OF THE ABUTMENTS IS DEPENDENT UPON THEIR CONNECTION TO THE REST OF THE MONOLITH. AFTER PEAK LOADING, IT IS RECOMMENDED THAT THE FOOTERS BE INSPECTED FOR CRACKS, AND ANY DELAMINATION BE REMEDIED.

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Comp. By [illegible] Date JAN 08 Project CID MO Sheet 179 of 437

Chkd. By PJM Date NOV 11 Subject TORSION Sheet ____ of ____

SPREAD FOOTERS:

THOUGH THE FOOTER WILL BE ANALYZED RIGIDLY IN STABILITY CALCS, THE LOCAL LOAD AT THE ABUTMENTS CREATES A TWISTING OF THE FOOTER'S CROSS-SECTION. THIS TWISTING IS NOT PRESENT IN THE FOOTER'S CROSS-SECTION UNDER THE STOP LOGS. FOR A SPREAD FOOTER, THE TWISTING IS THE SOLE RESULT OF THE DRIVING WATER LOAD AT THE ABUTMENT, AS ALL UPLIFT AND RESISTING FORCES ARE EQUIVALENT BETWEEN THE TWO CROSS-SECTION LOCATIONS.

MONOLITH 10' ABUTMENT LENGTH = 2'

ABUTMENT TORSION: $\frac{1}{2}\gamma_{H_2O}\left[(9'-9'')^3/3+(6'-7'')^2(18'/2')(5.36')\right] = \underline{45.9\ K\text{-}ft/ft}$

$(9'-9'')-(6'-7'')+(6'-7'')/3 = 5.36'$

DEDUCT LOAD ON SILL:

$-\left(\frac{1}{2}\gamma\left[(9'-9'')^3/3 - (6'-7'')^2(5.36')\right]\right) = \underline{-2.4\ K\text{-}ft/ft}$

LOAD FACTORS = 1 FOR RELIABILITY

$\underline{\underline{T_u = T_{max} = 43.5\ K\text{-}ft}}$

TORSIONAL THRESHOLD (τ_e):

USING ACI 11.6.1: $\phi = 1$ FOR RELIABILITY

$\tau_e = \phi\sqrt{f'_c}\,(A_{cp}^2/p_{cp})$ $f'_c = 3{,}000$ psi

$w = 11'-0''$ $h = 1'-3''$

$A_{cp} = (11')(1.25') = 13.75\ ft^2 = 1980\ in^2$

$p_{cp} = 2(11'+1.25') = 294\ in$

NEED H.D 2' BELOW TOW FOR FS = 3, 2.5 WITH 10% REDUCTION IN STRENGTH DUE TO DETERIORATION

$\underline{\underline{\tau_e = 60.9\ K\text{-}ft/ft}}$

$\underline{\underline{\tau_e > T_u \quad OK}}$ $\underline{\underline{FS = 1.40}}$

FS < 1.5 ; FS > 1.0. OK FOR RISK ANALYSIS, BUT NOT GOOD ENOUGH TO PASS FOR FUTURE CONDITIONS. ACI CRITERIA NOT MET. ADEQUATE LOAD FACTORS NOT USED.

ANY TORSIONAL SHEAR STRESS BEYOND $\frac{1}{4}T_{CR}$ MIGHT BE ADEQUATELY HANDLED BY REBAR AT CRITICAL MOMENT IF FS FOR FLEXURE ≥ 1.5. LONGITUDINAL REBAR IS ASSUMED ADEQUATE (ACI 11.6.3.7).

US ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT

PM 4/30/08
181/437

US Army Corps of Engineers

Probability of Failure
CID MO Monolith 10
Torsion in Pile Cap

Comp by:FRS
Chkd by: ~~[illegible]~~ PMuller

I. Objective

The computations below show the process used to calculate the Reliability and the Probability of Failure.

II. References

1. *Reliability-Based Design in Civil Engineering* by Milton E. Harr, Dover Publications Inc. 1996
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength

III. Situation

1. CID MO ~~M32 sill~~ M10 FOOTER does not meet the strength 1.5 factor of safety for which it has been determined 99.8% reliability can be assigned. See mathcad analysis of the wall for existing condition strength check.
2. FEMA 310, Section 4.2.4.4, states, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength.
3. Material Properties used:

   Mean Concrete Strength, $f'_{cM} := 3750 \cdot psi$

   Mean Steel Strength, $F_{yM} := 50 \cdot ksi$

4. From *Reliability Based Design in Civil Engineering* by Milton E. Harr, pg 31, the coefficient of variation for Reinforced Concrete Grade 40 is 14%.

IV. Variable Definitions

FS_D = Factor of Safety under mean material parameters
FS_{Fyu} = Factor of Safety due to the upper bound value of the Steel Yield Strength
FS_{Fyl} = Factor of Safety due to the lower bound value of the Steel Yield Strength
FS_{fcu} = Factor of Safety due to the upper bound value of the Concrete Compresive Strength
FS_{fcl} = Factor of Safety due to the lower bound value of the Concrete Compresive Strength
ΔF_{UW} = Difference in Factors of Safety due to the change in Steel Yield Strength
ΔF_S = Difference in Factors of Safety due to the change in Concrete Compresive Strength
σ_F = Standard Deviation of the Factor of Safety
V_F = Coefficient of Variation of the Factor of Safety
β_{LN} = Lognormal Reliability Index
R = Reliability
P_F = Probability that the factor of safety is less than 1.0 (Probability of Failure)

V. *Caclulating Factors of Safety*

Condition under consideration from strength check: Tu for Pile Cap (from hand-computation analysis). $T_u := 43.5 kip \cdot ft$

Design Concrete Strength

$\phi_B := 1.0$ Strength Reduction Factors not used in Risk and Uncertainty Analysis

$w_{cap} := 11ft$ Width of Pile Cap

$T_{toe} := 1.25ft$ Thickness of concrete section

$b := 12\frac{in}{ft}$ 1 ft strip of wall analyzed

$A_{cp} := T_{toe} \cdot w_{cap} = 1.98 \times 10^3 \cdot in^2$

$P_{cp} := 2 \cdot (T_{toe} + w_{cap}) = 294 \cdot in$

Mean Concrete Strength

$\phi\tau_{IM} := \phi_B \cdot \sqrt{f_{cM} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{IM} = 68.048 \cdot kip \cdot ft$

$FS_D := \frac{\phi\tau_{IM}}{T_u}$ $FS_D = 1.564$

Upper Concrete Strength

For reinforced concrete structures a 14% standard deviation based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

$f_{cU} := f_{cM} + f_{cM} \cdot 0.14$ $f_{cU} = 4275 \cdot psi$

$\phi\tau_{IU} := \phi_B \cdot \sqrt{f_{cU} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\phi\tau_{IU} = 72.66 \cdot kip \cdot ft$

$FS_{fcu} := \frac{\phi\tau_{IU}}{T_u}$ $FS_{fcu} = 1.67$

Lower Concrete Strength

$f'_{cL} := f'_{cM} - f'_{cM} \cdot 0.14$ $\quad f'_{cL} = 3225 \cdot psi$

$\phi\tau_{IL} := \phi_B \cdot \sqrt{f'_{cL} \cdot psi} \cdot \left(\frac{A_{cp}^2}{P_{cp}}\right)$ $\quad \phi\tau_{IL} = 63.11 \cdot kip \cdot ft$

$FS_{fcl} := \frac{\phi\tau_{IL}}{T_u}$ $\quad FS_{fcl} = 1.451$

VI. Probability of Failure Calculation

$\Delta F_{fc} := FS_{fcu} - FS_{fcl}$ $\quad \Delta F_{fc} = 0.22$

$\sigma_F := \sqrt{\left(\frac{\Delta F_{fc}}{2}\right)^2}$ $\quad \sigma_F = 0.11$

$V_F := \frac{\sigma_F}{FS_D}$ $\quad V_F = 0.07$

$\beta_{LN} := \frac{\ln\left(\frac{FS_D}{\sqrt{1 + V_F^2}}\right)}{\sqrt{\ln\left(1 + V_F^2\right)}}$ $\quad \beta_{LN} = 6.349$ Taylor Series method. See ETL 1110-2-547, pg B-8.

$R := cnorm(\beta_{LN})$ $\quad R = 100 \cdot \%$

cnorm (x) is a Mathcad function that returns the cumulative probability distribution with mean 0 and variance 1.

$P_F := 1 - R$ $\quad P_F = 1.08 \times 10^{-8} \cdot \%$

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Comp. By [illegible] Date JAN 08 Project CLO MO Sheet 187 of 437

Chkd. By RCM Date NOV 11 Subject STABILITY NOTES FOR CLOSURES Sheet ____ of ____

MONOLITH 10: STA 5+23.95

PER GEOTECH DATA:

	DRAINED	UNDRAINED
FOUNDATION = BLANKET	$\phi = 22°$ $C = 0$	$C = 600$ psf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

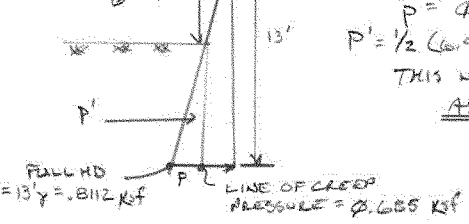
ULTIMATE BEARING, $q_u = 3400$ psf

RESULTS:

$q_{max} = 1.06$ ksf FOR FS = 2 $q_a = 1.7$ ksf

$q_a > q_{max}$ OK

$FS_{SLIDE} = 1.35$ WITH DRAINED STRENGTHS. THIS CASE ASSUMES SOME AMOUNT OF HORIZONTAL AND UPLIFT RELIEF WITH THE LINE OF CREEP METHOD. IT IS THE ENGINEER'S JUDGMENT TO TEST THE SLIDING WITH FULL HORIZONTAL PRESSURE (NO CHANGE MADE TO UPLIFT). SINCE THIS IS A VERY CONSERVATIVE ANALYSIS, A SLIDING FS > 1.1 WILL BE ACCEPTABLE. THIS FILE IS SAVED AS M10 FLHD.



HT OF SOIL = 13' − (6'-7") = 6.417'

p = 0.8112 − 0.685 = 0.1262 ksf

P' = ½ (6.417')(p) = 0.40 ksf

THIS WILL BE A SURCHARGE APPLIED AT EL 747.37'

CTWALL $FS_{SLIDE} = 1.18$ OK

MONOLITH 18: STA 8+67.90

PER GEOTECH DATA:

	DRAINED	UNDRAINED
USE FOUNDATION = BLANKET	$\phi = 22°$	$C = 0.6$ ksf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

$q_u = 3663$ psf.

MONOLITH 32: STA 14+80.69

PER GEOTECH DATA:

	DRAINED	UNDRAINED
USE FOUNDATION = BLANKET	$\phi = 22°$	$C = 0.6$ ksf

$\gamma_{SAT} = 115$ pcf $\gamma_m = 110$ pcf

$q_u = 4000$ psf

NOTE: ANALYSIS IN CTWALL DOES NOT ACCOUNT FOR 'Z' SHAPE OF MONOLITH (IN PLAN). THUS ACTUAL SLIDING 8-377S > CTWALL'S COMPUTATION.

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By [illegible] Date DEC 07 Project CID MO Sheet 188 of 437

Chkd. By FM Date NOV 11 Subject STABILITY Sheet ____ of ____

SPREAD FOUNDATION

MONOLITH 16 CONSISTS OF A FLOOD WALL AND CLOSURE ON ONE FOOTING. IT IS ASSUMED THAT THE FOOTER BEHAVES RIGIDLY.

THEREFORE THE CONCENTRATION OF H_2O LOADS AT ABUTMENTS WILL NOT CAUSE A CONCERN; THEY WILL DISTRIBUTE ACROSS THE MONOLITH LENGTH. FURTHERMORE, THE DIFFERENCE BETWEEN A RESOLVED TOTAL FORCE AND ECCENTRICITY FROM C.G. WILL NOT BE A CONCERN, IT ONLY MEANS THAT IF THE MONOLITH BEGINS TO SLIDE, IT WILL ALSO ROTATE (IN PLAN). THIS WILL ACTUALLY HELP STABILITY BY INCREASING SIDE FRICTION (IGNORED IN ANALYSIS) FROM BEARING ON ADJACENT MONOLITHS.

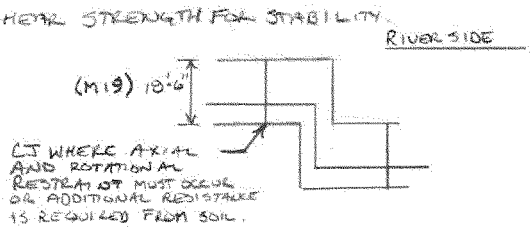
MONOLITH 16 IS THE ONLY MONOLITH WITH STOP LOG GAP ON A SPREAD FOOTING WITHOUT "Z" GEOMETRY.

→ A GENERAL 2-D STABILITY ANALYSIS WILL BE SUFFICIENT FOR CALCULATING SLIDING FS, BEARING, AND OVERTURNING.

MONOLITHS 18 & 32:

THESE ARE THE ONLY TWO MONOLITHS ON SPREAD FOOTINGS WITH STOP LOG GAPS. THEY ARE 'Z' SHAPED IN PLAN AND HAVE SIMILAR FOOTPRINT SIZES. THE GAP WIDTH IS THE SAME (17'-6"), BUT THE HEIGHT OF M32 (11'-11") IS MUCH TALLER THAN M18 (8'-7¾").

IT WILL BE INVESTIGATED HERE IF THIS 'Z' SHAPE WILL REQUIRE MORE SHEAR STRENGTH FOR STABILITY.



PDM 4/30/08
SEE COMMENTS ON DRAINED W/ ADDED WATER PRESS RUN
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********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.13.57

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Drained Case, Rigid Body Analysis
Computed by: FRS

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25	ft
Height of stem (HTS)	=	9.75	ft
Thickness top of stem (TTS)	=	1.00	ft
Thickness bottom of stem (TBS)	=	1.50	ft
Dist. of batter at bot. of stem (TBSR)	=	.00	ft
Depth of heel (THEEL)	=	3.25	ft
Distance of batter for heel (BTRH)	=	.00	ft
Depth of toe (TTOE)	=	1.25	ft
Width of toe (TWIDTH)	=	3.50	ft
Distance of batter for toe (BTRT)	=	.00	ft
Width of base (BWIDTH)	=	11.00	ft
Depth of key (HK)	=	2.00	ft
Width of bottom of key (TK)	=	1.00	ft
Dist. of batter at bot. of key (BTRK)	=	.50	ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

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(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
22.00	.000	.110	.115	11.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
22.00	.000	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = 22.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 22.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:
o Crack depth is to be calculated
o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

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Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

```
***** Summary of Results *****

Project name: CID MO Closures

***************     *** Satisfied ***
* Overturning *     Required base in comp. =  75.00 %
***************     Actual base in comp.   =  96.14 %
                    Overturning ratio      =   1.54

Xr (measured from toe) =   3.53 ft
Resultant ratio        =  .3205
Stem ratio             =  .3182
Base pressure at x=  10.58 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0569 ksf

***********     *** Satisfied ***
* Sliding *     Min. Required =   1.33
***********     Actual FS     =   1.35


********************** Output Results **********************

Date: **/01/17                                   Time: 10.13.57

Company name:

Project name:
  CID MO Closures
Project location:

Wall location:
  M10 Drained Case, Rigid Body Analysis
Computed by: FRS

***************************
**  Overturning Results  **
***************************

Solution converged in  1 iterations.

SMF used to calculate K's =     .6667
Alpha for the SMF         = -34.3032
Calculated earth pressure coefficients:
  Driving side at rest K    =    .4788
  Driving side at rest Kc   =    .8497
  Resisting side at rest K  =    .6254
  Resisting side at rest Kc =    .7908
  At-rest K's for resisting side calculated.

Depth of cracking =       .00 ft
```

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** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
758.25	.0000
752.17	.3794
745.25	.6850

Earth pressures:

Elevation (ft)	Pressure (ksf)
752.17	.0000
745.25	.2347

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.3564
747.25	.5106
745.25	.6688

Earth pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.0950

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
747.25	1.7357
745.25	1.7357

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.6850
1.00	.6688
1.50	.5106
11.00	.3564

** Forces and moments **

Part	Force (kips)		Mom. Arm	Moment
	Vert.	Horiz.	(ft)	(ft-k)

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Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.000	.00	.00
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.812	.31	.25
Shear (due to delta).....	.158		-11.00	-1.74
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		4.836	2.51	12.16
Resisting side:				
Effective earth loads.....		-.210	1.48	-.31
Balancing earth load.....		-3.471	-.99	3.45
Water loads.....		-1.967	-.04	.09
Foundation:				
Vertical force on base.....	-5.589		-3.53	19.70
Uplift.....	-5.090		-6.03	30.71
** Statics Check ** SUMS =	.000	.000		.00

Angle of base = 10.30 degrees
Normal force on base = 6.119 kips
Shear force on base = 2.416 kips
Max. available shear force = 2.861 kips

Base pressure at x= 10.58 ft from toe = .0000 ksf
Base pressure at toe = 1.0569 ksf

Xr (measured from toe) = 3.53 ft
Resultant ratio = .3205
Stem ratio = .3182
Base in compression = 96.14 %
Overturning ratio = 1.54

Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

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```
***********************
**  Sliding Results  **
***********************

Solution converged.  Summation of forces = 0.
```

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	3.632
2	1.153	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.3794	.6850		
2			.0000	.6850
2			11.0000	.3564
3	.0000	.3564		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-35.859	11.813	3.810	11.813	6.287
2	10.305	11.180	9.091	11.180	5.822
3	36.877	7.365	1.497	7.365	1.312

Wedge number	Net force (kips)
1	-4.215
2	2.822
3	1.393
SUM =	.000

```
+-----------------------------+
| Factor of safety =    1.351 |
+-----------------------------+
```

PDM 4/30/08

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********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.11.24

Company name:

Project name:
CID Missouri Closures
Project location:

Wall location:
Monolith 10 Drained Case, Rigid Body Analysis
Computed by: FRS + FULL HEAD ON U/S FACE.

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25	ft
Height of stem (HTS)	=	9.75	ft
Thickness top of stem (TTS)	=	1.00	ft
Thickness bottom of stem (TBS)	=	1.50	ft
Dist. of batter at bot. of stem (TBSR)	=	.00	ft
Depth of heel (THEEL)	=	3.25	ft
Distance of batter for heel (BTRH)	=	.00	ft
Depth of toe (TTOE)	=	1.25	ft
Width of toe (TWIDTH)	=	3.50	ft
Distance of batter for toe (BTRT)	=	.00	ft
Width of base (BWIDTH)	=	11.00	ft
Depth of key (HK)	=	2.00	ft
Width of bottom of key (TK)	=	1.00	ft
Dist. of batter at bot. of key (BTRK)	=	.50	ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

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(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
22.00	.000	.110	.115	11.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
22.00	.000	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = 22.00 (deg)
c for soil-structure interface = .000 (ksf)
phi for soil-soil interface = 22.00 (deg)
c for soil-soil interface = .000 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Horizontal line load data:

TO ESTABLISH FULL HD ON DRIVING SIDE

Elevation (ft)	Force (kips)
747.37	.40

CONCUR

CONSERVATIVE TO ASSUME NO CRACK (DRAINED) + FULL PRESSURE ON R.S + THIS WILL MAKE UPLIFT INCONSISTENT. SUGGEST CHANGING CRACK OPTION TO FORCE A CRACK TO HEEL OR DON'T ADD LOAD

USED ON PREVIOUS RUN

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:

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CAN BE CHANGED TO FORCE CRACK

```
  o  Crack depth is to be calculated
  o  Computed cracks *will* be filled with water

Strength mobilization factor =     .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

Forces on the resisting side *are used* in the sliding analysis.

*Do* iterate in overturning analysis.



***** Summary of Results *****

Project name: CID Missouri Closures

***************     *** Satisfied ***
* Overturning *     Required base in comp. =  75.00 %
***************     Actual base in comp.   =  93.85 %
                    Overturning ratio      =   1.53

Xr (measured from toe) =   3.44 ft
Resultant ratio        =  .3128
Stem ratio             =  .3182
Base pressure at x=  10.32 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0827 ksf

***********     *** Not Satisfied ***
* Sliding *     Min. Required =   1.33
***********     Actual FS     =   1.18

  To increase stability try one or a combination
  of the following:
    1.  Increase the base width
    2.  Slope the base of the structure
    3.  Lower the wall base
    4.  Add a key
```

CONCUR.

THIS MAY BE CONSERVATIVE

→ EXTRA H_2O PRESS WITH DRAINED CONDITION

– COMPUTED CRACK CASE (NO CRACK) IS MORE REALISTIC

```
********************** Output Results ***********************

Date: **/01/17                                  Time: 10.11.24

Company name:

Project name:
  CID Missouri Closures
Project location:

Wall location:
  Monolith 10 Drained Case, Rigid Body Analysis
Computed by: FRS

***************************
**  Overturning Results  **
```

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```
****************************

Solution converged in  1 iterations.

SMF used to calculate K's =     .6667
Alpha for the SMF          = -34.3032
Calculated earth pressure coefficients:
  Driving side at rest K    =     .4788
  Driving side at rest Kc   =     .8497
  Resisting side at rest K  =     .6254
  Resisting side at rest Kc =     .7908
  At-rest K's for resisting side calculated.

Depth of cracking =       .00 ft

** Driving side pressures **

  Water pressures:
    Elevation    Pressure
      (ft)        (ksf)
    ======================
      758.25       .0000
      752.17       .3794
      745.25       .6850

  Earth pressures:
    Elevation    Pressure
      (ft)        (ksf)
    ======================
      752.17       .0000
      745.25       .2347

** Resisting side pressures **

  Water pressures:
    Elevation    Pressure
      (ft)        (ksf)
    ======================
      751.67       .0000
      747.25       .3564
      747.25       .5106
      745.25       .6688

  Earth pressures:
    Elevation    Pressure
      (ft)        (ksf)
    ======================
      751.67       .0000
      747.25       .0950

  Balancing earth pressures:
    Elevation    Pressure
      (ft)        (ksf)
    ======================
      747.25      1.9357
      745.25      1.9357

** Uplift pressures **

  Water pressures:
     x-coord.    Pressure
```

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(ft)	(ksf)
.00	.6850
1.00	.6688
1.50	.5106
11.00	.3564

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.400	.12	.05
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.812	.31	.25
Shear (due to delta).....	.158		-11.00	-1.74
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		4.836	2.51	12.16
Resisting side:				
Effective earth loads.....		-.210	1.48	-.31
Balancing earth load.....		-3.871	-1.00	3.87
Water loads.....		-1.967	-.04	.09
Foundation:				
Vertical force on base.....	-5.589		-3.44	19.23
Uplift.....	-5.090		-6.03	30.71
** Statics Check ** SUMS =	.000	.000		.00

Angle of base = 10.30 degrees
Normal force on base = 6.191 kips
Shear force on base = 2.809 kips
Max. available shear force = 2.890 kips

Base pressure at x= 10.32 ft from toe = .0000 ksf
Base pressure at toe = 1.0827 ksf

Xr (measured from toe) = 3.44 ft
Resultant ratio = .3128
Stem ratio = .3182
Base in compression = 93.85 %
Overturning ratio = 1.53

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Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Wedge Number	Horizontal Loads (kips)	Vertical Loads (kips)
1	.000	3.379
2	1.553	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.3794	.6850		
2			.0000	.6850
2			11.0000	.3564
3	.0000	.3564		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = .00 ft

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	-37.846	11.279	3.544	11.279	6.003
2	10.305	11.180	9.091	11.180	5.822
3	35.753	7.565	1.560	7.565	1.348

Wedge number	Net force (kips)
1	-4.155
2	2.711
3	1.445
SUM =	.000

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```
+-------------------------------+
| Factor of safety =     1.181 |
+-------------------------------+
```

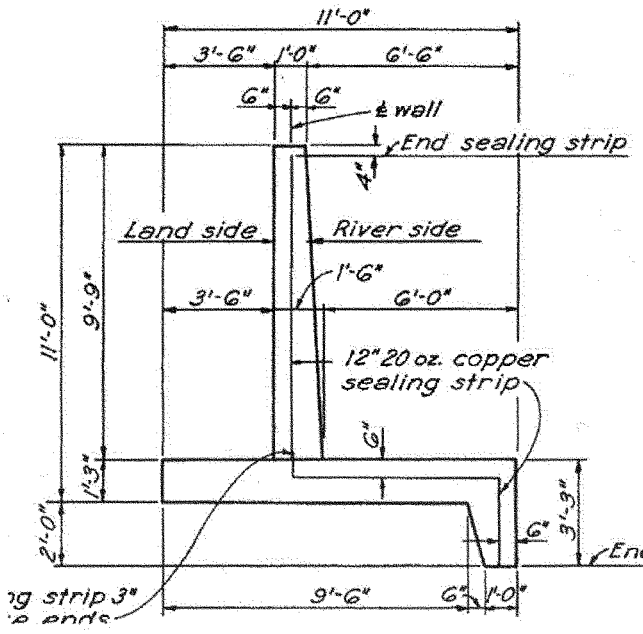
204/457 TDM 4/30/08

US Army Corps of Engineers.

CID Missouri
Monolith 10 Stoplog Gap
Footer and Wall Strength Analysis
Kansas Citys Levees

Comp by: FRS
Chkd by: [redacted] P Muller

DRAINED



Typical Floodwall Section

Assumptions Taken from Record Drawings File No. A-10-1176

1. The length of the base in bearing compression must be less than (D+E+F+G+H). Otherwise, the bending moment, M_H will be incorrect.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side Water EL is the EL of the top of resisting grade.

4. Temperature and Shrinkage steel assumed sufficient.

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Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: M10DRR.OUT

Elevation of top of stem (ELTS)	ELTS := 758.25ft
Height of stem (HTS)	HTS := 9ft + 9in
Thickness top of stem (TTS)	TTS := 1.0ft
Thickness bottom of stem (TBS)	TBS := 1.5ft
Dist. of batter at bot.of stem LS (TBSR)	TBSR := 0ft
Depth of heel (THEEL)	THEEL := 3ft + 3in
Distance of batter for heel (BTRH)	BTRH := 0ft *
Depth of toe (TTOE)	TTOE := 1ft + 3in *
Width of toe (TWIDTH)	TWIDTH := 3ft + 6in
Distance of batter for toe (BTRT)	BTRT := 0ft *
Width of base (BWIDTH)	BWIDTH := 11ft
Depth of key (HK)	HK := 2ft + 0in *
Width of bottom of key (TK)	TK := 1ft
Dist. of batter at bot. of key (BTRK)	BTRK := 0ft + 6in
Driving side soil elevation (ELSTDS)	ELSTDS := 752.17ft
Resisting side soil and water elevation (ELSTRS)	ELSTRS := 751.67ft
Driving side water elevation (WATELD)	WATELD := 758.25ft

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

$L := ELTS - ELSTRS$

$M := TTS$

* Equations in this file are established for a horizontal footer with uniform thickness.

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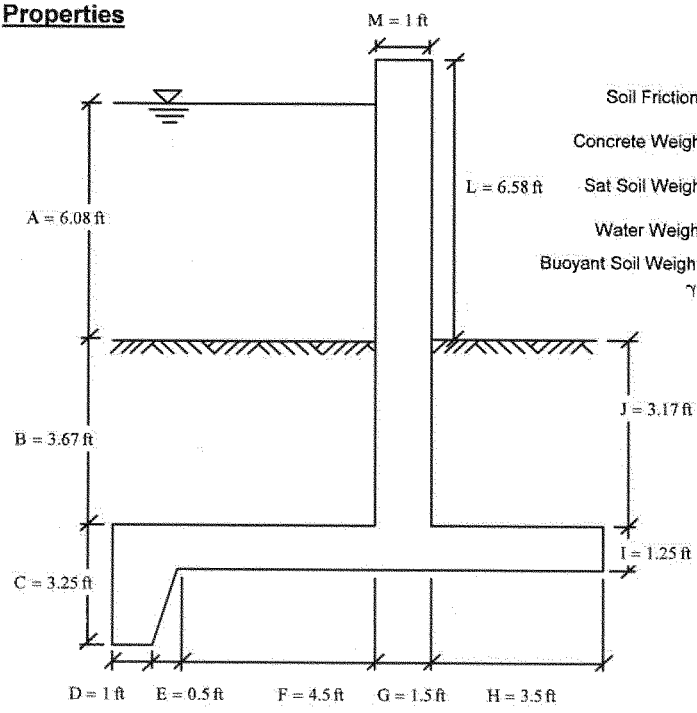
Properties



Soil Friction $\phi := 22 \cdot deg$

Concrete Weight $\gamma_c := 150pcf$

Sat Soil Weight $\gamma_{sat} := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

Buoyant Soil Weight

$\gamma_b := \gamma_{sat} - \gamma_w = 52.6 \cdot pcf$

$Base := D + E + F + G + H = 11\,ft$

$Verify := if(BWIDTH = Base, \text{"OKAY"}, \overline{\text{"Check Dimensions"}})$

Verify = "OKAY"

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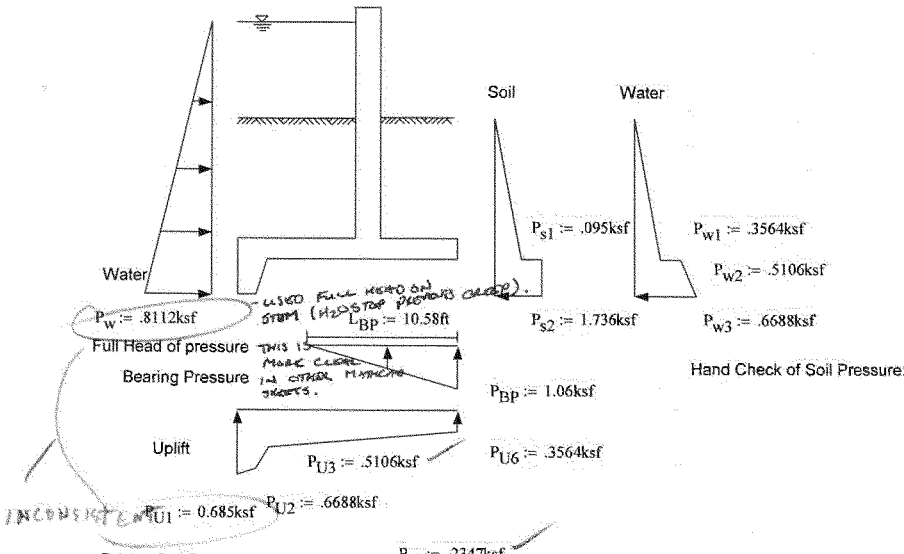
Pressures below were taken from corresponding CTWALL output.



Driving Soil Pressure at bottom of Key: $P_{s3} := .2347ksf$

Driving Soil Pressure at top of footer:

$$P_{s4} := \frac{B}{B + C} \cdot P_{s3} = 0.12 \cdot ksf$$

Driving Soil Pressure at base of footer:

$$P_{s5} := \frac{B}{(B + I)} \cdot P_{s3} = 0.18 \cdot ksf$$

Note, the presence of any cutoff wall could impact the uplift pressures. Reducing the uplift would cause an uncoservative design for the heel.

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3000 psi is used conservatively until a reliability calculation is performed
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$ Steel Properties $F_y := 40.0ksi$

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Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1.0$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1.0$	
Load Factors		
Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1.0$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

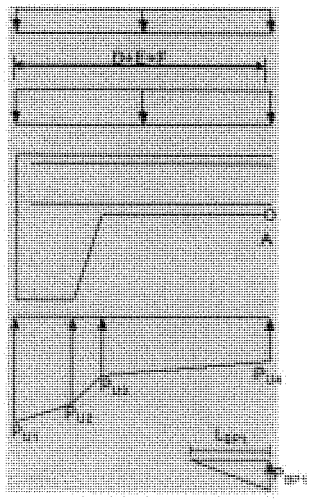
○ Location where moment is taken about.

Water

Soil

Slab

Uplift



$$W_w := \gamma_w \cdot (A + B) \cdot (D + E + F) \qquad W_w = 3650 \frac{lb}{ft}$$

$$W_s := \gamma_b \cdot B \cdot (D + E + F) \qquad W_s = 1158.25 \frac{lb}{ft}$$

Drag on edge of Key due to tendency to rotate:

$$\delta_p := \left[\frac{1}{2} \cdot \left(P_{s4} + P_{s3}\right) \cdot C \cdot \tan(\phi)\right] = 235.81 \frac{lb}{ft}$$

$$\delta_d := \text{if}\left(L_{BP} < BWIDTH, \delta_p, 0\right) = 235.81 \frac{lb}{ft}$$

$$P_{U4} := \left(P_{U3} - P_{U6}\right) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$$P_{U4} = 437.56 \frac{lb}{ft^2}$$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

$$L_{BP1} = 5.58\,ft$$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$$P_{BP1} := \frac{L_{BP1}}{L_{BP}} \cdot P_{BP} \qquad P_{BP1} = 559 \frac{lb}{ft^2}$$

Heel and Key Centroid

$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}} \qquad X_H = 3.59\,ft$$

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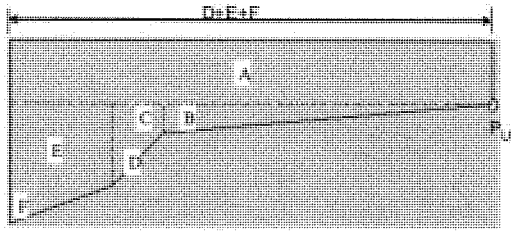
- HEEL check (cont'd)

Heel and Key Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot \gamma_c \qquad W_H = 1500\,\frac{lb}{ft}$$

Uplift Centroid



AREA	CENTROID
$A_A := (D + E + F)\cdot P_{U4}$	$C_A := \frac{D + E + F}{2}$
$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$	$C_B := \frac{2\cdot F}{3}$
$A_C := E\cdot(P_{U3} - P_{U4})$	$C_C := F + \frac{E}{2}$
$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$	$C_D := F + \frac{2\cdot E}{3}$
$A_E := D\cdot(P_{U2} - P_{U4})$	$C_E := \left(\frac{D}{2} + E + F\right)$
$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$	$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.24\,ft$$

Uplift on Heel $\quad W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 3105\,\frac{lb}{ft}$

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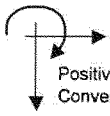
- HEEL check (cont'd)

Loading: Note, since the amount of base in compression is > 75%, bearing of water-side of key against soil due to rotation should be marginally different from counter-moment produced from sliding force on key. Water loads on each side of key are also similar and so their effects on the heel are ignored.

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[(W_w + W_s) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H + \delta_d \cdot (D + E + F) \right] \right]$$



$$M_H = -8275.73 \cdot \frac{ft \cdot lb}{ft}$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

$$M_{uH} := |M_H| \qquad M_{uH} = 8.3 \cdot \frac{kip \cdot ft}{ft}$$

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \left(W_w + W_s + \delta_d + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right)$$

$$V_H = 1879.6 \frac{lb}{ft} \qquad V_{uH} := |V_H| \qquad V_{uH} = 1.9 \cdot \frac{kip}{ft}$$

Capacity:
Flexural Capacity

Top Steel: $A_{Heel1} := .44 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$ Bottom Steel: $A_{Heel2} := .35 \frac{in^2}{ft}$ $C_{CH2} := 3.375in$

$$A_{s1} := A_{Heel1} \qquad d_{H1} := I - C_{CH1} \qquad A_{s2} := A_{Heel2} \qquad d_{H2} := I - C_{CH2}$$

$$b := 12 \frac{in}{ft} \quad a1 := \frac{A_{s1} \cdot F_y}{0.85 f_c \cdot b} \quad a1 = 0.58 \cdot in \qquad b_c := 12 \frac{in}{ft} \quad a2 := \frac{A_{s2} \cdot F_y}{0.85 f_c \cdot b_c} \quad a2 = 0.46 \cdot in$$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 16.63 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 13.3 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1}) \qquad \phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b_c \cdot d_{H2} \cdot \sqrt{f_c \cdot psi}$$

$$\phi V_{c1} = 15281 \frac{lb}{ft} \qquad \phi V_{c2} = 15281 \frac{lb}{ft}$$

$$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1}) \qquad \phi V_H = 15.28 \cdot \frac{kip}{ft}$$

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- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$ $\quad FS_1 = 2.01$

$M_{uH} = 8.28 \cdot \frac{kip \cdot ft}{ft}$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Shear $\phi V_H = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$

$V_{uH} = 1.88 \cdot \frac{kip}{ft}$ $\quad FS_2 = 8.13$

$Check2 := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 2.01$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Key Check

Hydrostatic Load is similar to each side of key, and thus neglected in analysis.

$M_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[(P_{s5} - P_{s2}) \cdot \frac{(C-I)^2}{2} + \frac{1}{2} \cdot (P_{s3} - P_{s5}) \cdot \frac{2(C-I)^2}{3} \right] = -3.04 \cdot kip \cdot \frac{ft}{ft}$ $\quad M_K > 0$ means water-side steel is in tension.

$V_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[(P_{s5} - P_{s2}) \cdot (C-I) + \frac{1}{2} (P_{s3} - P_{s5}) \cdot (C-I) \right] = -3062.23 \frac{lb}{ft}$ $\quad M_{uK} := |M_K| = 3042.35\ lb$

$V_{uK} := |V_K| = 3062.23 \frac{lb}{ft}$

Steel in Tension: $A_{Key} := 0.16 \frac{in^2}{ft}$ $\quad C_K := 3.375in$ $\quad d_K := (D + E) - C_K$

$b := 12 \frac{in}{ft}$ $\quad a_k := \frac{A_{Key} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_k = 0.21 \cdot in$ $\quad \phi M_K := \phi_B A_{Key} \cdot F_y \left(d_K - \frac{a_k}{2} \right)$ $\quad \phi M_K = 7.74 \cdot \frac{kip \cdot ft}{ft}$

Shear:

$\phi V_K := \phi_V \cdot 2 \cdot b \cdot d_K \cdot \sqrt{f'_c \cdot psi}$ $\quad \phi V_K = 19225 \frac{lb}{ft}$

$FS_{KM} := \frac{\phi M_K}{M_K} = -2.55$ $\quad Check_M := if(\phi M_K > 1.5 M_{uK}, "OKAY", "NO GOOD")$

$Check_M = "OKAY"$

$FS_{KV} := \frac{\phi V_K}{V_K} = -6.28$ $\quad Check_V := if(\phi V_K > 1.5 V_{uK}, "OKAY", "NO GOOD")$

$Check_V = "OKAY"$

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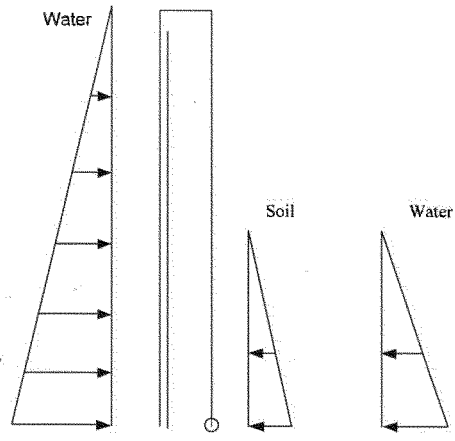
- STEM Check



Note:
P_{ss2} = Resistive Soil pressure present at base of stem
P_{ws2} = Resistive Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 608\,\frac{lb}{ft^2}$

$P_{ss2} := \frac{J}{I + J}\cdot P_{s1}$ $\qquad P_{ws2} := J\cdot\gamma_w$

Driving Soil Pressure at base of Stem:

$P_{ss1} := P_{s4} = 0.12\cdot ksf$

$P_{ss2} = 0.07\cdot ksf$ $\qquad P_{ws2} = 0.2\cdot ksf$

Waterside Steel: $A_{Stem1} := .51\,\frac{in^2}{ft}$ $\quad C_{CS1} := 2.81in$ $\quad d_{S1} := G - C_{CS1}$

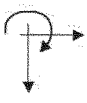
Resistive Side Steel: $A_{Stem2} := 0.2\cdot\frac{in^2}{ft}$

Loading:

Bending

$$M_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} + \frac{P_{ss1}\cdot B^2}{2\cdot 3} - \frac{(P_{ss2} + P_{ws2})\cdot(J^2)}{2\cdot 3}\right]$$

Note:
IF: M_{uS} is > 0
THEN: Riverside steel is in tension



$M_{uS} = 9.5\cdot\frac{kip\cdot ft}{ft}$

Positive Sign Convention

Shear

$$V_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\left[\frac{P_{ws1}\cdot(A + B)}{2} + \frac{P_{ss1}\cdot B}{2} - \frac{(P_{ss2} + P_{ws2})\cdot(J)}{2}\right]$$

$V_{uS} = 2.8\cdot\frac{kip}{ft}$

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- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.67 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

NICE!

$\mu := 1.0$

$\phi V_S := \phi_V \cdot \left(A_{Stem2} \cdot F_y \cdot \mu\right) = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 9.47 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 = 2.67$

$Check3 := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check3 = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 2.77 \cdot \frac{kip}{ft}$ $\quad FS_4 = 2.89$

$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO\ GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_3, FS_4\right)$ $\quad FS_S = 2.67$

Note: No bar cut-offs checks are made for the reliability Study.

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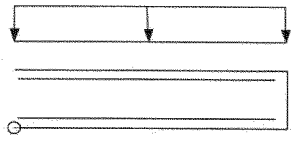
- TOE Check

Soil

Slab

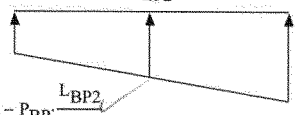


$W_s := \gamma_{sat} \cdot H \cdot (J)$ $\quad W_s = 1275.92 \frac{lb}{ft}$

$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5\,ft$

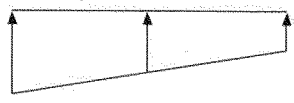
Bearing Pressure



$P_{BP2} := P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}$ $\quad P_{BP} = 1060 \frac{lb}{ft^2}$

$P_{BP2} = 709.34 \frac{lb}{ft^2}$

Uplift



$P_{U6} = 356.4 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[(P_{U3} - P_{U6}) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$P_{U5} = 413.21 \frac{lb}{ft^2}$

Bearing Pressure

$$L_{bp} := \frac{(L_{BP2} \cdot P_{BP2}) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}}$$

$L_{bp} = 2.8\,ft$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{(P_{BP} - P_{BP2}) \cdot (L_{BP2})}{2}$$

$W_{bp} = 3096 \frac{lb}{ft}$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{(P_{U5} - P_{U6}) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}}$$

$L_u = 1.71\,ft$

$$W_u := H \cdot P_{U6} + \frac{(P_{U5} - P_{U6}) \cdot H}{2}$$

$W_u = 1346.82 \frac{lb}{ft}$

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Toe Check (continued)

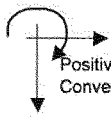
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot l) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -7.6 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 7.59 \cdot \frac{kip \cdot ft}{ft}$$



Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_s + (H \cdot l) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.51 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 2.51 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Area of bottom (tension) steel: $A_{Toe} := .352 \frac{in^2}{ft}$ $\quad C_{CT} := 3.375in$ $\quad d_T := 1 - C_{CT} = 11.63 \cdot in$

$$b := 12 \frac{in}{ft} \qquad A_{s1} := A_{Toe}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b} \qquad a1 = 0.46 \cdot in$$

$$\phi M_T := \phi_B A_{s1} \cdot F_y \left(d_T - \frac{a1}{2} \right) \qquad \phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_T := \phi_V \cdot 2 \cdot b \cdot d_T \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_T = 15.28 \cdot \frac{kip}{ft}$$

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Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_5 := \frac{\phi M_T}{M_{uT}}$

$M_{uT} = 7.59 \cdot \frac{kip \cdot ft}{ft}$

$FS_5 = 1.76$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO GOOD")$

Check5 = "OKAY"

Shear

$\phi V_T = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_6 := \frac{\phi V_T}{V_{uT}}$

$V_{uT} = 2.51 \cdot \frac{kip}{ft}$

$FS_6 = 6.09$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO GOOD")$

Check6 = "OKAY"

Controlling Factor of Safety

$FS_T := \min(FS_5, FS_6)$

$FS_T = 1.76$

End of Analysis

PDM 4/30/08
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********** Echoprint of Input Data **********

Date: **/01/17 Time: 10.16.16

Company name:

Project name:
CID MO Closures
Project location:

Wall location:
M10 Undrained Case, Rigid Body Analysis
Computed by: FRS

Structural geometry data:

Elevation of top of stem (ELTS)	=	758.25	ft
Height of stem (HTS)	=	9.75	ft
Thickness top of stem (TTS)	=	1.00	ft
Thickness bottom of stem (TBS)	=	1.50	ft
Dist. of batter at bot. of stem (TBSR)	=	.00	ft
Depth of heel (THEEL)	=	3.25	ft
Distance of batter for heel (BTRH)	=	.00	ft
Depth of toe (TTOE)	=	1.25	ft
Width of toe (TWIDTH)	=	3.50	ft
Distance of batter for toe (BTRT)	=	.00	ft
Width of base (BWIDTH)	=	11.00	ft
Depth of key (HK)	=	2.00	ft
Width of bottom of key (TK)	=	1.00	ft
Dist. of batter at bot. of key (BTRK)	=	.50	ft

Structure coordinates:

x (ft)	y (ft)
.00	745.25
.00	748.50
6.00	748.50
6.50	758.25
7.50	758.25
7.50	748.50
11.00	748.50
11.00	747.25
1.50	747.25
1.00	745.25

NOTE: X=0 is located at the left-hand side of the structure. The Y values correspond to the actual elevation used.

Structural property data:
Unit weight of concrete = .150 kcf

Driving side soil property data:

Phi	c	Moist Unit wt.	Saturated unit wt.	Delta	Elev. soil

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(deg)	(ksf)	(kcf)	(kcf)	(deg)	(ft)
.00	.600	.110	.115	.00	752.17

Driving side soil geometry:

Soil point	Batter (in:1ft)	Distance (ft)
1	.00	500.00
2	.00	.00
3	.00	500.00

Driving side soil profile:

Soil point	x (ft)	y (ft)
1	-1493.81	752.17
2	6.19	752.17

Resisting side soil property data:

Phi (deg)	c (ksf)	Moist Unit wt. (kcf)	Saturated unit wt. (kcf)	Elev. soil (ft)	Batter (in:1ft)
.00	.600	.110	.115	751.67	.00

Resisting side soil profile:

Soil point	x (ft)	y (ft)
1	7.50	751.67
2	507.50	751.67

Foundation property data:
phi for soil-structure interface = .00 (deg)
c for soil-structure interface = .300 (ksf)
phi for soil-soil interface = .00 (deg)
c for soil-soil interface = .600 (ksf)

Water data:
Driving side elevation = 758.25 ft
Resisting side elevation = 751.67 ft
Unit weight of water = .0624 kcf
Seepage pressures computed by Line of Creep method.

Minimum required factors of safety:
Sliding FS = 1.33
Overturning = 75.00% base in compression

Crack options:
o Crack *is* down to bottom of heel
o Computed cracks *will* be filled with water

Strength mobilization factor = .6667

At-rest pressures on the resisting side *are used*
in the overturning analysis.

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Forces on the resisting side *are used* in the sliding analysis.

Do iterate in overturning analysis.

```
***** Summary of Results *****

Project name: CID MO Closures

***************     *** Satisfied ***
* Overturning *     Required base in comp. =  75.00 %
***************     Actual base in comp.   =  78.63 %
                    Overturning ratio      =   1.33

Xr (measured from toe) =    2.88 ft
Resultant ratio        =  .2621
Stem ratio             =  .3182
Base pressure at x=   8.65 ft from toe =     .0000 ksf
Base pressure at toe                   =    1.0609 ksf

***********     *** Satisfied ***
* Sliding *     Min. Required =   1.33
***********     Actual FS     =   5.89


********************** Output Results **********************

Date: **/01/17                                    Time: 10.16.16

Company name:

Project name:
  CID MO Closures
Project location:

Wall location:
  M10 Undrained Case, Rigid Body Analysis
Computed by: FRS

***************************
**  Overturning Results  **
***************************

Solution converged in  1 iterations.

SMF used to calculate K's =     .6667
Alpha for the SMF         =     .0000
Calculated earth pressure coefficients:
  Driving side at rest K    =    .0000
  Driving side at rest Kc   =    .0000
  Resisting side at rest K  =   1.0000
  Resisting side at rest Kc =   1.0000
  At-rest K's for resisting side calculated.

Depth of cracking =      6.92 ft
```

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Crack extends to bottom of base of structure.

** Driving side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
758.25	.0000
745.25	.8112

** Resisting side pressures **

Water pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.3921
747.25	.6147
745.25	.7878

Earth pressures:

Elevation (ft)	Pressure (ksf)
751.67	.0000
747.25	.1162

Balancing earth pressures:

Elevation (ft)	Pressure (ksf)
747.25	1.3735
745.25	1.3735

** Uplift pressures **

Water pressures:

x-coord. (ft)	Pressure (ksf)
.00	.8112
1.00	.7878
1.50	.6147
11.00	.3921

** Forces and moments **

Part	Force (kips) Vert.	Force (kips) Horiz.	Mom. Arm (ft)	Moment (ft-k)
Structure:				
Structure weight.....	4.266		-5.34	-22.79
Structure, driving side:				
Moist soil.....	.000		.00	.00

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Saturated soil.....	2.572		-7.95	-20.45
Water above structure.....	.000		.00	.00
Water above soil.....	2.407		-7.83	-18.84
External vertical loads....	.000		.00	.00
Ext. horz. pressure loads..		.000	.00	.00
Ext. horz. line loads.....		.000	.00	.00
Structure, resisting side:				
Moist soil.....	.000		.00	.00
Saturated soil.....	1.276		-1.75	-2.23
Water above structure.....	.000		.00	.00
Water above soil.....	.000		.00	.00
Driving side:				
Effective earth loads.....		.000	.00	.00
Shear (due to delta).....	.000		.00	.00
Horiz. surcharge effects...		.000	.00	.00
Water loads.....		5.273	2.33	12.29
Resisting side:				
Effective earth loads.....		-.257	1.47	-.38
Balancing earth load.....		-2.747	-1.01	2.77
Water loads.....		-2.269	-.08	.19
Foundation:				
Vertical force on base.....	-4.588		-2.88	13.23
Uplift.....	-5.932		-6.10	36.21

==========

** Statics Check ** SUMS = .000 .000 .00

Angle of base = 10.30 degrees
Normal force on base = 5.005 kips
Shear force on base = 1.882 kips
Max. available shear force = 6.708 kips

Base pressure at x= 8.65 ft from toe = .0000 ksf
Base pressure at toe = 1.0609 ksf

Xr (measured from toe) = 2.88 ft
Resultant ratio = .2621
Stem ratio = .3182
Base in compression = 78.63 %
Overturning ratio = 1.33

Volume of concrete = 1.05 cubic yds/ft of wall

NOTE: The engineer shall verify that the computed bearing pressures below the wall do not exceed the allowable foundation bearing pressure, or, perform a bearing capacity analysis using the program CBEAR. Also, the engineer shall verify that the base pressures do not result in excessive differential settlement of the wall foundation.

****
** Sliding Results **
****

Solution converged. Summation of forces = 0.

Horizontal Vertical

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Wedge Number	Loads (kips)	Loads (kips)
1	.000	.000
2	5.273	2.407
3	.000	.000

Water pressures on wedges:

Wedge number	Top press. (ksf)	Bottom press. (ksf)	x-coord. (ft)	press. (ksf)
1	.0000	.0000		
2			.0000	.8112
2			11.0000	.3921
3	.0000	.3921		

Points of sliding plane:
Point 1 (left), x = .00 ft, y = 745.25 ft
Point 2 (right), x = 11.00 ft, y = 747.25 ft

Depth of cracking = 6.92 ft
Crack extends to bottom of base of structure.

Wedge number	Failure angle (deg)	Total length (ft)	Weight of wedge (kips)	Submerged length (ft)	Uplift force (kips)
1	.000	.000	.000	.000	.000
2	10.305	11.180	9.091	11.180	6.727
3	45.128	6.237	1.118	6.237	1.223

Wedge number	Net force (kips)
1	.000
2	-2.024
3	2.024
SUM =	.000

Factor of safety = 5.887

PDM 4/30/08
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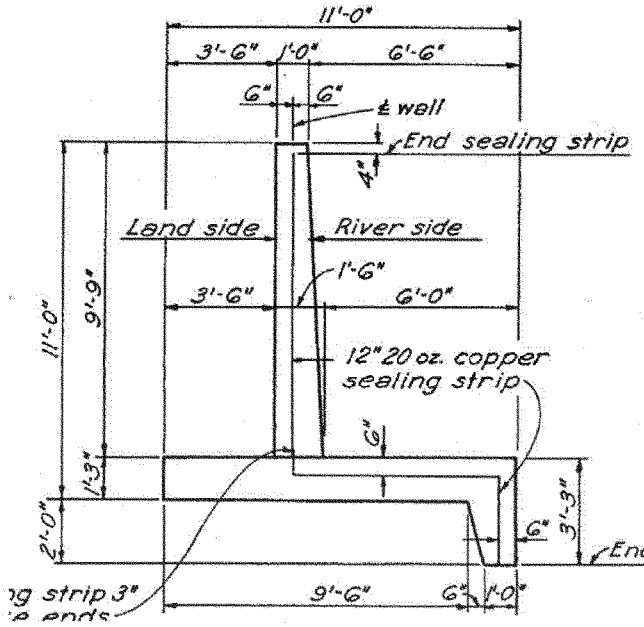
US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Footer and Wall Strength Analysis
Kansas Citys Levees

Comp by:FRS
Chkd by:[redacted]
PMiller

UNDRAINED



Typical Floodwall Section

Assumptions Taken from Record Drawings File No. A-10-1176

1. The length of the base in bearing compression must be less than (D+E+F+G+H). Otherwise, the bending moment, M_H will be incorrect.

2. Equations in this file are established for a horizontal footer with uniform thickness.

3. Resisting Side Water EL is the EL of the top of resisting grade.

4. Temperature and Shrinkage steel assumed sufficient.

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Variables

$kip := 1000lb$ $plf := \frac{lb}{ft}$ $psf \equiv \frac{lb}{ft^2}$ $ksf \equiv \frac{1000lb}{ft^2}$ $psi \equiv \frac{lb}{in^2}$ $ksi \equiv \frac{1000lb}{in^2}$ $pcf := \frac{lb}{ft^3}$

CTWALL INPUT FILE NAME: M10UNR.OUT

Elevation of top of stem (ELTS)	$ELTS := 758.25ft$
Height of stem (HTS)	$HTS := 9ft + 9in$
Thickness top of stem (TTS)	$TTS := 1.0ft$
Thickness bottom of stem (TBS)	$TBS := 1.5ft$
Dist. of batter at bot.of stem LS (TBSR)	$TBSR := 0ft$
Depth of heel (THEEL)	$THEEL := 3ft + 3in$
Distance of batter for heel (BTRH)	$BTRH := 0ft$ *
Depth of toe (TTOE)	$TTOE := 1ft + 3in$ *
Width of toe (TWIDTH)	$TWIDTH := 3ft + 6in$
Distance of batter for toe (BTRT)	$BTRT := 0ft$ *
Width of base (BWIDTH)	$BWIDTH := 11ft$
Depth of key (HK)	$HK := 2ft + 0in$ *
Width of bottom of key (TK)	$TK := 1ft$
Dist. of batter at bot. of key (BTRK)	$BTRK := 0ft + 6in$
Driving side soil elevation (ELSTDS)	$ELSTDS := 752.17ft$
Resisting side soil and water elevation (ELSTRS)	$ELSTRS := 751.67ft$
Driving side water elevation (WATELD)	$WATELD := 758.25ft$

$A := WATELD - ELSTDS$

$B := HTS - (ELTS - ELSTDS)$

$C := THEEL$

$D := TK$

$E := BTRK$

$F := BWIDTH - (TWIDTH + TBS + BTRK + TK)$

$G := TBS$

$H := TWIDTH$

$I := TTOE + BTRT$

$J := HTS - (ELTS - ELSTRS)$

$L := ELTS - ELSTRS$

$M := TTS$

* Equations in this file are established for a horizontal footer with uniform thickness.

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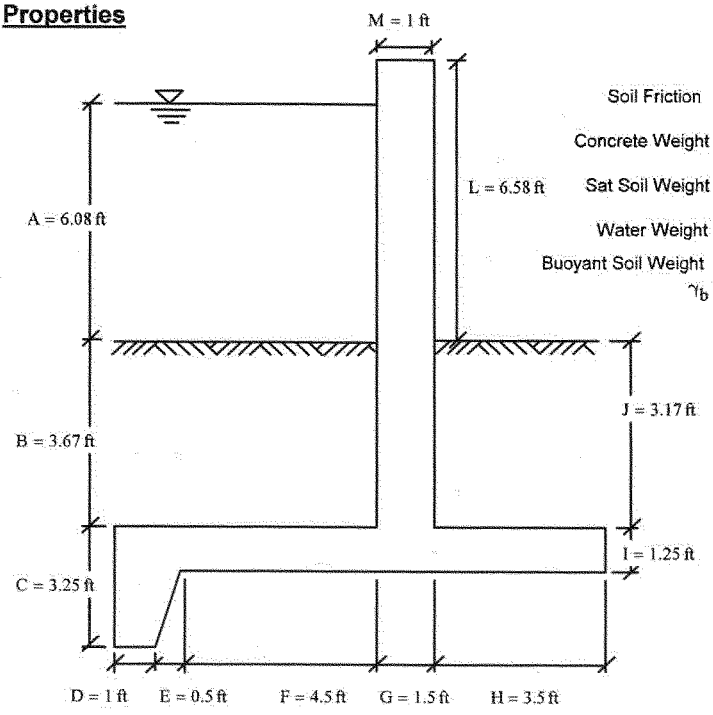
Properties



Soil Friction $\phi := 0 \cdot deg$

Concrete Weight $\gamma_c := 150pcf$

Sat Soil Weight $\gamma_{sat} := 115pcf$

Water Weight $\gamma_w := 62.4pcf$

Buoyant Soil Weight

$\gamma_b := \gamma_{sat} - \gamma_w = 52.6 \cdot pcf$

$Base := D + E + F + G + H = 11\,ft$

$Verify := if(BWIDTH = Base, "OKAY", "Check\ Dimensions")$

$Verify = "OKAY"$

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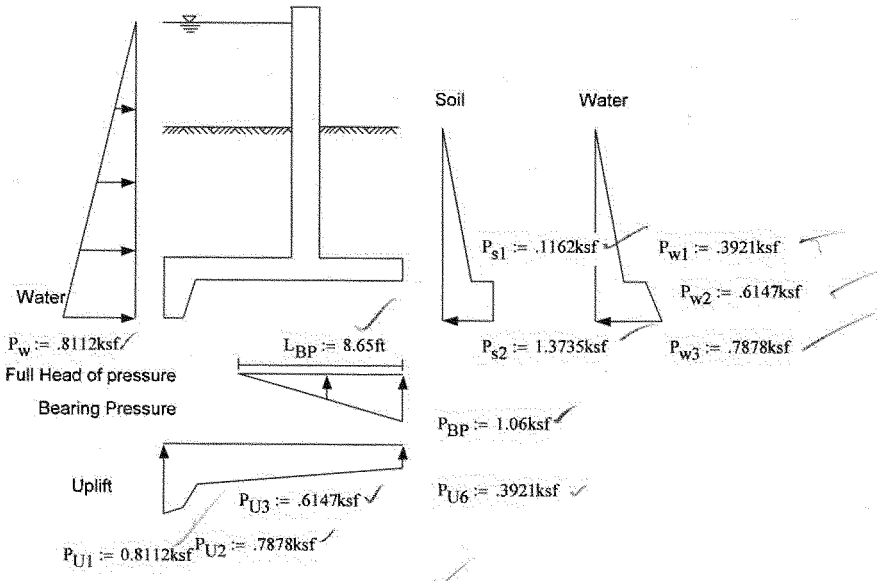
Pressures below were taken from corresponding CTWALL output.



Driving Soil Pressure at bottom of Key: $P_{s3} := 0ksf$

Driving Soil Pressure at top of footer:

$$P_{s4} := \frac{B}{B + C} \cdot P_{s3} = 0 \cdot ksf$$

Driving Soil Pressure at base of footer:

$$P_{s5} := \frac{B}{(B + I)} \cdot P_{s3} = 0 \cdot ksf$$

Note, the presence of any cutoff wall could impact the uplift pressures. Reducing the uplift would cause an uncoservative design for the heel.

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3000 psi is used conservatively until a reliability calculation is performed
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$ Steel Properties $F_y := 40.0ksi$

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Load & Resistance Factor Design

Strength Reduction Factors

Shear Strength	$\phi_V := 1.0$
Flexural Strength	$\phi_B := 1.0$

Load Factors

Dead and Live Load Factor	$\gamma_L := 1.0$
Hydraulic Load Factor	$\gamma_H := 1.0$
Extreme Case Factor	$\gamma_X := 1.0$

Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.

Load Multiplication Factor EM 1110-2-2104 (3-1)

Hydraulic Factor EM 1110-2-2104 (3-2)

Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

Reinforcement Checks

- HEEL

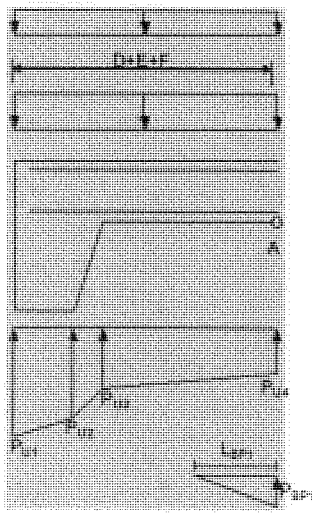
○ Location where moment is taken about.

Water

Soil

Slab

Uplift



$$W_w := \gamma_w \cdot (A + B) \cdot (D + E + F)$$

$$W_w = 3650 \frac{lb}{ft}$$

$$W_s := \gamma_b \cdot B \cdot (D + E + F)$$

$$W_s = 1158.25 \frac{lb}{ft}$$

Drag on edge of Key due to tendency to rotate:

$$\delta_p := \left[\frac{1}{2} \cdot \left(P_{s4} + P_{s3}\right) \cdot C \cdot \tan(\phi)\right] = 0$$

$$\delta_d := \text{if}\left(L_{BP} < BWIDTH, \delta_p, 0\right) = 0$$

$$P_{U4} := \left(P_{U3} - P_{U6}\right) \cdot \left(\frac{G + H}{F + G + H}\right) + P_{U6}$$

$$P_{U4} = 509.26 \frac{lb}{ft^2}$$

$$L_{BP1} := \text{if}\left[\left[L_{BP} - (G + H)\right] \geq 0ft, \left[L_{BP} - (G + H)\right], 0\right]$$

$$L_{BP1} = 3.65\,ft$$

Bearing Pressure, P_{BP1} acting on the heel at location "A"

$$P_{BP1} := \frac{L_{BP1}}{L_{BP}} \cdot P_{BF} \qquad P_{BP1} = 447 \frac{lb}{ft^2}$$

Heel and Key Centroid

$$X_H := \frac{(D + E + F) \cdot I \cdot \left(\frac{D + E + F}{2}\right) + (C - I) \cdot D \cdot \left(E + F + \frac{D}{2}\right) + \left[\frac{(C - I) \cdot E}{2}\right] \cdot \left(F + \frac{2 \cdot E}{3}\right)}{(D + E + F) \cdot I + (C - I) \cdot D + \frac{(C - I) \cdot E}{2}} \qquad X_H = 3.59\,ft$$

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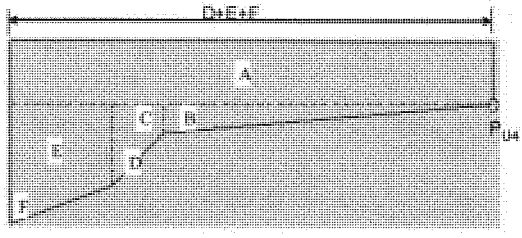
- HEEL check (cont'd)

Heel and Key Weight

$$W_H := \left[(D + E + F)\cdot I + (C - I)\cdot D + \frac{(C - I)\cdot E}{2}\right]\cdot \gamma_c \qquad W_H = 1500\,\frac{lb}{ft}$$

Uplift Centroid



AREA	CENTROID
$A_A := (D + E + F)\cdot P_{U4}$	$C_A := \frac{D + E + F}{2}$
$A_B := \frac{F\cdot(P_{U3} - P_{U4})}{2}$	$C_B := \frac{2\cdot F}{3}$
$A_C := E\cdot(P_{U3} - P_{U4})$	$C_C := F + \frac{E}{2}$
$A_D := \frac{E\cdot(P_{U2} - P_{U3})}{2}$	$C_D := F + \frac{2\cdot E}{3}$
$A_E := D\cdot(P_{U2} - P_{U4})$	$C_E := \left(\frac{D}{2} + E + F\right)$
$A_F := \frac{D\cdot(P_{U1} - P_{U2})}{2}$	$C_F := \left(\frac{2\cdot D}{3} + E + F\right)$

$$X_U := \frac{A_A\cdot C_A + A_B\cdot C_B + A_C\cdot C_C + A_D\cdot C_D + A_E\cdot C_E + A_F\cdot C_F}{A_A + A_B + A_C + A_D + A_E + A_F} \qquad X_U = 3.24\,ft$$

Uplift on Heel $\quad W_U := A_A + A_B + A_C + A_D + A_E + A_F \qquad W_U = 3679\,\frac{lb}{ft}$

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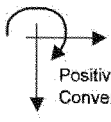
- HEEL check (cont'd)

Loading: Note, since the amount of base in compression is > 75%, bearing of water-side of key against soil dut to rotation should be marginally different from counter-moment produced from sliding force on key. Water loads on each side of key are also similar and so their effects on the heel are ignored.

Bending

$$M_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[W_U \cdot X_U + \frac{P_{BP1} \cdot L_{BP1}}{2} \cdot \frac{L_{BP1}}{3} - \left[(W_w + W_s) \cdot \left(\frac{D + E + F}{2} \right) + W_H \cdot X_H + \delta_d \cdot (D + E + F) \right] \right]$$



Positive Sign Convention

$$M_H = -6884.06 \cdot \frac{ft \cdot lb}{ft}$$

Note:
IF: M_H is < 0
THEN: Steel in top of heel is in tension

$$M_{uH} := |M_H| \qquad M_{uH} = 6.9 \cdot \frac{kip \cdot ft}{ft}$$

Shear

$$V_H := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(W_w + W_s + \delta_d + W_H - W_U - \frac{P_{BP1} \cdot L_{BP1}}{2} \right)$$

$$V_H = 1813.33 \frac{lb}{ft} \qquad V_{uH} := |V_H| \qquad V_{uH} = 1.8 \cdot \frac{kip}{ft}$$

Capacity:
Flexural Capacity

Top Steel: $A_{Heel1} := .44 \frac{in^2}{ft}$ $C_{CH1} := 3.375in$ Bottom Steel: $A_{Heel2} := .35 \frac{in^2}{ft}$ $C_{CH2} := 3.375in$

$A_{s1} := A_{Heel1}$ $d_{H1} := I - C_{CH1}$ $A_{s2} := A_{Heel2}$ $d_{H2} := I - C_{CH2}$

$b := 12 \frac{in}{ft}$ $a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $a1 = 0.58 \cdot in$ $b_c := 12 \frac{in}{ft}$ $a2 := \frac{A_{s2} \cdot F_y}{0.85 f'_c \cdot b_c}$ $a2 = 0.46 \cdot in$

$$\phi M_{H1} := \phi_B A_{s1} \cdot F_y \cdot \left(d_{H1} - \frac{a1}{2} \right) \qquad \phi M_{H2} := \phi_B A_{s2} \cdot F_y \cdot \left(d_{H2} - \frac{a2}{2} \right)$$

$$\phi M_{H1} = 16.63 \cdot \frac{kip \cdot ft}{ft} \qquad \phi M_{H2} = 13.3 \cdot \frac{kip \cdot ft}{ft}$$

$$\phi M_H := if(M_H > 0, \phi M_{H2}, \phi M_{H1}) \qquad \phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_{c1} := \phi_V \cdot 2 \cdot b \cdot d_{H1} \cdot \sqrt{f'_c \cdot psi} \qquad \phi V_{c2} := \phi_V \cdot 2 \cdot b_c \cdot d_{H2} \cdot \sqrt{f'_c \cdot psi}$$

$$\phi V_{c1} = 15281 \frac{lb}{ft} \qquad \phi V_{c2} = 15281 \frac{lb}{ft}$$

$$\phi V_H := if(M_H > 0, \phi V_{c2}, \phi V_{c1}) \qquad \phi V_H = 15.28 \cdot \frac{kip}{ft}$$

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- HEEL check (cont'd)

Factors of Safety

Bending $\phi M_H = 16.63 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_1 := \frac{\phi M_H}{M_{uH}}$ $\quad FS_1 = 2.42$

$M_{uH} = 6.88 \cdot \frac{kip \cdot ft}{ft}$

$Check1 := if(\phi M_H > 1.5 M_{uH}, "OKAY", "NO GOOD")$ $\quad Check1 = "OKAY"$

Shear $\phi V_H = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_2 := \frac{\phi V_H}{V_{uH}}$ $\quad FS_2 = 8.43$

$V_{uH} = 1.81 \cdot \frac{kip}{ft}$

$Check2 := if(\phi V_H > 1.5 V_{uH}, "OKAY", "NO GOOD")$ $\quad Check2 = "OKAY"$

Controlling Factor of Safety

$FS_H := \min(FS_1, FS_2)$ $\quad FS_H = 2.42$

Controlling Mechanism

$Comment_H := if(FS_1 > FS_2, "Shear in Heel", if(M_H > 0, "Flexural Bottom Steel in Heel", "Flexural Top Steel in Heel"))$

$Comment_H = "Flexural Top Steel in Heel"$

Key Check

Hydrostatic Load is similar to each side of key, and thus neglected in analysis.

$M_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[(P_{s5} - P_{s2}) \cdot \frac{(C-I)^2}{2} + \frac{1}{2} \cdot (P_{s3} - P_{s5}) \cdot \frac{2(C-I)^2}{3} \right] = -2.75 \cdot kip \cdot \frac{ft}{ft}$ $\quad M_K > 0$ means water-side steel is in tension.

$V_K := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[(P_{s5} - P_{s2}) \cdot (C-I) + \frac{1}{2} (P_{s3} - P_{s5}) \cdot (C-I) \right] = -2747 \frac{lb}{ft}$ $\quad M_{uK} := |M_K| = 2747 lb$

$V_{uK} := |V_K| = 2747 \frac{lb}{ft}$

Steel in Tension: $A_{Key} := 0.16 \frac{in^2}{ft}$ $\quad C_K := 3.375 in$ $\quad d_K := (D + E) - C_K$

$b := 12 \frac{in}{ft}$ $\quad a_k := \frac{A_{Key} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a_k = 0.21 \cdot in$ $\quad \phi M_K := \phi_B A_{Key} \cdot F_y \cdot \left(d_K - \frac{a_k}{2} \right)$ $\quad \phi M_K = 7.74 \cdot \frac{kip \cdot ft}{ft}$

Shear:

$\phi V_K := \phi_V \cdot 2 \cdot b \cdot d_K \cdot \sqrt{f'_c \cdot psi}$ $\quad \phi V_K = 19225 \frac{lb}{ft}$

$FS_{KM} := \frac{\phi M_K}{M_K} = -2.82$ $\quad Check_M := if(\phi M_K > 1.5 M_{uK}, "OKAY", "NO GOOD")$

$Check_M = "OKAY"$

$FS_{KV} := \frac{\phi V_K}{V_K} = -7$ $\quad Check_V := if(\phi V_K > 1.5 V_{uK}, "OKAY", "NO GOOD")$

$Check_V = "OKAY"$

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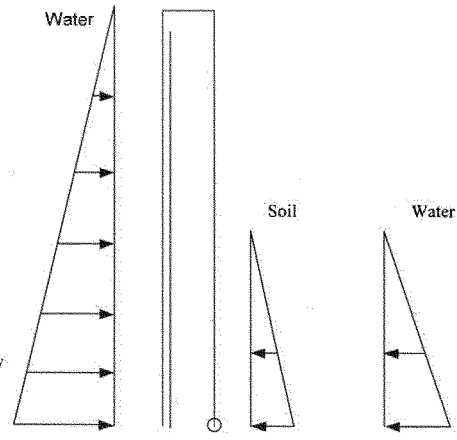
- STEM Check



Note:
P_{ss2} = Resistive Soil pressure present at base of stem
P_{ws2} = Resistive Water pressure present at base of stem

$P_{ws1} := (A + B)\cdot\gamma_w$

$P_{ws1} = 608\,\frac{lb}{ft^2}$

$P_{ss2} := \frac{J}{I + J}\cdot P_{s1}$ $\qquad P_{ws2} := J\cdot\gamma_w$

Driving Soil Pressure at base of Stem:
$P_{ss1} := P_{s4} = 0\cdot ksf$

$P_{ss2} = 0.08\cdot ksf$ $\qquad P_{ws2} = 0.2\cdot ksf$

Waterside Steel: $A_{Stem1} := .51\,\frac{in^2}{ft}$ $\qquad C_{CS1} := 2.81in$ $\qquad d_{S1} := G - C_{CS1}$

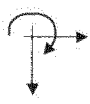
Resistive Side Steel: $A_{Stem2} := .20\cdot\frac{in^2}{ft}$

Loading:

Bending

$$M_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2}\cdot\frac{A + B}{3} + \frac{P_{ss1}\cdot B^2}{2\cdot 3} - \frac{(P_{ss2} + P_{ws2})\cdot(J^2)}{2\cdot 3}\right]$$

Note:
IF: M_{uS} is > 0
THEN: Riverside steel is in tension



$M_{uS} = 9.2\,\frac{kip\cdot ft}{ft}$

Positive Sign Convention

Shear

$$V_{uS} := \gamma_L\cdot\gamma_H\cdot\gamma_X\cdot\left[\frac{P_{ws1}\cdot(A + B)}{2} + \frac{P_{ss1}\cdot B}{2} - \frac{(P_{ss2} + P_{ws2})\cdot(J)}{2}\right]$$

$V_{uS} = 2.5\,\frac{kip}{ft}$

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- STEM Check (Cont'd)

Capacity:

Flexural Capacity

$A_{s1} := A_{Stem1}$

$b := 12\frac{in}{ft}$

$a1 := \frac{A_{s1} \cdot F_y}{0.85 f'_c \cdot b}$ $\quad a1 = 0.67 \cdot in$

$\phi M_S := \phi_B A_{s1} \cdot F_y \cdot \left(d_{S1} - \frac{a1}{2}\right)$ $\quad \phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$

Shear Capacity

$\mu := 1.0$

$\phi V_S := \phi_V \cdot A_{Stem2} \cdot F_y \cdot \mu = 8000\frac{lb}{ft}$ $\quad \phi V_S = 8 \cdot \frac{kip}{ft}$

Factors of Safety

Bending

$\phi M_S = 25.26 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 := \frac{\phi M_S}{M_{uS}}$

$M_{uS} = 9.17 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_3 = 2.75$

$Check3 := if\left(\phi M_S > 1.5 M_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check3 = "OKAY"$

Shear

$\phi V_S = 8 \cdot \frac{kip}{ft}$ $\quad FS_4 := \frac{\phi V_S}{V_{uS}}$

$V_{uS} = 2.52 \cdot \frac{kip}{ft}$ $\quad FS_4 = 3.17$

$Check4 := if\left(\phi V_S > 1.5 V_{uS}, "OKAY", "NO GOOD"\right)$ $\quad Check4 = "OKAY"$

Controlling Factor of Safety

$FS_S := \min\left(FS_3, FS_4\right)$ $\quad FS_S = 2.75$

Note: No bar cut-offs checks are made for the reliability Study.

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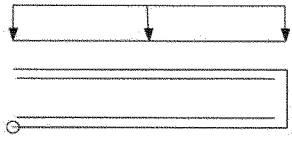
- TOE Check

Soil

Slab

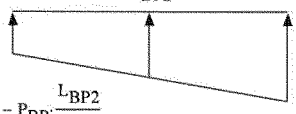


$W_s := \gamma_{sat} \cdot H \cdot (J)$ $\quad W_s = 1275.92 \frac{lb}{ft}$

$L_{BP2} := if(H \le L_{BP}, H, L_{BP})$

$L_{BP2} = 3.5\,ft$

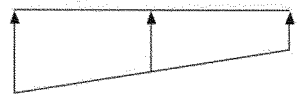
Bearing Pressure



$P_{BP2} := P_{BP} - P_{BP} \cdot \frac{L_{BP2}}{L_{BP}}$ $\quad P_{BP} = 1060 \frac{lb}{ft^2}$

$P_{BP2} = 631.1 \frac{lb}{ft^2}$

Uplift



$P_{U6} = 392.1 \frac{lb}{ft^2}$

$$P_{U5} := \left[\left[\left(P_{U3} - P_{U6}\right) \cdot \frac{H}{F + G + H} + P_{U6}\right]\right]$$

$P_{U5} = 474.11 \frac{lb}{ft^2}$

Bearing Pressure

$$L_{bp} := \frac{\left(L_{BP2} \cdot P_{BP2}\right) \cdot \left(\frac{L_{BP2}}{2}\right) + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2} \cdot \left(\frac{2L_{BP}}{3}\right)}{L_{BP2} \cdot P_{BP2} + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2}}$$

$L_{bp} = 2.77\,ft$

$$W_{bp} := L_{BP2} \cdot P_{BP2} + \frac{\left(P_{BP} - P_{BP2}\right) \cdot \left(L_{BP2}\right)}{2}$$

$W_{bp} = 2959 \frac{lb}{ft}$

Uplift

$$L_u := \frac{H \cdot P_{U6} \cdot \frac{H}{2} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2} \cdot \frac{H}{3}}{H \cdot P_{U6} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2}}$$

$L_u = 1.69\,ft$

$$W_u := H \cdot P_{U6} + \frac{\left(P_{U5} - P_{U6}\right) \cdot H}{2}$$

$W_u = 1515.87 \frac{lb}{ft}$

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Toe Check (continued)

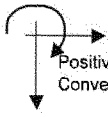
Loading:

Bending

$$M_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[\left[W_s + (H \cdot I) \cdot \gamma_c \right] \cdot \frac{H}{2} - \left(W_{bp} \cdot L_{bp} + W_u \cdot L_u \right) \right]$$

Note:
IF: $M_T < 0$
THEN: Bottom steel is in tension

$$M_T = -7.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{uT} := \left| M_T \right| \qquad M_{uT} = 7.38 \cdot \frac{kip \cdot ft}{ft}$$



Shear

$$V_T := \gamma_L \cdot \gamma_H \cdot \gamma_X \left[W_s + (H \cdot I) \cdot \gamma_c - W_{bp} - W_u \right]$$

$$V_T = -2.54 \cdot \frac{kip}{ft} \qquad V_{uT} := \left| V_T \right| \qquad V_{uT} = 2.54 \cdot \frac{kip}{ft}$$

Capacity:

Flexural Capacity

Area of bottom (tension) steel: $A_{Toe} := .352 \frac{in^2}{ft}$ $\quad C_{CT} := 3.375in$ $\quad d_T := I - C_{CT} = 11.63 \cdot in$

$$b := 12 \frac{in}{ft} \qquad A_{s1} := A_{Toe}$$

$$a1 := \frac{A_{s1} \cdot F_y}{0.85 f_c \cdot b} \qquad a1 = 0.46 \cdot in$$

$$\phi M_T := \phi_B A_{s1} \cdot F_y \cdot \left(d_T - \frac{a1}{2} \right) \qquad \phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$$

Shear Capacity

$$\phi V_T := \phi_V \cdot 2 \cdot b \cdot d_T \sqrt{f_c \cdot psi} \qquad \phi V_T = 15.28 \cdot \frac{kip}{ft}$$

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Toe Check (continued)

Factors of Safety

Bending

$\phi M_T = 13.37 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_5 := \frac{\phi M_T}{M_{uT}}$

$M_{uT} = 7.38 \cdot \frac{kip \cdot ft}{ft}$ $\quad FS_5 = 1.81$

$Check5 := if(\phi M_T > 1.5 \cdot M_{uT}, "OKAY", "NO\ GOOD")$ $\quad Check5 = "OKAY"$

Shear

$\phi V_T = 15.28 \cdot \frac{kip}{ft}$ $\quad FS_6 := \frac{\phi V_T}{V_{uT}}$

$V_{uT} = 2.54 \cdot \frac{kip}{ft}$ $\quad FS_6 = 6.01$

$Check6 := if(\phi V_T > 1.5 V_{uT}, "OKAY", "NO\ GOOD")$ $\quad Check6 = "OKAY"$

Controlling Factor of Safety

$FS_T := \min(FS_5, FS_6)$ $\quad FS_T = 1.81$

End of Analysis

US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Analyzed at Top of Footer
Kansas Citys Levees

Comp by: Sheffield
Chkd by: K. Marx 9/28/11

281/437

- Guide Post Strength Analysis

Tributary length of load,

$$B := \frac{24.02ft}{2}$$

Depth of Section, $D := 30in$

Width of section, $b := 24in$
Width for shear, $b_s := 17in$

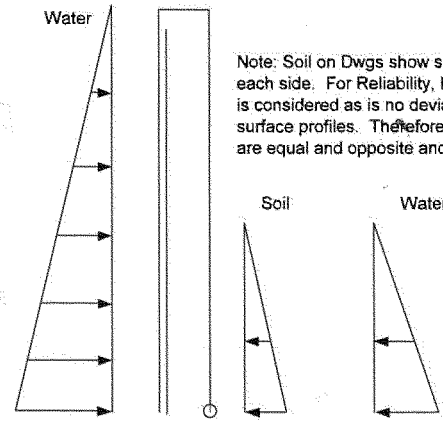
Height of water driving:

$$H_{w1} := 6ft + 7in$$

Note: Vertical Cracks are assumed on each side of Guide Post, reducing sill's ability to resist load.

Full Height of Guide:

$$H_w := 9ft + 9in$$



$\gamma_w := 62.4pcf$

Note: Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Height of water resisting:

$$H_{w2} := 3ft + 2in$$

Resistance at Footer:

$$w_{w1} := (H_{w1} \cdot \gamma_w \cdot B)$$

$$w_{w1} = 4934 \cdot plf$$

4 No 7 $A_s := 2.41in^2$

$$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{7}{8}\right)in \quad d_1 := D - C_1$$

$$d_1 = 27.0625 \cdot in$$

$$w_{w2} := H_{w2} \cdot \gamma_w \cdot b$$

$$w_{w2} = 395 \cdot plf$$

6 No 4 Area of dowels, $A_v := 1.18in^2$

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Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3750 psi is only used for the median strength in reliability calcs; 3000 psi is used for the following initial calculations.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 40.0ksi$ $E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	

Load Factors:

Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

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- Flexural Check of Stop Log Guide

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{1}{2} \cdot w_{w1} \cdot H_{w1} \cdot \left[\frac{H_{w1}}{3} + \left(H_w - H_{w1}\right)\right] + \frac{1}{2} H_{w1} \cdot \gamma_w \cdot b \cdot \left(H_w - H_{w1}\right)^2 + \frac{1}{6} \cdot \left(H_w - H_{w1}\right)^3 \cdot \gamma_w \cdot b - \frac{1}{6} \cdot w_{w2} \cdot H_{w2}^{\ 2}\right]$$

$M_u = 91.2 \cdot kip \cdot ft$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[\frac{1}{2} \cdot w_{w1} \cdot H_{w1} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2} + H_{w1} \cdot \gamma_w \cdot \left(H_w - H_{w1}\right) \cdot b + \frac{1}{2} \cdot \left(H_w - H_{w1}\right)^2 \cdot \gamma_w \cdot b\right]$$

$V_u = 18.8419 \cdot kip$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\quad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\quad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_1}{b \cdot d_1} = 0.0037$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Not Good; however, requirement need not be met if As provided is at least one-third greater than required. So for flexural capacity, 3/4 factor will be applied.

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_1 F_y}{0.85 \cdot f_c \cdot b} = 1.5752 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[\frac{3}{4} A_1 F_y \cdot \left(d_1 - \frac{a}{2}\right)\right] = 158.3064 \cdot kip \cdot ft$$

3/4 As is used since ρ_{min} is not met.

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- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity: $\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_n := \phi_V \cdot (A_v \cdot F_y \cdot \mu) = 47200\,lbf$ $\phi V_n = 47 \cdot kip$

FACTORS OF SAFETY

Bending:

$\phi M_n = 158.3064 \cdot kip \cdot ft$

$M_u = 91.1845 \cdot kip \cdot ft$

$FS_B := \frac{\phi M_n}{M_u} = 1.7361$ Check := if($FS_B > 1.5$, "OKAY", "NO GOOD") $FS_B = 1.7$ Check = "OKAY"

Shear:

$\phi V_n = 47.2 \cdot kip$

$V_u = 18.8419 \cdot kip$

$FS_V := \frac{\phi V_n}{V_u}$ Check1 := if($FS_V > 1.5$, "OKAY", "NO GOOD") $FS_V = 2.5051$ Check1 = "OKAY"

Controlling Factor of Safety

$FS_S := \min(FS_B, FS_V)$ $FS_S = 1.7$

US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap

Comp by: Sheffield 285/437
Chkd by: K. Marx 9/28/11

Kansas Citys Levees

- Guide Post Strength Analysis

Tributary length of load,

$$B := \frac{24.02ft}{2}$$

Depth of Section,

Width of section,

Width for shear, $b_s := 17in$

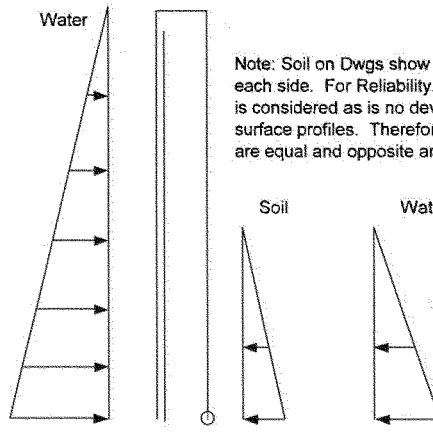
Height of water driving:

$$H_{w1} := 6ft + 7in$$

Note: Width of reinforced sill is assumed to strengthen abutment such that its small height and additional load will not control.

$$w_{w1} := H_{w1} \cdot \gamma_w \cdot B$$

$$w_{w1} = 4934 \cdot plf$$



$$\gamma_w := 62.4pcf$$

Note: Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Height of water resisting:

$$H_{w2} := 0ft$$

$$w_{w2} := H_{w2} \cdot \gamma_w \cdot b$$

$$w_{w2} = 0 \cdot plf$$

4 No 7

$$A_s := 2.41in^2$$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

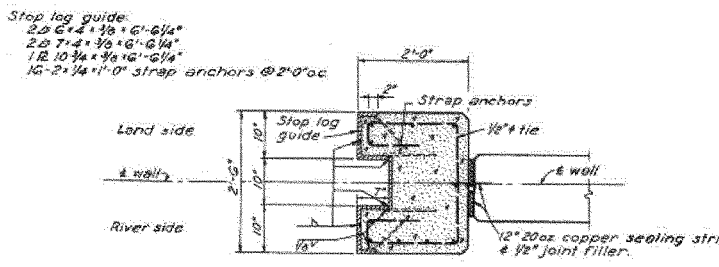
$$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{7}{8}\right)in$$

$$d_1 := D - C_1$$

$$d_1 = 27.0625 \cdot in$$

Area of stirrup, $A_v := .20in^2 \cdot 2$

Stirrup spacing, $s_s := 12in$



From Record Drawings
File No. A-10-1176

286/437

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete. As concrete ages it continues to hydrate and gain compressive strength. FEMA 310, Section 4.2.4.4, states, "Unless calculated otherwise the expected strength shall be assumed equal to the nominal strength multiplied by 1.25." Using FEMA's guidance the concrete strength becomes 3750 psi. However, 3750 psi is only used for the median strength in reliability calcs; 3000 psi is used for the following initial calculations.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $f'_c := 3000 \cdot psi$

Steel Properties $F_y := 40.0ksi$ $E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	

Load Factors:

Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

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- Flexural Check of Stop Log Guide

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{w1} \cdot H_{w1} \cdot \frac{H_{w1}}{3} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2} \cdot \frac{H_{w2}}{3}\right) \qquad M_u = 35.6 \cdot kip \cdot ft$$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{w1} \cdot H_{w1} - \frac{1}{2} \cdot w_{w2} \cdot H_{w2}\right) \qquad V_u = 16.2401 \cdot kip$$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\qquad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\qquad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_l}{b \cdot d_l} = 0.0037$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Guide Post does not meet Minimum Steel Ratio, ρ_{min}, thus should the section crack, immediate failure may occur!

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_l F_y}{0.85 \cdot f'_c \cdot b} = 1.5752 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[A_l F_y \cdot \left(d_l - \frac{a}{2}\right)\right] = 211.0752 \cdot kip \cdot ft$$

Since ρ_{min} is not met, compute ϕM_n based upon Unreinforced Section (ACI 22.5):

$$S_m := \frac{b \cdot D^2}{6} = 3600 \cdot in^3$$

$$\phi M_n := 5 \cdot \sqrt{f'_c \cdot psi} \cdot S_m = 82.1584 \cdot kip \cdot ft$$

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- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity:

$$\phi V_n := \phi_V \cdot \left(2 \cdot b_s \cdot d_1 \cdot \sqrt{f'_c \cdot psi} + \frac{A_v \cdot F_y \cdot d_1}{s_s}\right)$$ $\phi V_n = 86 \cdot kip$

FACTORS OF SAFETY

Bending:

$\phi M_n = 82.1584 \cdot kip \cdot ft$

$M_u = 35.638 \cdot kip \cdot ft$

$FS_B := \frac{\phi M_n}{M_u}$ Check := if(FS_B > 1.5, "OKAY", "NO GOOD") $FS_B = 2.3$ Check = "OKAY"

Shear:

$\phi V_n = 86.4807 \cdot kip$

$V_u = 16.2401 \cdot kip$

$FS_V := \frac{\phi V_n}{V_u}$ Check1 := if(FS_V > 1.5, "OKAY", "NO GOOD") $FS_V = 5.3251$ Check1 = "OKAY"

Controlling Factor of Safety

$FS_S := \min(FS_B, FS_V)$ $FS_S = 2.3$

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US Army Corps of Engineers

CID Missouri
Monolith 10 Stoplog Gap
Sill Analysis

Kansas Citys Levees

Comp by: Sheffield
Chkd by: K. Marx 9/28/11

Analysis will be performed on a per foot basis. Soil on Dwgs show same EL each side. For Reliability, No Passive is considered as is no deviation from surface profiles. Therefore soil loads are equal and opposite and ignored.

Depth of Section, $D := 30in$

$\gamma_w := 62.4pcf$

Height to top of sill:

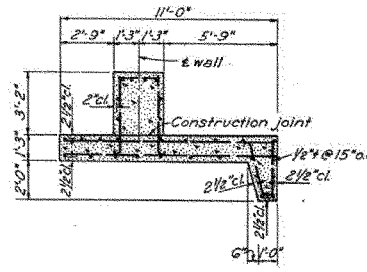
$H_{w1} := 6ft + 7in$ 6.58 $\quad w_{w1} := H_{w1} \cdot \gamma_w = 410.8 \cdot psf$

Height of sill:

$h_{sill} := 3ft + 2in$ 9.75

$H_T := H_{w1} + h_{sill} = 9.75\,ft$

$w_{wT} := H_T \cdot \gamma_w = 608.4 \cdot psf$



No 4 @ 12" OC $\quad A_1 := 0.20in^2$

Note: Compression steel is ignored since $\rho < 0.75\,\rho_b$

$C_1 := 2in + \frac{1}{2}in + \frac{1}{2} \cdot \left(\frac{4}{8}\right)in$

$d_1 := D - C_1 \qquad d_1 = 27.25 \cdot in$

From Record Drawings
File No. A-10-1176

Area of dowels, $\quad A_v := .20in^2$ $\quad$ No 4 @ 12" OC

Assumptions

- Concrete strengths were not specified in any of the information available and ACI recommends the use of 3000 psi nominal concrete strengths for older concrete.
- The Portland Cement Association pamphlet, *Engineered Concrete Structures*, Dec. 1997 Vol. 10 No. 3. recommends using 40 ksi yield strengths for rebar of this time period.

Concrete Properties $\quad f'_c := 3000 \cdot psi$

Steel Properties $\quad F_y := 40.0ksi \qquad E_s := 29000ksi$

Load & Resistance Factor Design

Strength Reduction Factors:

Shear Strength	$\phi_V := 1$	Note: Strength Reduction Factors (.85 for shear, 0.90 for bending) and Load Factors (1.6 live load and 1.3 for hydraulic structures) not applied in analysis of existing conditions.
Flexural Strength	$\phi_B := 1$	
Load Factors:		
Dead and Live Load Factor	$\gamma_L := 1.0$	Load Multiplication Factor EM 1110-2-2104 (3-1)
Hydraulic Load Factor	$\gamma_H := 1$	Hydraulic Factor EM 1110-2-2104 (3-2)
Extreme Case Factor	$\gamma_X := 1.0$	Short Duration (Extreme Condition) EM 1110-2-2104 (3-4)

All set to equal one for existing conditions reliability analysis

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- Flexural Check of Sill:

DEMAND

Bending:

$$M_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left[w_{w1} \cdot \frac{(H_T - H_{w1})^2}{2} + \frac{1}{2} \cdot (w_{wT} - w_{w1}) \cdot \frac{(H_T - H_{w1})^2}{3} \right] \cdot 12in \quad M_u = 2.4 \cdot kip \cdot ft$$

Shear:

$$V_u := \gamma_L \cdot \gamma_H \cdot \gamma_X \cdot \left(\frac{1}{2} \cdot w_{wT} \cdot H_T - \frac{1}{2} \cdot w_{w1} \cdot H_{w1} \right) \cdot 12in \qquad V_u = 1.6137 \cdot kip$$

CAPACITY

Flexural Capacity (of tension steel only):

For fy = 40,000 psi and f'c = 3,000 psi:

$roe_b := 0.0371$ $\quad roe_{max} := \frac{1}{2} \cdot roe_b = 0.0186$ $\quad roe_{min} := 0.0050$

Note: Compression steel is ignored since $\rho < 0.75\ \rho_b$

$$roe := \frac{A_1}{12in \cdot d_1} = 0.0006$$

Check Steel Ratios (0 is a false Statement, 1 is a true Statement):

$roe > roe_{min} = 0$ Not Good; however, requirement need not be met if As provided is at least one-third greater than required. So for flexural capacity, a 3/4 factor will be applied.

$roe \le roe_{max} = 1$ OK

Calculate Capacity:

$$a := \frac{A_1 F_y}{0.85 \cdot f_c \cdot 12in} = 0.2614 \cdot in$$

$$\phi M_n := \phi_B \cdot \left[\frac{3}{4} A_1 F_y \cdot \left(d_1 - \frac{a}{2} \right) \right] = 13.5596 \cdot kip \cdot ft$$

351/437

- Flexural Check of Stop Log Guide (cont'd)

Shear Capacity: $\mu := 1.0$ For Concrete placed against roughened concrete.

$\phi V_n := \phi_V \cdot (A_v \cdot F_y \cdot \mu) = 8000\,lbf$ $\phi V_n = 8 \cdot kip$

FACTORS OF SAFETY

Bending:

$\phi M_n = 13.5596 \cdot kip \cdot ft$

$M_u = 2.39 \cdot kip \cdot ft$

$FS_B := \frac{\phi M_n}{M_u}$ $Check := if(FS_B > 1.5, "OKAY", "NO GOOD")$ $FS_B = 5.7$ $Check = "OKAY"$

Shear:

$\phi V_n = 8 \cdot kip$

$V_u = 1.6137 \cdot kip$

$FS_V := \frac{\phi V_n}{V_u}$ $Check1 := if(FS_V > 1.5, "OKAY", "NO GOOD")$ $FS_V = 4.9574$ $Check1 = "OKAY"$

Controlling Factor of Safety

$FS_S := min(FS_B, FS_V)$ $FS_S = 5$

Chapter A-9

KANSAS RIVER BRIDGES

CHAPTER A-9
KANSAS RIVER BRIDGES

A-9.1 INTRODUCTION

There are a number of bridges over the Kansas River within the Kansas Citys Protection System (Armourdale, CID-Kansas, and Argentine) that currently have low beam below the top of existing levee. A couple of these structures were modified in the 1960s to be raised in a flood event, and one or two were permanently raised. The KC Terminal Railroad Bridge can no longer be raised because the raise mechanism was not maintained by the owner. As noted in the tables included in this report as Exhibits A-9.1 through A-9.4 in the Supplemental Exhibits section at the end of this chapter, the depth of water varies for each of the bridges over the Kansas River and that depth increases from existing to future with project conditions. Increasing superstructure depth underwater increases load due to flow on the structure. The superstructures, substructures, and foundations for the Kansas River bridges were most likely not originally designed for stream flow lateral loads that occur during a 500-year event. Impacts to the bridges as a result of the future with project levee raise, as compared to existing conditions, is assessed in the Feasibility Report by doing a qualitative assessment of the failure modes. Probability of failures are then assigned to the critical structures. A detailed structural analysis of each bridge was not completed due to the excessive cost of doing so. Some background information on decisions and assumptions made follow within this chapter. Exhibits A-9.1 through A-9.4 provide specific background to the assessments and corresponding structural failure rating.

HNTB performed a bridge reconnaissance and provided the COE with a letter of opinion in November 2008. During the reconnaissance, a significant amount of bridge data was collected and a list of potential failure modes was developed. The 5 failure modes in the HNTB letter of opinion are used in this qualitative assessment of the bridges. The HNTB letter is included as Exhibit A-9.5 at the end of this chapter.

A-9.2 FAILURE MODES

A-9.2.1 Structural Failure

Two structural failure modes were identified in the HNTB letter of opinion and have been modified slightly here to account for the fact that overstress is not a failure mode. Overstress is considered to be unacceptable damage, but not necessarily bridge failure.

1. Superstructure Failure (Bearing and anchor bolt failure) - The analysis evaluates the bearings and anchor bolts for failure that result from the uplift and lateral hydraulic forces on the superstructure.
2. Substructure Failure (Pier section shear and bending) - The analysis evaluates the pier section for shear and bending failure that result from the lateral forces.

A-9.2.2 Geotechnical Failure

Three geotechnical failure modes were identified in the HNTB letter of opinion and have been modified slightly here to account for the fact that overstress is not a failure mode. Overstress is considered to be unacceptable damage, but not necessarily bridge failure.

1. Substructure Failure (Overturning) - The analysis evaluates the substructure for exceeding ultimate overturning resistance that results from lateral hydraulic forces.
2. Substructure Failure (Sliding) - The analysis evaluates the substructure for exceeding ultimate sliding resistance that results from lateral forces.
3. Foundation Failure - The analysis evaluates the substructure foundation for failure that results from the lateral forces. Tension or pile capacity of the timber piles as well as bearing capacity of the piers on rock are to be evaluated as part of the foundation failure assessment.

Although scour was not listed as a specific failure mode, scour does impact the probability of the 3 geotechnical failure modes. Existing conditions scour ratings were available from the bridge underwater inspection reports and are as indicated in Exhibits A-9.1 through A-9.4. It was assumed that the future scour rating will be reduced by 1 for the future with project condition because scour will likely get worse with increased velocities. It is assumed that the future scour rating is not impacted by general river degradation. It is assumed that river degradation will happen over a long period of time and repairs or modifications will have been made over the life of the structure and will not be directly impacted by the Kansas Citys levees future with project condition.

A-9.3 BRIDGE FAILURE ASSUMPTIONS AND IMPACTS TO LEVEE FAILURE

There are 4 bridges that could fall on and damage floodwalls in an event of the bridge superstructure failure. Of the 4, only 2 of these bridges are underwater for the N500+3 water surface elevations. The Kansas Avenue East Bridge and the I-670 Bridge are the only two bridges that should be considered as possibly causing levee failure as a direct result of bridge failure. The bridges were assumed not to have superstructure spans over flood protection if:

- The superstructure is over a floodwall and the portion of the superstructure over the floodwall would not be underwater in an event
- The superstructure is over earthen levee, because it was assumed that if the structure were to fall on the earthen levee, it will not fail the levee.
- Piers on overbanks are in the levee footprint, because it is assumed the overbank piers would not see enough force to pullout and fail the earthen levee.
- Ends of the bridge are stoplog gaps, because it is assumed that superstructure failure will not fail the gap.

The Superstructure failure mode by failure of bearing anchor bolts is assumed not to impact the bridge abutment structural integrity. It is also assumed that the abutment or gap at this location will not fail if the superstructure fails. The substructure failure (structural and geotechnical) is assumed to occur at the river piers because these piers are typically the most prone to scour and damage during a flood.

There is a high probability of one bridge that has failed causing another bridge downstream to fail as well. The first failed bridge would cause a dam effect upstream of the second bridge. This would increase the pressure on the second bridge causing failure to be more likely than if the first bridge had not failed.

Ultimately, there is a higher risk of hydraulic overtopping for all but the 2 bridges listed above. For the 2 bridges listed above it is possible that structural failure could directly cause levee failure. It is assumed with the exception of these 2 bridges, the other bridge failures would cause a higher risk of levee failure as a result of a failed bridge causing a dam and resulting in a higher water surface elevation, but still lower than the proposed raise. Refer to Exhibit A-9.6 at the end of this chapter for a discussion on the hydraulic evaluation conducted to quantify the effects of a bridge failure.

A-9.4 EXISTING AND FUTURE WITHOUT PROJECT

There are currently 11 bridges below existing top of levee (TOL). Of those 11, 7 bridges are greater than 3 ft below existing TOL. This includes all bridges currently spanning the river, active or abandoned. Exhibit A-9.1 lists the data associated with the existing conditions and future without project conditions. This table gives an indication of the level of risk and reasoning for that assessment for each of the failure modes. Each failure mode is then categorized by color to indicate level of risk from a graphical standpoint.

- Green = low risk
- Yellow = moderate risk
- Red = high risk

A-9.5 FUTURE WITH PROJECT

There are 5 bridges with low chord above the N500+0 water surface elevation but below the N500+3 top of levee. There are 7 bridges with low chord below the N500+0 water surface elevation. Exhibit A-9.2 lists the data associated with the future with project conditions. The table gives an indication of the level of risk and reasoning for that assessment for each of the failure modes. Each failure mode is then categorized by color to indicate level of risk from a graphical standpoint.

- Green = low risk
- Yellow = moderate risk
- Red = high risk

Although it is assumed that removal of abandoned bridges at the future with project condition could address issues with these bridges, they are still included in the risk assessment to identify if they should be considered problem bridges.

A-9.6 FAILURE MODE ASSESSMENT

A major factor in determining the level of risk for each of the failure modes is the superstructure and substructure type. The following structure type considerations were made when assigning a failure mode risk assessment:

- A continuous span superstructure is less likely than a simple span structure to result in failure
- A continuous span superstructure is also less likely to result in collapse due to scour of the bridge piers
- A truss structure is likely to be able to convey some water through the superstructure
- A solid shaft pier is very unlikely to fail in shear or bending, due to volume of concrete and reinforcing in the structure
- A substructure on piles or drilled shafts in rock is less likely to fail than a substructure on spread footing on rock; there is some likelihood that the spread footing founding material could be negatively impacted by scour despite the fact that it is rock

Another major consideration given was with respect to how much of the length of the superstructure is underwater at the indicated water surface elevation. If the structure is a haunched structure, the amount of water actually on the superstructure is likely to be for only a short length of the bridge because the beam or truss varies in depth. The grade of the bridge also has impacts on the length of the superstructure underwater. The bridge plans were reviewed to determine superstructure types and elevations to determine the approximate length of the bridge underwater. Those with haunched superstructures or with minimal lengths of the bridge underwater were noted in Exhibits A-9.1 and A-9.2.

A-9.7 FAILURE MODE RATING

Values from 1 to 9 were assigned for red to green failure mode designations respectively, refer to Exhibits A-9.3 and A-9.4 for the table of values. These values are also dependent on the water height above low steel. A weighted average for the two structural failure modes was calculated and a weighted average for the three geotechnical failure modes was calculated. The weighted average for the geotechnical failure modes was then adjusted with respect to the scour rating. Then a total weighted average was calculated that resulted in the total rating value for that bridge. The bridges with the lowest total rating value are the highest risk structures.

The scour adjustment rating was loosely based on the probability of failure tables in the HAZUS MR4 technical manual, included as Exhibit A-9.7 at the end of this chapter. A scour rating of 6 and above is considered to have no impact on the failure mode analysis. A scour rating of 4 or 5 indicate some distress and has been identified, but no significant impact to the failure modes. A scour rating of 3 and below is considered to greatly impact the geotechnical failure modes.

A-9.7.1 EXISTING CONDITIONS

The bridges with low chord below the existing top of levee (TOL) O&M designated water surface elevation with any yellow or red failure mode in Exhibit A-9.1 were translated to Exhibit A-9.3. The table in Exhibit A-9.3 shows a failure mode rating considering the likelihood of each of the different type of failure modes. For each bridge failure mode, a value was assigned in the table in Exhibit A-9.3 according to the colors and descriptions from the table in Exhibit A-9.1.

The Kansas City Terminal Railroad Bridge is under the most water for future conditions. No probability of structural failure was calculated for existing conditions.

A-9.7.2 FUTURE CONDITIONS

The bridges with low chord below the N500+3 water surface elevation with any yellow or red failure mode in Exhibit A-9.2 were translated to the table in Exhibit A-9.4. The table in Exhibit A-9.4 shows a failure mode rating considering the likelihood of each of the different type of failure modes. For each bridge failure mode, a value was assigned in the table in Exhibit A-9.4 according to the colors and descriptions from the table in Exhibit A-9.2.

The Kansas City Terminal Railroad Bridge is under the most water for future conditions. This bridge was investigated for the probability of structural failure because it appeared to be the worst case bridge. The structural analysis reviewed the uplift and stream forces on the bridge versus the resisting dead load. The bridge dead load was approximated from the bridge drawings. The dead load was listed on the drawings and this was the dead load used in the analysis. The dead load was significant enough to exceed the forces due to uplift and forces due to uplift and stream forces combined. Therefore the structural reliability was assigned a value of 99.8% and the rest of the bridges on the river, which have less water on them, are assumed to be at a reliability of 99.8% as well. Consideration was given to the fact that other structure's dead load may not exceed the uplift and stream forces. The other bridges that are high risk are all trusses and are likely to perform similarly due to large dead load compared to minimal uplift pressures due to minimal water displacement. Note that the probability of failure was not investigated for the geotechnical or scour failure modes. These are assumed not to control based on the geotechnical and scour weighted ratings typically being greater than the structural failure

weighted rating, indicating that the likely failure mode is structural. Supporting calculations are provided in Exhibit A-9.8.

A-9.8 SUMMARY

The probabilistic analysis discussed in this chapter indicates a very low probability of failure of a Kansas River bridge (0.2%). The Kansas Avenue East Bridge is the only bridge that could directly result in levee failure should the bridge fail by falling on a structural component of the existing flood protection system (floodwall). This analysis is a very broad look at the potential failure modes and likelihood of failure. More detailed analysis, though costly, would better indicate probability of failures for these bridges.

A-9.9 SUPPLEMENTAL EXHIBITS

EXHIBIT A-9.1

Existing and Future Without Project Conditions
Probabilistic Approach to Bridge Failure

EXISTING CONDITIONS AND FUTURE WITHOUT PROJECT CONDITIONS - PROBABILISTIC APPROACH TO BRIDGE FAILURE

					CID	ARMOURDALE	ARGENTINE					superstructure type			5 failure modes					Substructure type			Scour Rating	Risk - Scour Rating			
#	Bridge ID	RM	Owner	Low Chord over Channel (NGVD29)	surveyed low chord over channel (NGVD29)	Existing TOL O&M (NGVD29)	Existing TOL O&M (NGVD29)	Existing TOL O&M (NGVD29)	Dist low chord is below Existing TOL O&M	superstructure spans over flood protection (Y/N)	Existing gap (Y/N)	investigate	type	simple span or continuous span	n100 channel velocities at the investigation locations (fps)	Bearing or anchor bolt failure due to tension	Pier section failure due to shear or bending	Substructure overturning	Substructure sliding	Foundation failure due to tension or bearing in piles or bearing of footings	River Piers	Overbank Piers	Scour notes	Existing scour rating	Risk of bridge failure without project	Risk of bridge failure with project	Risk if bridge fails then levee fails	NOTES
1	MO Pac RR #1	0.26	Union Pacific RR	756.0	758.1	760.9	--	--	2.8	N	N	1 river pier and 1 pier in 4th span	4-span truss	simple	12	medium risk - simple span bridge but only 2.4ft under water. Structure is a truss bridge with significant Dead wt compared to ht of water	low risk - solid shaft pier	low risk for overbank piers (piles on rock); low risk for river piers; minimal water on superstructure	low risk for overbank piers (piles on rock); low risk for river piers; minimal water on superstructure	low risk due to minimal water on superstructure	concrete bearing on rock	concrete on timber piles driven to rock	timber piles exposed during low water; general channel degradation noted	5	Low risk - unlikely to fail	Low risk - unlikely to fail	Low risk - unlikely to fail	
2	MO Pac RR #2	0.26	MDOT	761.6		760.9	758.1	--	2.8	Y	N				10										5	Low risk - unlikely to fail	Low risk - unlikely to fail	Low risk - unlikely to fail	
3	MO Pac RR #3	0.26	MDOT	761.6		760.9	758.1	--	2.8	Y	N				10										5	Low risk - unlikely to fail	Low risk - unlikely to fail	Low risk - unlikely to fail	
4	Frontage Rd	0.4		761.6		760.9	758.1	--	2.8	Y	N				10										5	Low risk - unlikely to fail	Low risk - unlikely to fail	Low risk - unlikely to fail	
5	KC Southern	0.81	KC Southern RR	761.6	762.6	762.1	760.7	--	1.4	N	N	1 river pier and bearings	5-span truss	simple	15	medium risk - simple span bridge. 3.3ft under water. Structure is a truss bridge with significant Dead wt compared to ht of water	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock							Abandoned Bridge
6	Central Ave	1.11	UG	760.6	760.2	762.2	761.7	--	1.0	N	N	2 river pier and bearings	truss	simple	13	low risk - simple span truss bridge 1.8ft (that structure ht) under water (fixed bearings at one end of each simple span)	low risk - solid shaft pier	low risk - minimal water on superstructure	low risk - minimal water on superstructure	low risk - minimal water on superstructure	concrete bearing on rock	concrete floating bearing on piles	top and sides of footings are exposed 2-4ft, however solid footings are 15ft thick according to drawings	5				
7	I-470	1.4		761.6		762.2	761.6	--	1.5	Y	N	river plate girder	continuous		12						concrete on solid shaft; exposed pier cap			5 for 18 days; 3 for 365				
8	MO Pac RR #72	1.61	Union Pacific RR	758.0		762.9	762.5	--	0.4	N	N/Y	1 river pier and bearings	4-span truss	simple	19	medium risk - simple span truss bridge 3.6ft under water-no brg details available (fixed at one end)	low risk - appear to be solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock		timber piles exposed during low water; general channel degradation noted; possible scour evidence at Bent 4					
9	UP RR #3	1.66	Union Pacific RR	758.0		762.9	762.5	--	0.4	N	--	1 river pier and bearings	4-span truss	simple	19	medium risk - simple span truss bridge 3.4ft (10% of ht of structure) under water-no brg details available (fixed at one end)	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock		general channel degradation noted					Bridge Raised 8' in 1993 Flood, Low Chord Reported in Down Position
10	CRP RR	2.3	City of Kansas City, MO	762.8	763.1	764.8	763.6	--	1.7	N	--	1 river pier on rock and 1 river pair on piles	3-span truss	simple	12	low risk - simple span truss bridge 1.7ft (that structure ht) under water-no brg details available (fixed at one end)	low risk - appear to be solid shaft pier	low risk for substructure piers (piles on rock); low risk for river piers; minimal water on superstructure	low risk for river piers; minimal water on superstructure	low risk for river piers; minimal water on superstructure	concrete bearing on rock	concrete on timber piles	general channel degradation noted; based on exposed wooden forms					Raised to Current Position, Gap Permanently Closed, and Abandoned
11	KS Ave East	2.17	UG	760.8		763	764.4	--	1.4	Y	N	west river pier and bearings	3-span truss	continuous	13	medium risk - continuous truss - bridge 4.2ft (12% of ht of structure) under water-no brg details available	low risk - solid shaft pier	low risk for river piers	low risk for shafts on rock	low risk for shafts on rock	concrete shafts on rock		no evidence of scour at piers	5				
12	KC Terminal	2.53	KC Terminal RR	758.3		765.9	765.5	--	0.4	N	Y	1 river pier and bearings	3-span truss	simple	12	medium risk - simple span double deck truss bridge 10ft (17% of ht of structure) under water-no brg details available (fixed at one end)	low risk - solid shaft pier	moderate risk for river piers	moderate	moderate	concrete bearing on rock		exposed caisson at the bottom					End Spans Raised 20' in 1993, but currently not operable. Low Chord Reported in Down Position

EXISTING CONDITIONS AND FUTURE WITHOUT PROJECT CONDITIONS - PROBABILISTIC APPROACH TO BRIDGE FAILURE

						CID	ARMOURDALE	ARGENTINE					Superstructure type			5 failure modes					Substructure type			Scour Rating	Risk - Scour Rating			
#	Bridge ID	BM	Owner	Low Chord over Channel (NGVD29)	Surveyed low chord over channel (NGVD29)	Existing TOL O&M (NGVD29)	Existing TOL O&M (NGVD29)	Existing TOL O&M (NGVD29)	Dist low chord is below Existing TOL O&M	superstructure spans over flood protection (Y/N)	Existing gap (Y/N)	investigate	type	simple span or continuous span	n500 channel velocities at the investigation locations (fps)	Bearing or anchor bolt failure due to tension	Pier section failure due to shear or bending	Substructure overturning	Substructure sliding	Foundation failure due to tension or bearing in piles or bearing of footings	River Piers	Overbank Piers	Scour notes	Existing scour rating	Risk of bridge failure without project	Risk of bridge failure with project	Risk if bridge fails then levee fails	NOTES
13	7th St NB	3.40	KDOT	774.6	774.4	---	768.1	---	6.0	N	N	east river pier and bearings	3-span truss	continuous	10	low risk - continuous 3-span truss bridge 5.3ft under water at movem point (assumed no big details available (0.3 million pounds of steel) river on - sta sheet)	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock		post scour - repaired by driving sheet pile around E river pier and concrete filled; general channel degradation noted; exposed sheet pile	7	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	
14	7th St SB	3.40	KDOT	775.6			768.1		7.5	N	N			continuous	10														
15	12th St	4.28		747.7	747.5		769.5	---	2.0	N	N		steel plate girder	continuous	10	low risk most of superstructure is out of the water (only a small portion of 1st span under water)	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	on drilled shafts in rock	spread ftg on piles	new piers ~1990	8	LOW			
16	18th St	4.96	KDOT	781.9	781.5		771	771	9.0	N	N	east river pier and bearings	5-span truss	continuous	9	medium to low risk 1082ft continuous 3spans, 6ft under water (abt 28%of 36 of structure) - continuous span makes very likely to fail- no big details available	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock		no evidence of scour at piers	8				
17	43 Ave West	4.82		772.7	774.4		772.7	772.7	11.6	N	N		steel plate girder	continuous	7										8				
18	45th St NB	7.30	KDOT	781.3				775.8	5.5	N	N				6						concrete ftg on steel pier - 2 piers - concrete ftg on steel pier with 2 piers - 5 piers - concrete spread footing abt 6ft into rock				8	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	
19	45th St SB	7.30	KDOT	781.3				775.8	5.5	N	N				6						concrete ftg on steel pier with 2 piers - 5 piers - concrete spread footing abt 6ft into rock				7	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	
20	Turner Bridge NB (K-32 Wyo)	9.10	KDOT	779.3	779.3			779.3	0.0	N	N		steel plate girder	continuous	5						concrete drilled shafts in rock								

abandoned
bold bridges reviewed by HNTB in 2008 bridge recon study
light blue bridges - superstructure not under water

Bridges With Low Chord Below TOL > 5ft
Bridges With Low Chord Below TOL < 3ft

EXHIBIT A-9.2

Future With Project Conditions – Probabilistic Approach to Bridge Failure

FUTURE WITH PROJECT - PROBABILISTIC APPROACH TO BRIDGE FAILURE

						CIS		AMERICANDALE		ARGENTINE																							
				Low Chord over Channel (BELOW)	Waterpad low chord over channel (ABOVE)	Existing TOL OBAR (BELOW)	Proposed ADD-3 TOL (BELOW)	Existing TOL OBAR (ABOVE)	Proposed ADD-3 TOL (ABOVE)	Existing TOL OBAR (ABOVE)	Proposed ADD-3 TOL (ABOVE)		ISHL low chord is below ADD-3 TOL	Superstructure spans over flood passection (Y/N)	Existing gap (Y/N)	Proposed Gap (Y/N)	Superstructure type																	
#	Bridge ID	Rte	Owner									WSSE WSEL	(BELOW)				Inventoryclass	Type	Simple span or continuous span	>500 channel velocities at the bore alignmen locations (fps)	Retaining or anchor bolt failure due to overflow	Pier section failure due to shear or bending	Substructure overstressing	Substructure sliding	Foundation failure due to erosion or bearing in piles or bearing of footings	Pier: Piers	Abutment: Piers	Issue notes	Existing issue rating	Future condition issue rating	Risk of bridge failure without project	Risk of bridge failure with project	Risk of bridge fails then leaves flow	PDF due to wave (FOR SHOW EVENTS)(remark)	NOTES
1	MO Pac RR #1	0.26	Union Pacific RR	739.0	758.1	768.6	764.5	---	---	---	---	758.9	2.8	N	N	N	1 river pier and 1 abu is dim gain	4 span truss	simple	12	medium risk - simple span bridge, sea rely, 2.4ft under water, structure is a truss bridge with significant Dead un-compared to bt of water	low risk - solid foundation	low risk for overbank piers - piles on rock; moderate risk for river piers	low risk for overbank piers - moderate risk for river piers	moderate	concrete footing on rock	concrete on timber piles driven to rock	timber piles exposed during low water; general channel degradation noted							
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
5	NO Mauthern	0.02	NE Dept Roads RR	760.5	780.5	782.1	782.3	801.7	802.1	---	---	802.9	3.3	N	N	N	1 river pier and bearings	3 span truss	simple	12	medium risk - simple span bridge, 5.5ft under water, structure is a truss bridge with significant Dead un-compared to bt of water	low risk - solid foundation	low risk of erosion - rock	moderate risk for rock	low risk for piers on rock	concrete on timber piles driven to rock							Abutment Bridge
6	Central Ave	1.11	UG	788.0	796.7	792.1	792.3	792.7	792.5	---	---	796.0	3.3	N	N	Y	1 river pier and bearings	truss	simple	12	medium risk - simple span bridge, 3.3ft under water, 100% of upstream structure heights. structure is a truss bridge with significant Dead un-compared to bt of water	low risk - solid foundation	low risk for overbank piers - piles; low risk for river piers	low risk for overbank piers - moderate risk for river piers	moderate risk for overbank piers	concrete footing bearing on rock	concrete footing bearing on piles	top and bottom of footing just exposed 2 ft, because added footings 2 ft thick according to drawings	5	6				0	
7	I1 470	1.4	MDOT	802.6		792.1	792.9	792.7	793.5	---	---	793.7	1.1			N	steel plate girder	continuous		12	low risk - continuous - 15ft/20ft/15ft - simple span bridge, only 1.1ft under water, only 1 span, structure is a truss bridge with significant Dead un-compared to bt of water	low risk - solid foundation	low risk for shallow foundation on rock	low risk for shallow foundation on rock	low risk for shallow foundation on rock	concrete on timber piles driven to rock			5 for 18 inch & for 9 ft	4			0	
8	MO Pac RR #22	1.61	Union Pacific RR			792.1	793.5	792.5	793.9	---	---	792.7	0.2	N	N/Y	N/Y	1 river pier and bearings	4 span truss	simple	12	medium risk - simple span truss bridge 6.5ft under water; no bug details available (based on same end)	low risk - against to far solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piers on rock	concrete on timber piles driven to rock		Timber piles exposed during low water; general channel degradation noted; possible scour evidence at bent 5						
9	UP RR #3	1.94	Union Pacific RR	798.0		792.1	793.1	792.5	793.5	---	---	792.7	0.6	N	Y	Y	1 river pier and bearings	4 span truss	simple	12	medium risk - simple span bridge 3.5ft under water; no bug details available (based on same end); 1 bridge is a truss structure, underwater bug details available (based on same end)	low risk - solid foundation	low risk piles on rock	low risk piles on rock	low risk for piers on rock	concrete on timber piles driven to rock		general channel degradation noted						Bridge Raised 8' in 1955 Flood. Low Chord Exposed in Down Position
10	Olive Rd	2.1	City of Genoa City, WI	798.0		792.1	793.1	792.5	793.5	---	---	793.3	0.2	N		N	1 river pier on rock and 1 abu pier on rock	3 span truss	simple	12	medium risk - simple span bridge 4.6ft (15ft) of structure bp under water no bug details available (based on same end)	low risk - against to far solid shaft pier	low risk for overbank piers - piles on rock; moderate risk for river piers	moderate	moderate	concrete footing on rock	concrete on timber piles	general channel degradation noted; exposed and exposed concrete frame						Raised to Current Position. Gap Permanently Closed, and abandoned
11	93 Ave Bend	2.2	UG	798.0		797	798	794.3	798	---	---	794.7	1.7			N	2 mid river pier and bearings	5 span truss	continuous	12	medium risk - continuous - lower Bridge 7.2ft (21ft) of structure bp under water no bug details available	low risk - solid foundation	low risk for river piers	low risk for shallow foundation	low risk for shallow foundation	concrete shallow on rock		no evidence of scour or pier	5	4				0	
12	KC Terminal	2.63	KC Terminal RR	798.0		792.9	793.7	792.5	793.5	---	---	794.1	0.4	N	Y	Y	1 river pier and bearings	3 span truss	simple	12	medium risk - simple span bridge 3.5ft under water (based on 12.4ft under water at least at least at least channel velocity) bug details available; 1 river pier is on shallow rock or shallow footing (based on same end)	low risk - solid foundation	moderate risk for river piers	moderate risk for river piers	moderate	concrete footing on rock		exposed caissons at the bottom		4				End Spans Raised 30' in 1955, but currently not operable. Low Chord Exposed in Down Position
13	7th St NB	8.42	MDOT	798.0	798.0	---	---	768.0	773.6	---	---	802.8	34.8	N	N	N	steel deck pier and bearings	3 span truss	continuous	30	low risk - continuous 5 span - lowest bp (12.4ft under water at least at least at least channel velocity) bug details available; 1 river pier is on shallow rock or shallow footing (based on same end)	low risk - solid foundation	low risk piers on rock	low risk piers on rock	low risk for piers on rock	concrete on timber piles driven to rock		scour scour - repaired by driving sheet pile around 1 river pier and concrete filled, general channel degradation noted; exposed sheet pile	7	6				0	

FUTURE WITH PROJECT - PROBABILISTIC APPROACH TO BRIDGE FAILURE

#	Bridge ID	RM	Owner	Low Chord over Channel (NGVD29)	surveyed low chord over channel (NGVD29)	CID: Existing TOL O&M (NGVD29)	CID: Proposed N500+3 TOL (NGVD29)	ARMOURDALE: Existing TOL O&M (NGVD29)	ARMOURDALE: Proposed N500+3 TOL (NGVD29)	ARGENTINE: Existing TOL O&M (NGVD29)	ARGENTINE: Proposed N500+3 TOL (NGVD29)	N500 WSE (NGVD29)	Dist low chord is below N500+3 TOL	superstructure spans over flood protection (Y/N)	Existing gap (Y/N)	Proposed Gap (Y/N)	Investigate	superstructure type: type	superstructure type: simple span or continuous span	n500 channel velocities at the investigation locations (fps)	5 failure modes: Bearing or anchor bolt failure due to tension	5 failure modes: Pier section failure due to shear or bending	5 failure modes: Substructure overturning	5 failure modes: Substructure sliding	5 failure modes: Foundation failure due to tension or bearing in piles or bearing of footings	Substructure type: River Piers	Substructure type: Overbank Piers	Scour notes	Scour Rating: Existing scour rating	Scour Rating: Future conditions scour rating	Risk - Scour Rating: Risk of bridge failure without project	Risk - Scour Rating: Risk of bridge failure with project	Risk - Scour Rating: Risk if bridge fails then levee fails	POF due to scour FOR 500YR EVENT (in percent)	NOTES
14	[illegible]	[illegible]	KDOT	[illegible]				[illegible]	[illegible]	---	---	[illegible]	[illegible]	N	N	N			continuous	10									7	6	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	0	
15	12th St	4.28		[illegible]	[illegible]			764.5	774.1	---	---	[illegible]	[illegible]	N	N	Y		steel plate girder	continuous	10	low risk most of superstructure is out of the water (only 40ft of N span under water)	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	low risk most of superstructure is out of the water	on drilled shafts in rock	spread ftg on piles	new piers ~1998	8	7	LOW			0	
16	18th St	4.96	KDOT	[illegible]	[illegible]			771	775.9	771	775.9	[illegible]	[illegible]	N	N	N	cast over pier and bearings	3-span truss	continuous	9	medium to low risk - 100ft continuous 3 spans, 10.9ft under water (about [illegible] of ht of structure) - continuous span makes less likely to fail due to uplift; no brg. details available	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete on timber piles driven to rock		no evidence of scour at piers	8	7				0	
17	KS Ave West	5.82		773.7	774.8			773.2	777.4	773.4	777.4	774.4	2.6	N	N	N		steel plate girder	continuous	7	low risk - 142ft continuous 5 spans, only 2.6ft under water; haunched girders and continuous span makes less likely to fail due to uplift; no brg. details available	low risk - solid shaft pier	low risk piles on rock	low risk piles on rock	low risk for piles on rock	concrete ftg on steel piles			8	7				0	
18	[illegible]	[illegible]	KDOT	[illegible]				[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	N	N	N				8						[illegible]			8	7	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	0	
19	[illegible]	[illegible]	KDOT	[illegible]				[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	N	N	N				8						[illegible]			8	7	LOW NOT UNDER WATER	LOW NOT UNDER WATER	LOW NOT UNDER WATER	0	
26	Turner Bridge (K-132 Hwy)	9.5	KDOT	778.9	779.3			---	---	776.8	781.1	778.1	1.8	N 4	N	N		steel plate girder	continuous	7	low risk - 440ft continuous span, only 1.8ft ([illegible] of ht of girders) of water on girders; no brg. details available	low risk, superstructure only under water 1.8ft and piers have webwall between shafts	low risk, piers on shafts into rock	low risk, piers on shafts into rock	low risk for shafts on rock	concrete drilled shafts in rock									

abandoned
bold bridges reviewed by HNTB in 2008 bridge recon study
Grey Background - superstructure not under water

Bridges With Low Chord Below n500 Water Surface Elevation
Bridges With Low Chord Above n500 Water Surface Elevation but Below n500+3 TOL

EXHIBIT A-9.3

KANSAS RIVER BRIDGE EXISTING CONDITIONS PROBABILISTIC FAILURE RATING

Failure Mode	Water Height Above Low Steel
1	2
2	3
4	5
5	6
7	8
8	9

SCOUR RATING	SCOUR RATING multiplier
8	1
7	1
6	1
5	0.95
4	0.9
3	0.7
2	0.6
1	0.4

			FAILURE MODES						STRUCTURAL FAILURE Rating		GEOTECHNICAL FAILURE Rating		SCOUR ADJUSTED GEOTECHNICAL FAILURE Weighted Rating	Total Rating		POF of structural failure	levee failure as direct result of bridge failure
			SUPERSTRUCTURE	SUBSTRUCTURE													
				STRUCTURAL FAILURE	GEOTECHNICAL FAILURE												
Bridge	Dist low chord is below existing TOL O&M	Bridge Type	Bearing or Anchor Bolt	Pier Failure	Overturning	Sliding	Foundation	FUTURE CONDITIONS Scour Rating	Raw Average	Weighted Average	Raw Average	Weighted Average		Raw Average	Weighted Average		
KC Terminal	10	Simple	4	7	4	4	4	5	5.50	2.35	4.00	1.80	1.71	4.75	2.03	0.20%	0
UP RR#3	8.6	Simple	1	7	7	7	7	5	4.00	0.85	7.00	3.15	2.99	5.50	1.92	0.20%	0
CRIP RR*	1.7	Simple	4	7	4	4	4	5	5.50	2.35	4.00	1.80	1.71	4.75	2.03	0.20%	0
KC Southern*	3.5	Simple	4	7	7	7	7	5	5.50	2.35	7.00	3.15	2.99	6.25	2.67	0.20%	0
MO Pac RR#2	3.6	Simple	4	7	7	7	7	5	5.50	2.35	7.00	3.15	2.99	6.25	2.67	0.20%	0
MO Pac RR#1	2.8	Simple	5	8	5	8	5	5	6.50	2.90	6.00	2.70	2.57	6.25	2.73	0.20%	0
KC Ave East**	4.2	Continuous	5	7	7	7	7	4	6.00	2.85	7.00	3.15	2.84	6.50	2.84	0.20%	1
Central Ave	1.9	Simple	5	8	8	8	6	4	6.50	2.90	7.33	3.30	2.97	6.92	2.94	0.20%	0
18th St	6	Continuous	5	7	7	7	7	7	6.00	2.85	7.00	3.15	3.15	6.50	3.00	0.20%	0
Weighting Percentage			50%	5%	15%	15%	15%			50%			50%				

Notes:
Unshaded values in structural rating table for simple, shaded values for continuous.
Red values in Scour Rating column are assumed.
*Abandoned Bridges
**Structural failure could directly cause levee failure on CID-KS side only

EXHIBIT A-9.4

KANSAS RIVER BRIDGE FUTURE WITH PROJECT PROBABILISTIC FAILURE RATING

Failure Mode \ Water Height Above Low Steel		
	1	2
	2	3
	4	5
	5	6
	7	8
	8	9

SCOUR RATING	SCOUR RATING multiplier
8	1
7	1
6	1
5	0.95
4	0.9
3	0.7
2	0.6
1	0.4

			FAILURE MODES															
			SUPERSTRUCTURE	SUBSTRUCTURE					STRUCTURAL FAILURE Rating		GEOTECHNICAL FAILURE Rating		SCOUR ADJUSTED GEOTECHNICAL FAILURE Weighted Rating	Total Rating		POF of structural failure	levee failure as direct result of bridge failure
				STRUCTURAL FAILURE	GEOTECHNICAL FAILURE												
Bridge	Dist low chord is below N500+3 TOL	Bridge Type	Bearing or Anchor Bolt	Pier Failure	Overturning	Sliding	Foundation	FUTURE CONDITIONS Scour Rating	Raw Average	Weighted Average	Raw Average	Weighted Average		Raw Average	Weighted Average		
KC Terminal	13.8	Simple	1	7	4	4	4	5	4.00	0.85	4.00	1.80	1.71	4.00	1.28	0.20%	0
UP RR#3	11.6	Simple	1	7	7	7	7	5	4.00	0.85	7.00	3.15	2.99	5.50	1.92	0.20%	0
CRGP RR*	4.6	Simple	4	7	4	4	4	5	5.50	2.35	4.00	1.80	1.71	4.75	2.08	0.20%	0
KC Southern*	3.5	Simple	4	7	7	7	7	5	5.50	2.35	7.00	3.15	2.99	6.25	2.67	0.20%	0
MO Pac RR#2	6.5	Simple	4	7	7	7	7	5	5.50	2.35	7.00	3.15	2.99	6.25	2.67	0.20%	0
MO Pac RR#1	2.6	Simple	5	8	5	8	5	5	6.50	2.90	6.00	2.70	2.57	6.25	2.73	0.20%	0
KS Ave Gas*	7.2	Continuous	5	7	7	7	7	4	6.00	2.85	7.00	3.15	2.84	6.50	2.84	0.20%	1
Central Ave	3.3	Simple	5	8	8	8	6	4	6.50	2.90	7.33	3.30	2.97	6.92	2.94	0.20%	0
18th St	10.9	Continuous	5	7	7	7	7	7	6.00	2.85	7.00	3.15	3.15	6.50	3.00	0.20%	0
Weighting Percentage			50%	5%	15%	15%	15%			50%			50%				

Notes:
Unshaded values in structural rating table for simple, shaded values for continuous
Red values in Scour Rating column are assumed.
*Abandoned Bridges
**Structural failure could directly cause levee failure on CID-KS side only

EXHIBIT A-9.5

HNTB 2008 Bridge Reconnaissance – Letter of Opinion

HNTB Federal Services Corporation
Engineers Architects Planners

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Date
11/7/08

To
Lamar McKissack, COE

HNTB

From
Dave Renetzky, HNTB

PROJECT CORRESPONDENCE

Subject
Kansas City Levees Phase 2
Bridge Reconnaissance – HNTB Task 3a
Letter of Opinion

Introduction

The bridge analysis task of the Kansas City Levees Phase 2 Study to determine potential impacts to existing bridges, railroad crossings, gaps, and closures on the lower Kansas River has many aspects to it. The Kansas River units of the feasibility study include Argentine, Central Industrial District (CID), and Armourdale. Task 3 of the HNTB Scope of Work dated 13 August 2008 provides an overview of technical requirements to complete the effort, including field data collection, stakeholder coordination, and detailed analysis and documentation. It was decided that breaking the task into two parts, Task 3a and Task 3b, would accomplish initial goals while bringing insight into the effort necessary to complete the overall task. Task 3a was considered bridge reconnaissance, while Task 3b will be performance of the analyses.

The general purpose of the bridge reconnaissance was to:

- Research and understand the bridge configurations
- Observe structural conditions of the bridges
- Observe site conditions with respect to scour, geotechnical aspects, and closure gaps
- Relate low steel elevations to the proposed water surface elevations
- Form an approach to the lateral load analysis
- Provide an opinion of base and option tasks in the scope

Nine bridges were identified by the Corps to include in the reconnaissance effort. Dave Renetzky (HNTB), Chuck Steffensmeier (HNTB), Kevin Kuhl (HNTB), Alma Baratta (HNTB), Bill McFarland (COE), Eddie Fernandez (COE), and Ernie Quinto (Kaw Valley Drainage District) visited the following bridges on 14 October 2008:

- Missouri Pacific #1
- Missouri Pacific #72
- Union Pacific #3
- Kansas City Terminal

On 15 October 2008, Dave Renetzky (HNTB), Chuck Steffensmeier (HNTB), Tom Westerman (HNTB), Alma Baratta (HNTB), Bill McFarland (COE), and Ernie Quinto (KVDD) visited the remaining bridges:

- Kansas Avenue East
- 7th Street Northbound
- 18th Street

- Chicago Rock Island & Pacific
- Kansas City Southern

Site visit notes and reports are included in the appendix to this document, along with numerous photos from each bridge on CD. Each bridge was documented with a structural observation sheet, a stream stability analysis sheet, and a geotechnical site visit report.

Structural Analysis Overview

The superstructures, substructures and foundations for the bridges contained in this study were most likely not originally designed for stream flow lateral loads that occur during a 500-year flood event. The substructures for all of the bridge river piers are either concrete piers that bear directly on rock or concrete piers that are supported by timber piles that have been driven to rock. The five failure modes that are shown on the next page and explained below should be evaluated for each bridge for the stream flow lateral load that result from the 500-year and 500-year + 3' flood event. The analysis of all five failure modes will determine if modifications are needed to decrease risk of failure for the bridge.

It is recommended that the railroad bridge substructures be evaluated with the load combinations, which include stream flow, from chapter 8 of the 1995 edition of the AREMA manual for Railway Engineering. The highway bridge substructures should be evaluated for the load combinations, which include stream flow, from the 2002 17th edition of the AASHTO Standard Specifications for Highway Bridges. All of the bridges will be evaluated by either the Load Factor or Service Load design method.

Another part of the recommended study is scour analysis to be performed on the bridge piers. The results of the scour analysis should be included in the structural evaluation of the river pier foundations for both storm events.

The five potential failure modes of the bridges include:

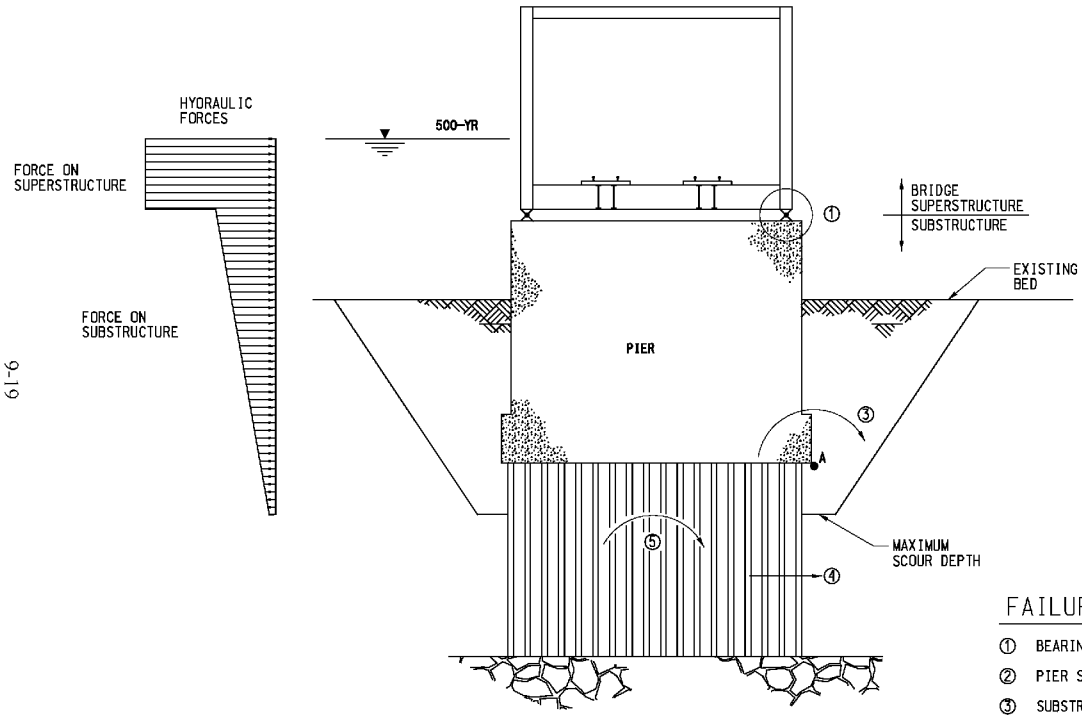
1. Bearing and anchor bolt overstress - The analysis will evaluate the bearings and anchor bolts for overstress that result from the uplift and lateral hydraulic forces on the superstructure.

2. Pier section overstress - The analysis will evaluate the pier section for shear and bending overstress that result from the lateral forces.

3. Substructure Overturning - The analysis will evaluate the substructure for exceeding allowable overturning resistance that results from lateral hydraulic forces.

4. Substructure Sliding - The analysis will evaluate the substructure for exceeding allowable sliding resistance that results from lateral forces.

5. Foundation overstress - The analysis will evaluate the substructure foundation for overstress that results from the lateral forces. Tension or pile capacity of the timber piles as well as bearing capacity of the piers on rock are to be evaluated as part of the foundation overstress.



FAILURE MODES

① BEARING AND ANCHOR BOLT OVERSTRESS

② PIER SECTION OVERSTRESS

③ SUBSTRUCTURE OVERTURNING

④ SUBSTRUCTURE SLIDING

⑤ FOUNDATION OVERSTRESS

HNTB

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9-19

Analysis should be completed for all five failure modes. Based on hydraulic forces, the most likely failure mode will be either substructure overturning or substructure sliding. However, each failure mode will need to be investigated to ensure the limiting failure mode has been identified and appropriate modifications are identified. Forces calculated for each flood event will be used for the analysis of each failure mode. The failure analysis calculations are not needed for the cases where the n500 water surface elevation does not exceed the elevation of the low chord. The lateral forces exerted on the bridge for this case are much less because the force from the water is not exerted along the length of the bridge. While it is possible that scour depth could be greater with a lower water surface elevation (and resultant higher velocity), the evaluation can be bypassed unless there is a known scour problem at a particular bridge.

The analysis may determine that the bridge or substructure component is overstressed. This analysis is based upon an allowable stress, the ultimate stress is typically much greater, but considering material variability an allowable stress that is lower than the ultimate stress is used. Allowable stress varies according to the age of the bridge and the process used for the original material, a recommendation of the allowable stress for the material used in each bridge will be included in a design memorandum and agreed upon as part of the future 3b tasks. The result of these stress calculations is that the member may be overstressed but not fail, so a retrofit will not be required in the case where the ultimate stress is not reached. In cases where the ultimate stress is reached, the member would fail and a retrofit or replacement will be recommended.

Failure modes; 3- Substructure overturning, 4-Substructure sliding and 5- Foundation overstress are a function of the maximum scour at the pier. Scour analysis for each river pier is recommended to ensure that the worst-case scenario is identified and included in the analysis.

For purposes of this study, it is assumed that failures resulting from existing maintenance or the life span of the structure are the responsibility of the owner. Future analysis will only consider the possible failures as a result of the proposed impacts of this study.

Ownership of the railroad structures is provided in the following sections. The U.S. Army Corps will provide a letter to each of the owners that will describe the study purpose and ask that several team members meet with them to discuss the study. During the meeting with the owners, the team will provide additional information regarding the study and answer questions that they may have. The team will request copies of the owner's bridge plans during the meeting.

1. Missouri Pacific #1 Bridge (River mile 0.26)

Research - The Missouri Pacific bridge is a single track, 4-span through truss with 82' deck plate girder on north end (3 @200', 1 @ 233', 1 @82' = 915'). The river piers are concrete bearing on rock. The fourth span of the bridge was added in 1917, with the substructure for the additional span constructed of concrete on timber piles driven to rock.

The low steel elevation of the bridge is at elevation 759.0. The Q500 WSEL is 756.9, 2.1 feet below the low steel elevation of the bridge. The Q500+3' WSEL is 760.9, which pressurizes flow 1.9' above the low steel elevation of the bridge.

The owner of the bridge is Union Pacific Railroad. The contact person and information is as follows:

David McKernan
Manager Industry & Public Projects
Phone: 314-331-0682

Opinion - The failure mode analyses should include the investigation of one river pier and a pier for the new span of the bridge that was added in 1917. The Q500 WSEL is below the low steel of the bridge and does not need to be investigated. No underwater inspection will be completed; the analysis will include a relative evaluation of a pile section that is agreed upon prior to the analysis.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by a minimum of 1.9', converting the bridge to a lift span, reinforcing the bearings and reinforcing the piers.

Depending upon the mitigation measures for this bridge, the stoplogs on the west end of the bridge will need to tie into the high ground to the west. The proposed floodwall on the east end of the bridge will tie into the existing embankment.

2. Kansas City Southern Bridge (River mile 0.81)

Research - The Kansas City Southern bridge was built in 1916 and is not in use today. The bridge is a single track, 3-span through truss (2 @360', 1 @ 230'= 950'). The river piers are constructed of concrete on timber piles driven to rock.

The low steel elevation of the bridge is at elevation 759.4. The Q500 WSEL is 759.0, 0.4 feet below the low steel elevation of the bridge. The Q500+3' WSEL is 762.1, which pressurizes flow 2.7' above the low steel elevation of the bridge.

The owner of the bridge is the Kansas City Southern Railroad. The contact person and their information is as follows:

Srikanth Honnur
Director Track and Bridge Construction
Phone: 816-983-1138
Fax: 816-983-1186
shonnur@kcsouthern.com

Opinion - The failure mode analyses should include the investigation of one river pier and bearings. The Q500 WSEL is below the low steel of the bridge and does not need to be investigated. No underwater inspection will be completed; the analysis will include a relative evaluation of a pile section that is agreed upon prior to the analysis.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by 2.7', removing the bridge, reinforcing the bearings and reinforcing the piers. If the analysis results in a bridge failure, complete removal of the existing bridge would be the most advantageous option from a river hydraulics standpoint.

The existing levee wall surrounds the abutments of this bridge and will need to be raised. No stoplogs are needed at this bridge.

3. Missouri Pacific 72 Bridge (River mile 1.62)

Research - The Missouri Pacific bridge was built in 1945 and is a double track 4-span through truss (3 @180', 1 @ 196'on W. end =736'). The piers and abutments are concrete on timber pile driven to rock.

The low steel elevation of the bridge is at elevation 758.9. The Q500 WSEL is 762.2, 2.3 feet above the low steel elevation of the bridge. The Q500+3' WSEL is 765.5, 6.6' above the low steel elevation of the bridge.

The owner of the bridge is Union Pacific Railroad. The contact person and information is as follows:

David McKernan
Manager Industry & Public Projects
Phone: 314-331-0682

Opinion - The failure mode analyses should include the evaluation of one river pier and bearings for both flood elevations since they are both above the low steel elevation of the bridge. Because this bridge is in close proximity to the Union Pacific Bridge (River mile 1.64) bearing overstress of the soil due to additional loads from the anticipated water elevations may be a possible mode of failure. No underwater inspection will be completed; the analysis will include a relative evaluation of a pile section that is agreed upon prior to the analysis.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by a minimum of 6.6', converting the bridge to lift spans, reinforcing the bearings and reinforcing the piers.

Stoplogs at both the east and west abutment of the bridge will need to accommodate the increased height of the floodwall. Conceptual layouts of the stoplogs and where they tie to the floodwall or levee are shown in the original scope documents. Design of the stoplogs will need to accommodate a minimal disruption to train traffic. A micropile with a precast cap may be a good alternative that can be installed in a relatively short period of time and the installation equipment is mobile e.g. a large excavator can be used to install the micro-piles instead of a crane used to install driven piles or drilled shafts. This alternative will be dependant upon the soil conditions and loading on the stoplog. Alternates to stoplogs are a swing gate or miter gate for quicker closure.

4. Union Pacific Bridge (River mile 1.64)

Research - The Union Pacific bridge was built in 1945 and is a double track 4-span through truss (3 @180', 1 @ 196'on W. end =736'). The bridge has been retrofitted to raise 8.8' vertically on a tension screw lifting device and was last raised for the 1993 flood. The piers and abutments are constructed of concrete on timber piles driven to rock.

The low steel elevation of the bridge in a raised position is at elevation 762.7. The Q500 WSEL is 762.5, 0.2 feet below the low steel elevation of the bridge in a raised position. The Q500+3'

WSEL is 765.6, a minimum of 2.9' above the low steel elevation of the bridge in a raised position.

The owner of the bridge is Union Pacific Railroad. The contact person and information is as follows:

David McKernan
Manager Industry & Public Projects
Phone: 314-331-0682

Opinion - The failure mode analyses should include the investigation of one river pier and bearings for both flood elevations since they are both above the low steel elevation of the bridge. No underwater inspection will be completed; the analysis will include a relative evaluation of a pile section that is agreed upon prior to the analysis.

Tension screw lifting devices are a relatively reliable lifting mechanism and require limited maintenance. However, in the past the electric cables that provide power to the mechanisms has been stolen, rendering the lifting mechanism temporarily inoperable.

Stoplogs at both the east and west abutment of the bridge will need to accommodate the increased height of the floodwall. Conceptual layouts of the stoplogs and where they tie to the floodwall or levee are shown in the original scope documents. Design of the stoplogs will need to accommodate a minimal disruption to train traffic. A micropile with a precast cap may be a good alternative that can be installed in a relatively short period of time and the installation equipment is mobile; e.g. a large excavator can be used to install the micropiles instead of a crane used to install driven piles or drilled shafts. This alternative will be dependant upon the soil conditions and loading on the stoplog.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by a minimum of 2.9', increasing the lift on the bridge from 8.8' to 11.7', reinforcing the bearings and reinforcing the piers.

5. Chicago Rock Island & Pacific Bridge (River mile 2.10)

Research - The Chicago Rock Island and Pacific bridge was built is a single track, 3-span through truss (2 @ 302', 1 @ 119'on E. end =723') and is not in use today. The center river pier is constructed of concrete on rock and the two adjacent piers are constructed of concrete on timber piles. The bridge is in the lifted position and is permanently shored to secure the raised position.

The low steel elevation of the bridge is at elevation 762.8. The Q500 WSEL is 764.3, a minimum of 1.5 feet above the low steel elevation of the bridge. The Q500+3' WSEL is 767.7, a minimum of 4.9' above the low steel elevation of the bridge.

The owner of the bridge is the City of Kansas City, Missouri. The contact person and information is:

Tom Degenhardt, P.E., R.L.S.
Assistant City Engineer
KCMO Public Works
19th Floor City Hall, 414 East 12th Street

Kansas City, Missouri 64106-2795
816-513-2506
816-513-2712 Fax
816-719-0191 Cell
tom_degenhardt@kcmo.org

Opinion - The failure mode analyses should include the investigation of one river pier this is on rock and one river pier that is concrete on timber piles driver to rock. The Q500 WSEL and Q500 +3 are both above the low steel of the bridge and both flood events will need to be investigated. No underwater inspection will be completed; the analysis will include a relative evaluation of a pile section that is agreed upon prior to the analysis.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by a minimum of 4.5', removing the bridge, reinforcing the bearings and reinforcing the piers. If the analysis results in a bridge failure, complete removal of the existing bridge would be the most advantageous option from a river hydraulics standpoint.

The existing levee wall surrounds the abutments of this bridge and will need to be raised. No stoplogs are needed at this bridge.

6. Kansas City Terminal Bridge (River mile 2.53)

Research - The Kansas City Terminal bridge was built in 1914 and revised to a lift bridge in 1967 and is a bi-level, double track 3-span through truss (2 @300', 1 @ 132'on W. end =732'). The bridge has been retrofitted to raise 10.5' vertically with hydraulic cylinders. Two of the three spans were raised in the 1993 flood. The river piers are concrete on rock and the abutments are concrete piers on timber pile driven to rock.

The low steel elevation of the bridge in a raised position is at elevation 766.4. The Q500 WSEL is 766.7, a minimum of 0.3 feet above the low steel elevation of the bridge in a raised position. The Q500+3' WSEL is 766.7, a minimum of 3.3' above the low steel elevation of the bridge in a raised position.

The owner of the bridge is the Kansas City Terminal Railroad. The contact person and information is as follows:

Chuck Mader
Chief Engineer
4501 Kansas Avenue
Kansas City, MO 66106
Phone: 913-551-2596
cemader@kctrailway.com

Opinion - The failure mode analyses should include the investigation of one river pier and bearings for both flood elevations since they are both above the low steel elevation of the bridge. No underwater inspection will be completed.

There is a large amount of hydraulic fluid that has leaked onto the bridge piers and there is evidence that the hydraulic mechanisms do not operate adequately to lift the bridge.

Possible mitigation measures for the bridge include; raising the bridge on the existing alignment by a minimum of 3.3', increasing the lift on the bridge from 10.5' to 13.8', reinforcing the bearings and reinforcing the piers.

Stoplogs at both the east and west abutment of the bridge and yard immediately east of the bridge will need to accommodate the increased height of the floodwall. Conceptual layouts of the stoplogs and where they tie to the floodwall are included in the original scope documents. Design of the stoplogs will need to accommodate a minimal disruption to train traffic. A micropile with a precast cap may be a good alternative that can be installed in a relatively short period of time and the installation equipment is mobile e.g. a large excavator can be used to install the micropiles instead of a crane used to install driven piles or drilled shafts. This alternative will be dependant upon the soil conditions and loading on the stoplog.

7. East Kansas Avenue Bridge (River mile 2.17)

Research - The river bridge is a steel 3-span deck truss (1 @ 267.34', cantilever arm @ 48', 1 @ 317.5', cantilever arm @ 88' = 720.84'). The out to out bridge width 52'-6" and it has a concrete deck that will accommodate two lanes of traffic in each direction. The river piers are concrete shafts that bear on rock. The original bridge plans were not included with the plans provided but the 1953 plans for the 5' bridge raise were included. The owner of the bridge is The Unified Government of Wyandotte County, the contact is Fred Backus. The bridge is currently in service.

The low steel elevation of the bridge is 760.8. The Q500 WSEL is 764.9, a minimum of 4.1' above the low steel elevation of the bridge. The Q500+3' WSEL is 767.9, a minimum of 7.1' above the low steel elevation of the bridge.

Opinion - The potential failure mode analysis will include the investigation of the west river pier. This should be the pier that is most heavily loaded. Both flood events will need to be investigated since both events will have water on the low steel.

Potential mitigation measures for the bridge include strengthening the bearing devices, adding bearing bumper blocks, strengthening the pier and strengthening the foundation. Raising the bridge is also an option, but the height of raise will need to be carefully considered.

8. 7th Street Northbound Bridge (River mile 3.42)

Research - The river bridge is a steel 3-span continuous deck truss (1 @ 272.25', 1 @ 334.5', 1 @ 272.25' = 879'). The out to out bridge width in unknown since the plans provided did not have this information but it has a concrete deck that will accommodate two northbound lanes of traffic. The river piers are concrete piers that are supported by timber piles that have been driven to rock. The original bridge plans are dated 1933. The owner of the bridge is KDOT, the contact is Loren Risch. The bridge is currently in service.

The low steel elevation of the bridge is 756.0. The Q500 WSEL is 769.8, a minimum of 13.8' above the low steel elevation of the bridge. The Q500+3' WSEL is 772.8, a minimum of 16.8' above the low steel elevation of the bridge.

Opinion - The potential failure mode analysis should include the investigation of the east river pier. This should be the pier that is most heavily loaded and it receives direct flow from the

river. It was also observed in the field that at some time sheet pile had been placed around this pier and concrete had been added to fill the void between the pier and sheet piling. Both flood events will need to be investigated since both events will have excessive water on the low steel.

Potential mitigation measures for the bridge include strengthening the bearing devices, adding bearing bumper blocks, strengthening the pier and strengthening the foundation.

9. 18th Street Bridge (River mile 4.96)

Research - The river bridge is a steel 3-span continuous deck truss ((1 @ 329.88', 1 @ 424.12', 1 @ 330.86' = 1084.86'). The out to out bridge width is 62'-6" and it has a concrete deck that will accommodate two lanes of traffic in both directions. The river piers are concrete piers that are supported by timber piles that have been driven to rock. The original bridge plans are dated 1957. The owner of the bridge is KDOT, the contact is Loren Risch. The bridge is currently in service.

The low steel elevation of the bridge is 764.3. The Q500 WSEL is 772.9, 8.6' above the low steel elevation of the bridge. The Q500+3' WSEL is 775.9, 11.6' above the low steel elevation of the bridge.

Opinion - The potential failure mode analysis should include the investigation of the east river pier. This should be the pier that is most heavily loaded and it receives direct flow from the river. Both flood events will need to be investigated since both events will have water on the low steel.

Potential mitigation measures for the bridge include strengthening the bearing devices, adding bearing bumper blocks, strengthening the pier and strengthening the foundation. Raising the bridge is also an option, but the height of raise will need to be carefully considered.

Hydraulics/Scour Analysis Overview

The total scour at any waterway is comprised of three components: long-term scour (aggradation/degradation), contraction scour, and local scour. Long-term scour (aggradation/degradation) is the change in the river bottom that occurs over time, either naturally or due to man-made activities. Usually, detailed historical data for the study site is used to estimate how the stream bed will behave over a period of time. Observations and discussions with the Kaw Valley Drainage District indicate that bed degradation may have occurred in the Kansas River at one or more of these bridges. Original construction plans and Operation and Maintenance manuals will provide some insight to the original bed levels. More recent information from the Corps is currently being sought. In addition to more recent soundings or bed levels that may be available from the COE, the Kansas Department of Transportation should have some data for their bridges. In a worst case scenario, soundings would need to be taken to obtain a recent bed profile during the next phase of the study.

Contraction scour is a result of flow contraction created by narrowing of the floodplain at the bridge crossing location. Due to the existence of levees (or high ground) on both sides of the river and the span of the bridge from levee to levee, there is no hydraulic constriction created from an overall floodplain standpoint. Therefore there is generally no contraction scour present.

Local scour is caused by the vortices which occur when the flow impacts the support posts or footings. The action of the vortex removes bed material from around the structure. With the transport rate of sediment away from the structure greater than the transport rate into the region, a scour hole develops. With the increased water surface from the potential project improvements, local scour will increase directly. The parameters needed for the local scour calculations will be obtained from bridge plans, field observations, and the COE hydraulic models. The hydraulic models are assumed to be essentially complete, with the exception of updates to the low steel elevations and potential changes to the bed profile information.

Scour analysis typically consists of data collection and field inspection, hydrology and hydraulics, scour calculations and assessment, and countermeasure design. During the recent site visits for the Kansas City Levees Study, observations were made about scour potential. This included looking at evidence of present or past scour, channel stability, the approach angle of flow, soil composition, and existing countermeasures. Refer to the field notes in the Appendix for specific items noted at each bridge. Many photos were taken to document the observations as well.

Scour calculations will be performed using methodologies outlined in HEC-18. An updated procedure has been recently developed by FHWA for calculating pier scour in pressure flow situations due to apparent flaws in the latest published calculations. Although not yet released as an updated HEC-18 document, our opinion is that the procedure will be the most appropriate for the bridges in this study.

Geotechnical Analysis Overview

The role of the geotechnical staff will be to aid in the structural analysis of the failure modes which include the foundations. Depending upon foundation type, one of the following analyses may be performed; 1) calculating the tension capacity of existing piles or 2) providing an assessment of the allowable and ultimate bearing capacity of the piers founded on rock. In addition to the geotechnical evaluation of the foundations, we will provide grain size values for use in the calculation of scour. Soil grain size values will be obtained from the boring and laboratory testing information made available to HNTB by the U.S. Army Corps of Engineers, Kansas City District.

As part of the information gathering for this phase of the project, plans for the 12th Street Bridge over the Kansas River and W.B. Viaduct were obtained from HNTB files. Based on these plans, shale of the Pleasanton Group is the formation that supports the bridges included in this report. The rock referred to in this report is most likely the shale of the Pleasanton Group.

The purpose of our field reconnaissance was to observe the existing conditions near the bridges for slope instability, record the soil type at the ground surface and to observe existing conditions such as exposed piles or other observations that would have an impact on subsequent analyses. The reports of our observations are documented in the Geotechnical Site Visit Reports included in the Geotechnical Field Forms Appendix.

Conclusions

Five potential bridge failure modes should be evaluated for each bridge: bearing and anchor bolt overstress, pier section overstress, substructure overturning, substructure sliding, and foundation overstress. In order to potentially recommend a proper modification, all modes of failure should be tested. One retrofit could protect against one type of failure while leaving the bridge susceptible to another.

Scour analysis should be performed for all piers in the river to ensure that the worst-case scenario is identified and included in the structural analysis. The estimate of structural analysis effort should include opinions of how many piers will be analyzed for each bridge.

None of the bridges in the study were likely designed for an n500 water surface elevation, so they are all likely to be overstressed under proposed conditions from the information that is currently known. Some analyses for the n500 water surface elevations were left off of the Task 3b scope, which is reasonable for a feasibility level effort since they are below the low steel of the bridge. Subtasks were generally listed as OPTIONs for the n500 WSEL and BASE for the n500+3 WSEL if both are above the low steel of the bridge. The OPTION subtasks can be eliminated if the BASE analysis does not show an overstress failure. Other subtasks are broken out based on the distinction between operational immediately following an event and simply rendering it repairable at some point in the future. The recommended analysis for the study will not identify that distinction and therefore eliminate the need for several OPTION subtasks.

The embankment behind the abutments of several bridges is above the proposed flood wall elevation and no modification will be needed. The two bridges that have been abandoned have floodwalls that extend around the perimeter of the bridge abutment and the wall will need to be raised. The Missouri Pacific #72 bridge, Union Pacific Bridge and KCT Bridge all have stop gaps that will need to be modified or built at a new location.

It is desired that the organization of the Task 3b proposal minimize and narrow the scope of bridge analysis effort wherever possible, as long as it does not compromise formulation and economic decision making. The duration of Task 3b is estimated to be approximately six months, but can be expedited if necessary due to the ability to separate individual subtasks.

APPENDIX

Kansas City Levees Feasibility Study
Bridge Reconnaissance
10/23/08

Bridge Fact Sheet

1. Missouri Pacific 1 Bridge (River mile 0.26)

- 4-span through truss with 82' DPG on north end (3 @200', 1 @ 233', 1 @82' = 915')
- Single Track Bridge
- River piers- Concrete on rock
- 4th span added in 1917
- Low Steel Elevation : 759.0
- n500 WSEL: 756.9 (2.1' below low steel)
- n500+3' WSEL: 760.9 (1.9' above low steel)
- Owner: Union Pacific Railroad
- Contact: David McKernan
  Manager Industry & Public Projects
  Phone: 314-331-0682

2. Kansas City Southern Bridge (River mile 0.81)

- 3-span through truss (2 @360', 1 @ 230'= 950') – Not used
- Single Track Bridge
- River piers- Concrete on timber piles
- Constructed in 1916
- Low Steel Elevation : 759.4
- n500 WSEL: 759.0 (0.4' below low steel)
- n500+3' WSEL: 762.1 (2.7' above low steel)
- Owner:Kansas City Southern Railroad
- Contact: Srikanth Honnur
  Director Track and Bridge Construction
  Phone: 816-983-1138
  Fax: 816-983-1186
  shonnur@kcsouthern.com

3. Missouri Pacific 72 Bridge (River mile 1.62)

- 4-span through truss (3 @180', 1 @ 196'on W. end =736')
- Double Track Bridge
- Piers- Concrete on timber piles to rock

- Constructed in 1945
- Low Steel Elevation : 758.9
- n500 WSEL: 762.2 (2.3' above low steel)
- n500+3' WSEL: 765.5 (6.6' above low steel)
- Owner: Union Pacific Railroad
- Contact: David McKernan
  Manager Industry & Public Projects
  Phone: 314-331-0682

4. Union Pacific Bridge (River mile 1.64)

- 4-span through truss (3 @180', 1 @ 196'on W. end =736')
- Bridge raises 8.8' vertically on a tension screw lifting device
- Double Track Bridge
- Piers- Concrete on timber piles
- Constructed in 1945
- Low Steel Elevation : 753.9 *(762.7 in raised position)*
- n500 WSEL: 762.5 (8.6' above low steel) *(0.2' below low steel in raised position)*
- n500+3' WSEL: 765.6 (11.7' above low steel) *(2.9' above low steel in raised position)*
- Owner: Union Pacific Railroad
- Contact: David McKernan
  Manager Industry & Public Projects
  Phone: 314-331-0682

5. Chicago Rock Island & Pacific Bridge (River mile 2.10)

- 3-span through truss (2 @ 302', 1 @ 119'on E. end =723') Not used
- Bridge is in raised position with permanent shoring
- Single Track Bridge
- Piers- Concrete on timber piles to rock, river pier is on rock
- Constructed in 1945
- Low Steel Elevation : 762.8 (Current raised position)
- n500 WSEL: 764.3 (1.5' above low steel)
- n500+3' WSEL: 767.7 (4.9' above low steel)
- Owner: KCMO
- Contact: Tom Degenhardt, P.E., R.L.S.
  Assistant City Engineer
  KCMO Public Works
  19th Floor City Hall, 414 East 12th Street
  Kansas City, Missouri 64106-2795
  816-513-2506
  816-513-2712 Fax

816-719-0191 Cell
tom_degenhardt@kcmo.org

6. Kansas City Terminal Bridge (River mile 2.53)

- 3-span through truss (2 @300', 1 @ 132'on W. end =732')
- Bridge raises 10.5' vertically on hydraulic cylinders
- Double Track Two Level Ballast Deck Bridge
- River Piers – Concrete on Rock Abutments – Concrete on piles
- Constructed in 1914, revised to lift spans in 1967
- Low Steel Elevation : 755.9 *(766.4 in raised position)*
- n500 WSEL: 766.7 (10.8' above low steel) *(0.3' above low steel in raised position)*
- n500+3' WSEL: 769.7 (13.8' above low steel) *(3.3' above low steel in raised position)*
- Owner: Kansas City Terminal Railroad
- Contact: Chuck Mader
  Chief Engineer
  4501 Kansas Avenue
  Kansas City, MO 66106
  Phone: 913-551-2596
  cemader@kctrailway.com

7. East Kansas Avenue Bridge (River mile 2.17)

- 3-span deck truss (1 @ 267.34', cantilever arm @ 48', 1 @ 317.5', cantilever arm @ 88' = 720.84')
- Bridge raised 5' in 1953
- Out to out bridge width 52'-6". Two lanes each direction.
- River Bridge – West abutment on piles – River piers concrete shaft on rock
- Original plans not available. 1953 bridge modification plans available.
- Low Steel Elevation : 760.8
- n500 WSEL: 764.9 (4.1' above low steel)
- n500+3' WSEL: 767.9 (7.1' above low steel)
- The owner of the bridge is The Unified Government of Wyandotte County, the contact is Fred Backus
- The bridge is currently in service.

8. 7th Street Northbound Bridge (River mile 3.42)

- 3-span continuous deck truss (1 @ 272.25', 1 @ 334.5', 1 @ 272.25' = 879')
- Out to out bridge width unknown. Accommodates two lanes northbound.
- River Bridge – River piers on piles
- Original plans dated 1933.

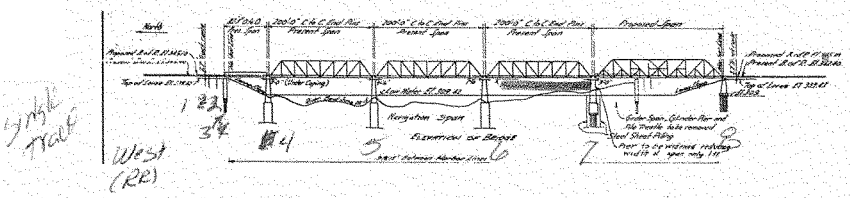
- Low Steel Elevation : 756.0
- n500 WSEL: 769.8 (13.8' above low steel)
- n500+3' WSEL: 772.8 (16.8' above low steel)
- The owner of the bridge is KDOT, the contact is Loren Risch
- The bridge is currently in service

9. 18th Street Bridge (River mile 4.96)

- 3-span continuous deck truss (1 @ 329.88', 1 @ 424.12', 1 @ 330.86' = 1084.86')
- Out to out bridge width is 62'-6". Accommodates two lanes of traffic in both directions.
- River Bridge – River piers on piles
- Original plans dated 1957.
- Low Steel Elevation : 764.3
- n500 WSEL: 772.9 (8.6' above low steel)
- n500+3' WSEL: 775.9 (11.6' above low steel)
- The owner of the bridge is KDOT, the contact is Loren Risch
- The bridge is currently in service

Observation Date: 10/14/08
Bridge: Missouri Pacific RR BR #1
Stream/ River: Kansas River - 0.26 City/ County:

Superstructure: Through TNSS, TPG on West Approach with 2 Span Concrete Girder on Concrete Piles
Substructure: - Assumed on Plan
No observed displacement of Bearings
Access: West end of Levee along tracks - East Levee & Along bike Path



Low Steel Elevation: 759.0 Source: NGVD29
Q500 WSEL: 756.9 Height from Low Steel: 2.1' Below L.S.
Q500 +3' WSEL: 760.9 Height from Low Steel: 1.9' Above Low Steel
V<12 FT/SEC

Condition Observed

Left Abutment: 2 Span Precast Concrete with TPG -
Pier 2: 3 Pile Concrete
Pier 3: 2 Pile with Concrete Cap
Pier 4: Concrete on Timber Piles
Pier 5: 6 & 7 Concrete on Timber Piles - Some concrete spalling
Right Abutment: - No observation

West Approach: Track goes under highway
East Approach: goes under I-70 & curved

Observations: - West approach - no stop log - would tie into to High ground further west.

- Ernie Quinto - said that Timber Piles are exposed at low water
East approach may need stop logs added

Signature: [signature] Date: 10/14/08

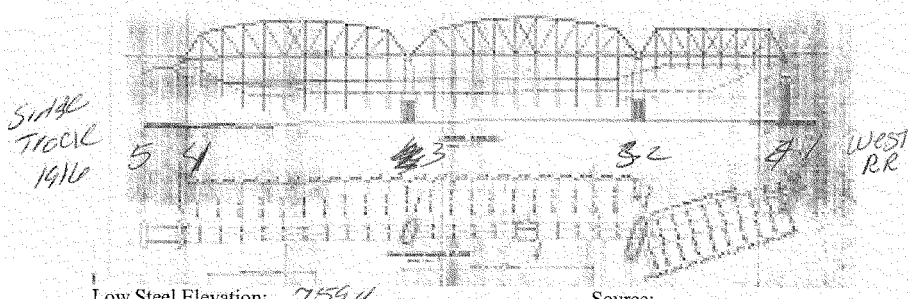
6

Observation Date: 10/15/08
Bridge: Kansas City Southern – Abandoned
Stream/ River: Kansas River - 0.81 City/ County:

Superstructure: As Shown on Plans

Substructure: Concrete on Timber Piles

Access: Left & Right Levee



Low Steel Elevation: 759.4 Source:
Q500 WSEL: 759.0 Height from Low Steel: 0.4' Below LS
Q500 +3' WSEL: 762.1 Height from Low Steel: 2.7' Above L.S.
V=13 FT/sec

Condition Observed

Left Abutment: Some Spalling
Pier 2: Some Spalling & Horizontal Cracks
Pier 3: " " " "
Pier 4: " " " "
~~Pier 5:~~
Right Abutment: Some Spalling

West Approach: NA
East Approach: NA

Observations: West Abutment - Levee Surrounds Abutment
East Abutment ties with Levee

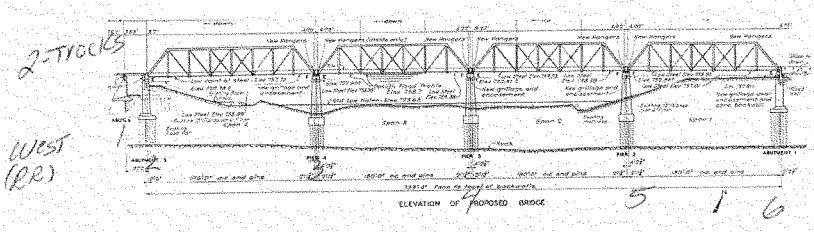
Signature: Date: 10/15/08

2

Observation Date: 10/14/08
Bridge: Missouri pacific Railroad Bridge 72
Stream/ River: Kansas River - 1.62 City/ County:

Superstructure: As Shown on Plan
- Small Beam Span Bridge @ West Approach
Substructure: Concrete on timber Piles - As Shown on Plans

Access:



Low Steel Elevation: 758.9 Source: NGVD 29
Q500 WSEL: 762.2 Height from Low Steel: 2.3' Above L.S.
Q500 +3' WSEL: 765.5 Height from Low Steel: 6.6' Above L.S.
V=13 Ft/sec

Condition Observed

Left Abutment: Some Spalling & Horizontal Cracks
Pier 2: " " " "
Pier 3: " " " " Piers have been Raised ≈8'
Pier 4: " " " "
Pier 5: " " " "
Right Abutment: No Observation

West Approach: Curve to West Allows Raise - Turn out ≈50' West
East Approach: goes under Highway

Observations: West Approach was Previously used as a RR under Bridge

- Ernie stated that he has observed exposed Timber Piles at Channel Piers

- No Displacement of Bridge Substructure evident

Signature: [signature] Date: 10/14/08

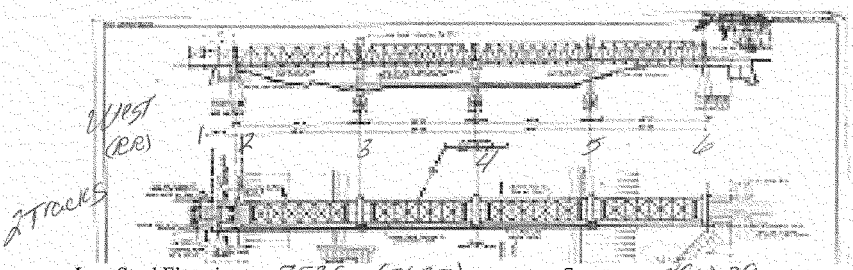
3

Observation Date: 10/14/08
Bridge: Union Pacific Railroad
Stream/ River: Kansas River 1.64 City/ County:

Superstructure:

Substructure:

Access: Levee Rd,



Low Steel Elevation: 753.9 (762.7) Source: NGVD 29
Q500 WSEL: 762.5 Height from Low Steel: 8.6' Above LS
Q500 +3' WSEL: 765.6 Height from Low Steel: 11.7' Above LS
V= 13 ft/sec

Condition Observed

Left Abutment: Abutment has some spalling
Pier 2: Added to Pier for movable Bridge support
Pier 3:
Pier 4: - Inline with Mo Pack Pier
Pier 5:
Right Abutment:

West Approach: Track could be Raised - T.O. @ Abutment
East Approach: Track goes under Roadway

Observations: Bridge was last Raised in 93 flood
- No stop block on west end
- Rail is continuous through Abutment ≈ 2.5' Below top

Stop logs - Non on E approach for Mo. Pac
- Concrete flood wall @ UPRR

- Bridge moves 8.8' vertically with a tension screw lifting Device

Signature: [signature] Date: 10/14/08

5

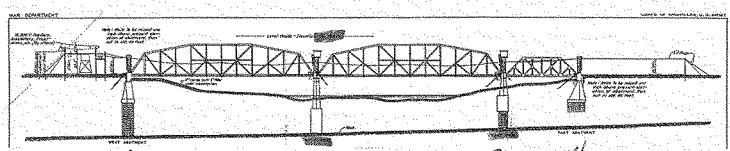
Observation Date: _____
Bridge: Chicago Rock Island & Pacific - Abandoned
Stream/ River: Kansas River 2.10 City/ County: _____

Superstructure: As shown on Plans

Substructure: As shown on Plans

Access: - Left & Right Levee

Single Track



1 2 3 4

(NA - Already up)

Low Steel Elevation: 762.8 ~~(770.8)~~ Source: NGVD 29
Q500 WSEL: 764.3 Height from Low Steel: 1.5' Above L.C.
Q500 +3' WSEL: 767.7 Height from Low Steel: 4.9' Above L.C.
V = 13 ft/sec

Condition Observed

Left Abutment: No major deterioration noted -
Pier 2: Some spalling & Horizontal Cracks - exposed formwood
Pier 3: " " " "
~~Pier 4:~~
~~Pier 5:~~
Right Abutment: Some spalling & Horizontal Cracks

West Approach: NA
East Approach: NA

Observations:
Utilities on Bridge
- Bridge ~~is in~~ is in Raised Position & Shored under bearings
- Tension Screws appear to be in ~~of~~ full upward Position
- Levee wall surrounds Abutments

Bridge moves 8' vertically with a tension screw lifting device.

Signature: [signature] Date: 10/15/08

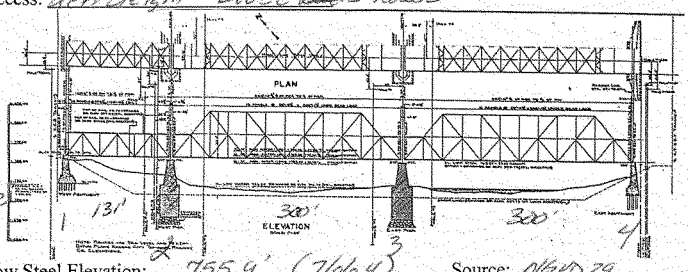
4

Observation Date: 10/14/08
Bridge: Kansas City Terminal
Stream/ River: Kansas River - 2.53 City/ County:

Superstructure: Double track - 2 level through Truss as shown on Plans
Substructure: Concrete Piers & Abutments as shown on Plans

Access: Left & Right Levee ~~Roads~~ Roads

2 Tracks Upper & lower w/ Gage



Low Steel Elevation: 755.9' (766.4) Source: NGVD 29
Q500 WSEL: 766.7' Height from Low Steel: 10.8' Above L.C.
Q500 +3' WSEL: 769.7' Height from Low Steel: 13.8' Above L.C.
V = 12 FT/SEC

Condition Observed

Left Abutment: Under stop log
Pier 2: Concret in good condition
Pier 3: " " " "
~~Pier 4:~~
~~Pier 5:~~
Right Abutment: Covered - Not observed

West Approach: Curved Approach Bridge -
East Approach: Curve to North for lower track
- Upper tracks curve to South - No Immediate vertical obstruction

Observations:
ground ≈ 12' below existing ~~flow~~ water line
Raised 1 Portion of Bridge in 93 - Right Bank
Stop logs @ Both ends of Bridge @ Abutments
- Stop logs ~~to~~ further upstream @ West end of Yard
Bridge moves 10.5 feet vertically with Hydraulic Cylinders

Signature: [signature] Date: 10/14/08

Observation Date: 10-15-2008
Bridge: Kansas Avenue East
Stream/ River: Kansas River
City/ County: Kansas City, KS Wyandotte County

Superstructure: Recently rehabbed. Bottom chord repaired and partially painted. New deck recently installed.

Substructure: Riverpiers fair condition. Some concrete spalling and cavitation at waterline. One hole 12" deep.

Access:

Low Steel Elevation: 760.8 Source:
Q500 WSEL: 764.9 Height from Low Steel: 4.1'
Q500 +3' WSEL: 767.9 Height from Low Steel: 7.1

Condition Observed

Pier 1:

Pier 2:

Pier 3:

West Approach:

East Approach:

Observations:

Signature: Thomas J. Westerman Date: 10-15-2008

Observation Date: 10-15-2008
Bridge: 7th Street Northbound
Stream/ River: Kansas River
City/ County: Kansas City, KS Wyandotte County

Superstructure: Fair to good condition.

Substructure: Fair to good condition. At some time sheet piling added around east river pier and concrete placed between pier and sheet piling.

Access:

Low Steel Elevation: 756.0 Source:
Q500 WSEL: 769.8 Height from Low Steel: 13.8'
Q500 +3' WSEL: 772.8 Height from Low Steel: 16.8'

Condition Observed

Pier 5:

Pier 6:

Pier 7:

Pier 8:

North Approach:

South Approach:

Observations:

Signature: Thomas J. Wortmann Date: 10-15-08

Observation Date: 10-15-2008
Bridge: 18th Street
Stream/ River: Kansas River
City/ County: Kansas City, KS Wyandotte County

Superstructure: Fair to good condition

Substructure: The substructure in fair to good condition. Bridge currently being jacked and having bearings replaced.

Access:

Low Steel Elevation: 764.3 Source:
Q500 WSEL: 772.9 Height from Low Steel: 8.6
Q500 +3' WSEL: 775.9 Height from Low Steel: 11.6

Condition Observed

Pier 7: some debris present

Pier 8:

Pier 9:

Pier 10:

North Approach:

South Approach:

Observations:

Signature: Thomas J Westerman Date: 10-15-2008

STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Missouri Pacific #1 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) # 12-17) *mostly stable* *some near vertical banks on outside of bend* *fairly good vegetation cover*	a. Banks (Photo(s) # 18-25) *stable* *some roots exposed* *fairly good vegetation cover*
b. Main Channel (Photo(s) #29) *clear channel* *point bar downstream on right bank* *may have degraded based on piers* *poor approach angle to piers ~10 degrees*	b. Main Channel (Photo(s) #19) *clear channel* *point bar downstream on right bank* *may have degraded based on piers* *poor approach angle to piers ~10 degrees*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) # 26; 34) *right bank just upstream of bridge*	d. Debris (Photo(s) # 17-18; 23) *left bank just downstream of bridge* *right bank just downstream of bridge*
e. Other Features (Photo(s) #_____) *dumped gravel/rock at abutment on north side*	e. Other Features (Photo(s) #_____)

3. Conditions at Bridge:

a. Structure (Photo(s) # 1-12)
round nose piers, square at the bottom
spalling and light cracks in river piers
no evidence of scour at piers

b. Channel Protection & Scour Countermeasures (Photo(s) # 14-18; 32-33)
woody debris and tree limbs/roots on outside of bend; some scouring of bank
riprap protection on left bank upstream with piles or buried tree branches sticking out
riprap protection on upper banks

c. Waterway Area (Photo(s) # 19-20; 29)
clear
point bar downstream toward confluence
no debris on piers

4. Additional Comments:
Ernie (KVDD) has seen some exposed piles during very low water in the river
Ernie (KVDD) said there was no damage to the bridge in the 1993 flood
observed about 1.3 fps velocity on day of visit

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Missouri Pacific #1 Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

GEOMORPHIC FACTORS

PROPERTIES						COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide)		Medium (100-500 ft or 30-150 m wide)		Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral	Intermittent		Perennial but flashy	Perennial	Perennial
BED MATERIAL	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)		Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)		Narrow (2 - 10X channel width)		Wide (>10X channel width)	Narrow between levees/floodwalls
NATURAL LEVEES	Little or none		Mainly on concave		Well developed on both banks	None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised			Probably Incised		Incised
CHANNEL BOUNDARIES (erosion)	Alluvial		Semi-alluvial		Non-alluvial	Alluvial
TREE COVER ON BANKS	<50 percent of bank line		50-90 percent		>90 percent	50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1-1.05)	Sinuous (1.06-1.25)	Meandering (1.26-2.0)		Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)		Locally braided (5 - 35 percent)		Generally braided (5 - 35 percent)	Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent)		Locally anabranched (5 - 35 percent)		Generally anabranched (5 - 35 percent)	Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Narrow Point Bars	Wider at bends	Wide Point Bars	Random Variation / Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures,* Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Missouri Pacific #1 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
MoPac#1_001	Armourdale	looking northwest at north abutment (Bent 1)
MoPac#1_002		looking northwest at Bent 2
MoPac#1_003		looking north at north abutment area, Bent 1, Bent 2, and Bent 3
MoPac#1_004		looking south at Bent 4
MoPac#1_005		looking east along Bent 4; no evidence of scour
MoPac#1_006		exposed bands of cable around Bent 4
MoPac#1_007		looking south at Bent 5
MoPac#1_008		looking south at Bent 5
MoPac#1_009		looking across the downstream face of the bridge from left bank; Bents 5, 6, and 7
MoPac#1_010		looking at Bent 5 and Bent 6 from downstream left bank
MoPac#1_011		looking at Bent 6 and Bent 7 from downstream left bank
MoPac#1_012		looking south at upstream right bank
MoPac#1_013		looking south at upstream right bank
MoPac#1_014		looking upstream from the downstream left bank; Bent 5 and Bent 6
MoPac#1_015		looking southwest (upstream) at downstream left bank
MoPac#1_016		looking southwest at downstream left bank, woody debris
MoPac#1_017		looking southwest at downstream left bank, woody debris
MoPac#1_018		looking northeast at downstream left bank
MoPac#1_019		looking downstream toward confluence
MoPac#1_020		looking southeast at downstream right bank toward confluence
MoPac#1_021		looking southeast at downstream right bank toward confluence
MoPac#1_022		looking southeast at downstream right bank
MoPac#1_023		looking southeast at downstream right bank; exposed tree roots
MoPac#1_024		looking southeast at downstream right bank
MoPac#1_025		looking north at downstream right bank
MoPac#1_026	CID	looking at the upsteam face of the bridge; Bent 4, Bent 5 and Bent 6
MoPac#1_027		looking at the upsteam face of the bridge; Bent 4, Bent 5 and Bent 6
MoPac#1_028		looking at the upsteam face of the bridge; Bent 4, Bent 5 and Bent 6
MoPac#1_029		looking upstream from below I-70 bridge
MoPac#1_030		looking northwest at the left bank upstream
MoPac#1_031		looking at the left bank just upstream of I-70 bridge; evidence of abutment scour
MoPac#1_032		looking at the upstream left bank; riprap and piling?
MoPac#1_033		looking at the upstream left bank
MoPac#1_034		looking at the downstream left bank
MoPac#1_035		looking at the downstream right bank
MoPac#1_036		looking downstream from upsteam of bridge
MoPac#1_037		looking northeast at right bank under bridge; Bent 7
MoPac#1_038		looking northeast at right bank under bridge

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas City Southern
Job No.: 45529
Inspected By: Dave Renetzky
Date: 10/14/08
Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *mostly stable* *good angle on bank material; some erosion* *good vegetation cover*	a. Banks (Photo(s) #) *stable* *some roots exposed* *fairly good vegetation cover*
b. Main Channel (Photo(s) #) *clear channel; minor woody debris* *some deposition on inside of bend* *pier is in a bend; decent approach angle*	b. Main Channel (Photo(s) #) *clear channel; minor woody debris* *some deposition on inside of bend*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #) *right bank just upstream of bridge* on pier	d. Debris (Photo(s) #) *left bank just downstream of bridge* *deposited material may indicate scour at pier*
e. Other Features (Photo(s) #_____)	e. Other Features (Photo(s) #_____)

3. Conditions at Bridge:

a. Structure (Photo(s) #)
round nose piers
spalling and light cracks in river piers

b. Channel Protection & Scour Countermeasures (Photo(s) #)
riprap protection on upper banks
soil entrenched riprap protection just upstream on outside of bend

c. Waterway Area (Photo(s) #)
clear
light debris on pier

4. Additional Comments:

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Kansas City Southern Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

PROPERTIES	GEOMORPHIC FACTORS				COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide)	Medium (100-500 ft or 30-150 m wide)	Wide (>500 ft or 150 m wide)		Wide
FLOW HABIT	Ephemeral	Intermittent	Perennial but flashy	Perennial	Perennial
BED MATERIAL	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)	Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)	Narrow (2 - 10X channel width)	Wide (> 10X channel width)		Narrow between levees/floodwalls
NATURAL LEVEES	Little or none	Mainly on concave	Well developed on both banks		None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised	Probably Incised			Incised
CHANNEL BOUNDARIES (erosion)	Alluvial	Semi-alluvial	Non-alluvial		Alluvial
TREE COVER ON BANKS	<50 percent of bank line	50-90 percent	>90 percent		50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1 -1.05)	Sinuous (1.06 -1.25)	Meandering (1.26 -2.0)	Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)	Locally braided (5 - 35 percent)	Generally braided (5 - 35 percent)		Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent)	Locally anabranched (5 - 35 percent)	Generally anabranched (5 - 35 percent)		Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Wider at bends	Random Variation	Irregular point and lateral bars	Equiwidth; some point bars
	Narrow Point Bars	Wide Point Bars			

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas City Southern
Job No.: 45529
Inspected By: Dave Renetzky
Date: 10/14/08
Sheet No: 3

HNTB

Photo	Levee Side	Description
KCS_001	CID	looking southwest across the bridge
KCS_002		looking at the original bridge design plate
KCS_003		looking at the original bridge design plate
KCS_004		looking southwest across the upstream face of the bridge
KCS_005		looking at the upstream right bank
KCS_006		looking south at the upstream right bank
KCS_007		looking at the channel and upstream right bank
KCS_008		looking at the upstream left bank
KCS_009		looking across the tracks from the middle of the bridge
KCS_010		looking west at the downstream left bank
KCS_011		looking further downstream on the left bank
KCS_012		looking further downstream on the left bank
KCS_013		looking further downstream on the left bank
KCS_014		looking at the downstream channel and right bank
KCS_015		looking at the right bank downstream from bridge
KCS_016		looking north at the upper bank protection downstream of the bridge
KCS_017		looking south at the abutment area (Bent 4)
KCS_018		looking northeast across the bridge
KCS_019		looking at upstream face of bridge
KCS_020		looking at upstream right bank
KCS_021		looking further upstream at the right bank
KCS_022		looking at the upstream channel and left bank
KCS_023		looking at the upstream channel and left bank
KCS_024	Armourdale	looking at the upstream left bank riprap protection
KCS_025		looking at the high bank protection on the downstream left bank
KCS_026		looking at the high bank protection on the downstream left bank
KCS_027		looking at the downstream left bank from just upstream of the bridge
KCS_028		looking at Bent 2
KCS_029		looking downstream at the channel
KCS_030		looking downstream at the channel
KCS_031		looking across the channel at Bent 3; note debris

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Missouri Pacific #72 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *mostly stable* *fairly good vegetation cover*	a. Banks (Photo(s) #) *fairly stable* *right bank has lost vegetation* *riprap protection just downstream*
b. Main Channel (Photo(s)) *clear channel* *may have degraded based on piers*	b. Main Channel (Photo(s) #) *debris along banks* *piers upstream and downstream* *good approach angle to piers ~10 degrees*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #) *on Bents of upstream UP#3 bridge*	d. Debris (Photo(s) #) *along downstream right bank*
e. Other Features (Photo(s) #_____) *UP#3 bridge piers line up*	e. Other Features (Photo(s) #_____) riprap on high banks

3. Conditions at Bridge:

a. Structure (Photo(s) #)
sharp nose piers, square at the bottom
spalling and light cracks in river piers
possible scour evidence at Bent 4

b. Channel Protection & Scour Countermeasures (Photo(s) #)
woody debris and some scouring of bank
riprap protection on right bank downstream
riprap protection on upper banks

c. Waterway Area (Photo(s) #)

4. Additional Comments:

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Missouri Pacific #72 Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

GEOMORPHIC FACTORS

PROPERTIES					COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide)	Medium (100-500 ft or 30-150 m wide)	Wide (>500 ft or 150 m wide)		Wide
FLOW HABIT	Ephemeral	Intermittent	Perennial but flashy	Perennial	Perennial
BED MATERIAL	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)	Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)	Narrow (2 - 10X channel width)	Wide (>10X channel width)		Narrow between levees/floodwalls
NATURAL LEVEES	Little or none	Mainly on concave	Well developed on both banks		None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not incised	Probably incised			Incised
CHANNEL BOUNDARIES (erosion)	Alluvial	Semi-alluvial	Non-alluvial		Alluvial
TREE COVER ON BANKS	<50 percent of bank line	50-90 percent	>90 percent		50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1-1.05)	Sinuous (1.06-1.25)	Meandering (1.26-2.0)	Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)	Locally braided (5 - 35 percent)	Generally braided (5 - 35 percent)		Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent)	Locally anabranched (5 - 35 percent)	Generally anabranched (5 - 35 percent)		Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Wider at bends	Random Variation	Narrow Point Bars / Wide Point Bars / Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Missouri Pacific #72 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
MoPac#72_001	Armourdale	looking north under the slot area
MoPac#72_002		looking north at Bent 5
MoPac#72_003		looking north at Bent 5
MoPac#72_004		looking north at Bent 5
MoPac#72_005		looking downstream at the right bank
MoPac#72_006	CID	looking downstream at the left bank
MoPac#72_007		looking downstream at the left bank
MoPac#72_008		looking downstream at the left bank
MoPac#72_009		looking downstream at the left bank
MoPac#72_010		looking northeast across the downstream face of the bridge
MoPac#72_011		looking at the downstream right bank
MoPac#72_012		looking north at the downstream channel
MoPac#72_013		looking north at the downstream channel
MoPac#72_014	Armourdale	looking at the right bank of the downstream bridge
MoPac#72_015		looking north at the downstream channel
MoPac#72_016		looking at the downstream left bank and slot area
MoPac#72_017		looking at the bank just downstream of the MoPac #72 bridge
MoPac#72_018		looking southwest across the downstream face of the bridge
MoPac#72_019	CID	looking southwest across the downstream face of the bridge
MoPac#72_020		looking southeast at the abutment area, Bent 5

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Union Pacific #3 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) # 10-12) *stable* *fairly good vegetation cover*	a. Banks (Photo(s) # 18-25) *stable* *fairly good vegetation cover* *steeper banks on right side*
b. Main Channel (Photo(s) #8-13) *clear channel* *in a straight reach* *may have degraded based on piers*	b. Main Channel (Photo(s) #19) *clear channel* *some deposited rocky material on right bank* *may have degraded based on piers*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #2-5) *on Bent 3*	d. Debris (Photo(s) #)
e. Other Features (Photo(s) #_____) *upstream of MoPac bridge; piers line up*	e. Other Features (Photo(s) #_____)

3. Conditions at Bridge:

a. Structure (Photo(s) #)
pionted nose piers, square at the bottom
spalling on river piers
debris could be an issue with square base

b. Channel Protection & Scour Countermeasures (Photo(s) #)
possible protecton on Bent 4
riprap protection on upper banks

c. Waterway Area (Photo(s) #)
clear, except for multiple piers in the area

4. Additional Comments:
replaced the jump span concrete a few years ago
was raised for 1993 flood event

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Union Pacific #3 Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

GEOMORPHIC FACTORS

PROPERTIES						COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide)		Medium (100-500 ft or 30-150 m wide)		Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral	Intermittent		Perennial but flashy	Perennial	Perennial
BED MATERIAL	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)	Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)		Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)	Narrow (2 - 10X channel width)	Wide (>10X channel width)			Narrow between levees/floodwalls
NATURAL LEVEES	Little or none	Mainly on concave	Well developed on both banks			None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised	Probably Incised				Incised
CHANNEL BOUNDARIES (erosion)	Alluvial	Semi-alluvial	Non-alluvial			Alluvial
TREE COVER ON BANKS	<50 percent of bank line	50-90 percent	>90 percent			50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1 -1.05)	Sinuous (1.06 -1.25)	Meandering (1.26 -2.0)	Highly Meandering (>2.0)		Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)	Locally braided (5 - 35 percent)	Generally braided (5 - 35 percent)			Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent)	Locally anabranched (5 - 35 percent)	Generally anabranched (5 - 35 percent)			Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Wider at bends	Random Variation	Narrow Point Bars	Wide Point Bars	Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Union Pacific #3 Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
UP#3_001	CID	looking north at the upstream face of the bridge
UP#3_002		looking north at the upstream face of the bridge; Bent 3 and Bent 4
UP#3_003		looking north at the upstream face of the bridge; Bent 3 is in the middle
UP#3_004		looking north at Bent 3
UP#3_005		looking north at Bent 3; debris
UP#3_006		looking north at Bent 4
UP#3_007		looking at the swirling water between Bent 3 and the MoPac bridge
UP#3_008		looking southwest at Bent 3
UP#3_009		looking southwest at debris on Bent 3
UP#3_010		looking southwest at upstream left bank
UP#3_011		looking further upstream at left bank
UP#3_012		looking upstream along right bank
UP#3_013		looking south through slot area
UP#3_014		looking south at bridge
UP#3_015	Armourdale	looking southeast at bridge
UP#3_016		looking northeast at bridge
UP#3_017		looking northeast along downstream face of bridge
UP#3_018		looking at Bent 4, possible scour protection
UP#3_019		looking at Bent 3
UP#3_020		looking at Bent 2
UP#3_021		looking at abutment area (Bent 1)
UP#3_022		looking northeast at the downstream right bank
UP#3_023		looking northeast at the downstream right bank
UP#3_024	CID	looking southwest across the upstream face of the bridge
UP#3_025		looking west at the bridge
UP#3_026		looking west at the bridge
UP#3_027		looking northwest at the left bank upstream
UP#3_028		looking north at the upstream channel

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: CRI&P Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *fairly stable* *fairly good vegetation cover* *deposited trees, roots, and woody debris*	a. Banks (Photo(s) #) *stable* *fairly good vegetation cover* *sparse riprap protection on outside of bend*
b. Main Channel (Photo(s) #) *upstream bridge piers* *deposited sediment on inside of bend* *fair approach angle on piers*	b. Main Channel (Photo(s) #) *woody debris*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #) *left bank just upstream of bridge* *wood debris; trees* *silt and debris on west bank*	d. Debris (Photo(s) #)
e. Other Features (Photo(s) #_____)	e. Other Features (Photo(s) #_____)

3. Conditions at Bridge:

a. Structure (Photo(s) #)
square nose piers, square at the bottom
spalling and a few fairly deep cracks in river pier
channel may be degraded based on exposed wooden forms

b. Channel Protection & Scour Countermeasures (Photo(s) #)
riprap protection on upper banks
sparse riprap on outside of bend

c. Waterway Area (Photo(s) #)
clear
deposition on inside of bend
no debris on piers

4. Additional Comments:
the bridge is abandoned
the bridge is in a permanently raised position

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: CRI&P Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

PROPERTIES	GEOMORPHIC FACTORS	COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide) / Medium (100-500 ft or 30-150 m wide) / Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral / Intermittent / Perennial but flashy / Perennial	Perennial
BED MATERIAL	Silt-Clay / Silt / Sand / Gravel / Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan / Low relief valley (<100 ft. or 30 m deep) / Moderate relief (100-1000 ft. or 30-300 m deep) / High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width) / Narrow (2 - 10X channel width) / Wide (>10X channel width)	Narrow between levees/floodwalls
NATURAL LEVEES	Little or none / Mainly on concave / Well developed on both banks	None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not incised / Probably incised	Incised
CHANNEL BOUNDARIES (erosion)	Alluvial / Semi-alluvial / Non-alluvial	Alluvial
TREE COVER ON BANKS	<50 percent of bank line / 50-90 percent / >90 percent	50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1 -1.05) / Sinuous (1.06 -1.25) / Meandering (1.26 -2.0) / Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent) / Locally braided (5 - 35 percent) / Generally braided (5 - 35 percent)	Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent) / Locally anabranched (5 - 35 percent) / Generally anabranched (5 - 35 percent)	Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth / Wider at bends / Random Variation / Narrow Point Bars / Wide Point Bars / Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures,* Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: CRI&P
Job No.: 45529
Inspected By: Dave Renetzky
Date: 10/15/08
Sheet No: 3

HNTB

Photo	Levee Side	Description
CRI&P_001	Armourdale	looking south at the west abutment (Bent 1)
CRI&P_002		looking at Bent 1
CRI&P_003		looking at abutment area and lift mechanism
CRI&P_004		looking at abutment area and lift mechanism
CRI&P_005		looking east along downstream bridge face
CRI&P_006		looking east just upstream of the bridge at the right bank
CRI&P_007		looking at the upstream channel
CRI&P_008		looking at the upstream left bank
CRI&P_009		looking at the deposition and debris on the left bank
CRI&P_010		looking at the storm sewer outlet on the left bank just downstream of the bridge
CRI&P_011		looking at the storm sewer outlet on the left bank just downstream of the bridge
CRI&P_012		looking at the erosion from the storm sewer outlet
CRI&P_013		looking at the erosion from the storm sewer outlet
CRI&P_014		looking downstream at the left bank
CRI&P_015		looking downstream at the channel and the right bank
CRI&P_016		looking across the channel at downstream right bank
CRI&P_017		looking across the channel at riprap protection under the bridge
CRI&P_018		looking at the cavitation and cracks in Bent 2
CRI&P_019		looking at the exposed wooden forms? around Bent 2
CRI&P_020		looking south at the upstream right bank from the east side of the bridge
CRI&P_021		looking at the downstream face of the bridge
CRI&P_022		looking at the downstream left and right banks
CRI&P_023		looking at the downstream channel
CRI&P_024	CID	looking further downstream of the bridge
CRI&P_025		looking south (upstream) toward the river meander
CRI&P_026		looking south (upstream) toward the river meander
CRI&P_027		looking southwest to the right bank of the bridge
CRI&P_028		looking at the lift mechanism under the abutment

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas City Terminal
Job No.: 45529
Inspected By: Dave Renetzky
Date: 10/14/08
Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) # 12-15) *mostly stable* *some lost vegetation and erosion* *fairly good vegetation cover*	a. Banks (Photo(s) # 18-22) *stable* *some erosion from local runoff on left bank* *fairly good vegetation cover*
b. Main Channel (Photo(s) #13) *clear channel* *decent approach angle to piers*	b. Main Channel (Photo(s) #19-20) *clear channel* *some deposited or placed material on right bank*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s)) *none*	d. Debris (Photo(s) # 20) *left bank just downstream of bridge*
e. Other Features (Photo(s) # 24) *high bank protection on left*	e. Other Features (Photo(s) # 31) high bank protection on left

3. Conditions at Bridge:

a. Structure (Photo(s) # 7-11)
square nose piers, exposed caisson at the bottom

b. Channel Protection & Scour Countermeasures (Photo(s))
riprap protection on upper banks

c. Waterway Area (Photo(s))
clear
no debris on piers

4. Additional Comments:
silty sand and sandy silt on the banks
clay lumps on the downstream side of the bridge
right span was raised in 1993

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Kansas City Terminal Job No.: 45529
Inspected by: Dave Renetzky Date: 10/14/2008 Sheet No: 2

HNTB

PROPERTIES	GEOMORPHIC FACTORS	COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide) / Medium (100-500 ft or 30-150 m wide) / Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral / Intermittent / Perennial but flashy / Perennial	Perennial
BED MATERIAL	Silt-Clay / Silt / Sand / Gravel / Cobble or boulder	Sandy Silt; Clay
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan / Low relief valley (<100 ft. or 30 m deep) / Moderate relief (100-1000 ft. or 30-300 m deep) / High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width) / Narrow (2 - 10X channel width) / Wide (>10X channel width)	Narrow between levees/floodwalls
NATURAL LEVEES	Little or none / Mainly on concave / Well developed on both banks	None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised / Probably Incised	Incised
CHANNEL BOUNDARIES (erosion)	Alluvial / Semi-alluvial / Non-alluvial	Alluvial
TREE COVER ON BANKS	<50 percent of bank line / 50-90 percent / >90 percent	50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1 -1.05) / Sinuous (1.06 -1.25) / Meandering (1.26 -2.0) / Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent) / Locally braided (5 - 35 percent) / Generally braided (5 - 35 percent)	Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent) / Locally anabranched (5 - 35 percent) / Generally anabranched (5 - 35 percent)	Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth / Wider at bends / Random Variation / Narrow Point Bars / Wide Point Bars / Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas City Terminal Job No.: 45529
Inspected By: Dave Renetzky Date: 10/14/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
KCT_001	Armourdale	looking along downstream face of bridge from northwest abutment (Bent 1)
KCT_002		looking southwest at Bent 1 and lift mechanism
KCT_003		looking southwest at riprap protection along abutment area
KCT_004		looking southwest at riprap protection along abutment area
KCT_005		looking under bridge at Bent 1
KCT_006		looking south at Bent 2
KCT_007		looking south across downstream face of bridge
KCT_008		looking at the north side of Bent 2
KCT_009		looking at the south side of Bent 2
KCT_010		looking at the south side of Bent 2 (local flow erosion along pier)
KCT_011		looking at the south side of Bent 2
KCT_012		looking north (upstream) at the left bank
KCT_013		looking north (upstream) at the banks
KCT_014		looking northwest (upstream) at the right bank
KCT_015		looking northwest across the channel at Bent 3 and upstream
KCT_016		looking northwest across the channel at Bent 3
KCT_017		looking northwest at Bent 3 and wooden caisson forms?
KCT_018		looking northwest across the channel at Bent 3 and downstream
KCT_019		looking west at the downstream right bank
KCT_020		looking downstream at the channel and banks
KCT_021		looking northwest at the left bank downstream of bridge
KCT_022		looking downstream at left bank erosion from Bent 2
KCT_023		looking at the outlet of OK Creek on downstream right bank
KCT_024	CID	looking upstream at the high bank riprap protection
KCT_025		looking at the upstream face from the southeast abutment (Bent 4)
KCT_026		looking across the channel at Bent 3 on the upstream side
KCT_027		looking southwest at Bent 4
KCT_028		looking at the downstream face from the southeast abutment (Bent 4)
KCT_029		looking across the channel at Bent 3 on the downstream side
KCT_030		looking north at the downstream channel
KCT_031		looking downstream along the high left bank protection
KCT_032		looking at the stoplog gap area
KCT_033		looking at the stoplog gap area

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas Avenue East Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *stable* *good vegetation cover*	a. Banks (Photo(s) #) *mostly stable* *fairly good vegetation cover* *some erosion, woody debris between bridges*
b. Main Channel (Photo(s) #) *clear channel* *poor approach angle to piers ~10 degrees*	b. Main Channel (Photo(s) #) *clear channel* *decent approach angle to piers - maybe 5 degrees*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #) *some deposition on left bank just upstream*	d. Debris (Photo(s) #) *left bank just downstream of bridge* *right bank downstream of bridge*
e. Other Features (Photo(s) #_____) *bridge just downstream* force main dumps in on left bank just upstream	e. Other Features (Photo(s) #_____) grouted riprap at outfall

3. Conditions at Bridge:

a. Structure (Photo(s) #)
round nose piers
heavy cracks, spalling, and cavitation on river pier
no evidence of scour at piers

b. Channel Protection & Scour Countermeasures (Photo(s) #)
sparse riprap downstream right bank
riprap protection under abutment
riprap protection on high banks

c. Waterway Area (Photo(s) #)
clear
small amount of woody debris
bridge in close proximity downstream

4. Additional Comments:
recent rehab to the lower portion of the superstructure

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: Kansas Avenue East — Job No.: 45529
Inspected by: Dave Renetzky — Date: 10/14/2008 — Sheet No: 2

HNTB

PROPERTIES	GEOMORPHIC FACTORS	COMMENTS
STREAM SIZE	Small (<100ft or 30 m wide); Medium (100-500 ft or 30-150 m wide); Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral; Intermittent; Perennial but flashy; Perennial	Perennial
BED MATERIAL	Silt-Clay; Silt; Sand; Gravel; Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan; Low relief valley (<100 ft. or 30 m deep); Moderate relief (100-1000 ft. or 30-300 m deep); High relief (>1000 ft or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width); Narrow (2 - 10X channel width); Wide (>10X channel width)	Narrow between levees/floodwalls
NATURAL LEVEES	Little or none; Mainly on concave; Well developed on both banks	None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised; Probably Incised	Incised
CHANNEL BOUNDARIES (erosion)	Alluvial; Semi-alluvial; Non-alluvial	Alluvial
TREE COVER ON BANKS	<50 percent of bank line; 50-90 percent; >90 percent	50-90 percent
SINUOSITY (ratio of curved channel length to straight length)	Straight (Sinuosity 1 -1.05); Sinuous (1.06 -1.25); Meandering (1.26 -2.0); Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent); Locally braided (5 - 35 percent); Generally braided (5 - 35 percent)	Not braided
ANABRANCHED STREAMS (substantial islands)	Not anabranched (<5 percent); Locally anabranched (5 - 35 percent); Generally anabranched (5 - 35 percent)	Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth; Wider at bends; Random Variation; Narrow Point Bars; Wide Point Bars; Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: Kansas Avenue East Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
KSAveE_001		looking east along the upstream face of the bridge
KSAveE_002		looking south along the upstream face of the bridge
KSAveE_003		looking east at Bent 2 and Bent 3
KSAveE_004		looking at the USGS gage on the bridge
KSAveE_005		looking at the old USGS station
KSAveE_006		looking east at Bent 2
KSAveE_007		looking east at upstream right bank
KSAveE_008		looking further upstream at the right bank
KSAveE_009	Armourdale	looking at the channel upstream of the bridge
KSAveE_010		looking south at the left high bank upstream of the bridge
KSAveE_011		looking at the left bank just downstream of the bridge
KSAveE_012		looking northeast across the channel at the downstream right bank
KSAveE_013		looking at a significant outfall along the downstream right bank
KSAveE_014		looking across the channel at Bent 2 and Bent 3
KSAveE_015		looking southeast at Bent 2
KSAveE_016		closeup of cavitation on Bent 2
KSAveE_017		closeup of cavitation on Bent 3
KSAveE_018	CID	looking at the east abutment area
KSAveE_019		looking at the east end of the bridge

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: 7th Street Northbound
Job No.: 45529
Inspected By: Dave Renetzky
Date: 10/15/08
Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *mostly stable* *fairly good vegetation cover* *some vegetation loss on left bank*	a. Banks (Photo(s) #) *stable* *some debris and rocks* *fairly good vegetation cover*
b. Main Channel (Photo(s) #) *clear channel* *point bar developed just upsteram of bridge* *may have degraded based on piers*	b. Main Channel (Photo(s) #) *clear channel* *may have degraded based on piers*
c. Floodplain (Photo(s) #_____) *levee on one side; high ground on the other*	c. Floodplain (Photo(s) #_____) *levee on one side; high ground on the other*
d. Debris (Photo(s) #) *left bank just upstream of bridge*	d. Debris (Photo(s) #) *left bank just downstream of bridge*
e. Other Features (Photo(s) #_____) *rock protecting levee slope at abutment*	e. Other Features (Photo(s) #_____)

3. Conditions at Bridge:

a. Structure (Photo(s) #)
round nose piers, square at the bottom
exposed sheet pile
evidence of possible scour at Bent 2

b. Channel Protection & Scour Countermeasures (Photo(s) #)

c. Waterway Area (Photo(s) #)
clear
deposition occurring upstream near left bank
no debris on piers

4. Additional Comments:

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: 7th Street Northbound Job No.: 45529
Inspected by: Dave Renetzky Date: Date: 10/15 Sheet No: 2

HNTB

GEOMORPHIC FACTORS

PROPERTIES					COMMENTS
a. Banks (Photo(s) #) SIZE	Small (<100ft or 30 m wide)	a. Banks (Photo(s) #) Medium (100-500 ft or 30-150 m wide)		Wide (>500 ft or 150 m wide)	Wide
FLOW HABIT	Ephemeral	Intermittent	Perennial but flashy	Perennial	Perennial
BED b. Main Channel (Photo(s) #)	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)	Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)	Narrow (2 - 10X channel width)	Wide (>10X channel width)		Narrow between levees/floodwalls
NATURAL d. Debris (Photo(s) #)	Little or none	Mainly on concave	Well developed on both banks		None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised	Probably Incised			Incised
CHANNEL BOUNDARIES (erosion)	Alluvial	Semi-alluvial	Non-alluvial		Alluvial
a. Structure (Photo(s) #) ON BANKS	<50 percent of bank line	50-90 percent	>90 percent		50-90 percent
SINUOSITY (ratio of curved channel length to b. Channel Protection & Scour Countermeasures (Photo(s) #)	Straight (Sinuosity 1 -1.05)	Sinuous (1.06 -1.25)	Meandering (1.26 -2.0)	Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)	Locally braided (5 - 35 percent)	Generally braided (5 - 35 percent)		Not braided
c. Waterway Area (Photo(s) #) STREAMS (substantial islands)	Not anabranched (<5 percent)	Locally anabranched (5 - 35 percent)	Generally anabranched (5 - 35 percent)		Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Wider at bends	Random Variation		Equiwidth; some point bars
	Narrow Point Bars	Wide Point Bars	Irregular point and lateral bars		

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures*, Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: 7th Street Northbound Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
7thStN_001	Armourdale	looking south across the downstream face of the bridge
7thStN_002		looking south across the downstream face of the bridge
7thStN_003		looking south across the downstream face of the bridge
7thStN_004		looking south across the downstream face of the bridge
7thStN_005		looking southeast across the upstream face of the bridge
7thStN_006		looking southeast at Bent 3
7thStN_007		looking along left upstream bank
7thStN_008		looking upstream from left bank
7thStN_009		looking at upstream right bank from left bank
7thStN_010		looking at downstream right bank just downstream of bridge
7thStN_011		looking at downstream right bank further downstream of bridge
7thStN_012		looking at downstream channel and left bank
7thStN_013		looking at downstream channel and left bank
7thStN_014		looking southeast at Bent 3
7thStN_015		looking southeast at Bent 3 sheetpile
7thStN_016		looking north from Bent 2 area
7thStN_017		looking at possible scour of Bent 2
7thStN_018		looking at possible scour of Bent 2
7thStN_019		looking at possible scour of Bent 2
7thStN_020		looking at possible scour of Bent 2
7thStN_021		looking at possible scour of Bent 3
7thStN_022		looking at possible scour of Bent on Southbound bridge; likely local flow erosion
7thStN_023		looking at possible scour of Bent on Southbound bridge; likely local flow erosion
7thStN_024		looking at possible scour of Bent on Southbound bridge; likely local flow erosion
7thStN_025		looking southeast toward Bent 2
7thStN_026		looking northwest at Bent 1
7thStN_027		looking southeast from Bent 1

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: 18th Street Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 1

HNTB

1. Upstream Conditions:	2. Downstream Conditions:
a. Banks (Photo(s) #) *stable* *fairly good vegetation cover*	a. Banks (Photo(s) #) *stable* *fairly good vegetation cover*
b. Main Channel (Photo(s) #) *vegetated point bar upstream* *piers are aligned with river*	b. Main Channel (Photo(s) #) *clear channel*
c. Floodplain (Photo(s) #_____) *levees on both sides*	c. Floodplain (Photo(s) #_____) *levees on both sides*
d. Debris (Photo(s) #) *right bank just upstream of bridge* heavy on piers	d. Debris (Photo(s) #)
e. Other Features (Photo(s) #_____) *high bank protection*	e. Other Features (Photo(s) #_____) high bank protection

3. Conditions at Bridge:

a. Structure (Photo(s) #)
round nose piers
spalling on the river piers
no evidence of scour at piers

b. Channel Protection & Scour Countermeasures (Photo(s) #)
some abutment riprap protections
riprap protection on upper banks

c. Waterway Area (Photo(s) #)
clear
point bar upstream dividing channel
heavy debris on piers

4. Additional Comments:
repair work going on at time of site visit; bearing repair/replacement

P:\jobs\45529_TO1_KC_Levees\Bridge\Deliverable_LetterOfOpinion\Appendix\3_H&HFieldNotes\[ScourFieldInspection18thSt.xls]Inspection

STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No.: 18th Street — Job No.: 45529
Inspected by: Dave Renetzky — Date: Date: 10/15 — Sheet No: 2

HNTB

GEOMORPHIC FACTORS

PROPERTIES					COMMENTS
a. Banks (Photo(s) #) SIZE	Small (<100ft or 30 m wide)	a. Banks (Photo(s) #) Medium (100-500 ft or 30-150 m wide)	Wide (>500 ft or 150 m wide)		Wide
FLOW HABIT	Ephemeral	Intermittent	Perennial but flashy	Perennial	Perennial
BED b. Main Channel (Photo(s) #)	Silt-Clay	Silt	Sand	Gravel	Cobble or boulder	Sand, Sandy Silt
VALLEY SETTING (flat, hilly, mountainous)	No valley; alluvial fan	Low relief valley (<100 ft. or 30 m deep)	Moderate relief (100-1000 ft. or 30-300 m deep)	High relief (>1000 ft. or 300 m deep)	Low relief valley
FLOOD-PLAINS (width)	Little or none (<2X channel width)	Narrow (2 - 10X channel width)	Wide (> 10X channel width)		Narrow between levees/floodwalls
NATURAL d. Debris (Photo(s) #)	Little or none	Mainly on concave	Well developed on both banks		None; man-made levees/floodwalls
APPARENT INCISION (bank height vs channel width)	Not Incised	Probably Incised			Incised
CHANNEL BOUNDARIES (erosion)	Alluvial	Semi-alluvial	Non-alluvial		Alluvial
a. Structure (Photo(s) #) ON BANKS	<50 percent of bank line	50-90 percent	>90 percent		50-90 percent
SINUOSITY (ratio of curved channel length to b. Channel Protection & Scour Countermeasures (Photo(s) #)	Straight (Sinuosity 1 -1.05)	Sinuous (1.06 -1.25)	Meandering (1.26 -2.0)	Highly Meandering (>2.0)	Meandering
BRAIDED STREAMS (multiple and interlacing channels)	Not braided (<5 percent)	Locally braided (5 - 35 percent)	Generally braided (5 - 35 percent)		Not braided
c. Waterway Area (Photo(s) #) STREAMS (substantial islands)	Not anabranched (<5 percent)	Locally anabranched (5 - 35 percent)	Generally anabranched (5 - 35 percent)		Not anabranched
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS (lateral stability)	Equiwidth	Wider at bends	Random Variation	Narrow Point Bars	Wide Point Bars	Irregular point and lateral bars	Equiwidth; some point bars

Performed in Field
Performed in Field and Office
Performed in Office

Source: "Figure 12. Hydraulic and location factors that affect stream stability." HEC-20, *Stream Stability at Highway Structures,* Federal Highway Administration, 1991

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STREAM STABILITY ANALYSIS SHEET

Stream Name: Kansas River
Bridge No: 18th Street Job No.: 45529
Inspected By: Dave Renetzky Date: 10/15/08 Sheet No: 3

HNTB

Photo	Levee Side	Description
18thSt_001		looking east from south abutment area (Bent 4)
18thSt_002		looking across the upstream face of the bridge
18thSt_003		looking down the bridge from the south abutment; Bent 3
18thSt_004		looking at the south abutment (Bent 4) and riprap
18thSt_005		looking at the upper bank protection along the right bank upstream
18thSt_006		looking at the right bank upstream
18thSt_007	Argentine	looking northwest at the upstream channel
18thSt_008		looking northwest at the upstream channel and left bank
18thSt_009		looking across the channel at Bent 2
18thSt_010		looking across the channel at Bent 4
18thSt_011		looking east from south abutment area at downstream left bank
18thSt_012		looking downstream at channel and right bank
18thSt_013		looking downstream at the high bank protection
18thSt_014		looking across the downstream face of the bridge from the north abutment (Bent 1)
18thSt_015		looking at equipment being used to replace bearings
18thSt_016		looking at Bent 1
18thSt_017		looking at Bent 1
18thSt_018	Armourdale	looking at upstream side of Bent 2 and Bent 3; note debris and spalling
18thSt_019		looking at upstream side of Bent 3; note debris
18thSt_020		looking at upstream side of Bent 2; note debris and spalling
18thSt_021		looking at the upstream right bank
18thSt_022		looking further upstream at the right bank

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Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Missouri Pacific #1
Job No. 45529
Date: 10/14/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Railroad Bridge

Notes: Pier footings supported on rock and piers supported on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Lean clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Roadway intersection	Upstream Right:	Interstate 70
Downstream Left:	Park	Downstream Right:	Buildings

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Buildings and Interstate Overpass

5. Additional Remarks:

Ernie Quinto from the KAW Valley Drainage District indicated that damage to the bridge was not evident after the 1993 flood.

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Kansas City Southern
Job No. 45529
Date: 10/15/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Abandoned Railroad Bridge

Notes: Piers supported on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Railyard	Upstream Right:	Railyard
Downstream Left:	Railyard	Downstream Right:	Railyard

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Railyards

5. Additional Remarks:

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Missouri Pacific #72
Job No. 45529
Date: 10/14/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Railroad Bridge

Notes: Piers founded on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Union Pacific #3 Bridge	Upstream Right:	Union Pacific #3 Bridge
Downstream Left:	Warehouse	Downstream Right:	Open Land

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Warehouses, highways

5. Additional Remarks:

Bank on right side showed signs of erosion.

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Union Pacific #3
Job No. 45529
Date: 10/14/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Railroad Bridge

Notes: Piers founded on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Warehouse	Upstream Right:	I-70/Open Land
Downstream Left:	Missouri Pacific #72 Bridge	Downstream Right:	Missouri Pacific #72 Bridge

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Warehouses, highways

5. Additional Remarks:

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Chicago Rock Island & Pacific
Job No. 45529
Date: 10/15/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Abandoned Railroad Bridge

Notes: Main river pier is a footing on rock, adjacent piers are founded on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Commercial Storage/Trucking	Upstream Right:	Commercial Building
	Road Bridge		Road Bridge
Downstream Left:	Commercial Buildings	Downstream Right:	Kemper Arena

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Commercial buildings, storage

5. Additional Remarks:

Timber grillage/sofit slab exposed at main river pier. In addition, right bank showed signs of erosion.

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: Kansas City Terminal
Job No. 45529
Date: 10/14/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Railroad Bridge

Notes: Main piers are footings on rock, abutments are supported on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Silty sand underlain by clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Commercial Buildings	Upstream Right:	Abandoned Building
Downstream Left:	Commercial Buildings	Downstream Right:	Railroad

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Commercial buildings, railroad

5. Additional Remarks:

The bridge span over the right bank was raised in 1993 due to the flood conditions.

Grillage/sofit slab exposed at main river pier.

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: East Kansas Avenue
Job No. 45529
Date: 10/15/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Highway Bridge

Notes: Main piers are footings on rock and abutments are supported on timber piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Clay. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left: Commercial Storage/Trucking	Upstream Right: Commercial Building
Downstream Left: Commercial Buildings	Downstream Right: Kemper Arena
Abandoned Railroad Bridge	

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Commercial buildings, storage

5. Additional Remarks:

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: 7th Street (Northbound)
Job No. 45529
Date: 10/15/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Highway Bridge

Notes: Bridge piers supported on footings or piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Sandy silt. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Vegetated Land	Upstream Right:	Railroad
Downstream Left:	Commercial Buildings	Downstream Right:	Railroad

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Commercial buildings, railroad

5. Additional Remarks:

Kansas City Levees Flood Risk
Management Project Feasibility Study
Phase 2

Bridge: 18th Street
Job No. 45529
Date: 10/15/08
Evaluated by: AKB
Reviewed by: KAK

Geotechnical Site Visit Report

Bridge over: Kansas River Description: Highway Bridge

Notes: Piers supported on piles.

1. Type of surface material of riverbank (sand, clay, silt, riprap, etc.) Any vegetation present?

Silty sand. Banks are well-vegetated. Vegetation consists of small trees and underbrush.

2. Any sliding earth fills or slopes at bridge site? Give details below:

None observed.

3. Describe the land use in each of the four quadrants near the bridge (pasture, crops, brush, developed, etc.)

Upstream Left:	Vegetated Land	Upstream Right:	Railroad
Downstream Left:	Commercial Buildings	Downstream Right:	Railroad

4. Note any improvements in the vicinity of the bridge, including residences, businesses, other buildings, crop fields, etc.

Railroad, commercial buildings

5. Additional Remarks:

Repairs are currently being made to bearings.

EXHIBIT A-9.6

Engineering Division Hydrology & Hydraulics Memorandum For Record (ED-HH MFR)

CENWK-ED-HH 24 February 2012 (Draft)

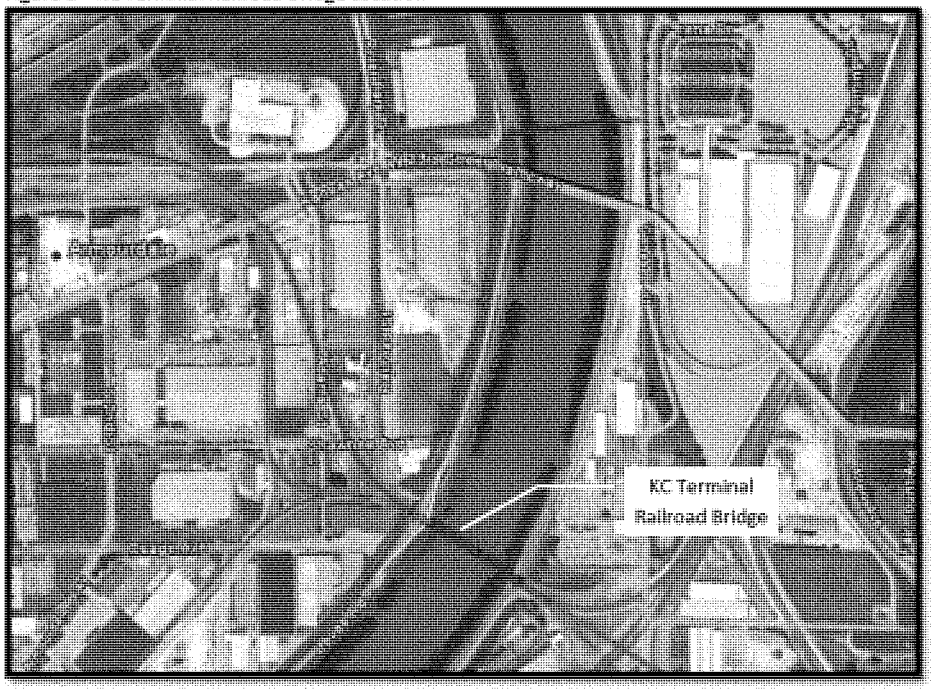
MEMORANDUM FOR Record

SUBJECT: Effects of Bridge Failure on Water Surface Elevations (KC Levees)

This hydraulic evaluation included quantifying the effects of a bridge failure on the 0.2% and 1.0% (500 and 100 year) water surface elevations on the Kansas River. There are several bridges over the Kansas River between the Kansas City Levees that could be impacted by flood waters.

The choice of which bridge to assume could fail during a 0.2% and 1.0% flood event was primarily based on the "Failure Modes" evaluation conducted by the structural group. The failure modes report indentified several bridges that would be overtopped by floodwaters during an extreme storm event. In addition, the report includes a table that provides probabilistic approach to potential bridge failure. Within the table the bridges within the reach of interest are given ratings based on the failure modes evaluated. This evaluation reveled that the bridge with the highest probability of failure during a 0.2% storm event was the KC Terminal Railroad Bridge (See Figure 1).

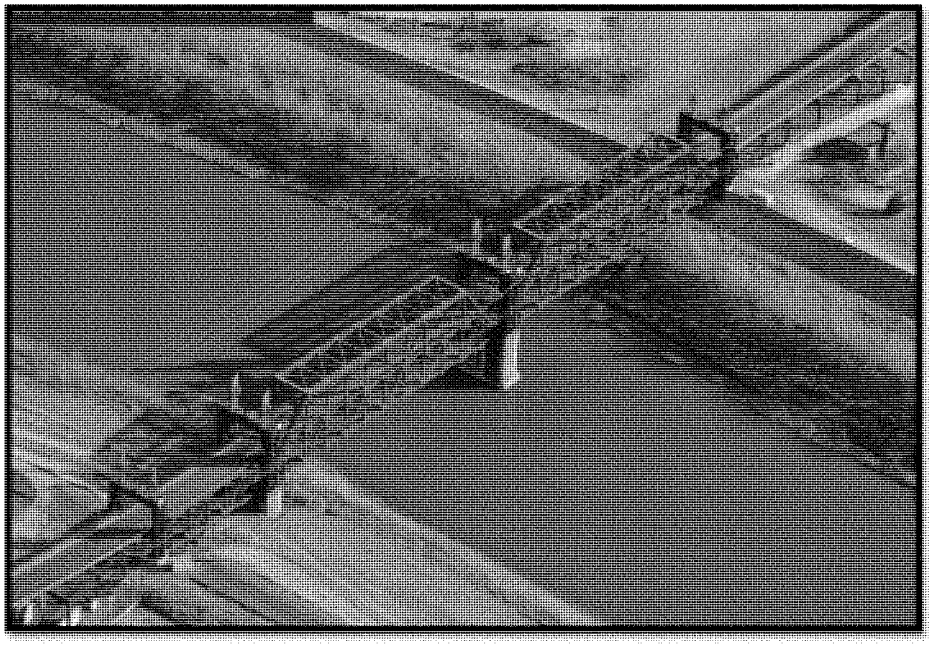
Figure 1 – KC Terminal Railroad Bridge Location



Bridge Failure Modeling Approach

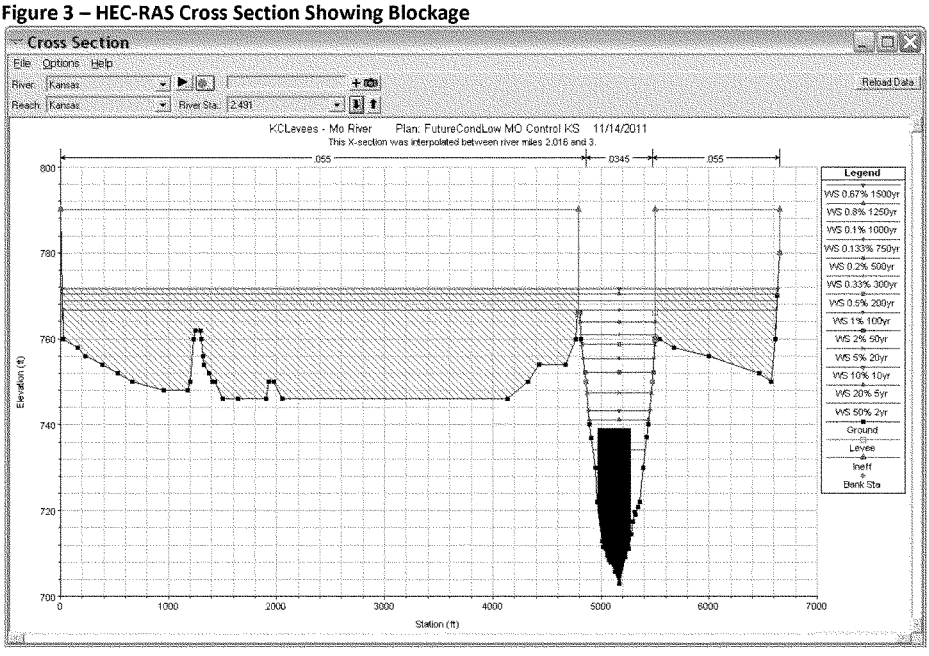
The evaluation of a potential bridge failure included modifying the proposed conditions HEC-RAS model to reflect the loss of one span from the KC Terminal Bridge. Based on photos from the 1951 Flood, the truss bridges that failed appeared to only lose the center span, while the other spans remained intact or failed in place. As such the assumption for the KC Terminal Bridge is that a failure could occur on the center span while the other two spans remain intact. A photo of the KC Terminal Bridge is shown in Figure 2.

Figure 1 – KC Terminal Railroad Bridge Photo



As noted above, the modeling approach assumes that only the center span of the KC Terminal Bridge will fail during a 0.2% flood event. For this effort it is assumed that center span failure would result in blockage of the Kansas River. In order to model the "blockage" a failed bridge would cause, a blocked obstruction was added to the cross section down stream of the bridge location (See Figure 3).

Figure 3 – HEC-RAS Cross Section Showing Blockage



Two blockage scenarios were evaluated. The first assumes that the failed center span would come to rest immediately at the bridge location, i.e. it would not be carried downstream. The second scenario assumes that the bridge would be carried/pushed downstream and trapped at the next river crossing, Kansas Avenue. To account for the open space between the bridge webbing, the height of the blocked obstruction was set to 36 feet, which is approximately one-half the height of the intact bridge structure. This height also accounts for some bridge deformation. It should also be noted that the center span of the bridge was removed from the "bridge cross section" within the HEC-RAS model to account for it not being there.

Results

The blocked obstruction resulted in an increase in water surface elevations upstream of the bridge failure. The worst case condition was a failure that remained near the bridge location. The hydraulic model indicated that for a 0.2% (500-year) event, water surface profiles would be increased by approximately 2.2 feet above non-failure conditions. The model indicates that the increase would be carried upstream and reduces to 0.7 feet above non-failure conditions 12 miles upstream of the failure location (end of the modeled reach). Comparison tables for both the 0.2% and 1.0% events are provided below.

Table 1 – 0.2% (500-year) Storm

River Station	Non-Fail Elevation	Bridge failure Elevation	Difference
14.62	782.29	782.96	0.67
14.25	782.15	782.82	0.67
13.65	781.96	782.64	0.68
13.3	781.68	782.39	0.71
12.94	781.43	782.16	0.73
12.4	781.22	781.95	0.73
11.85	780.84	781.61	0.77
11.35	780.29	781.09	0.80
10.9	779.85	780.69	0.84
10.6	779.00	779.9	0.90
10.4	778.80	779.72	0.92
9.82	778.59	779.52	0.93
9.505	778.03	778.98	0.95
9.49	777.99	778.93	0.94
9.04	777.30	778.3	1.00
8.42	776.74	777.79	1.05
7.65	775.64	776.78	1.14
7.364	775.30	776.45	1.15
7.35962	775.29	776.45	1.16
7.35087	775.26	776.42	1.16
7.34212	775.25	776.41	1.16
7.33337	775.22	776.39	1.17
7.329	775.21	776.38	1.17
6.88	774.97	776.17	1.20
5.967	774.46	775.7	1.24
5.831	774.49	775.73	1.24
5.811	774.46	775.7	1.24
5.52	773.96	775.23	1.27
5.506	773.92	775.19	1.27
4.97	773.40	774.72	1.32
4.949	773.17	774.49	1.32
4.48	771.78	773.22	1.44
4.284	771.42	772.9	1.48
4.276	771.23	772.69	1.46
3.96	770.71	772.23	1.52
3.427	769.75	771.39	1.64
3.405	769.43	771.06	1.63
3.05	768.54	770.26	1.72
2.536	766.64	768.59	1.95
2.5305	KC Terminal Bridge		
2.525	766.10	768.28	2.18

Table 2 – 1.0% (100-year) Storm

River Station	Non-Fail Elevation	Bridge failure Elevation	Difference
14.62	773.97	774.61	0.64
14.25	773.80	774.45	0.65
13.65	773.57	774.23	0.66
13.3	773.23	773.93	0.70
12.94	772.94	773.66	0.72
12.4	772.69	773.42	0.73
11.85	772.31	773.08	0.77
11.35	771.69	772.47	0.78
10.9	771.23	772.06	0.83
10.6	770.39	771.28	0.89
10.4	770.10	771	0.90
9.82	769.80	770.75	0.95
9.505	769.43	770.39	0.96
9.49	769.40	770.36	0.96
9.04	768.64	769.68	1.04
8.42	768.01	769.11	1.10
7.65	766.97	768.17	1.20
7.364	766.61	767.84	1.23
7.35962	766.60	767.84	1.24
7.35087	766.57	767.81	1.24
7.34212	766.56	767.8	1.24
7.33337	766.53	767.78	1.25
7.329	766.52	767.77	1.25
6.88	766.25	767.53	1.28
5.967	765.75	767.08	1.33
5.831	765.74	767.08	1.34
5.811	765.72	767.07	1.35
5.52	765.30	766.68	1.38
5.506	765.27	766.65	1.38
4.97	764.78	766.21	1.43
4.949	764.66	766.1	1.44
4.48	763.47	765.03	1.56
4.284	763.10	764.71	1.61
4.276	763.01	764.63	1.62
3.96	762.56	764.23	1.67
3.427	761.61	763.43	1.82
3.405	761.45	763.28	1.83
3.05	760.64	762.61	1.97
2.536	758.99	761.23	2.24
2.5305	KC Terminal Bridge		
2.525	758.75	761.08	2.33

Anthony Hall P.E.
Hydraulic Engineer

EXHIBIT A-9.7

HAZUS MR4 Flood Model Technical Manual Excerpt

Analyzing losses for lifeline facilities is for the most part similar to that discussed in the general building stock chapter, but the damage functions for lifelines really define the potential damage to components of the lifeline system that are either uniquely vulnerable to inundation (such as electrical components) or are expensive or difficult to repair/replace (such as motors, controllers, etc.).

7.2.1 Input Requirements and Output Format

7.2.2 Form of the Damage Functions

With the obvious exception of bridges (discussed below) the damage functions for lifelines are very similar to the depth to damage curves used to the general building stock. The depth of flooding within the facility is compared to the height of critical components and the amount of damage can be estimated. In most cases, the elevation of the equipment also provides for a depth of flooding at which point the functionality of the facility starts to become questionable.

7.2.3 Transportation Bridges

In discussions with the FHWA, no comprehensive database of bridge damage could be identified. As a result, the proposed damage relationships are estimates that should be calibrated once the overall flood module is operable.

HAZUS comes with the national bridge inventory database as part of the default data and the objective is to use as much of the information in that database as possible. This database includes all bridges in the U.S. with a span of 20-feet or greater. In discussions with an FHWA representative it was suggested that possible fields within the bridge inventory database include: scour potential, waterway adequacy, and span type (simple versus continuous). The FHWA also provided other references for further information.

The database field discussed, the scour potential rating, has values that range from 0 to 9, where 0 indicates the bridge is closed as a result of scour damage, and 9 indicates that the bridge is not over water. Scour ratings 4-9 are not used in HAZUS. Ratings of significance to the flood model are:

- 1 – closed
- 2 – existing problem
- 3 – 100-year flood could damage.

There appears to be a very low probability of failure due to flooding for bridges with a scour potential greater than 3. As bridges are designed for 500-year floods, for single span bridges over water (i.e. – scour potential < 9), the HAZUS Flood Model assumes a 1% probability of failure for floods with a return period of 100 years, and 1.5% probability for floods with a return period

of 1000 years. For continuous bridges, the HAZUS Flood Model uses 25% of the values for single span bridges.

The waterway adequacy gives some indication how much the bridge is restricting the channel, but there is no known correlation to bridge damage due to flooding.
Most bridge failures are simple spans. Scour of bridge piers on continuous spans does not usually result in collapse. Expected damage for continuous span bridges is taken to be 25% of that for single span bridges. Similarly, bridges with multiple piers provide redundancy that reduces their vulnerability. The span type is included in the National Bridge Database, but the number of piers is not.

The relationships for single-span and continuous-span bridge damage due to flooding is shown below.

Table 7.2 Highway Single-span Bridge Damage Relationship

Flood Return Period	Scour Potential[1]/Probability of Failure (percent)				
	1	2	3	4-8	9
100-year	5	2	1	0	N/A
500-year (2x 100-year probability)	10	4	2	0	N/A
1000-year (1.5x 500-year probability)	15	6	3	0	N/A

The Scour Potential is a field in the HAZUS Bridge database and is from the FHWA inventory of bridges

Table 7.3 Highway Continuous-Span Bridge Damage Relationship

Flood Return Period	Scour Potential[1]/Probability of Failure (percent)				
	1	2	3	4-8	9
100-year	1.25	0.5	0.25	0	N/A
500-year (2x 100-year probability)	2.5	1	0.5	0	N/A
1000-year (1.5x 500-year probability)	3.75	1.5	0.75	0	N/A

[1] The Scour Potential is a field in the HAZUS Bridge database and is from the FHWA inventory of bridges

In the future, it may be possible to develop damage relationships for different bridge span materials (concrete, steel, wood), but no data exists, and the focus is on the bridge foundation vulnerability rather than the span.

"Failure" is defined to be loss of function due to flood/scour damage. As there is very limited data, the preliminary recommendation is that "failure" represents a damage value of 25% of replacement cost. This is a mean value taking into account damage that could be scour/undercutting of a single pier, to collapse of a span. This same relationship is applied to rail and light rail bridges. The damage relationships are applicable to pipelines supported on highway bridges.

EXHIBIT A-9.8

KCT RR Bridge Structural Failure Hand Calculations

U.S. ARMY CORPS OF ENGINEERS - KANSAS CITY DISTRICT

Comp. By KSM Date 11/9/11 Project KC Levees feasibility study Sheet ____ of ____

Chkd. By ____ Date ____ Subject Kansas River bridges Sheet ____ of ____

Future w/ Project probabilistic failure rating

- take worst case bridge (amount of superstructure under water)

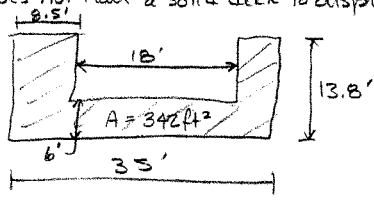
KC Terminal Bridge

on RR drawing DL w/ both floors ballasted = 25,500#/1 in ft (25.5 k/ft)

Check uplift

13.8' under water

Assume the following area contributing to uplift since the bridge does not have a solid deck to displace the water:

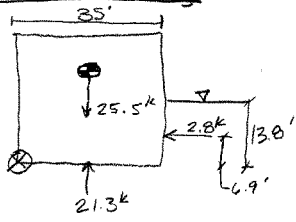


buoyant uplift = $342 ft^2 \times 62.4 \frac{\#}{ft^3} = 21.3\ k/ft$

$\frac{25.5^k}{21.3^k} = 1.2$ Factor of Safety

1.2 > 1.1 required FS for extreme load case

Check overturning



Stream forces:

V = 12 fps K = 1.4 for drift

$P = KV^2 = 1.4 \times 12^2 = 202\ psf = 0.2\ ksf$

$F = 13.8 ft \times 0.2 ksf = 2.8\ k/1 m ft$

Resist ↻ $M_{@} = 25.5^k \times \frac{35'}{2} = 446\ k\cdot ft$

Overt. ↺ $M_{@} = 21.3^k \times \frac{35'}{2} + 2.8^k \times 6.9' = 392\ k\cdot ft$

$\frac{446}{392} = 1.14 > 1.1$ for extreme event

OK Reliability = 99.8%

(g) Ice Pressure

In the absence of more precise data, the following shall be used as minimum design criteria:

1) Ice pressure is 150 psi.

2) Ice thickness is 1'-0".

3) Location of force on pier shall be midway between the ordinary high water elevation and the elevation at Q100.

Ice loads would normally not be considered except on larger rivers such as the Kansas or Missouri Rivers. The need to include ice pressure in the design of a structure should be made at the field check.

(h) Drift

Piers located in streams which are susceptible to transporting large amounts of drift and debris shall be designed to withstand the corresponding increase in stream pressure due to drift accumulation on the pier. The amount of drift build-up is a matter of judgment, but as a guide, the size of the drift may be approximated as follows: (Ref. 1993 AASHTO Interims)

(1) The drift width may be assumed as 10% of each span length contributory to the pier, but no less than 10 feet nor greater than 45 ft.

(2) The vertical depth could extend from the flow line to the computed high water elevation.

An estimate of the amount of drift to be expected should be discussed at Field Check.

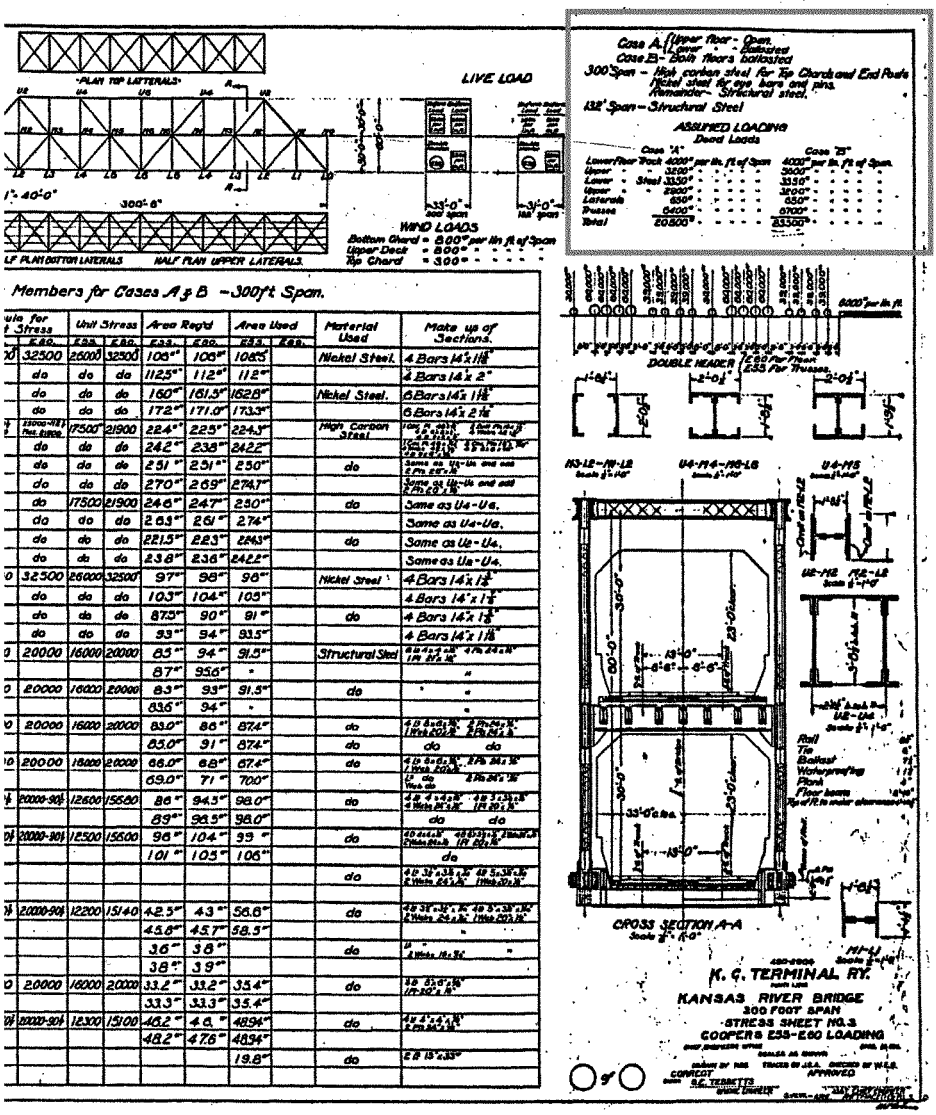
The stream flow pressure can be calculated by $P = KV^2$ (AASHTO Art. 3.18.1) with $K = 1.4$ for drift lodged against a pier.

Blockage of the flow area by drift build-up can increase the stream velocity and thus increase the scour depths through the bridge opening.

(i) Scour

Scour is not a load per se, however by changing the conditions of the channel its effects need to be considered in the design of the substructure. The bridge should

minal 2.47



Members for Cases A & B - 300 ft. Span.

ula for t Stress		Unit Stress		Area Req'd		Area Used		Material Used	Make up of Sections.
	E 90.	E 90.	E 90.	E 35.	E 60.	E 35.	E 60.		
00	32500	26000	32500	106"	106"	108.5		Nickel Steel.	4 Bars 14" x 1 1/8"
	do	do	do	112.5"	112"	112"			4 Bars 14" x 2"
	do	do	do	160"	161.3"	162.8"		Nickel Steel.	6 Bars 14" x 1 1/8"
	do	do	do	172"	171.0"	173.3"			6 Bars 14" x 2 1/8"
	23000-19% Pat. Block	17500	21900	224"	225"	224.3"		High Carbon Steel	[illegible]
	do	do	do	242"	238"	242.2"			[illegible]
	do	do	do	251"	251"	250"		do	Same as U2-U4 and see E Pn 25" x 1/2"
	do	do	do	270"	269"	274.7"			Same as U2-U4 and see E Pn 25" x 1/2"
	do	17500	21900	246"	247"	250"		do	Same as U4-U6.
	do	do	do	263"	261"	274"			Same as U4-U6.
	do	do	do	221.5"	223"	224.3"		do	Same as U6-U8.
	do	do	do	238"	236"	242.2"			Same as U6-U8.
0	32500	26000	32500	97"	98"	98"		Nickel Steel	4 Bars 14" x 1 1/8"
	do	do	do	103"	104"	105"			4 Bars 14" x 1 1/8"
	do	do	do	87.5"	90"	91"		do	4 Bars 14" x 1 1/8"
	do	do	do	93"	94"	93.5"			4 Bars 14" x 1 1/8"
0	20000	16000	20000	85"	94"	91.5"		Structural Steel	[illegible]
				87"	95.6"	"			"
0	20000	16000	20000	83"	93"	91.5"		do	"
				83.5"	94"	"			"
0	20000	16000	20000	83.0"	86"	87.4"		do	[illegible]
				83.0"	91"	87.4"		do	do do
0	20000	16000	20000	66.0"	68"	67.4"		do	[illegible]
				69.0"	71"	70.0"			U do 2 Pn 24" x 1/2" Web do
0	20000-90%	12600	15560	86"	94.5"	98.0"		do	[illegible]
				89"	98.5"	98.0"			do do
0	20000-90%	12500	15500	96"	104"	99"		do	[illegible]
				101"	105"	106"			do
								do	[illegible]
0	20000-90%	12200	15140	42.5"	43"	56.6"		do	[illegible]
				45.6"	45.7"	58.5"			"
				36"	38"			do	" " 2 Webs 16 x 3/8
				38"	39"				
0	20000	16000	20000	33.2"	33.2"	35.4"		do	[illegible]
				33.3"	33.3"	35.4"			
0	20000-90%	12300	15100	46.2"	46"	48.94"		do	[illegible]
				48.2"	47.6"	48.94"			
						19.8"		do	[illegible]

Chapter A-10

STRUCTURAL ANALYSIS ARMOURDALE

CHAPTER A-10
STRUCTURAL ANALYSIS - ARMOURDALE

A-10.1 OVERVIEW

This chapter of the engineering appendix addresses the structural features (floodwalls, gatewells, and closure structures) for the Armourdale Unit which includes both existing conditions and future with project conditions. The existing conditions of the structural features are addressed first followed by a separate evaluation of future alternatives for raising the current level of protection (N500+0, N500+3, and N500+5). The existing condition evaluation includes assigning reliability to the structures and recommending remedies if required. The future conditions section addresses the approach taken to meet the impacts due to each raise option. For general structural methodology refer to the "General Structural Analysis" chapter of this appendix. Elevations reference the NGVD 29 datum as information was obtained from record drawings unless otherwise stated.

A-10.2 SOIL MATERIAL PROPERTIES

The soil properties used for the Armourdale Unit calculations are located in Table A-10.1. These values were used for general blanket properties in the structural calculations. Soil to structure friction and cohesion interaction was typically neglected for stability and strength calculations. However, where necessary, this behavior was considered under geotechnical guidance.

TABLE A-10.1
Armourdale Soil Properties

Friction Angle	26°
Cohesion	-
Moist unit weight	110 pcf
Saturated unit weight	115 pcf
Wall friction coefficient (used in gatewell uplift)	0.25

A-10.3 EXISTING CONDITIONS

A-10.3.1 Floodwalls

Floodwalls on the Armourdale Unit consist of two spread footing floodwalls and one pile founded floodwall. For each of these walls, the existing condition was evaluated. The evaluation entailed analysis of existing reliability and recommending solutions for any existing condition deficiencies. See the "General Structural Analysis" chapter of this appendix for further information on reliability analysis.

The existing Armourdale floodwalls are as follows:

- Station 60+30 to 77+78 spread footing floodwall constructed in 1973
- Station 246+90 to 250+50 spread footing floodwall constructed in 1952 and raised in 1973
- Station 257+65 to 302+58 pile foundation floodwall constructed in 1952 and raised in 1973

This does not include walls that are part of a closure structure (see Closure Structures section).

A-10.3.1.1 Floodwall Station 60+30 to 77+78

The original spread footing floodwall from Station 60+30 to 77+78 was constructed in the 1970s as part of the 1962 Modification ('62 Mod). Refer to Exhibit A-10.1 for a photo and section drawing in the Supplemental Exhibits section. This stretch of wall has not been modified to date other than replacing the stoplog gap section under the West Kansas Avenue Bridge. Monoliths 1-4 and 10-47; (60+30 to 61+57 and 63+00 to 77+78) currently exist as the originally constructed typical section, and the section that replaced the stoplog gap (61+57 to 63+00) was constructed in the mid-80s to match that section. The result is an essentially uniform wall for which complete construction information is known. The only difference from the original and more recent construction was a greater reinforcement cover, but greater material strengths (F_y=60ksi and f'c=4ksi) led to a larger factor of safety than the original typical section (the rest of the wall).

The controlling section for this wall was the typical section (monoliths 1-4 & 10-47) from the early 1970s. See Table A-10.2, Armourdale Existing Floodwall Conditions, for stability and strength summary.

A-10.3.1.2 Floodwall Station 246+88 to 250+52

The original wall was constructed in 1952 and was raised as part of the '62 Mod which included adding buttresses. Refer to Exhibit A-10.2 for a photo and section drawing in the Supplemental Exhibits section. The original drawings refer to two sections as follows:

- Type "B": Station 246+88 to 247+93 (105-ft)
  - The original sandbag gap under the E. Kansas Ave. Br. from 147+36 to 147+86 was filled in with the '62 Mod. Where bridge bracing did not interfere, the wall beneath was extended, but where bracing existed, reinforced concrete was placed and isolated so as not to transfer load to the wall but rather to the bridge bracing. This patch was found to be acceptable for filling the gap, but should not be added to for a raise.

- Type "A": Station 247+93 to 250+52
  - Original sandbag gap from 249+37 to 249+72 replaced with a stop log gap with '62 Mod

- Stoplog gap filled in with concrete and bridge abandoned (found by field observation, date unknown)

Section "A" was found to be the critical section after analysis of both sections. This stretch of wall did not meet strength screening criteria and therefore, a reliability analysis was performed which computed high reliability. See Table A-10.2 for stability and strength summary. For the specifics on the stoplog gap in this stretch of wall, see the Closure Structures section of this chapter.

TABLE A-10.2
Armourdale Existing Floodwall Conditions

Spread Footing Floodwalls (Existing Condition with Water to Top)							
Station	Wall Cross Section	Overturning % Base in Compression (25% Req'd)	Bearing, % of Allowable (150 % Max)	Sliding Factor of Safety (1.3 Req'd)	Wall Strength Factor of Safety (1.5 Req'd)	Comments	Reliability
60+30 to 77+78	Complete Stretch of Walls	48%	100%	1.6	1.5	Toe Steel Governs	99.8% Assigned
247+88 to 250+50	Sect. "A"	51%	122%	>3	1.4	Toe Steel Governs	99.9% Calculated

Pile Foundation Floodwalls (Existing Condition with Water to Top)						
Station	Wall Cross Section	Factors of Safety			Wall/Pile Cap FS Control	Reliability
		Wall/ Pile Cap (1.5 Req'd)	Pile			
			Soil (1.7 Req'd)	Strength (1.5 Req'd)		
257+84 to 259+04	Mon. 6-9	2.2	5.4	6.7	Heel Flexure	99.8% Assigned
273+56 to 274+76	Mon. 93-95 (Slot)	1.5	5	5.6	Heel Flexure	99.8% Assigned
293+61 to 294+92	Mon. 93,94,95	1.5	4.9	5.5	Heel Flexure	99.8% Assigned
295+90 to 302+58	Mon. 97-111	1.8	6.7	6.9	Stem Flexure	99.8% Assigned

A-10.3.1.3 Floodwall Station 257+65 to 302+58

This floodwall was constructed in1953 and was raised with the '62 Mod. The foundations consist of various configurations of 18-in x 18-in precast concrete bearing piles with a 12-in concrete sheet piling cutoff at the heel. The concrete bearing piles appear to be at least 30-ft long based on the penetration table in the drawings for bearing piles not driven to design depth or refusal. The '62 Mod included a raise varying from 2.7-ft to 1.1-ft and included a stem thickening on the landside for most of the wall monoliths.

Original design allowable loads for the piles can be found in "Analysis of Design for Construction of Flood Wall, Levee and Appurtenances" which accompanied the drawings. The capacities were reevaluated during this feasibility effort and compared to the original values. The original values are as follows:

- 18-in x 18-in precast concrete bearing piles
  - Max allowable load of 35 tons
  - Max horizontal thrust of 7 tons

- 1-ft thick x 16-ft long concrete sheet piling
  - Max allowable load of 4.5 tons

- Foundation piles (both types) allowable tension of 200 lbs per sq. ft of surface area

From 257+65 to 302+58 the wall sections vary significantly along with the soil elevations. Total heights range from 26-ft to 14-ft and exposed stem heights range from 23.6-ft to 0.0-ft. This can be seen in the photos and figures provided in Exhibit A-10.3 and Exhibit A-10.4 at the end of this chapter.

All of the wall monoliths met screening factors of safety, and therefore, the wall was assigned a reliability of 99.8%. The factors of safety for the critical sections are provided in Table A-10.2.

A-10.3.2 Drainage Structures

The results for the complete analysis of the Armourdale gatewells existing conditions are presented in Table A-10.3. It is important to note that complete information, wall thicknesses, reinforcement size, and spacing, was not available for all gatewells. See the table for locations. The results show that where complete information was available all gatewells met the screening criteria factor of safety.

It follows that if a comparable reinforcement ratio is chosen based on the gatewells for which information is known, then all the gatewells with that assumed ratio will pass. Continuing that thought process, all gatewells were assigned a reliability of 99.8% for the economic analysis. This assigned reliability is for economic use only and does not certify the reliability of the structure. To provide actual conclusions regarding the gatewells that lack information field testing will be required. This must include checking wall thicknesses, reinforcement size, and spacing.

For further information regarding the analysis approach and examples see the General Structural Analysis chapter of this appendix.

TABLE A-10.3
Armourdale Existing Gatewell Conditions

Existing Conditions with Water to Top of Protection			
Station	Uplift Factor of Safety (>1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism
0+75 UE	1.4	2.4	Base Slab Flexure
7+50 UE	1.5	4.9	Wall Shear
12+79 UE	1.4	1.6	Base Slab Flexure
15+33[b]	1.5	0.7	Base Slab Flexure
32+80	1.3	2.7	Wall Shear
64+71	1.4	1.9	Wall Shear
76+83		Refer to Osage Pump Station Analysis	
79+60[b]	1.4	0.8	Wall Flexure
79+80[b]	1.1	1.4	Wall Flexure
90+79	1.5	4.5	Wall Shear
91+76	1.4	3.4	Wall Shear
108+95[b]	1.1	2.9	Wall Shear
129+20	1.5	1.9	Wall Flexure
156+75	1.5	1.9	Wall Flexure
185+70[b]	1.4	2.3	Wall Flexure
186+74	1.5	2.9	Wall Shear
194+60[b]	1.6	5.0	Wall Flexure
212+76	1.5	3.3	Base Slab Flexure
220+64	1.4	3.2	Wall Shear
230+77		Refer to Shawnee Pump Station Analysis	
240+73	1.4	1.9	Wall Flexure
244+70	1.5	3.5	Wall Flexure
246+53	1.5	4.4	Wall Flexure
250+31[a]	1.5	4.9	Wall Flexure
253+43[a]	1.8	4.7	Wall Flexure
256+71	1.5	3.8	Wall Flexure
260+00[a,b]	1.1	0.9	Wall Flexure
260+84[a]	1.3	7.0	Wall Flexure
262+89[a]	1.1	6.9	Wall Flexure
266+76[a]	1.3	10.0	Wall Flexure
276+79		Refer to KC Southern Pump Station Analysis	
281+50[c]	N/A (on piles)	2.1	Wall Shear
286+59[a]	1.1	4.3	Wall Flexure
290+52[a]	1.5	3.4	Wall Flexure
295+45[b]			
295+52	1.5	3.1	Wall Flexure
299+20		Refer to Central Ave Pump Station Analysis	
311+11	1.4	1.9	Wall Flexure
315+10[b]	1.3	1.5	Wall Flexure
41+45[b]	1.2	1.8	Wall Flexure

[a] top below level of protection

[b] Incomplete information (complete drawings were not found)

[c] Integral with floodwall on piles. Simplified analysis checked wall strength only

A-10.3.3 Closure Structures

The reliability for the Armourdale stop log gaps are given in Table A-10.4, Armourdale Existing Stop Log Gap Conditions.

TABLE A-10.4
Armourdale Existing Stop Log Gap Conditions

Stoplog Gap Reliability: Spread Footing Floodwalls (Existing Condition with Water to Top)					
Station	Crossing	Overturning % Base in Compression (25% Req'd)	Bearing, % of Allowable (150 % Max)	Sliding Factor of Safety	Wall Strength Factor of Safety
0+40 to 2+90	Upper End (Railroad)	48%	129%	1.35	2.0
226+75	KC Terminal Bridge	94%	82%	>3	1.8
249+54	CRI&P Bridge	81%	73%	>3	1.7

Stoplog Gap Reliability, Continued (Existing Condition with Water to Top)					
Station	Crossing	Factors of Safety			
		Stoplog Post	Stoplog Guide	Stoplog	Post Well
0+40 to 2+90	Upper End (Railroad)	3	1.8	2.1	1.3
226+75	KC Terminal Bridge	2.6	2.7	OK[1]	2.2
249+54	CRI&P Bridge	N/A	1.94	?	N/A

[1]Checked against NDS specifications

Stoplog Gaps Reliability (Existing Condition with Water to Top)			
Station	Crossing	Reliability	Comments
0+30 to 2+90	Upper End (Railroad)	99.9%	Complete drawings available
226+75	KC Terminal Bridge	99.8%	FS = 1.65 calculated for steel backwall plate
249+54	CRI&P Bridge	99.8%	Assigned reliability. However, the details of concrete placed to fill in the stoplog gap are unknown. Sponsor noted that concrete fill was submitted to and approved by USACE

A-10.3.3.1 CRI&P Bridge Station 249+55

This bridge and the tracks this gap closed have been abandoned. In conjunction with the abandonment of the bridge, the gap was filled in with concrete. The details for that concrete fill were not found. All other information for this crossing was available. The results showed the stop log gap to be reliable. The concrete fill concern was discussed with the sponsor who verified that the plans were submitted to the Corps of Engineers and accepted.

A-10.4 FUTURE WITH PROJECT CONDITIONS

Future conditions include the alternatives and recommendations for raising the unit elevation to the river stage of the N500+0, N500+3, and the N500+5. The primary emphasis for the evaluation was on the N500+3 with abbreviated approach to the N500+0 and N500+5. This was achieved by using the results from the N500+3 and the existing conditions to estimate the N500+0 and N500+5.

A-10.4.1 Floodwall Future Conditions at Existing Floodwall Stations

The following discussion and results are for stations where walls currently exist. The following alternatives are a summary of the thought process that created the final recommendation for the N500+3 event, along with a brief discussion of the N500+0 and N500+5, where walls currently exist. Refer to the structural methodology for more information on formulation of the general floodwall alternatives. Also, for background information on existing walls, refer to the Existing Conditions section of this chapter.

A-10.4.1.1 Floodwall Station 60+30 to 77+78

Nominal 500+3, 4.25-ft Raise

Alternative 1: Landside Earthen Levee

This was found to be impractical due to land requirements. The wall was only able to retain 2-ft of additional landside soil (with 1 on 3 batter) and therefore, it was not recommended to be used to retain landside soil (acting the opposite direction for which it was designed).

Alternative 2: New Construction/Modification

Overturning and sliding became a severe problem in considering modification, therefore, a stem extension alone was found to be impossible. In addition, the stem and base became highly overstressed and required extensive widening, thickening, and addition of buttresses which would include a significant doweling effort. Such modifications proved to be unreasonable in comparison to the cost of new construction.

Alternative 3: Riverside Levee Raise

Although the general approach in this study has been to avoid any riverside impacts, a riverside levee raise was investigated for this stretch. However, this was eventually eliminated for several reasons.

Conclusions:

a. A landside levee is not feasible because of site restrictions.
b. Riverside levee raise was eliminated.
c. Raising the existing floodwall is impractical.
d. New floodwall is recommended. This will require measures to maintain the current level of protection (i.e. stage construction).

N500+0 and N500+5 Events

N500+0

The N500+0 event extension, 1.25-ft, created instability and strength problems (the existing condition factor of safety was only 1.5 for strength). By utilizing the allowable

additional landside backfill, stability could be achieved, however strength continued to be a concern. This could potentially be mitigated with buttresses, but the recommendation is to construct a new wall. Therefore, a new wall is recommended where a riverside levee raise is not possible.

N500+5
For this 6.25-ft raise, new floodwall construction with auger cast piles was determined to be the most effective solution.

A-10.4.1.2 Floodwall Station 246+88 to 250+52

Nominal 500+3, 3.5-ft Raise

Alternative 1: Landside Earthen Levee
This existing floodwall was investigated to determine if it could retain additional landside soil (landside levee construction). The two typical sections were investigated (refer to existing conditions for more information), and it was found that the section from 246+88 to 247+93 can retain soil to the top of wall but the section from 247+93 to 250+52 cannot retain more than the 11-ft already retained. A landside levee was found to be impractical because placing a levee under the East Kansas Bridge buried existing bridge steel. Therefore, a new wall is recommended under the bridge.

Alternative 2: New Construction/Modification
A simple stem extension will not work due to overturning and strength problems, and because foundation modifications would be required to achieve stability. Furthermore, the portion under the bridge has a questionable existing patch which cannot effectively be raised (see existing conditions). Considering that this wall would require heavy modification and has had a previous extension including buttresses, new construction is clearly the best alternative. The new floodwall will be placed landside of the existing wall. Refer to Exhibit A-10.5 of the Supplemental Exhibits section at the end of this chapter for a section sketch.

Conclusions:

a. Raising the existing floodwall is impractical.
b. The recommendation for this stretch is a new wall landside of the existing wall. Refer to the Maps section of the Feasibility Report for an orientation sketch.

N500+0 and N500+5 Events

N500+0
For the N500+0 event extension, 1.1-ft, a stem extension is acceptable for the section past 247+93 (just downstream of the E Kansas Ave Bridge). However, the beginning section, 246+88 to 247+93, which runs under the bridge, should be replaced. This is because the wall under the bridge and the patched in sand bag gap are not acceptable for an extension or any sound modification.

N500+5
For this 6.25-ft raise, new floodwall construction with auger cast piles was determined to be the most effective solution. The approach was the same outlined for the N500+3

A-10.4.1.3 Floodwall Station 257+65 to 302+58

Nominal 500 + 3

Alternative 1: Landside Earthen Levee

Not feasible due to site constraints.

Alternative 2 : New Construction/Modification

A simple stem extension will not work due to strength problems. Buttresses are required along the entire length of wall to alleviate stem and heel deficiencies. Also, the slot on the landside of the wall must be filled to the adjacent landside elevation.

In addition to the initial structural analysis and foundation analysis using CPGA, a computer program used to analyze pile groups, a more refined geotechnical analysis was run to evaluate the foundation piles and the wall deflection, see Exhibit A-10.6 at the end of this chapter. The geotechnical analysis showed that a new row of piles was required at stations where the existing landside elevation could not be raised enough to alleviate the additional load from the raise. Therefore, away from the deeper portion of the slot, new piles are required. Refer to the Armourdale solution matrix for detailed stationing and Exhibit A-10.5 for a section sketch.

Conclusions

Modifying the wall is recommended over new construction. The cost analysis of the Fairfax BPU floodwall alternatives in Phase 1 of this feasibility study showed that modifying that floodwall was more cost effective than replacement, and in that case, the new wall was to be built behind the existing wall. In this case, real estate restrictions require that the wall be replaced in the same footprint as the existing wall which would require temporary protection, sequenced construction and building around the existing piles. Therefore, modifying the wall is recommended except for a new stop log gap and wall from Station 274+36 to 277+21. Refer to the Armourdale solution matrix for detailed stationing and Exhibit A-10.5 for a typical section sketch. The structural sketches in Exhibits A-10.7 and A-10.8 were used for cost estimating and they are located in the Supplemental Exhibits section.

N500+0 and N500+5 Events

N500+0

This raise is less than 1-ft and can be accomplished with a simple stem extension and backfilling the wall to 751ft.

N500+5

For this 5-ft raise, modification was recommended. Site constraints and the existing level of protection made placing a landside levee impractical. In addition, new floodwall construction would be too costly requiring special measures to maintain the level of protection. To provide the raise, modification required a new row of auger-cast piles, a toe extension, and buttresses.

A-10.4.2 New Construction of T-Walls on Existing Levees

In several locations placing T-Walls on existing levees has been recommended for raising the level of protection; refer to the Maps section of the Feasibility Report. This solution is recommended in locations where an existing levee could not be raised because of spatial constraints. By using a T-Wall on the existing levee, a new level could be reached without adding soil and the associated driving forces. Even so, the use of a T-Wall still required use of a landside berm for global stability. The typical section used for the N500+3 cost estimate is given in Exhibit A-10.5 of the Supplemental Exhibits section at the end of this chapter.

A-10.4.3 New Floodwall Construction

Often, after exhausting all other solutions, floodwalls were recommended due to land availability. In some locations with existing levee, the levee is to be removed to prevent global stability concerns and replaced with a floodwall.

A-10.4.4 N500+3 Floodwall Summary

For the locations of proposed N500+3 floodwalls refer to the Maps section of the Feasibility Report.

A-10.4.5 Drainage Structures

The results for the complete analysis of the Armourdale gatewells N500+3 future conditions are presented in Table A-10.5. Flotation was checked based on the extreme FS from EM 110-2-2100. Strength was checked in a manner similar to existing conditions for simplicity and to provide comparable results. Rather than providing a typical LRFD check, a factor of safety for each gatewell was compared to an equivalent required minimum FS based on LRFD factors. The equivalent minimum FS of 2.6 is based on the following:

$FS \geq HF*LF/\phi = 1.3*1.7/0.85 = 2.6$ where

HF = hydraulic factor,
LF = single load factor,
ϕ = resistance factor (lower value for shear assumed).

The gatewells which meet the strength and stability requirements need only a simple raise. Otherwise, the modification is stated in the "Notes" column of the results in Table A-10.5.

It is important to note that complete information regarding wall thicknesses, reinforcement size, and spacing, was not available for all gatewells, refer to Table A-10.5 for locations. Those gatewells were analyzed based on known information from drawings, field measured data, and assumptions. Assumptions were made conservatively. For example, wall thickness was assumed to be constant for the full depth (the top could be measured in the field) and the reinforcement was assumed to be the minimum found in gatewells. Verification is required unless the analysis with conservative assumptions showed that the gatewell meets criteria. For further

information regarding the analysis approach and examples see the "General Structural Analysis" chapter of this appendix.

TABLE A-10.5
N500+3 Gatewell Results

N500+3 with Water to Top of Protection					
Station	Proposed Raise	Uplift Factor of Safety (>1.1 Req'd)	Strength Factor of Safety (> 2.6 Req'd)	Controlling Structural Mechanism	Notes
0+75 UE	SL Gap	1.4	2.0	Base Slab Flexure	Replace (new alignment)
7+50 UE	RS levee	1.4	4.1	Wall Shear	
12+79 UE	FW	1.3	1.5	Base Slab Flexure	Replace
15+33[b]	levee	1.5	0.6	Base Slab Flexure	Modification assumed. Newer than '62 Mod (no prev. raise)
32+80	levee	1.2	2.6	Wall Shear	
64+71	FW	1.3	1.6	Wall Shear	Replace. Slurry cut-off wall
76+83		Refer to Osage Pump Station Analysis			
79+60[b]		To be abandoned			
79+80[b]		To be abandoned			
90+79	levee	1.5	4.0	Wall Shear	
91+76	levee	1.4	2.9	Wall Shear	
108+95[b]	levee	1.1	2.7	Wall Shear	Raise only. Adequate with conservative assumptions
129+20	levee	1.4	1.7	Wall Flexure	Modify for strength
156+75	Twall	1.4	1.7	Wall Flexure	Modify for strength
185+70[b]	FW	1.4	2.1	Wall Flexure	Modification assumed. FS > 2 w/ conservative assumptions. No previous raise and O&MM PL 222 shows thicker walls at base.
186+74	FW	1.5	2.7	Wall Shear	
194+60[b]		To be abandoned			
212+76	FW	1.5	3.1	Base Slab Flexure	
220+64	FW	1.4	2.8	Wall Shear	
230+77		Refer to Shawnee Pump Station Analysis			
240+73	Twall	1.4	1.7	Wall Flexure	Modify for strength
244+70	Twall	1.5	3.2	Wall Flexure	
246+53	FW	1.5	3.8	Wall Flexure	
250+31[a]	FW	1.3	4.8	Wall Flexure	
253+43[a]	FW	Abandoned: no action			
256+71	FW	Abandoned: grout pipes, fill gatewell			
260+00[a,b]	FW	1	0.9	Wall Flexure	Replacement assumed. Pre-1950 construction. Recommend testing to verify
260+84[a]	FW	Abandoned: grout pipes, fill gatewell			
262+89[a]	FW	1	6.6	Wall Flexure	Add concrete collar at top for weight
266+76[a]	FW	1	9.0	Wall Flexure	Add concrete collar at top for weight
276+79	FW	Refer to KC Southern Railroad Pump Station Analysis			
281+50[c]	FW	See note c			
286+59[a]	FW	0.9	4.1	Wall Flexure	Abandon
290+52[a]	FW	1.1	3.2	Wall Flexure	
295+45[b]	FW				
295+52	FW	1.4	3.0	Wall Flexure	
299+20	FW	Refer to Central Ave Pump Station Analysis			
311+11	levee	1.4	1.7	Wall Flexure	Modify for strength
315+10[b]	levee	1.2	1.4	Wall Flexure	Modification assumed. No prev. raise. N500+3' Raise ~ 1.5ft
41+45[b]	levee	1.2	1.6	Wall Flexure	Replacement assumed

[a] top below level of protection
[b] Incomplete information (complete drawings were not found) - conservative assumptions used
[c] Integral with floodwall, founded on piles. No analysis performed, assumed adequate but needs to be analyzed with floodwall modification

A-10.4.6 RCBs and Pipes Associated with Gatewells

The results for the conduits associated with gatewells can be found in Table A-10.6 with additional information for the box culverts (factors of safety) given in Table A-10.7. In addition, the following paragraphs discuss the approach to cast iron pipes and estimating the associated work for future conditions. For more information on analysis of RCBs and pipes refer to the General Structural Analysis chapter and to the structural exhibits for examples.

Two main concerns exist for the existing cast iron pipes (CIPs). First, they may see additional load with a raise in levee elevation (not in the case of some floodwalls as described below). Secondly, CIPs are susceptible to corrosion. For this study, assumptions were made based on available data, but during formulation of plans and specs these pipes will require a detailed inspection and may require wall thickness measurements.

Without wall thickness an analysis could not be performed to verify the ability of each pipe to withstand additional loading invoked by a levee raise. However, in locations where new floodwalls replaced existing levees, it was assumed that enough soil would be removed to offset the increase in water loading. In those cases, if a detailed inspection reveals an adequate condition, no additional work will be required.

In addition to strength issues, determining the pipe's condition is of utmost importance. CIPs are no longer used in levee projects because they are vulnerable to corrosion, and the CIPs under consideration were constructed during use of leadite joints which have had severe corrosion problems. Unless completely replaced, these pipes will require a detailed, high quality inspection.

Due to the lack of knowledge on pipe conditions and wall thicknesses, cost for this study was a rough estimation. The estimation included:

- Inspection of all pipes.
- Measurement of thickness for all of the pipes that do not have "None" under the action column, refer to Table A-10.6
- Replacement of 25% of the pipes that do not have "None" under the action column, refer to Table A-10.6
- Slip liner of 25% of the pipes that do not have "None" listed under the action column, refer to Table A-10.6

TABLE A-10.6
N500+3 Gatewell Conduits

N500+3 Results: Conduits Associated with Gatewells		
Station	Conduit Size & Type	Action*
0+75 UE	36" DIP (pressure)	Replace section for new alignment
7+50 UE	24" RCP	None
12+79 UE	12'x8' RCB	None
15+33	8'x8' RCB	No Details found: Use new wall to remove enough soil to offset additional water load
32+80	42" RCP	None
64+71	42" RCP	None
76+83	Refer to Osage Pump Station Analysis	
79+60	84" RCP	Grout
79+80	60" RCP	Grout
90+79	30" DIP	None
91+76	30" CIP	See CIP Paragraph
108+95	42" RCP	None
129+20	Refer to 12th Street Pump Station Analysis	
156+75	Refer to Mill St Pump Station Analysis	
185+70	Refer to 5th St Pump Station Analysis	
186+74	30" DIP	None
194+60	18" & 12" CIPs	Grout and Abandon
212+76	24" CIP	None
220+64	24" CIP	None
230+77	Refer to Shawnee Pump Station Analysis	
240+73	48" RCP	None
244+70	24" CIP	See CIP Paragraph
246+53	30" CIP (pressure)	See CIP Paragraph
250+31	12" CIP	See CIP Paragraph
253+43	Abandoned and grouted	
256+71	Abandoned	Grout
260+00	30" CIP	See CIP Paragraph
260+84	Abandoned	Grout
262+89	12" CIP	See CIP Paragraph
266+76	16" CIP	See CIP Paragraph
276+79	Refer to KC Southern Railroad Pump Station Analysis	
281+50	42" RCP	None
286+59	24" CIP	Grout and Abandon
290+52	24" CIP	See CIP Paragraph
295+45	18" RCP	None
295+52	Refer to National Beef Pump Station Analysis	
299+20	Refer to Central Ave Pump Station Analysis	
311+11	7.5'x7.5' RCB	None
315+10	5'x4x RCB	No details: assumed adequate. Must be verified with testing
41+45	48" CIP (Pressure)	None (No previous raises, N500+3 ~1 ft)

*Complete inspection required for all pipes

TABLE A-10.7
N500+3 RCB Analysis Results

Armourdale RCB Results: N500+3 Levee Elevations							
Station	Size	Bending			Shear		
		FS	Load Case	Member	FS	Load Case	Member
12+79	12x8	2.3	3	Wall	1.9	3	Floor
15+33	8x8	No information found					
311+11	7.5x7.5	2.7	3	Wall	2.4	2	Wall
315+10	5x4	No information found					

Load Case	C_V	C_H
1	1	0.4
2	1	0.57
3	1.5	0.5

For cost purposes, it is assumed that all of the RCBs will receive liners under the levee since the two with drawings do not meet design criteria (FS>2.6 for HF=1.3, LF=1.7, ϕ=0.85).

A-10.4.7 Closure Structures

A summary of the closure structures required for the N500+3 event is given below in Table A-10.8, N500+3 Closure Structures.

TABLE A-10.8
N500+3 Closure Structures

Station	Crossing	Type	Notes
0+40 to 2+90	Upper End (Railroad)	Stop Log	New gap riverside with new gatewell, Pile foundation.
130+00	12th St. Bridge	Sand Bag	Minor work necessary
226+75	KC Terminal Bridge	Stop Log	New wall and gap landside of existing on abandoned RR, Pile foundation
275+42 & 276+26	Union Pacific RR & Missouri Pacific RR	Stop Log	New wall and gap landside of slot (fill slot), Pile Foundation
295+50	Central Ave	Sand Bag	
42+50 LE	Lower End Termination	Sand Bag	

The following notes are with regard to the future condition consideration for closure structures:

- Locations with existing stop log gaps (Upper End Crossing and KC Terminal Bridge):
  - The UE crossing required extensive modification for the raise (foundation and stem modification) which can be seen by existing condition concerns. Therefore, complete replacement was recommended which includes a new gatewell as an accessible riverside valve is required for the existing utility.

  - The complete details for most of the Kansas City Terminal Bridge stop log gaps were found near the end of this study. The existing condition was updated based on the drawings but the raise was not re-evaluated. The assumption for cost purposes is a complete replacement, but modification of the gap should be evaluated in the design phase to save cost and impact to the railroad.

- New stop log gap at the Union Pacific and Missouri Pacific Railroad
  - The existing sandbag gaps are in between the bridge trusses and physical restraints prevent construction of a stop log gap in the current line of protection.

  - Existing foundations in the slot from the abutments for the bridge over the Kansas River and for the bridge over the slot prevent construction of the new stop log gap in the slot.

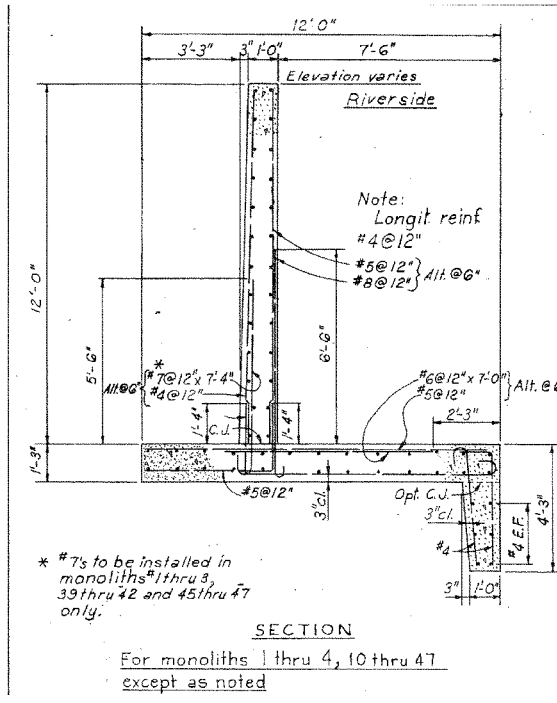
  - Due to the above reasons, the stop log gap was moved to the landward side of the slot with a new floodwall transitioning from the existing line of protection to the location of the new gap. A pile founded floodwall was anticipated to match the deep foundations of the existing walls. HTRW restrictions at PBI Gordon prevented use of levee as a transition to the new gap.

A-10.5 REFERENCES:

1. Record Drawings; Volume I; Operations and Maintenance Manual; Armourdale Unit; Sheets dated Dec. 1951
2. Armourdale Unit, Kansas City, KS-1962 Modification. Plans for Construction of Flood Protection Works; May 1973
3. Armourdale Unit, Analysis of Design for Construction of Flood Wall, Levee and Appurtenances; July 1950
4. Plan and Profile of Proposed State Highway Federal Aid Project, Kansas City, KS, Wyandotte County; K-132, Kansas Avenue, 1982

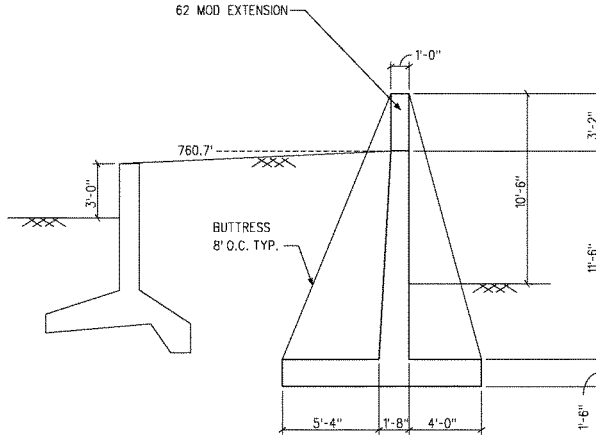
A-10.6 SUPPLEMENTAL EXHIBITS

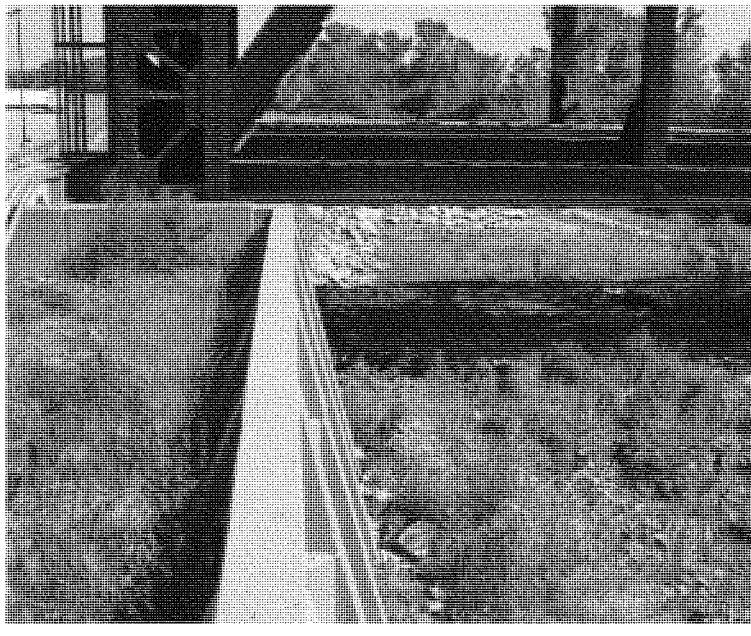
EXHIBIT A-10.1



Armourdale Structural Exhibit 1: 60+30 to 77+78 Floodwall

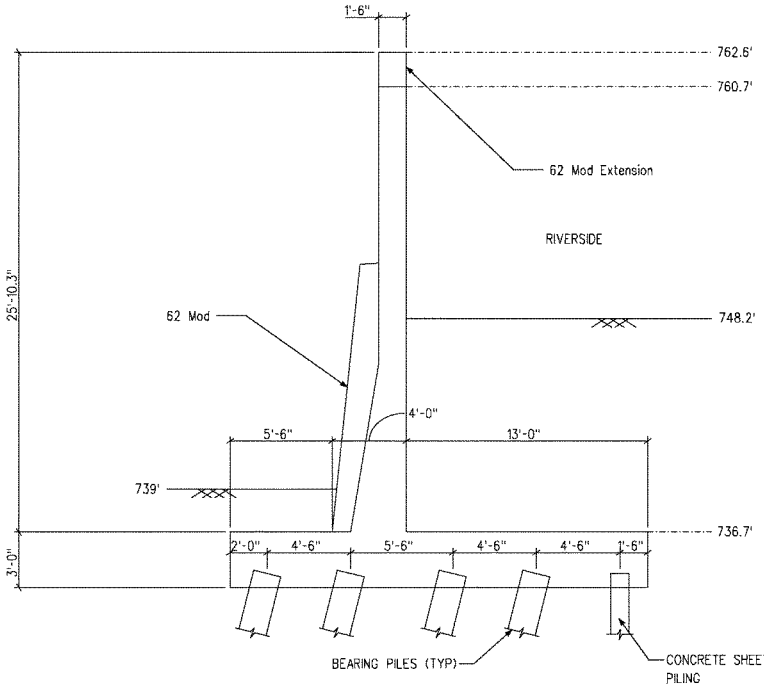
EXHIBIT A-10.2

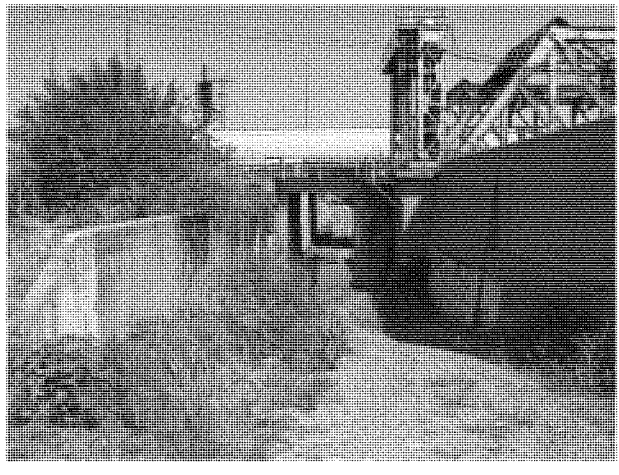




Armourdale Structural Exhibit 2: Floodwall Station 247+93 to 250+50
Riverside Right (Photo looking downstream)

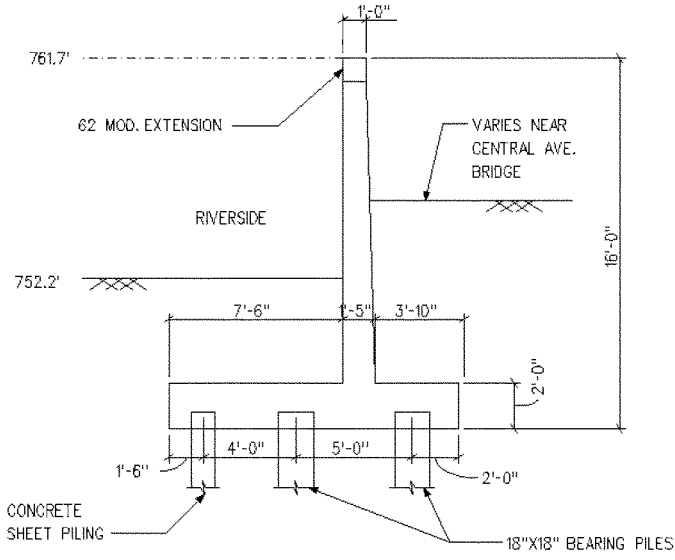
EXHIBIT A-10.3





Armourdale Structural Exhibit 3: 274+00, Slot Riverside Right (Photo looking downstream)

EXHIBIT A-10.4

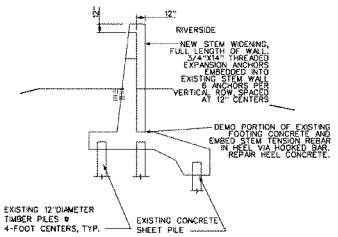


Armourdale Structural Exhibit 4: 294+00, Floodwall

(Photo from 295+00 looking upstream)

EXHIBIT A-10.5

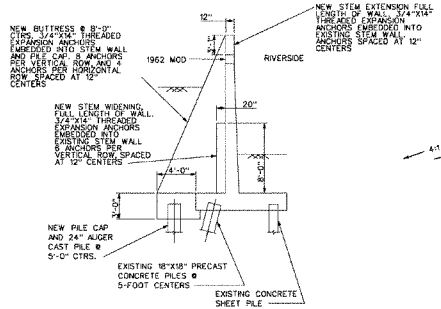
Proposed Typical Wall Sections



A TYPICAL MODIFIED FLOODWALL

NOT TO SCALE

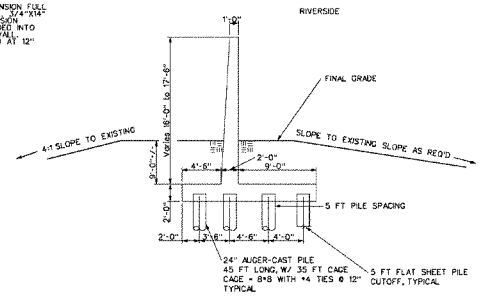
STATIONS: 28+72.66 TO 40+31.25



B MODIFIED FLOODWALL WITH NEW PILES

NOT TO SCALE

STATIONS: 74+35.94 TO 75+29



C TYPICAL NEW FLOODWALL: 16' TO 17'-6"

NOT TO SCALE

STATIONS: 75+76 TO 77+27.75

US Army Corps of Engineers
Kansas City District

Revision	Date

Designed by: K. Martin
Drawn by: K. Martin
Checked by: M. Wilson
Submitted by: B. Grandquist

Date: NOV 2009
File No.
Proj. Mgr: M. Wilson
CADD File Name: 12-CAD02_S1.DGN

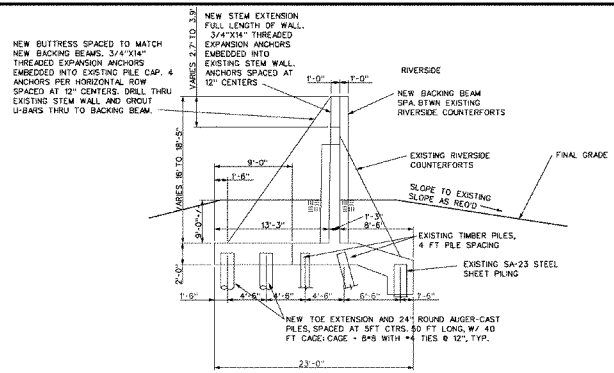
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

KANSAS CITY - KANSAS UNIT
CENTRAL INDUSTRIAL DISTRICT

FLOODWALL SECTIONS

Sheet Reference Number
S1

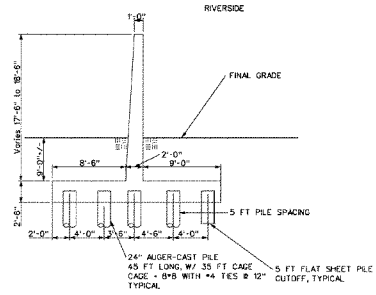
K:\MCSSIONPROJECTS\CI\KANSAS CITY\STRUCTURES\PHASE 2\CAD2\MAPBOOK FILES



D TYPICAL MODIFIED FLOODWALL: 16' TO 18'-5"

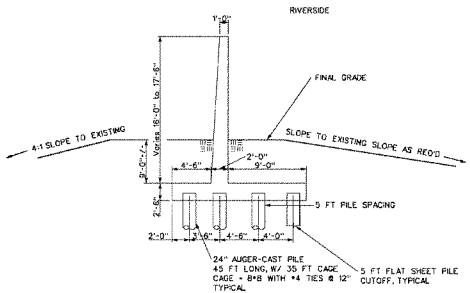
NOT TO SCALE

STATIONS: 102+73.38 TO 166+25



E TYPICAL TIEBACK WALL: 17'-6" TO 18'-5" IN HEIGHT

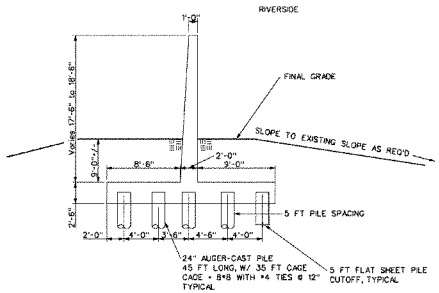
NOT TO SCALE



F TYPICAL NEW FLOODWALL: 16' TO 17'-6"

NOT TO SCALE

STATIONS: 102+73.38 TO 135+00



G TYPICAL NEW FLOODWALL: 17'-6" TO 18'-5"

NOT TO SCALE

STATIONS: 135+00 TO 166+25

US Army Corps of Engineers Kansas City District

SYMBOL	DESCRIPTION	DATE

Drawn by: R. MARK	Date: MAY 2006
Reviewed by: R. MARK	File: 4C
Approved by: R. MARK	PM: Moore
Checked by: R. MARK	PE:
Submitted by: E. Schwaab	CADD File name: KC-CSWG-S2.DGN

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI

KANSAS CITY, KANSAS UNIT
CENTRAL INDUSTRIAL DISTRICT

FLOODWALL SECTIONS

Sheet Reference Number:
S2

EXHIBIT A-10.6

Geotechnical Evaluation of Piles

23 January 2008

Kansas City Levees - Armourdale: Evaluation of Existing Floodwalls for the n500+3 Design Condition

1. General:

A detailed geotechnical analysis was performed using the existing Armourdale floodwall and adding the n500+3 raise to the structure. Preceding this, loads on the existing pile foundations had been analyzed for the n500+3 loading condition and the foundations were found to be adequate with respect to axial loading and bending of the piles. However, a detailed analysis of the lateral deflection was requested. This was deemed critical since the wall had been raised once in the 1970's with no foundation modifications and the n500+3 design condition will raise the wall again (for a total raise of 3 to 6 feet along the length of the wall). Excessive movement of the wall would allow an uncontrolled release of water into the protected area, thus would constitute failure of the protection even without a catastrophic collapse.

A lateral deformation analysis for the n500+3 raise was performed at two locations; station 258+00 and station 277+00. The two sections analyzed represent two ends of the spectrum for the existing wall, station 258+00 is the smallest section and station 277+00 is the tallest section. Most of this wall was originally constructed to maintain the railroad "slot", however much of the slot has been filled in. This analysis assumed that any remaining slot would be filled in to the adjacent existing ground surface. The analysis was performed using the Ensoft software GROUP, which performs a 3 dimensional analysis of a pile group, and provides lateral deflections as well as lateral and axial loads on the piles.

2. Results:

Station 258+00. The lateral deflection was computed to be 0.6 inches for the undrained loading case and pinned pile head connection, and 2.4 inches for the drained loading condition. A limit of 1-inch of lateral deflection is the design target. Axial and lateral load capacities were not exceeded. These deflections are considered to be excessive, so a wall modification was investigated. An additional pile (same type as the existing) was added to the wall foundation on the land side at the same spacing as the existing piles. This reduced

computed wall deflections to 0.3 and 1.1 inches respectively. This is considered acceptable.

Station 277+00. The lateral deflection was computed to be 0.15 inches for the undrained loading case and pinned pile head connection and 0.30 inches for the drained loading case. Axial and lateral load capacities were not exceeded.

3. Recommendations:

a. The floodwall foundation at station 258+00 is inadequate for the n500+3 raise, and the foundation at station 277+00 is adequate. The wall foundation for the section at 258+00 can be modified for the n500+3 raise. The next step was determining the extent of wall that requires additional piles.

b. Comparing the net change in exposed wall stem (stem above the landside ground surface) for the n500+3 raise and accounting for filling all low landside slot areas to adjacent existing ground, the wall at station 258+00 showed a net stem increase of about 4.8 feet and the wall at station 277+00 showed a net stem decrease of 8.6 feet.

c. All walls with a net wall stem increase greater than 2.5 feet should be modified by adding an additional land side pile to the foundation. This would be reflected in the feasibility study construction cost estimate. The wall system could be evaluated in more detail during PED, hopefully to reduce the amount of wall that requires foundation modification.

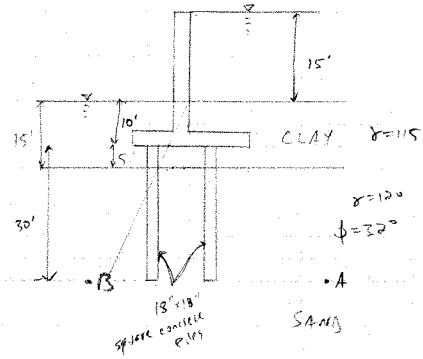
Wall foundation modification locations:

Sta 257+65 to Sta 261+50
Sta 283+50 to Sta 302+60*

*The wall downstream of Central Avenue, station 295+50 to 302+58 is a wall on a single pile. It meets the net stem raise criteria as provided in the above paragraph, but should evaluated for lateral deflection during PED.

Armourdale Levee | Pile Capacity | GMB 06/08/07
Ck RSK 6/11/07

EM - 1110-2-2906 Pile Foundation Design



Calculate Ultimate Capacity for Extreme Event (MAX LOAD)
N500+3 water Loading, Existing Piles.

Assume End Bearing only.

$q_t = \bar{\sigma}_v N_q$ ✓ $\bar{\sigma}_v$ calculated : $\bar{\sigma}_v$ = effective stress @ pile tip

N_q from Figure 4-4 in EM.

$N_q = 30$ ✓

$q_t = (2229\ psf)(30)$

$q_t = 66870\ psf$

Pt A $\bar{\sigma}$

$\sigma = (15)(62.4) + (15)(115) + (25)(120)$

$\sigma = 5661\ psf$

✓ $u = (15+15+25)(62.4)$ } SLIGHTLY CONSERVATIVE

$u = 3432\ psf$

$\bar{\sigma} = \sigma - u$
$= 5661 - 3432$

$\bar{\sigma}_v = 2229\ psf$

Ultimate Pile capacity =
$(66870\ psf)(1.5\ ft \times 1.5\ ft)$
$= 150{,}457\ lbs$ (67,000 lb)
$= 150\ kips$

From EM, Factor of safety recommended for extreme condition is 1.7. However, I recommend using minimum F.S. = 2.0 for extreme event.

- Ultimate Pile Capacity = 150 kips
- Allowable Pile Capacity = 75 kips

Original designers recommended 35 ton (70 kip) Allowable Pile Capacity.

Interesting!

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

where

c_a = adhesion between the clay and the pile

α = adhesion factor

c = undrained shear strength of the clay from a Q test

The values of α as a function of the undrained shear are given in Figure 4-5a.

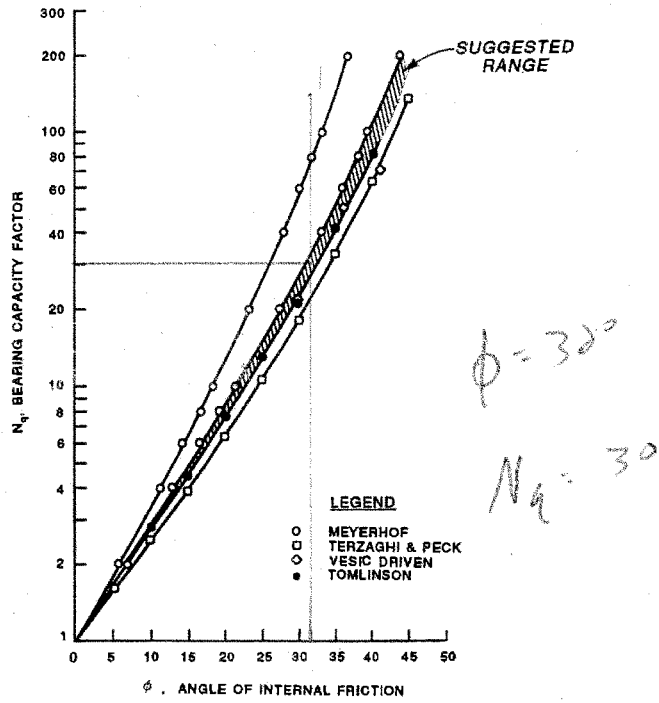


Figure 4-4. Bearing capacity factor

An alternate procedure developed by Semple and Rigden (Item 56) to obtain values of α which is especially applicable for very long piles is given in Figure 4-5b where:

$$\alpha = \alpha_1 \alpha_2$$

and

$$f_a = \alpha c$$

Soil-pile properties should, therefore, be determined for each type of loading to be considered.

c. Factor of Safety for Pile Capacity. The ultimate axial capacity, based on geotechnical considerations, should be divided by the following factors of safety to determine the design pile capacity for axial loading:

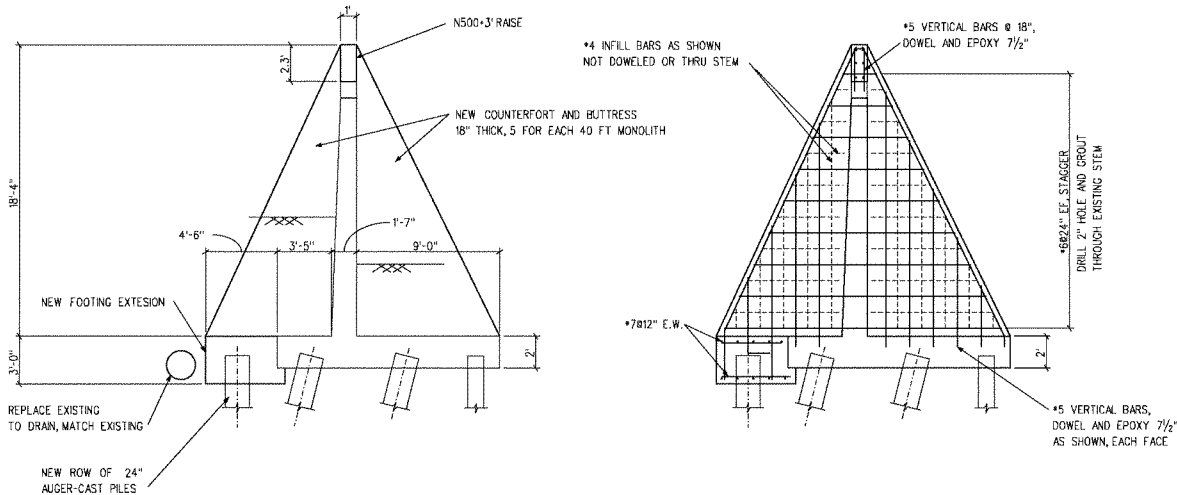
Method of Determining Capacity	Loading Condition	Minimum Factor of Safety Compression	Minimum Factor of Safety Tension
Theoretical or empirical prediction to be verified by pile load test	Usual	2.0	2.0
	Unusual	1.5	1.5
	Extreme	1.15	1.15
Theoretical or empirical prediction to be verified by pile driving analyzer as described in Paragraph 5-4a	Usual	2.5	3.0
	Unusual	1.9	2.25
	Extreme	1.4	1.7
Theoretical or empirical prediction not verified by load test	Usual	3.0	3.0
	Unusual	2.25	2.25
	Extreme	1.7	1.7

The minimum safety factors in the table above are based on experience using the methods of site investigation, testing and analysis presented herein and are the basis for standard practice. Deviations from these minimum values may be justified by extensive foundation investigations and testing which reduce uncertainties related to the variability of the foundation material and soil strength parameters to a minimum. Such extensive studies should be conducted in consultation with and approved by CECW-ED. These minimum safety factors also include uncertainties related to factors which affect pile capacity during installation and the need to provide a design capacity which exhibits very little nonlinear load-deformation behavior at normal service load levels.

d. Allowable Stresses in Structural Members. Allowable design stresses for service loads should be limited to the values described in the following paragraphs. For unusual loadings as described in paragraph 4-2b(2), the allowable stresses may be increased by one third.

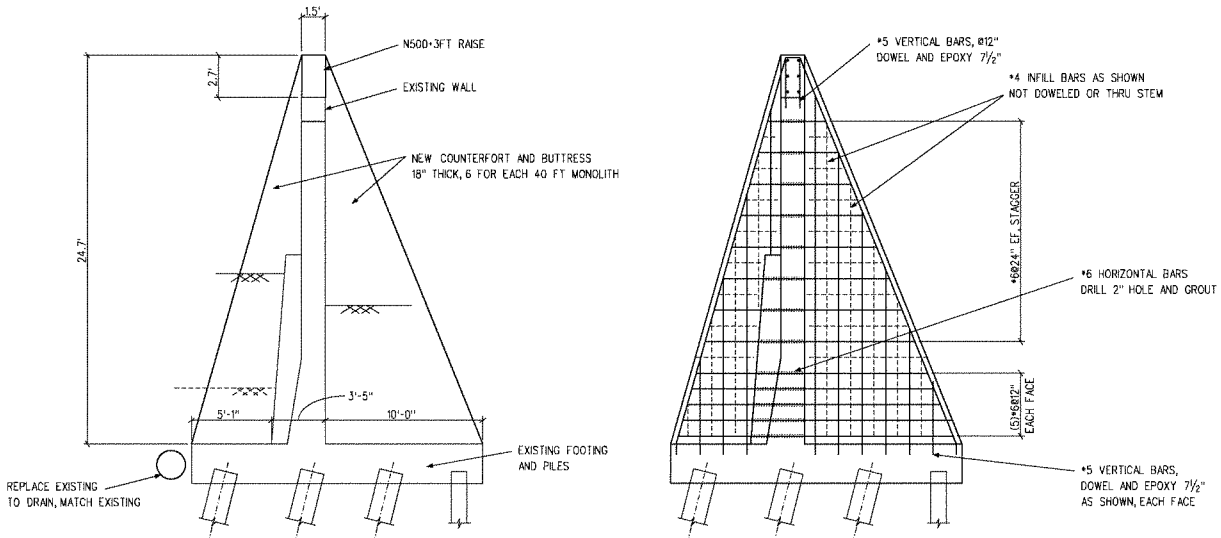
(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowable stress should reflect a high factor of safety. The distribution of allowable axial tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region is usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a

EXHIBIT A-10.7
Wall Modification With New Piles



AVERAGE FLOODWALL MODIFICATION FOR
257+65 TO 261+50 AND 283+50 TO 302+58
(BASED EXISTING MON. 83 AND AVG RAISE)

EXHIBIT A-10.8
Wall Modification Without New Piles



AVERAGE FLOODWALL MODIFICATION FOR
261+50 TO 274+36 AND 277+21 TO 283+50
(EXISTING MON. 62 & AVG RAISE)

Chapter A-11

STRUCTURAL ANALYSIS CID-KS

CHAPTER A-11
STRUCTURAL ANALYSIS CID-KS

A-11.1 OVERVIEW

This chapter addresses the structural features (floodwalls, gatewells, and closure structures) for the Central Industrial District-Kansas (CID-KS) Unit, which includes both existing conditions and future with project conditions. The existing conditions of the structural features are addressed first, followed by a separate evaluation of future alternatives for raising the current level of protection (N500+0, N500+3, and N500+5). The existing condition evaluation includes assigning reliability to the structures. The future conditions section addresses the approach taken to meet the impacts due to each raise option. For general structural methodology refer to the "General Structural Analysis" chapter of this appendix. Elevations reference the NGVD29 datum as information was obtained from record drawings unless otherwise stated.

A-11.2 ASSUMPTIONS

See the "General Structural Analysis" chapter for general KC Levees Structural Feasibility Assumptions. The following assumptions are in addition to those general assumptions, applying specifically to the CID-Kansas Unit.

1. Due to smaller geotechnical strengths, only the long-term load case of the seemingly critical closure or floodwall was analyzed with consideration of pile strengths.

2. The footer's heel and toe maximum moments occur at the stem.

3. For a consistent risk and economic comparison with other units' T-wall results, axial tension in the heel of T-walls was not analyzed.

4. Heels of T-walls were analyzed without contribution of key forces producing a pure moment into the heel. This moment is usually small in comparison with heel forces. Also, the heel was analyzed for net vertical pressures only.

5. Material properties could not be determined from existing documentation for a majority of the structures on the CID-KS Unit. The floodwalls from Station 27+84 to 36+62 and from 102+74.60 to Station 180+00 were constructed in the mid 1930s. Material properties could not be determined for these structures. As a result, estimated steel strengths, concrete strengths, and standard deviations were used for all strength analysis and risk computations for these structures on the CID-KS Unit as noted in a previous chapter on structural analysis methodology. For the floodwall from Station 74+36 to 77+28, the material properties as designated in the "CID Kansas Section; Analysis of Design to accompany Construction Plans" was used for analysis. The floodwalls from Station 26+72 to 27+84 and from 36+62 to 40+31 would have

also been designated by this documentation, but the older floodwall from Station 27+84 to 36+62 controlled the analysis for this section of floodwall.

6. The following checks were not performed on the floodwalls because they were assumed likely not control: tension pile embedment, punching shear of pile through cap, vertical and horizontal bearing of pile and cap, and horizontal shear in cap (at sheetpile) and in all piles.

7. During initial floodwall calculations using the CASE CPGA software, default values for C33, the axial stiffness modifier for embedded portion of the pile, were used based on the User's Guide for CPGA. These default values used were 1 and 2 for full tip bearing and full skin friction respectively. Upon further investigation during other project design phases, it was found that variations from these scenarios results in redistribution of loads in the analysis. Additional steps were taken to calculate some site specific C33 values for different pile types and the calculated C33 values were used for all of the existing conditions evaluations and the most critical future condition evaluation. It was determined that smaller calculated values of C33 were appropriate based on site specific data. The smaller C33 values typically resulted in a more even distribution of load and therefore a larger factor of safety for most of the piles than when using the default C33 values. Any calculations with a value of 2.0 or 1.0 for C33 should be reviewed during design phase and site specific information should be used for the calculation of C33 in lieu of the default values.

A-11.3 SOIL MATERIAL PROPERTIES

The soil properties used for the CID-KS calculations are located in Table A-11.1. These values were used for general blanket properties in the structural calculations. Soil to structure friction and cohesion interaction was typically neglected for stability and strength calculations. However, where necessary, this behavior was considered under geotechnical guidance.

TABLE A-11.1
CID-KS Soil Properties

	Foundation Blanket	Fill (cinders/rubble)
Friction Angle (drained shear strength)	24°	21°
Cohesion	-	-
Moist unit weight	110 pcf	110 pcf
Saturated unit weight	115 pcf	115 pcf
Wall to soil friction coefficient (used in gatewell uplift)	0.25	0.25

A-11.4 EXISTING CONDITIONS

A-11.4.1 Floodwalls

Floodwalls on the CID-KS Unit consist of two separate floodwalls on timber piles and one concrete pile founded floodwall. For each of these walls the existing condition was evaluated. The evaluation entailed analysis of existing reliability and recommending solutions for any existing condition deficiencies. See the General Structural Analysis chapter of the engineering appendix for further information on reliability analysis.

The existing CID-KS floodwalls are as follows:

- Station 27+84 to 36+62 timber pile founded floodwall constructed in 1937 and raised approximately 3-ft in 1950
- Station 26+72 to 27+84 and 36+62 to 40+31 timber pile founded floodwall constructed in 1950
- Station 74+36 to 77+28 concrete pile founded floodwall constructed in 1950 and raised approximately 1.8-ft as part of the 1962 Modification (62 Mod) in 1979
- Station 102+74.60 to 180+00 timber pile founded floodwall constructed in 1937 and raised up to approximately 2-ft in 1950
- Station 102+74.60 to 166+25 timber pile founded floodwall raised approximately 3-ft as part of the 1962 Modification (1962 Mod) in 1979; Wall from 166+25 to 180+00 was not raised

This does not include walls that are part of a closure structure (see Closure Structures section).

A-11.4.1.1 Floodwall Station 26+72 to 40+31

The original floodwall on timber piles from Station 27+86.16 to Station 36+64.17 was constructed around 1937. The wall was originally constructed on 12-in diameter timber piles with provisions for a future raise (Type "C" floodwall).

In 1950, the original wall was raised, the heel was extended, and a precast concrete sheet pile cutoff wall was added to the heel. Also in 1950, additional walls upstream and downstream of this wall were constructed from Station 26+72.66 to 27+86.16 (Type "D" floodwall) and from Station 36+64.17 to Station 40+31.25 (Type "D" floodwall). These walls were also constructed on timber piles and included a 12-in precast concrete sheet pile cutoff wall. Information from "Analysis of Design to accompany Construction Plans" Feb 1946, which accompanied the drawings, provided design assumptions for the piles. See Exhibit A-11.1 for details of this wall at the end of this chapter.

The result is a uniform wall from Station 27+86.16 to 40+31.25 for which most construction information is known. The designs of the two different eras are very similar and we assumed the only significant difference from the original and more recent construction was possibly greater material strengths. Assumed material strengths used for analysis were based on historical data for the original construction in 1937.

A portion of floodwall from Station 39+66 to 40+06 was modified in October 2000 "Central Industrial District Flood Wall Rehabilitation PL 84-99". Deformation noted at the stem was the reason for the modification. The modification involved exposing the landside toe, adding a new repair stem, and repairing the waterstop. During the rehab work, the tops of the piles were exposed. The exterior of the piles were noted not to have any deficiencies, but no pile testing was done.

The controlling section of wall for analysis was the section from Station 40+06 to 40+31 because it has the greatest pile spacing.

For existing conditions, with water to top of wall, the monolith analyzed for this floodwall reach met the screening factors of safety. The factors of safety are provided in Table A-11.2.

TABLE A-11.2
Existing Floodwall Conditions

CID-KS Floodwalls - Pile Foundation Floodwalls - Existing Conditions with Water to Top of Wall									
		Factors of Safety							
			Concrete Sheet Piling		Concrete Pile		Timber Pile		
Station	Controlling Wall Cross Section	Wall/ Pile Cap (1.5 Req'd)	1.5 Req'd for strength	Meets allowable Axial Compression and Tension loads	1.5 Req'd for strength	Meets allowable Axial Compression and Tension loads	Axial (ALF<1), Combined Compression and Bending (CBF<1), Shear meet allowables?	Comments	Reliability
26+72 to 40+31	40+06 to 40+31	1.9	7.2	YES	NA	NA	ALF=0.60 CBF=0.33 SHEAR OK	Wall flexural steel riverside controlling mechanism	
74+36 to 77+28	NA	3.5	23.8	YES	30.5	YES	NA	acts more as a retaining wall than a floodwall	
102+73 to 179+81	138+95 to 162+91	2.2	NA	NA	NA	NA	ALF=0.65 CBF=0.93 Vallow/Vu=0.8 - NG	Timber Pile shear POF analysis	99.8% calculated

A-11.4.1.2 Floodwall Station 74+36 to 77+28

The original wall was constructed in 1950 and was raised in 1979 as part of the 1962 Mod. Refer to Exhibit A-11.2 for details at the end of this chapter. The wall is essentially a tie-back wall for two railroad bridges; the Missouri Pacific Railroad (MoPac) and the Union Pacific Railroad with levees at either end of the floodwall. This wall actually consists of three walls and is broken up as follows:

- From Station 74+35.94 to the MoPac Railroad Bridge abutment wall at approximately Station 75+25 (Type "A" floodwall)
- From Station 75+76 to 75+92

- From the Union Pacific Railroad bridge abutment wall at approximately Station 76+42 to Station 77+27.75 (Type "A" floodwall)

The record drawings from the 1950 construction and the record drawings from the 1962 Mod do not show the details of the floodwall. The 1962 Mod record drawings show a very rough sketch of that wall, but no details were found. These walls are acting more as retaining walls than floodwalls. There is very little exposed stem on the landside.

There was also a retaining wall inland of the MoPac Railroad bridge that was detailed and constructed in 1950. This wall retained fill for the railroad slot that paralleled the floodwall and went under the MoPac Railroad and toward the stockyards upstream of this location. This retaining wall was buried during the 1962 Mod and was not analyzed for this feasibility study.

The controlling section of wall for analysis was the Type "A" floodwall. The Type "A" floodwall is founded on 18-inx18-in precast concrete piles with a 12-in precast concrete sheet pile cutoff wall at the heel. As shown in the results the capacity of the floodwall far exceeds what is required. It is likely that the walls were designed more as retaining walls with large railroad live loads than as floodwalls. This floodwall with water to top of wall met the screening factors of safety. The factors of safety are provided in Table A-11.2.

A-11.4.1.3 Floodwall Station 102+74 to 179+81

This floodwall was originally constructed in 1937, it was raised in 1950 and a portion of it was raised again in 1979 (1962 Mod). The original wall starts at Station 102+73.38 and ends at 179+81.15 and was raised in 1950 for that whole length. For the 1962 Mod raise, it was raised from Station 102+73.38 to Station 166+25 and a stop log gap was constructed perpendicular to the channel at Station 166+25. The remainder of the wall from Station 166+25 to 179+81.15 was not raised with the 1962 Mod. Refer to Exhibit A-11.3 for details at the end of this chapter. The original construction from 1937 consisted of two types of floodwalls on timber piles with provisions for a future raise and is detailed as follows:

- Station 102+73.38 to 120+01.12 Type "C" floodwall
- Station 120+01.12 to 162+90.66 Type "B" floodwall
- Station 162+90.66 to 175+66.22 Type "C" floodwall
- Station 175+66.22 to 179+81.15 Type "B" floodwall

In 1950 the stem was raised varying from approximately 0.6-ft to 2-ft.

In 1979 with the 1962 Mod, the following modifications were made from Station 102+73.38 to Station166+25:

- stem was raised up to 4.5-ft
- counterforts and buttresses were added
- riverside base was extended
- steel sheet pile cutoff wall was constructed at the heel

The floodwall upstream of the stoplog gap at Station 166+25 that was not modified with the 1962 Mod and was not analyzed for this feasibility study because it does not provide the existing required level of protection. The floodwall sections from 102+74 to 166+25 are very similar with very minor soil elevation variations. Total heights range from 13-ft to 15-ft and exposed stem heights range from 5-ft to 13-ft. Information from "Analysis of Design to accompany Construction Plans" Feb 1946, which accompanied the drawings, provided design assumptions for the piles.

The monolith between Station 162+39.20 and Station 162+91.20 (Type "B" floodwall) was the controlling monolith for analysis because of the smaller footing thickness, wider counterfort spacing, smaller stem width at the base, and the tallest Type "B" floodwall monolith.

For existing conditions, with water to top of wall, this monolith did not meet the screening factors of safety and a reliability calculation was performed as shown in Table A-11.2.

A-11.4.1.4 Floodwall Summary

A pile investigation contract was awarded in 2010 to determine the capacity and condition of the floodwall's existing timber piles. The contract scope was to determine the overall timber piles condition through visual inspection and geophysical testing. The piles were exposed in 2 locations and the following information was collected: integrity of the pile cap/soil interface along the base of the floodwall, lengths of the partially exposed piles, overall condition of the partially exposed piles utilizing sonic echo/impulse response (SE/IR) methods, pile core samples for laboratory testing. The piles were found to be in good condition. SE/IR test methods provided pile length and condition information. The 20-ft pile length shown on the Record Drawings was verified by the SE/IR test to be the minimum pile length. The sonic impulse velocity (longitudinal and across the diameter) indicated the condition of most of the piles were sound. Some piles were noted as cracked or split, but are described as surficial cracking and assumed to have been in that condition since construction. The timber species was determined to be southern pine and the creosote penetration depth was determined to be acceptable for the majority of the piles through laboratory testing of core samples. Low levels of fungal colonization were also noted from the core samples. It was concluded that, in general, the investigation results indicate that the piles are in excellent physical condition with sufficient preservative retention levels to produce satisfactory service. At most locations along the floodwall investigation locations the pile cap/soil interface was intact and in other locations up to 3/4" of roofing was observed. The roofing extended riverward to the floodwall key. Refer to the "Investigation and Testing of Existing Flood Wall Piles, Pile Assessment Report", March 2011 by HNTB for more detailed information.

A-11.4.2 Drainage Structures

The results for the complete analysis of the CID-KS gatewells existing conditions are presented in Table A-11.3. Most information (wall thicknesses, reinforcement size, and spacing) was available for the gatewells.

TABLE A-11.3
Existing Gatewell Conditions

CID-KS Gatewells - Existing Conditions with Water to Top of Protection				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Reliability
83+52 CID-MO STA.	2.5	1.6	Wall Shear	
87+60 CID-MO STA.	1.8	1.7	Wall Shear	
10+81	1.7	1.9	Wall Bending	
19+90	2.5	1.8	Base Slab Moment	
37+06	1.972	11.196	Base Slab Moment	
50+98	1.6	5.4	Base Slab Moment	
52+07	1.5	8.2	Base Slab Moment	
56+91.5	2.1	4.8	Base Slab Moment	
58+12	2.6	2.8	Wall Flexural Steel	
62+78	1.4	5.1	Base Slab Moment	
67+65	0.9	2.6	Base Slab Moment	10% calculated
71+70	1.3	3.3	Base Slab Moment	
74+21.5	1.58	4.374	Base Slab Moment	
77+80	1.2	1.8	Base Slab Moment	
80+90	3.7	2.1	Base Slab Moment	
84+90	3.9	1.9	Base Slab Moment	
88+19	3.8	2	Base Slab Moment	
94+32	3.6	2.6	Base Slab Moment	
98+05	3.4	2	Base Slab Moment	
102+52	3.1	3.4	Base Slab Moment	
106+49	2.2	1.3	Wall Bending	99.8% calculated
124+83	*	*	*	
138+29	2.5	2.5	Base Slab Moment	
152+28	1.8	2.5	Base Slab Moment	
159+70	1.2	3.7	Base Slab Moment	
167+95	NA	5.6	Wall Bending	

*Gatewell at Station 124+83 is located at the Turkey Creek Sewer and is used to operate the emergency gates for the Turkey Creek Sewer. The gatewell was constructed on top of, and integral with, the existing sewer culvert. The gates are at the upstream end of the sewer, therefore when the Kansas River level rises, the water rises in the gatewell as well so the water is equalized on both sides of the gatewell walls. Therefore the gatewell structure does not need to be checked for uplift or strength.

All of the gatewells, with the exception of the gatewell at Station 67+65 and 106+49, met the screening factors of safety. A reliability analysis was done for the gatewells at Station 67+65 and 106+49 and the results are as shown in Table A-11.3.

The gatewell at Station 124+83 is located at the Turkey Creek Sewer and is used to operate the emergency gates for the Turkey Creek Sewer. The gatewell was constructed on top of, and is integral with, the existing sewer culvert. The gates are at the upstream end of the sewer, therefore when the Kansas River level rises the water rises in the

gatewell and around the gatewell. So the water is equalized on both sides of the gatewell walls. Therefore, the gatewell structure was not analyzed for uplift or strength.

To address the existing conditions reliability issue for the gatewell at Station 67+65, a concrete collar with a mass of approximately 15 kips, or 100 cubic feet of concrete, would need to be added to the top of the gatewell for additional structure weight to improve the uplift factor of safety to the 1.1 required. No modifications to the gatewell at Station 106+49 are recommended for existing conditions.

For further information regarding the typical gatewell analysis approach and examples see the General Structural Analysis chapter of this appendix.

A-11.4.3 Closure Structures

The reliability for the CID-KS closure structures are given in Table A-11.4. Some brief points and conclusions follow.

A-11.4.3.1 CRI&P RR Bridge Station 104+51.50

This bridge and the tracks at this stoplog gap have been abandoned. In conjunction with the abandonment of the bridge, the gap was filled in with concrete. The details for that concrete fill were not found. The concern that the concrete fill may not be adequate was discussed with the sponsor who verified that the plans were submitted to the Corps of Engineers and accepted. All other information for this crossing was available. For existing conditions with water to top of wall, this stoplog gap did not meet the screening factor of safety for bearing and reliability was calculated. The factors of safety and reliability values are provided in Table A-11.4.

A-11.4.3.2 KC Terminal Bridge Station 132+20

The details of this closure structure were included in the KC Terminal Bridge and Approach Alteration Plans which are separate from the O&M drawings. The As-built plans date to May 1964. The plans show the construction of the stoplog gap is integral to the bridge abutment. Therefore, this closure structure was not analyzed for bearing, overturning or sliding. For existing conditions with water to top of wall, this stoplog gap did not meet the screening factors of safety and a reliability analysis was completed. The factors of safety and reliability values are provided in Table A-11.4.

In order to address the existing conditions reliability issue for this stoplog gap, the back wall would need to be thickened by 18-in and a layer of reinforcing would need to be added to the thickened portion of the back wall. The wall would need to be thickened from approximately Elev. 753.17 to approximately 4.75-ft from the top of the wall for a total height of 8-ft.

A-11.4.3.3 Upper End Stoplog Gap Station 166+31

This stoplog gap is perpendicular to the floodwall at this station and ties into the bluff on the landside. Plans for this structure were available in the O&M drawings for the 1962 Mod. For existing conditions with water to top of wall, this stoplog gap did not meet the

screening factor of safety for bearing and reliability was calculated. The factors of safety and reliability values are provided in Table A-11.4.

TABLE A-11.4
Existing Closure Structure Conditions

CID-KS Stoplog Gaps - Spread Footing (Existing Condition with Water to Top)					
Station	Crossing	Overturning % Base in Compression (25% Req'd)	Bearing, Factor of Safety (2.0 Req'd)	Sliding Factor of Safety (1.1 Req'd)	Wall Strength Factor of Safety (1.5 Req'd)
104+51.50	CRI&PRR bridge	100%	1.78	144.8	3.0
132+20	KC Terminal Bridge	---	---	---	0.7
166+31	Upper End Gap	89%	1.41	1.6	1.6

CID-KS Stoplog Gap - Continued (Existing Condition with Water to Top)					
Station	Crossing	Factors of Safety			
		Stoplog Post (1.5 Req'd)	Stoplog Guide (1.5 Req'd)	Stoplog (1.5 Req'd)	Post Well (1.5 Req'd)
104+51.50	CRI&PRR bridge	---	2.9	NOI*	---
132+20	KC Terminal Bridge	1.82	2.22	meets criteria	3.65**
166+31	Upper End Gap	1.87	4.6	2.0	1.7

Note: NOI = not enough information
*Stoplog gap was filled in with concrete; therefore, no stoplogs to analyze. No plans have been found that detail the reinforcing put into that concrete in-fill; calculated that the reinforcing required to meet the 1.5 FS would be #8 @ 8".
** The post well is integral with the bridge abutment, so the well is assumed to be stout for bending and shear, but the post well was checked for bearing and shear due to loads from post

CID-KS Stoplog Gaps Reliability (Existing Condition with Water to Top)			
Station	Crossing	Reliability	Comments
104+51.50	CRI&PRR bridge	94% calculated	Details of the concrete placed to fill in the stoplog gap are unknown
132+20	KC Terminal Bridge	0.6% calculated	Calculated Reliability; 12" back wall is not substantial enough to take water to the top of the wall
166+31	Upper End Gap	80% calculated	Complete drawings and specifications available

A-11.5 FUTURE WITH PROJECT CONDITIONS

Future conditions include the alternatives and recommendations for raising the unit elevation to the river stage of the N500+0, N500+3, and the N500+5. The primary emphasis for the evaluation was on the N500+3 with abbreviated approach to the N500+0 and N500+5. This was achieved by using the results from the N500+3 and the existing conditions to estimate the N500+0 and N500+5.

A-11.5.1 Floodwall Future Conditions at Existing Floodwall Stations

The following discussion and results are for stations where walls currently exist. The following alternatives are a summary of the thought process that was used to develop the final recommendation for the N500+3, along with a brief discussion of the N500+0 and N500+5, where walls currently exist.

Based on the 2010 pile investigation it was assumed that the existing timber piles are in good condition and are still structurally adequate to handle a floodwall raise. The pile investigation was limited in nature and does not eliminate uncertainty of the condition of the remaining existing timber piles. It is assumed that for all options with timber piles a more complete sampling and testing measure will need to be implemented during design.

In addition to timber pile testing, a representative sampling of concrete should be performed during design to validate the material strength used in calculations.

Refer to the structural methodology for more information on formulation of the general floodwall alternatives. Also, for background information on existing walls, refer to the Existing Conditions section of this chapter.

A-11.5.1.1 Floodwall Station 26+72 to 40+31

N500+3, 1-ft raise

Alternative 1: Floodwall Raise

A floodwall raise for this portion of floodwall is feasible from an existing structural capacity standpoint. The analysis shows that for the stem to be extended, the stem would need to be widened by 12 inches near the base on the riverside face of the wall. Widening the stem to the landside face increases axial load on that pile under it and then the axial load exceeds the capacity. The pile capacity, for a proposed raise, changes very little in this reach because the raise is very small. Based on the CPGA analysis, the existing piles meet allowable loads. Based on the Group analysis, the deflections for a floodwall raise on existing piles are less than an inch.

Alternative 2: Levee with floodwall acting as retaining wall (DEEMED NOT APPLICABLE)

This existing floodwall was investigated to determine if it could act as a retaining wall for the additional landside soil (landside levee construction). The controlling typical section was investigated (refer to existing conditions for more information), and it was found that the wall cannot retain enough soil to construct a landside levee.

Alternative 3: Replace floodwall with new levee (DEEMED NOT APPLICABLE)

Real estate is not available for this option.

Alternative 4: Replace floodwall with new floodwall (staged construction)

A new floodwall will be constructed in-line with the existing floodwall by way of staged construction. This method of construction will require temporary protection, sequenced construction and building around existing piles. Special considerations may need to be made where the Mistletoe Pump Station is located on this wall.

Alternative 5: Replace floodwall with new floodwall (offset construction)

A new floodwall will be constructed offset landward of the existing floodwall. This method of construction will require additional right-of-way and possibly utility relocations. Special wall transitions will be needed at the Mistletoe Pump Station and at

each end of the floodwall where it ties into the James Street Bridge abutment and the KCS Bridge abutment.

Recommendation
Alternative 1: Since the existing timber piles appear to be in satisfactory condition, raising the existing floodwall should be most cost effective. See the Maps section of the Feasibility Report for location and Exhibit A-11.4 of the Supplemental Exhibits section at the end of the chapter for a typical section.

N500+0 and N500+5 Events
N500+0
For the N500+0 event, the wall would not need to be raised and the existing structure is adequate for this event.

N500+5
For this 3-ft raise, raising the existing floodwall would require thickening the stem at the base, extending the toe and adding a row of auger cast piles. Another alternative would be to replace the wall, but may be less cost effective.

A-11.5.1.2 Floodwall Station 74+36 to 77+28

N500+3, 3-ft Raise
Alternative 1: Floodwall Raise
A floodwall raise for this portion of floodwall is feasible from an existing structural capacity standpoint. The analysis shows that for the stem to be extended, the stem would be need to be widened by 20-in near the base, a 24-in auger cast pile would be added on the landside and the toe would be extended landward. New buttresses would be required at 8-ft centers to transfer forces at the toe extension.

Alternative 2: Floodwall Raise and Replacement
A floodwall raise is feasible as described in Alternative 1, however from a constructability standpoint it may be best to raise a portion of the floodwall and replace other portions. There are Missouri Pacific and Union Pacific railroad bridges that intersect this section of floodwall. This alternative specifies raising the floodwall from Station 74+35.94 to 75+25 and modifying that section of floodwall as described in Alternative 1 and replacing the floodwall from Station 75+76 to 77+27.75 with a T-wall type floodwall on auger cast piles. This will better facilitate the new construction of the stoplog gap at the Union Pacific railroad bridge by constructing new wall upstream of the new stoplog gap rather than trying to tie the old wall into the new gap structure. A special wall will need to be constructed between the bridges to allow for a sand bag gap on the current wall alignment at the Missouri Pacific railroad bridge and a stoplog gap landward of the existing sandbag gap at the Union Pacific railroad bridge.

Alternative 3: Floodwall Replacement
Although, the existing floodwall is structurally adequate to be raised for this project, it may be a cleaner solution to construct all new T-wall type floodwall on auger cast piles on alignment landward of the existing floodwall for this short portion of wall. There are

two railroad bridges to construct around and the new stoplog gap would need to be offset from the original alignment. To tie into the new stoplog gap and to construct a wall between the two bridges to transition from the new stoplog gap on offset alignment to the sand bag gap on existing alignment, may cause more constructability concerns rather than just replacing the whole length of wall along an offset alignment.

Recommendation
Alternative 2: This alternative minimizes cost by raising as much existing floodwall as possible and constructing new floodwall in locations where modifications would be less desirable. See the Maps section of the Feasibility Report for location and the sketch of typical floodwall Sections B and C in Exhibit A-11.4 at the end of the chapter.

N500+0 and N500+5 Events
N500+0
For the N500+0 event the wall would not need to be raised. The existing floodwall would not need to be modified or replaced.

N500+5
For this 5-ft raise, new floodwall construction with auger cast piles was determined to be the most effective solution. The approach is similar to that detailed in the typical sections of Exhibit A-11.4 in the Supplemental Exhibits section for new wall for the N500+3.

A-11.5.1.3 Floodwall Station 102+73 to 179+81

N500+3, up to 4-ft. raise
Alternative 1: Raise Floodwall from 102+73 to 131+00 and Construct Tieback Wall to Bluff (Section D and E respectively in Exhibit A-11.4 of the Supplemental Exhibits section)
Significant modifications to this floodwall have occurred in the past. A floodwall raise is feasible, but additional substantial modifications would be required. The following would be required:

- stem would be raised between 2.7-ft and 3.9-ft
- stem would be widened by 12-in from Station 120+01.12 to Station 131+00 (Original Type B wall needs stem widening, but Type C does not)
- the floodwall toe would need to be extended landward by 9-ft
- 2 new rows of 24-in round auger cast piles spaced at 5-ftcenters would need to be added to the new floodwall toe
- New counterforts on the landside and backing beams on the riverside would be required and would be spaced centered between the existing riverside counterforts
- New relief wells will be required for underseepage control; new relief wells ensure that pile capacity is as good as, if not better than, the existing pile capacity; based on the group analysis, the deflections for a floodwall raise on existing piles with two rows of new piles are less than an inch

The tieback wall would be new construction of a T-wall type floodwall on auger cast piles approximately 1100-ft in length, would begin at Station 131+00 and tie into the bluffs as shown in the Maps section of the Feasibility Report, and would include 4

stoplog gaps. This tieback wall will eliminate the need to replace approximately 2500-ft of floodwall upstream of this station, the need for a new stoplog gap at the KC Terminal Railroad Bridge and at the Upper End. During the feasibility process, it was anticipated that the railroads may have concerns with this alternative since the new floodwall and gaps could impose a sight distance problem at the railroad/floodwall junctions.

Alternative 2: Raise Floodwall from 102+73 to 166+25 (Section D in Exhibit A-11.4 of the Supplemental Exhibits section at the end of this chapter)
Significant modifications to this floodwall have occurred in the past. A floodwall raise is feasible, but additional substantial modifications would be required. The following would be required:

- stem would be raised between 2.7-ft and 3.9-ft
- stem would be widened by 12-in from Station 120+01.12 to Station 162+90.66 (Original Type B wall needs stem widening, but Type C does not)
- the floodwall toe would need to be extended landward by 9-ft
- 2 new rows of 24-in round auger cast piles spaced at 5-ft centers would need to be added to the new floodwall toe
- New counterforts on the landside and backing beams on the riverside would be required and would be spaced centered between the existing riverside counterforts
- New relief wells will be required for underseepage control; new relief wells ensure that pile capacity is as good as, if not better than, the existing pile capacity; based on the group analysis, the deflections for a floodwall raise on existing piles with two rows of new piles are less than an inch

Alternative 3: Raise Floodwall from 102+73 to 138+00 and Construct Tieback Wall to Bluff (Section D and E, respectively, in Exhibit A-11.4 of the Supplemental Exhibits section at the end of this chapter)
Significant modifications to this floodwall have occurred in the past. A floodwall raise is feasible, but additional substantial modifications would be required. The following would be required:

- stem would be raised between 2.7-ft and 3.9-ft
- stem would be widened by 12-in from Station 120+01.12 to Station 138+00 (Original Type B wall needs stem widening, but Type C does not)
- the floodwall toe would need to be extended landward by 9-ft
- 2 new rows of 24-in round auger cast piles spaced at 5-ft centers would need to be added to the new floodwall toe
- New counterforts on the landside and backing beams on the riverside would be required and would be spaced centered between the existing riverside counterforts
- New relief wells will be required for underseepage control; new relief wells ensure that pile capacity is as good as, if not better than, the existing pile capacity; based on the Group analysis, the deflections for a floodwall raise on existing piles with two rows of new piles are less than an inch

The tieback wall would be new construction of a T-wall type floodwall on auger cast piles approximately 520-ft in length and would include 1 stoplog gap. This tieback wall

will eliminate the need to replace approximately 2,300-ft of floodwall upstream of this station and would thereby eliminate substantial cost to the project. A disadvantage to this alternative is the impacts to the building and the railroad storage area as indicated in the Maps section of the Feasibility Report.

Alternative 4: Replace Floodwall from 102+73 to 166+25 (offset construction) – (Section F and G in Exhibit A-11.4 of the Supplemental Exhibits section at the end of the chapter)
A new T-wall floodwall on auger cast piles to be constructed on an offset alignment approximately 6-ft from the existing floodwall is recommended for this alternative. The existing floodwall would need to be removed and the new construction would overlap the area where the existing floodwall exists. The advantage of this alternative would be that little or no additional real estate would be required to construct this floodwall. However, one disadvantage would be that the new piles would need to be located during construction to clear existing piles. Existing piles are spaced longitudinally at 4-ft centers and transversely at 4.5-ft centers and the piles are 12-in in diameter. The existing pile spacing is very tight and will make new pile spacing difficult. Offsetting the new wall from the existing wall slightly will allow for better new pile arrangements. New relief wells will be required for underseepage control. This alternative would be additive to other alternative options (Alternative 1, 2 or 3) to replace the raise existing floodwall construction method.

Alternative 5: Floodwall Acting as Retaining Wall with Levee Behind (DEEMED NOT APPLICABLE)
This existing floodwall was investigated to determine if it could act as a retaining wall for the additional landside soil (landside levee construction). The controlling typical section was investigated (refer to existing conditions for more information), and it was found that the wall cannot retain enough soil to construct a landside levee.

Alternative 6: Replace Floodwall with New Floodwall (offset clear construction) (Section F and G in Exhibit A-11.4 of the Supplemental Exhibits section at the end of the chapter)
A new T-wall floodwall on auger cast piles on an offset alignment is recommended for this alternative. The advantage of this alternative would be that the new construction would be completely offset from the existing alignment and would be clear of the existing floodwall so new pile arrangement is independent of the existing pile arrangement. One disadvantage would be additional real estate will be required to construct and to maintain the new floodwall. Another disadvantage would be the additional costs associated with accessing or replacing existing gatewells. The gatewell at Station 106+49 is constructed in line with the existing wall and would need to be replaced in line with the new floodwall. A new method to access the existing gatewell for the Turkey Creek sewer at Station 124+83 would be required. The gatewell at Station 138+29 would need to be replaced so that it would be landward of the newly aligned floodwall. This alternative would be additive to other alternative options (Alternative 1, 2 or 3) to replace the existing alignment construction method.

Alternative 7: Replace Floodwall with Levee (DEEMED NOT APPLICABLE)
Real estate is not available to make this a viable alternative.

Recommendation
Note for both of these alternatives no improvements will be made past the stoplog gap at Station 166+25.

- Alternative 6 is the best structural alterative due to ease of construction, however likely to be very costly, so not the best overall project alternative
- Alternative 3 appears to be the next best alternative because the existing timber piles appear to be in satisfactory condition, raising the existing floodwall on existing alignment should be more cost effective than replacing the wall; this alternative minimizes the length of existing wall raise/modifications and is likely the most cost effective alternative

N500+0 and N500+5 Events
N500+0
For the N500+0 event the wall would need to be raised approximately one foot. The recommendation would be to raise the floodwall. It is likely a stem widening would not be required for this design event, but toe extension and new piles are required, however, the number of new piles may be able to be reduced.

N500+5
For this up to 6-ft raise, new floodwall construction with auger cast piles was determined to be the most effective solution. The approach is similar to that detailed in the Maps section of the Feasibility Report for new wall for the N500+3.

A-11.5.2 N500+3 Floodwall Summary

For the locations of proposed N500+3 floodwalls refer to the Maps section of the Feasibility Report accompanying the main body of the Feasibility Report.

A-11.5.3 Drainage Structures

The results for the complete analysis of the CID-KS gatewells N500+3 future conditions are presented in Table A-11.5. Those found to meet criteria need only a raise. For further information regarding the analysis approach and examples see the General Structural Analysis chapter of this appendix. It is possible for factors of safety to increase for future conditions if a refined analysis was warranted.

A-11.5.3.1 Recommended Actions

The gatewells requiring raises are as presented in Table A-11.5. Flotation was checked based on the extreme FS from EM 110-2-2100. Strength was checked in a manner similar to that for existing conditions for simplicity and to provide comparable results. Rather than providing a typical LRFD check, a factor of safety for each gatewell was compared to an equivalent required minimum FS based on LRFD factors. The factors of

safety were calculated as follows:

$$FS \geq HF*LF/\phi = 1.3*1.7/0.9 = 2.45 \text{ (Bending)}$$
$$FS \geq HF*LF/\phi = 1.3*1.7/0.85 = 2.6 \text{ (Shear)}$$

where: HF = hydraulic factor,
LF = single load factor,
ϕ = resistance factor (lower value for shear assumed).

TABLE A-11.5
N500+3 Gatewells Results

CID-KS - Gatewells (N500+3 with Water to Top of Protection)					
Station	Proposed Raise (ft)	Uplift Factor of Safety (>1.1 Req'd)	Strength Factor of Safety (> 2.46 Moment, >2.6 Shear Req'd)	Controlling Structural Mechanism	Notes
83+52 CID-MO STA.[R]	0	2.5	1.6	Wall Bending	Assume no raise required (no protection raise)
87+60 CID-MO STA.[R]	0	1.8	1.7	wall shear	Assume no raise required
10+81[R]	0	1.7	1.9	Wall Bending	no raise required - top of riverside gatewell at EL 760.99 (N500+3=EL760.9)
19+90[R]	0	2.5	1.8	Base Slab Moment	no raise required - top of riverside gatewell at EL 768.0 (N500+3=EL761.0)
37+06					
50+98*	0	1.6	5.4	Base Slab Moment	no raise required-GATE VALVE
52+07*	0	1.5	8.2	Base Slab Moment	no raise required-GATE VALVE
56+91.5*	3	2.4	4.1	Base Slab Moment	FILL REQUIRED (GATE VALVE)
58+12[R]	7	3.2	2.5	Wall Bending	7' raise required so that pressurized pipe can be replaced for pump station over the levee
62+78*	5	1.9	4.0	Base Slab Moment	FILL REQUIRED (GATE VALVE)
67+65*	6	1.4	2.5	Base Slab Moment	FILL REQUIRED (GATE VALVE)
71+70*	6	1.6	2.5	Base Slab Moment	FILL REQUIRED (GATE VALVE)
74+21.5					
77+80[R]	3	1.1	1.6	Base Slab Moment	Raise required - top of riverside gatewell at EL 763.25 (N500+3=EL 765.6). Modification required (FRP).
80+90	4	3.6	1.9	Base Slab Moment	Raise required - landside. Modification required (FRP).
84+90	4	3.6	1.7	Base Slab Moment	Raise required - landside. Modification required (FRP).
88+19	4	3.7	1.8	Base Slab Moment	Raise required - landside. Modification required (FRP).
94+32	4	3.3	2.3	Base Slab Moment	Raise required - landside
98+05	3	3.4	1.8	Base Slab Moment	Raise required - landside. Modification required (FRP).
102+52	3	3.2	3.0	Base Slab Moment	Raise required - landside
106+49	3	2.2	1.3	Wall Bending	Raise required - landside. Modification required (FRP).
124+83 (TC sewer)	4	**	**		Raise required
138+29	4	2.6	2.1	Base Slab Moment	Raise required
152+28*	0	1.8	2.5	Base Slab Moment	no raise required-secondary gate downstream
159+70*	0	1.1	3.6	Base Slab Moment	no raise required-secondary gate downstream
167+95	NEW GATEWELL	NEW GATEWELL	NEW GATEWELL		Replacement b/c it is attached to stoplog gap that will be replaced/relocated - OR elimination if not important

*top below level of protection on landside (no raise required)
[R] indicates gatewell is on the riverside of the protection
These gatewells were previously raised with the 1952 Mod

Some gatewells will need to be raised due to raise in protection; others need to be raised due to landside fill required. The gatewell at Station 67+65 does not need any of the modifications it did for existing conditions. There are 8 gatewells that will not meet design criteria if the flood protection is raised. However, because material strengths were assumed since they were not indicated on the drawings, reinforcing strength could be verified by testing during design to determine if the structure can remain in service for future conditions.

For future conditions, 14 gatewells will need to be raised and 8 will require modification. Fiber Reinforced Polymer (FRP) overlay to increase strength is a recommendation for modification for these gatewells. For construction cost estimating purposes, the average gatewell height raise is 4-ft and average size is 9-ft by 7-ft exterior dimensions.

The gatewell at Station 124+83, the Turkey Creek sewer will also need to be raised. The raise for this gatewell will be a more substantial cost than the other average gatewell raises because of the nature of the machinery on the platform.

It is assumed that the gatewell at Station 167+95 is necessary and will need to be relocated for Alternative 2 because the existing stoplog gap will be replaced upstream of the existing location. The cost to replace this gatewell could be eliminated if it is determined that the gatewell is no longer used or is of insignificant use.

A-11.5.4 RCBs and Pipes Associated with Gatewells

The results for the conduits associated with gatewells can be found in Table A-11.6 with additional information for the box culverts (factors of safety) given in Table A-11.7. In addition, the following paragraph discusses the approach to cast iron pipes and estimating the associated work for future conditions. For more information on analysis of RCBs and pipes refer to the General Structural Analysis chapter and to the structural exhibits for examples.

Two main concerns exist for the existing cast iron pipes (CIPs). First, they may see additional load with a raise in levee elevation (not in the case of some floodwalls as described below). Secondly, CIPs are susceptible to corrosion. For this study, assumptions were made based on available data, but during formulation of plans and specs these pipes will require a detailed inspection and may require wall thickness measurements.

Without wall thickness, an analysis could not be performed to verify the ability of each pipe to withstand additional loading invoked by a levee raise. However, in locations where new floodwalls replaced existing levees, it was assumed that enough soil would be removed to offset the increase in water loading. In those cases, if a detailed inspection reveals an adequate condition, no additional work will be required.

In addition to strength issues, determining the pipe's condition is of utmost importance. CIPs are no longer used in levee projects because they are vulnerable to corrosion, and the CIPs under consideration were constructed during use of leadite joints which have

TABLE A-11.6
N500+3 Gatewell Conduits

CID-KS - Conduits Associated with Gatewells			
Station	Conduit Size & Type	Levee Raise (ft)	Action*
83+52 CID-MO	42" RCP		refer to pump station analysis
87+60 CID-MO	48" CIP	0	see CIP paragraph
10+81	60" PCCP & 48" DIP	0	no action - no raise here
19+90	66" RCP	0.2	no action - minimal raise here
50+98	10" CIP	0.9	see CIP paragraph
52+07	10" CIP	1.3	see CIP paragraph
56+91.5	10" CIP	1.5	see CIP paragraph
58+12	84" RCP Class IV & 3-30" pipes		refer to pump station analysis
62+78	12" CIP	2.1	see CIP paragraph
67+65	18" CIP	2.7	see CIP paragraph
71+70	18" CIP	2.8	see CIP paragraph
77+80	42" RCP	3.3	None - based on assumptions adequate for raise
80+90	24" CIP	3.6	see CIP paragraph
84+90	18" CIP	3.6	see CIP paragraph
88+19	24" CIP	3.4	see CIP paragraph
94+32	18" CIP	3.4	see CIP paragraph
98+05	24" CIP	3.2	see CIP paragraph
102+52	12" CIP	2.9	see CIP paragraph
106+49	6'X6' RCB	2.9	
124+83 (TC sewer)	DOUBLE 17'X18' RCB	3.3	
138+29	36" CIP	3.8	see CIP paragraph
152+28	24" CIP	3.7	see CIP paragraph
159+70	42" CIP	3.8	see CIP paragraph
167+95	10" VCP	3.9	None - floodwall raise - no additional fill

*Complete inspection is required for all pipes

had severe corrosion problems. Unless completely replaced, these pipes will require a detailed, high quality inspection.

Due to the lack of pipe conditions and wall thicknesses, cost for this study was a rough estimation. The estimation included:

- Inspection of all pipes.
- Measurement of thickness for all CIP pipes
- Replacement of 25% of the CIP pipes
- Slip liner for 25% of the CIP pipes

TABLE A-11.7
N500+3 RCB Analysis Results

CID-KS LEVEE UNIT - RCB N500+3										
Station	Size	Bending				Shear				Reliability
		FS	Load Case	Member	Location	FS	Load Case	Member	Location	
106+49	6'X6' RCB	refer to pump station analysis								
124+83 (TC sewer)	DOUBLE 17'X18' RCB	3.5	2	11, 13	Centerline	1.1	2	11, 13	Left End	92%

Note: The factor of safety calculated by CORTCUL is divided by the strength reduction factor

Load Case	C_V	C_H
1	1	0.47
2	1	0.64
3	1.5	0.50

A-11.5.5 Closure Structures

A summary of the closure structures required for the N500+3 event is given in Table A-11.8.

Table A-11.8
N500+3 Closure Structures

CID-KS - Closure Structures (N500+3)					
Station	Crossing	Existing Type	Raise Height (ft)	Future Type	Description
19+00	Lewis and Clark Viaduct (I-70)	none	less than 1	none	no modifications necessary
26+30	James Street Bridge	none	1	none	was a sandbag gap and then the bridge was raised - now there is no gap - no modifications necessary
40+75	KCS RR bridge (abandoned)	none	1	none	probably nothing required. Levee about 3 feet higher than floodwall protection; construct levee raise thru here if needed
57+25	Central Avenue Bridge	none	~1.5	new sand bag gap	grated deck so a sandbag gap is required - will need new construction
75+46	MoPAC RR bridges	none	~3	new sand bag gap	will need new construction
76+16	UP RR bridge	Sand Bag	~3	new stop log gap	will need new construction
104+51.50	CRI&P RR bridge	closed stop log gap	~3	new floodwall	eliminate closed stoplog gap
132+20	KC Terminal Bridge	stop log gap	~4	will probably need new stop log gap	4' raise is substantial and would probably want to replace this gap with a new one, but it may be possible to modify existing - thicker backwalls, new stoplog post, new stop logs
166+31	Upper End (RR tracks)	stop log gap	~4	new stop log gap upstream of existing gap structure	will need new construction

All closure structures were assumed to be sand bag or stoplog gaps. Should the sponsor request a rolling gate in lieu of the assumed gap configuration, it is assumed they will fund that difference in cost. The following are additional notes regard the future condition consideration for closure structures:

- Union Pacific Railroad bridge gap (76+16)
  - This gap is currently a sand bag gap approximately 3-ft in height. The raise in this vicinity is approximately 3-ft, pushing this gap out of the range of a viable sand bag gap. Therefore a new stoplog gap structure is proposed and is proposed to be offset to the east of the existing sand bag gap for ease of construction.

- CRI&P Railroad bridge (104+51.50)
  - This closure structure was filled in with concrete. Therefore it is recommended that this be replaced with the new floodwall without any accommodations for a gap.

- KC Terminal Bridge (132+20)
  - The existing closure structure is integral to the bridge abutment, however there are existing conditions reliability issues. Therefore in lieu of rehabilitating the existing closure structure for a raise on the order of 4-ft it is recommended to construct a new stoplog gap offset landward of the existing closure structure.

- Upper End (railroad tracks – 166+31)
  - The existing stop log gap structure would require extensive modifications for the raise, which is evident from the existing condition assessment. In addition, because disruption to rail traffic would be the same regardless of a rehab or a replacement, the recommendation for this closure structure is to replace the existing stoplog gap just upstream of the existing structure. To replace the closure structure to the upstream side of existing, additional floodwall past Station 166+25 will need to be constructed. It is also feasible to replace this closure structure to the downstream side of existing, but the tracks start to spread in the downstream direction which could result in gaps of excessive widths.

A-11.6 REFERENCES

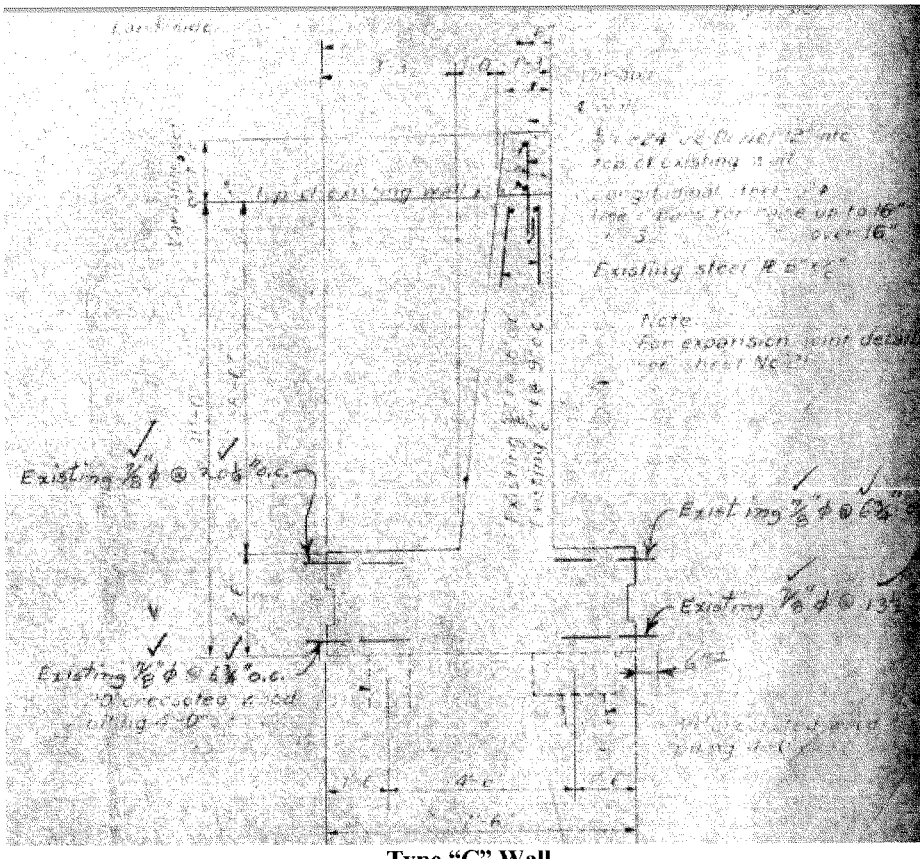
1. Record Drawings; Volume One; Operations and Maintenance Manual; Central Industrial District Unit – Kansas Section; Sheets dated Oct. 1950
2. Record Drawings; Volume Two; Operations and Maintenance Manual; Central Industrial District Unit – Kansas Section; Sheets dated Dec. 1979
3. CID Kansas Section; Analysis of Design to accompany Construction Plans; February 1946, Section 6
4. Investigation and Testing of Existing Flood Wall Piles, Pile Assessment Report, March 2011 by HNTB
5. 1954 Clarkson Construction shop drawings of Gatewell 83+52 (MO stationing).
6. May 1963 Treatment Plant Outfall, KC, KS plans of Gatewell (87+60 MO stationing).
7. City of KCK, Municipal Wastewater Treatment Plant #1 - Emergency Flow Diversion, 2/13/1990, plans detailing Gatewell at Station 10+81
8. City of KCK, Sewerage Improvements KS River Crossing Sewage Force Main, July 1965, plans detailing Gatewell at Station 19+90
9. Black and Veatch Contract 1 Storm Water Pumping Station Plans 1986, plans detailing Gatewell at Station 58+12
10. HNTB I-670 bridge deck drainage underground system plans, plans detailing Gatewell at Station 77+80
11. American Royal Arena Storm Sewer System plans by Shafer, Kline and Warren, September 1973, plans detailing Gatewell at Station 106+49

A-11.7 SUPPLEMENTAL EXHIBITS

EXHIBIT A-11.1

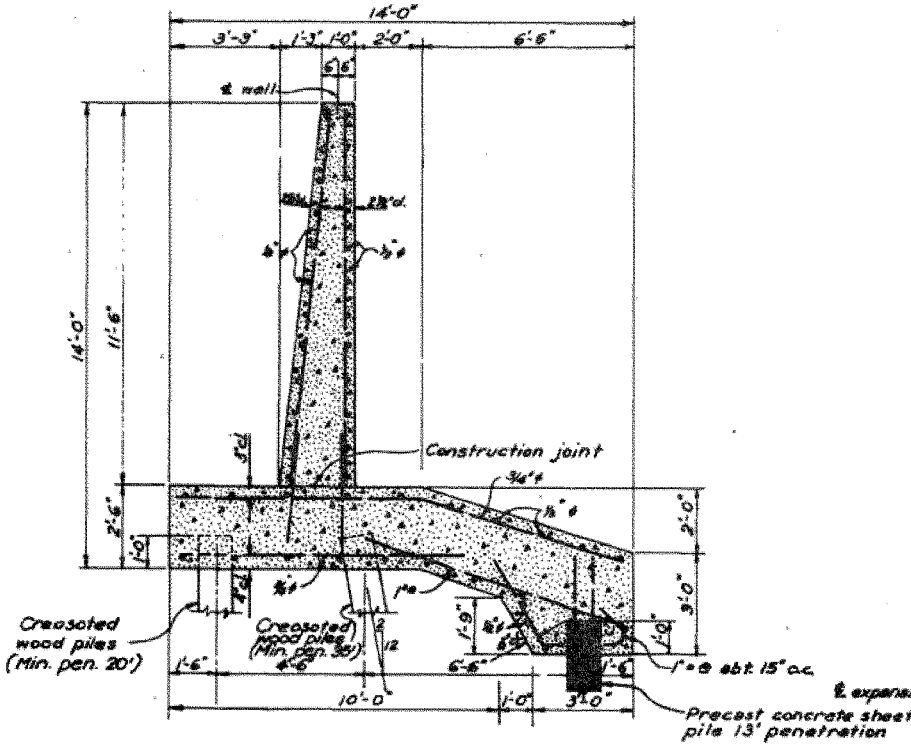
Floodwall Details – Station 26+72 to 40+31

CID-KS Sta. 27+86.16 to Sta. 36+64.17 Floodwall (1937)



Type "C" Wall

CID-KS Sta. 26+72.66 to 27+86.16 and
Sta. 36+64.17 to Sta. 40+31.25 Floodwall (1950)



SECTION A-A

TYPE "D" FLOODWALL

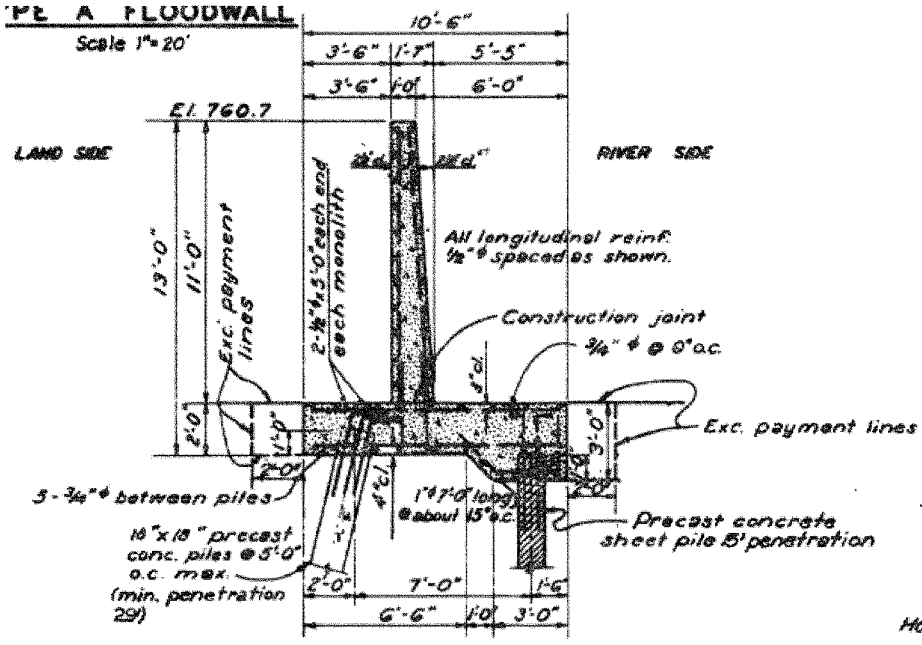
CID-KS 26+72 to 40+31 Floodwall (2007)

Looking Upstream

EXHIBIT A-11.2

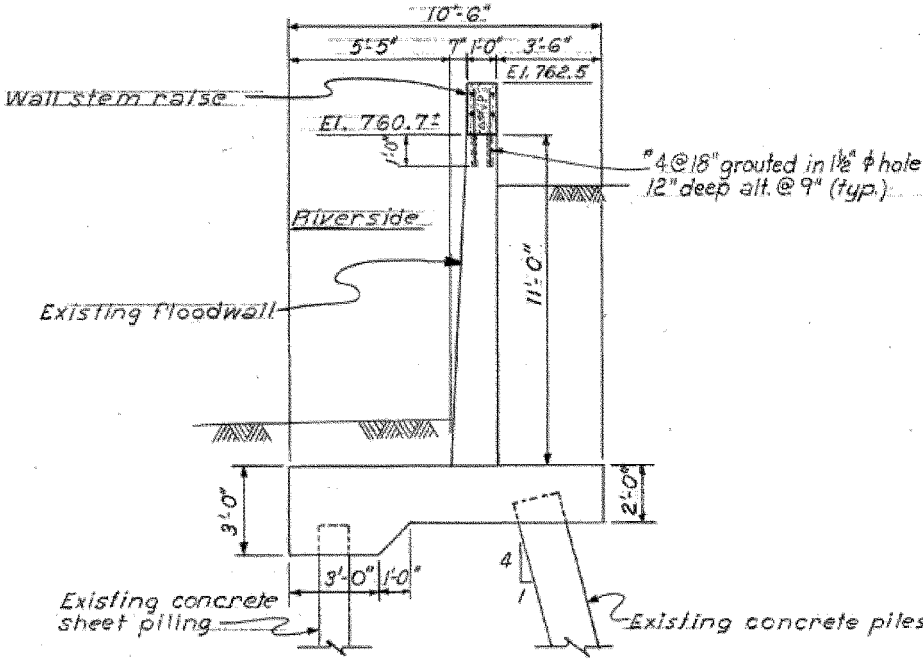
Floodwall Details – Station 74+36 to 77+28

CID-KS 74+36 to 77+28 Floodwall (1950)



TYPICAL SECTION

CID-KS 74+36 to 77+28 Floodwall (1979)



FLOODWALL SECTION

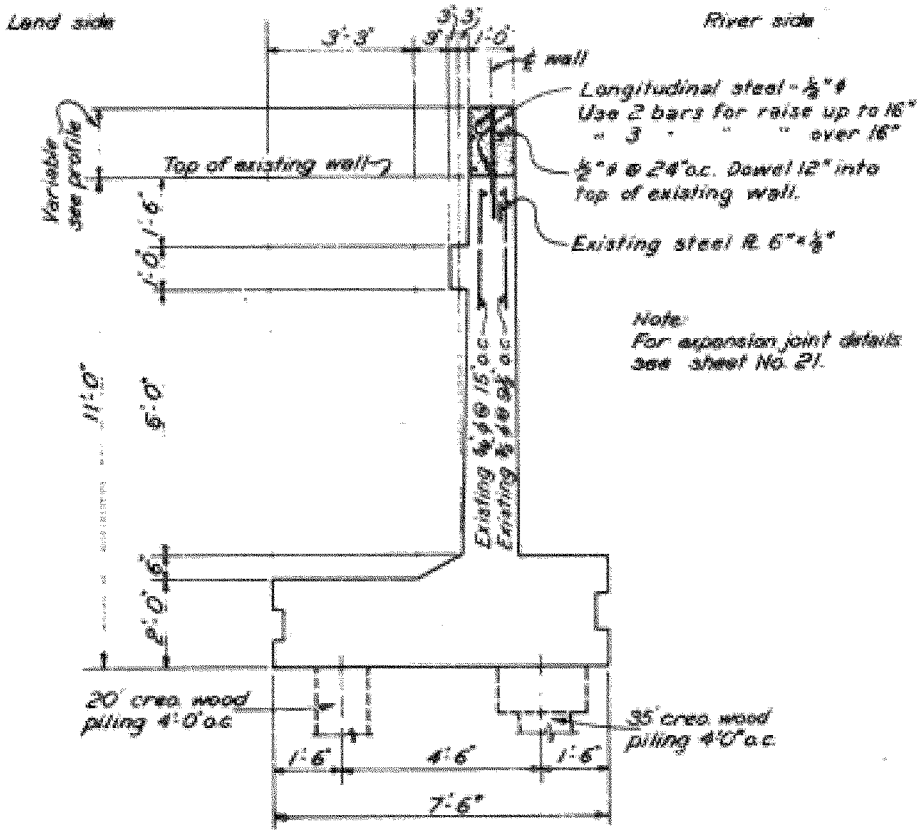
CID-KS 74+36 to 77+28 Floodwall (2007)

Looking Upstream

EXHIBIT A-11.3

Floodwall Details – Station 102+74 to 179+81

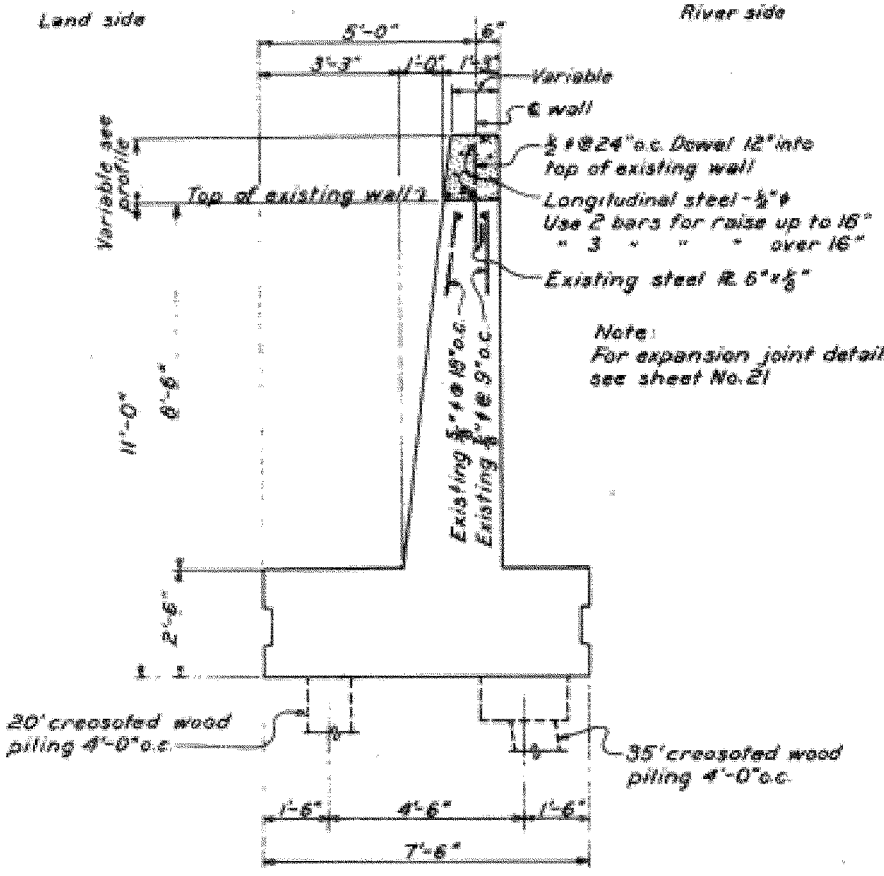
CID-KS 102+74 to 179+81 Floodwall (1950)



SECTION THRU EXISTING FLOOD WALL TYPE "B"

STA. 120+01.12 TO STA. 162+90.66 &
STA. 175+66.22 TO STA. 179+81.15

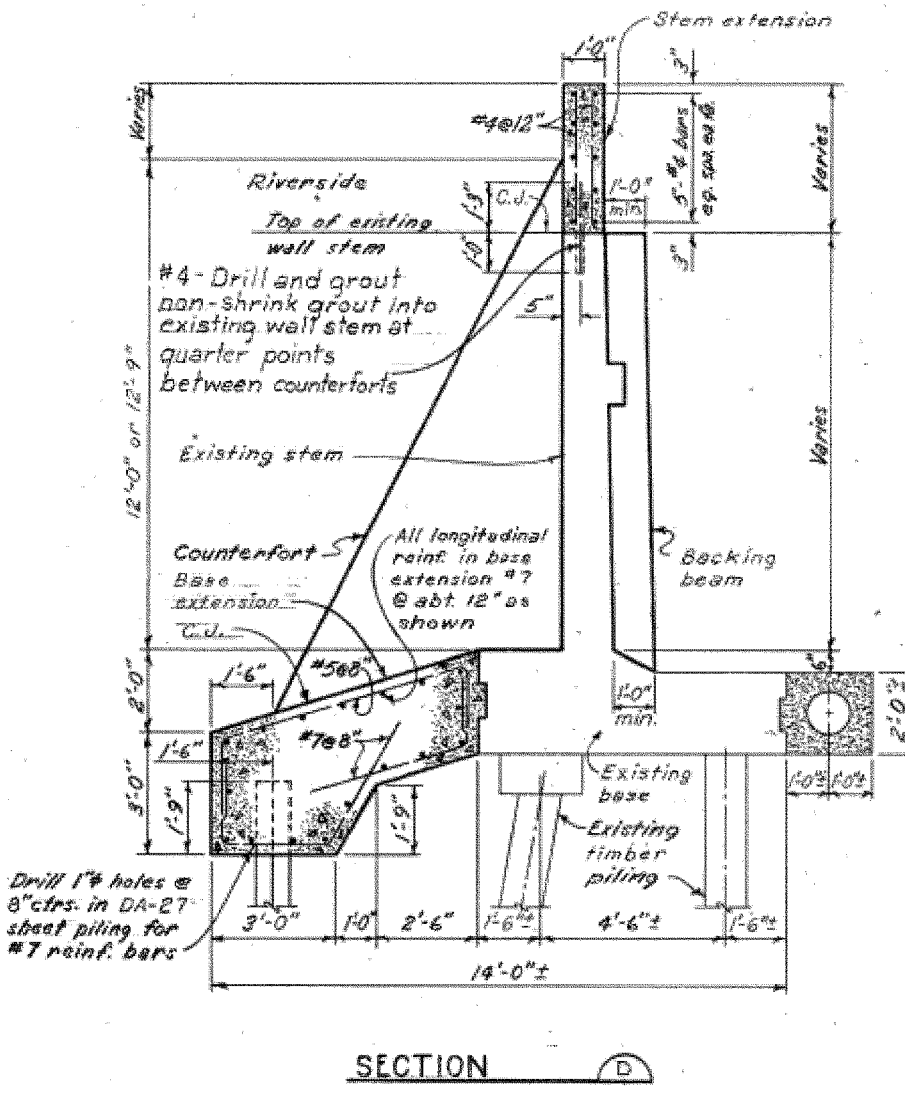
CID-KS 102+74 to 179+81 Floodwall (1950)



SECTION THRU EXISTING FLOOD WALL TYPE "C"

STA. 102+73.38 TO STA. 120+01.12 &
STA. 162+90.66 TO STA. 175+65.22

CID-KS 102+74 to 166+25 Floodwall (1979)



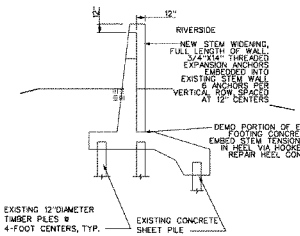
SECTION D

CID-KS 102+74 to 166+25 Floodwall (2007)

Looking Upstream

EXHIBIT A-11.4

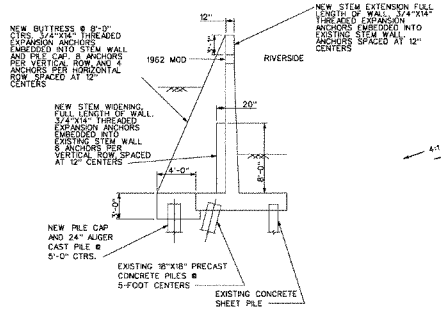
Proposed Typical Wall Sections



A TYPICAL MODIFIED FLOODWALL

NOT TO SCALE

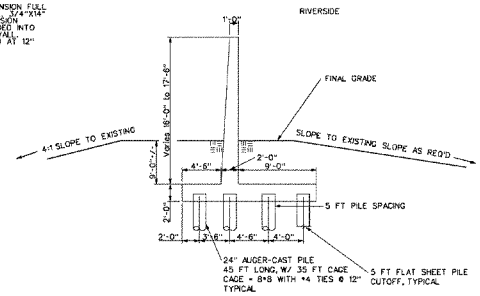
STATIONS: 28+72.66 TO 40+31.25



B MODIFIED FLOODWALL WITH NEW PILES

NOT TO SCALE

STATIONS: 74+35.94 TO 75+25



C TYPICAL NEW FLOODWALL: 16' TO 17'-6"

NOT TO SCALE

STATIONS: 75+76 TO 77+27.75

US Army Corps of Engineers
Kansas City District

Revisions	Date

Designed by K. MARK
Drawn by K. MARK
Checked by K. MARK
Reviewed by
Approved by
Submitted by B. STANSBURY

Date NOV 2009
File No.
Proj. Mgr.
Scale
CADD File Name 12-CAND_S1.DGN

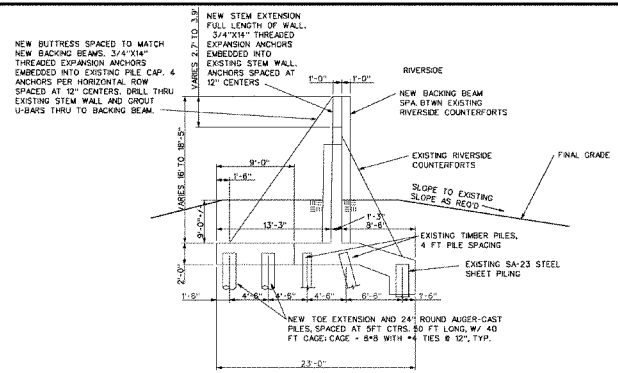
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

KANSAS CITY - KANSAS UNIT
CENTRAL INDUSTRIAL DISTRICT

FLOODWALL SECTIONS

Sheet Reference Number
S1

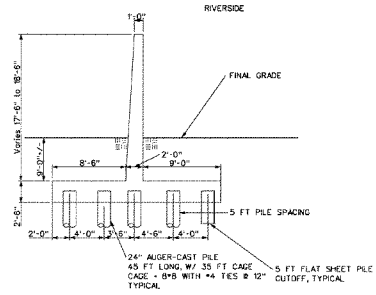
K:\MISSIONPROJECTS\CITY\KANSAS CITY\STRUCTURES\PHASE 2\CADD\MAPBOOK FILES



O TYPICAL MODIFIED FLOODWALL: 16' TO 18'-5"

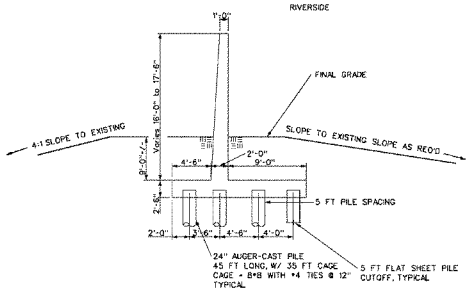
NOT TO SCALE

STATIONS: 102+73.38 TO 166+25



E TYPICAL TIEBACK WALL: 17'-6" TO 18'-5" IN HEIGHT

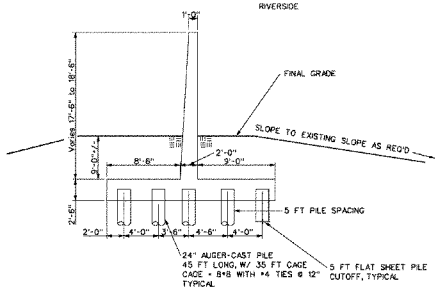
NOT TO SCALE



F TYPICAL NEW FLOODWALL: 16' TO 17'-6"

NOT TO SCALE

STATIONS: 102+73.38 TO 135+00



G TYPICAL NEW FLOODWALL: 17'-6" TO 18'-5"

NOT TO SCALE

STATIONS: 135+00 TO 166+25

US Army Corps of Engineers
Kansas City District

Date	Revision

Designed by: R. Mark	Date: MAY 2006
Drawn by: R. Mark	File:
Checked by: R. Mark	FWS Scale:
Approved by:	PM:
Submitted by: R. Schwabauer	CADD File Name: KC-CWIS-S2.DGN

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI

KANSAS CITY, KANSAS UNIT
CENTRAL INDUSTRIAL DISTRICT

FLOODWALL SECTIONS

Sheet Reference Number:
S2

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Chapter A-12

STRUCTURAL ANALYSIS
CID-MO

CHAPTER A-12
STRUCTURAL ANALYSIS CID-MO

A-12.1 OVERVIEW

This chapter addresses the structural features (floodwalls, gatewells, and closure structures) for the Central Industrial District - Missouri (CID-MO) Unit which includes existing conditions only because a raise is not being pursued at this time. The existing condition evaluation includes assigning reliability to the structures and recommending remedies if required. For general structural methodology refer to the General Structural Analysis chapter. Elevations reference the NGVD29 datum as information was obtained from record drawings unless otherwise stated.

A-12.2 ASSUMPTIONS

See the "General Structural Analysis" chapter for general KC Levees Structural Feasibility Assumptions. The following assumptions are in addition to those general assumptions, applying specifically to the CID-MO Unit.

1. To assess flexible behavior of a closure's pile cap and localized load effects on piles, a simplified torsional analysis is considered sufficient for the current scope and budget. A finite element model is required for more precision, and confidence in decisions regarding the reliability of the structures. A maximum width of 4-ft, centered about each abutment, was selected due to pile spacing, concentrated loads at the abutment, minimum horizontal steel, and the absence of torsion reinforcement. Using a 4-ft width assumes rotation so that torsion is minimized (note, torsion reliability, with respect to ¼ τ_{cr}, was less than 99.8% for water at top of the wall). The abutment width is only 2.5-ft wide, and placement of primary reinforcement was limited to within the abutment width. Considering sill heights, footer thicknesses, and that torsional reliability was near 99%; however, distribution of stresses beyond the 2.5-ft width is reasonable.
2. Due to smaller geotechnical strengths, only the long-term load case of the seemingly critical closure and floodwall was analyzed with consideration of pile strengths.
3. The footer's heel and toe maximum moments occur at the stem.
4. Loads are capable of distributing throughout the pile cap for a width of 4-ft, centered about the abutments. It is noted that the rebar detailing has primary steel only under abutment (typically 2-ft 6-in). USACE program CPGA was used to analyze the pile forces, which assumes a rigid cap.
5. The pile cap under the closure width is adequate for strength and in good condition.
6. For a consistent risk and economic comparison with other units' T-wall results, axial tension in the heel of T-walls was not analyzed, shear and flexure in the floodwall keys were not analyzed, and shear friction at construction joints with only residual rebar capacity not used to resist bending was not checked.

7. Heels of T-walls were analyzed without contribution of key forces producing a pure moment into the heel. This moment is usually small in comparison with heel forces. Also, the heel was analyzed for net vertical pressures only.
8. Based on closure structure analysis, the following checks were not performed on the floodwalls because they proved not to control for the closures, therefore likely not control for the floodwalls: tension pile embedment, punching shear of pile through pile-cap, vertical and horizontal bearing of pile and pile-cap, spread footer stability with short-term strengths, horizontal shear in pile-cap (at sheetpile) and horizontal shear in all piles.
9. For a consistent risk and economic comparison of gate well stability calculations with other units' results, a portion of soil skin friction is assumed aiding in the resistance of flotation. A crack is assumed in the top five feet of soil, which eliminates any contribution of the soil in resisting uplift.

A-12.3 SOIL MATERIAL PROPERTIES

The expected soil properties used for the CID-MO calculations are located in Table A-12.1. These values were used for general soil properties in the structural calculations. The allowable bearing capacity values were used in the structural calculations and were arrived at by dividing the ultimate bearing capacity from the table below by the factor of safety listed. Soil to structure friction and cohesion interaction was typically neglected for stability and strength calculations. However, for gatewells and spread footing founded floodwalls, this behavior was considered under geotechnical guidance.

TABLE A-12.1
CID-MO Soil Properties

	Embankment	Fill/Debris	Foundation Blanket	Foundation Sand
Friction Angle (drained shear strength)	29°	20°	22°	30°
Cohesion	-	-	-	-
Moist unit wt.	115 pcf	110 pcf	110 pcf	115 pcf
Saturated unit wt.	120 pcf	115 pcf	115 pcf	120 pcf
Wall friction coefficient (used in gatewell uplift)	0.25	0.25	0.25	0.25
Ultimate Bearing Capacity – compression (psf) FS=1.7 (Extreme Loading Condition)			3,400 (Station 0+00 to 7+00) 3,610 (Station 7+00 to 7+50) 3,663 (Station 7+50 to 10+25) 40,000 (Station 10+25 to 23+00)	

A-12.4 CID-MO EXISTING CONDITIONS

A-12.4.1 Floodwalls

Floodwalls on the CID-MO Unit consist of spread footing floodwalls and pile founded floodwalls. For each of these types of walls the existing condition was evaluated. The evaluation entailed analysis of existing reliability and recommending solutions for any existing condition deficiencies. The results can be seen in Table A-12.2 and Table A-12.3 for spread footing floodwalls and pile founded floodwalls, respectively. See the General Structural Analysis appendix chapter for further information on reliability analysis.

The existing CID-MO floodwalls are as follows:

- Station 0+00 to 22+31.71 spread footing floodwall constructed in 1947
- Station 22+81.46 to 78+12.22 pile foundation floodwall constructed in 1947

This does not include walls that are part of a closure structure (see Closure Structures section). The floodwall is interrupted in locations where the wall ties into bridge piers or roadway ramps. The floodwall ties into existing ground at Station 0+00 and ties into an earthen levee at Station 78+12.22.

The soil elevations both landside and riverside of the wall for the majority of the walls were field verified and where there were differences, the field noted elevations were used in the analysis.

Clarification to pile details on the drawings - The record drawing (A-10-825) shows only 18-in square piles with and without tapers and the precast concrete sheet pile section. The last revision date on that sheet is 6/23/1947. Another drawing that apparently preceded the record drawing shows 16-in and 18-in square piles without tapers and the precast concrete sheet pile section. All other drawings in the record drawings (as-builts) show that there are 16-in and 18-in piles that were used – including the driven pile lengths listed on record drawing A-10-1178. So the following assumptions were made for the feasibility analysis:

1. 16-in piles not tapered and as detailed
2. All 18-in piles tapered – looking at 21-ft long pile (each type wall has at least 1 monolith with the 21-ft long piles) (therefore used 15-inx15-in dimensions for properties –because of taper)
3. Used the precast sheet pile as detailed on the record drawings

The controlling probability of failure results for the spread footing walls is based on the controlling walls; "12-ft" wall for strength reliability (wall between Station 0+00 to 3+49.16) and "Especial" wall (wall between Station 20+23.77 and 20+65.77) for sliding reliability. The controlling probability of failure results for the pile footing walls is based on the wall "R" (wall between Station 22+81.46 to 24+54.76) which controlled for pile geotechnical compression capacity. The pile capacities changed very little as the water level lowered down the wall, so the probability of failure analysis assumed the same pile capacity at each location.

A-12.4.1.1 Floodwall Summary

TABLE A-12.2
CID-MO Existing Spread Footing Floodwall Conditions

CID-MO Floodwalls - Spread footing Floodwalls Existing Conditions with Water to Top of Wall							
Station	Wall Type	Factors of Safety					Comments
		OVERTURNING % Base in Compression (25% Req'd)	BEARING (2.0 Req'd)	SLIDING (1.3 Req'd)	STRENGTH Wall/ Heel/Toe (1.5 Req'd)	Mechanism	
0+00.00 to 3+49.16	12'	72%	3.2	1.1	0.9	Flexural Bottom Steel in Toe	86% reliability for strength calculated
4+09.81 to 7+03.97	11'	83%	3.3	1.0	1.6	Flexural Bottom Steel in Toe	
7+03.97 to 7+45.97	13'	100%	3.3	2.6	1.1	Flexural Top Steel in Heel	
7+45.97 to 8+29.97	15'	95%	2.8	1.4	1.1	Flexural Top Steel in Heel	
8+29.97 to 11+93.90	16'	68%	2.2	0.9	1.1	Flexural Top Steel in Heel	
11+93.90 to 13+61.90	L (Monolith 29)	90%	21.6	2.5	2.0	Flexural Bottom Steel in Toe	
13+61.90 to 14+45.90	M (Monolith 31)	100%	22.3	3.3	2.2	Flexural Bottom Steel in Toe	
15+19.77 to 16+87.77	J (Monolith 34)	77%	19.3	2.4	1.8	Flexural Bottom Steel in Toe	
16+87.77 to 18+13.77	H (Monolith 38)	80%	22.4	2.2	1.9	Flexural Bottom Steel in Toe	
18+13.77 to 18+97.77	G (Monolith 40)	61%	20.4	1.7	1.9	Flexural Bottom Steel in Toe	
18+97.77 to 19+39.77	F	33%	12.1	1.6	1.5	Flexural Bottom Steel in Toe	
19+39.77 to 20+23.77	F special	0%	0.0	0.4	2.2	Flexural Riverside Steel in Stem	
20+23.77 to 20+65.77	Especial	0%	0.0	0.0	2.0	Flexural Bottom Steel in Toe	0% reliability for sliding
20+65.77 to 22+31.71	E (Monolith 47)	0%	0.0	0.7	1.6	Flexural Bottom Steel in Toe	

TABLE A-12.3
CID-MO Existing Pile Footing Floodwall Conditions

CID-MO FLOODWALLS - Pile Foundation Floodwalls - Existing Conditions with Water to Top of Wall								
Wall Type	Factors of Safety							Comments
	STRENGTH Wall/ Pile Cap (1.5 Req'd)	Concrete Sheet Piling			Concrete Pile			
		STRENGTH (1.5 Req'd)	Geotech allowable Axial Compression and Tension loads (1.0 required)		STRENGTH (1.5 Req'd)	Geotech allowable Axial Compression and Tension loads (1.0 required)		
R	1.4	5	9.1	tension	11.4	0.94	compression	96% reliability calculated for concrete pile geotechnical compression capacity
Q	2.0	2.2	2.7	tension	4.0	1.23	compression	
Q-Monolith 54	1.2	2.4	1.2	tension	4.3	0.90	compression	
P	2.1	7	12.4	tension	7.4	1.54	compression	
N	2.2	2.2	3.4	tension	5.1	1.01	compression	
S	1.0	3	14.7	tension	19.4	1.46	compression	93% reliability calculated for heel strength

A-12.4.2 Floodwall Structures Remediation

As a result of the findings noted in the floodwall existing conditions analysis, repair is recommended for approximately 292-ft of flood wall (between Station 19+39.77 and 22+31.71). A preliminary recommended repair may be to:

1. Reconstruct the heel key by demolishing the end of the heel at the existing unreinforced key
2. Extending the heel key approximately 1-ft deeper than the existing key depth (total key depth of 2-ft) and reinforcing the key for E Special and F special walls
3. Extending the heel approximately 2-ft riverward and constructing a new reinforced key approximately 4-ft.

This repair will allow the key to be considered for sliding resistance for the stability of the floodwall.

The 12-ft spread footing floodwall has a reliability of 69%. The analysis was based upon the maximum exposed stem height based upon field verified measurements. Much of the 12-ft wall has much less exposed stem since it is near the end of the protection where the

groundline tapers up to the top of the wall (see record drawings). That coupled with a reasonably high reliability when water is to the top of the wall and the fact that once water is 1-ft down from the top of the wall, reliability is 98%, results in low risk of failure of the existing 12-ft spread footing floodwall. Similar results can be expected at the other walls with a low strength factor of safety. Some additional measures may be needed to address sliding in the area from Station 0+00 to 11+93, but they are expected to be minimal in nature.

A-12.4.3 Drainage Structures

The structural section of the Engineering Appendix for the Kansas City, Missouri and Kansas City, Kansas Flood Protection Project contains an evaluation of the existing gatewells' abilities to meet N500+0 flood event. The three CID-MO gatewells and one manhole were analyzed with water to top of levee (worst case) only to determine their reliability for strength and flotation. The results of this analysis are shown in Table A-12.4.

TABLE A-12.4
CID-MO Existing Gatewell Conditions

Existing Conditions with Water to Top of Protection HGL's - from Geotech May 2011					
Station	Flotation Factor of Safety (>1.1 Req'd)	Strength Factor of Safety (>1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability (%)	Comment
14+97 (GW)	N/A	1.4	Wall Bending	99.86	The bottom of GW is the floodwall toe, therefore uplift is not analyzed. The floodwall stability is analyzed separately.
14+99 (MH)	1.9	16.1	Base Slab Moment	N/A	MH is 38.5' upstream of the gatewell, Line of Creep Method used for uplift pressures.
24+76.9	1.3	2.0	Wall Bending	N/A	HGL change. Gatewell is attached to Broadway pump station. Uplift not a concern.
41+84.9	1.3	2.9	Base Slab Moment	N/A	No Change from previous HGL estimate. Gatewell attached to floodwall, uplift not a concern.
52+85	N/A	N/A	N/A	N/A	Gatewell removed and rebuilt in Santa Fe Pump Station (Sheet #7, A-10 -1617) Gatewell should be analyzed in unison with the Pump station.

The three gatewells and one manhole along the CID-MO Unit were analyzed for flotation and strength requirements to determine if the gatewells could withstand water seepage pressures (HGL's) with water to top of flood walls. Results are summarized in Table A-12.4. All values are based on the N500+0 or water to top of levee (worst case).

The gatewell at Station 14+97 showed a 1.4 factor of safety for wall flexure, slightly less than the minimum requirement of 1.5. A probability of failure (POF) analysis was conducted which resulted in the wall being 99.86% reliable for strength. Flotation was not analyzed for this gatewell because the bottom of the gatewell is the toe of the floodwall. The floodwall stability was analyzed separately.

A-12.4.4 Closure Structures

There were seven closure structures analyzed for the feasibility study:

1. M10 (Station 5+23.95)
2. M18 (Station 8+67.90)
3. M32 (Station 14+80.09)
4. M145 (Station 63+15.28)
5. M 158 (Station 68+90.14)
6. M162 (Station 70+70.66)
7. Levee closure (Station 80+18.62)

The Factor of Safety (FS) results from the deterministic analysis of key structural components are summarized in the Table A-12.5. Pertaining to pile-founded closures (M145, M158, M162, and the Levee Closure), considering a simplified approach to qualify torsion, the geotechnical pullout strength of the tension piles (precast sheetpile) was critical. Reliability is 36% with water elevation at Tow of Wall (TOW); 85% with water elevation at 1-ft below TOW; and 99.5% with water elevation at 2-ft below TOW. With water elevation below 2-ft from TOW, the reliability is considered 99.8%. M145 was the most critical of all the pile-founded monoliths, and its results were used to establish POF plots versus water elevation for all closures.

Since the closures were not found 99.8% reliable, without adjustment for structure degradation, a review in original design assumptions was performed. Like the present analysis, it seems that the original designers included water level to top of the wall. Unlike the present analysis, the original design apparently only used undrained geotechnical strengths for the pile capacities (whereas the feasibility study included lower drained strengths), considered the closure's pile caps as rigid (whereas torsion was qualified in the feasibility study), but very conservatively ignored any efficiency of the cutoff wall to mitigate seepage and uplift pressures (whereas the cutoff wall efficiency was assumed 50% in the feasibility study).

Table A-12.5
CID-MO Closure Summary of Results for Existing Conditions - Summary of Probable Failure Mode Results[1]

Monolith (FS[2])	Heel Trav Moment at Stem	Heel Trav Shear (stem or cut pile)	Pile Cap Heel Long Moment at Crit Pile (Pos or Neg)[3]	Stem Moment at Base[4]	Stem Shear at base	Toe Trav Moment at Stem[5]	Toe Trav Shear (stem or pile)	Tension Pile's flexor development (ctd) Pile Cap	Max Compressive Pile Punching Shear	Critical Pile Bearing (HCC Cap Loaded Vert or Horz)	Cutoff Pile Horz Shear through Pile Cap	Cutoff Pile Geotechnical Pullout	Cutoff Pile Combined Axial and Moment	Shear through Cutoff Pile	Max Compr Pile Geotech Bearing Capacity	Max Compr Pile Combined Axial and Moment	Shear through Square Pile	Spreadfooter Key Moment	Spreadfooter Key Shear	Guide Post Moment at Top of Footer	Guide Post Shear at Top of Footer	Guide Post Moment at Sill EL	Guide Post Shear at Sill EL	Pothwall Strength Analysis[6]	Interior Post Strength (shear or moment)	Stop logs Moment	Stop logs Shear	Stop logs bearing	Sill Strength (shear or moment)
M10	2.01	8.13	NA	2.67	2.89	1.76	6.01	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	2.55	6.28	1.70	2.51	2.30	5.33	NA	NA	CMLevee	CM32	18.20	4.96
M18	1.62	6.32	NA	2.51	1.50	1.59	5.94	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	3.10	5.77	2.48	1.55	7.80	4.70	NA	NA	CMLevee	CM32	18.20	2.11
M32	3.16	5.45	NA	2.78	2.15	1.36	4.36	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	2.00	1.94	1.60	2.88	NA	NA	CMLevee	4.00	18.20	4.20
M145	1.09	1.39	2.25	2.50	1.55	3.11	1.77	5.75	2.80	13.35	1.57	0.96	1.07	1.22	2.23	1.90	1.79	NA	NA	2.30	2.59	NA	NA	NA	NA	CMLevee	CM32	18.20	3.13
M158	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.48	1.57	3.30	2.75	NA	NA	CMLevee	CM32	18.20	3.50
M162	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.90	1.67	6.90	4.56	1.67	1.98	CMLevee	CM32	18.20	2.99
Mlevee	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	CM145	NA	NA	2.10	2.54	5.10	5.94	NA	NA	1.81	CM32	18.20	4.86

Pile Cap Torsion Assessment[7]

Monolith	FS	Reliability (at Water EL) TOW	TOW-1ft
M145	0.95	79.7%	99.8%
M158	1.05	98.9%	99.8%
M162	0.94	75.2%	99.8%
Mlevee	1.37	99.8%	

Spread Footer Torsion Assessment[7]

Monolith	FS	Reliability (at Water EL) TOW
M10	1.4	99.8%
M18	3.0	CM10
M32	3.6	CM10

Spread Footing Stability Results (Existing Condition with Water to Top of Wall)[8]:

Monolith	FS Sliding	Req FS	% Base Compressed	Req %	FS Bearing	Req FS
M10 (drained Strengths)	1.35	1.3	96%	25%	3.2	2
M10 (undrained Strengths)	5.89	1.3	79%	25%	3.2	2
M18 (drained Strengths)	2.84	1.3	100%	25%	2.62	2
M18 (undrained Strengths)	18.4	1.3	100%	25%	2.69	2
M32 (drained Strengths)	1.59	1.3	100%	25%	21.5	2
M32 (undrained Strenghts were judged from inspection to not control)						

Risk and Reliability Results:

The single most critical component of the gaps (lowest FS and highest COV) was the geotechnical pullout of the cutoff wall piles.

Cutoff Wall Pullout Reliability Results (based upon M145 only)

	TOW[10]	TOW-1ft	TOW-2ft	TOW-3ft
M145	36.4%	84.8%	99.5%	99.8%

Notes:

1 Some failure mechanisms are judged to be critical on one or two closures. For these mechanisms, a designation of CMxxx is used to refer the reader to the results of the critical monolith. Analysis was not performed for non-critical mechanisms.
2 FS reported are for water EL at Top of Wall. For Factors of Safety greater than 1.5, a reliability of 99.8% is assigned.
3 FS and Reliability is computed with respect to a capacity equal to 1/4 of cracking Torsion.
4 Longitudinal Shear in pile cap across center row of piles is adequate by inspection (by comparing to capacity in primary load direction).
5 The stem's Temperature and Shrinkage reinforcement is only 74% of current USACE criteria.
6 A 2D analysis in CTWALL was performed for results. Rotation in Plan due to eccentric loading on M18 and M32 (Z shape), was found to posit a minor stress on neighboring walls should shear resistance be exceeded by footer.
7 Strength FS for spread footer gaps is critical of drained and undrained results.
8 The stem at M32 has an abrupt change in cross-sectional area. Flexure was checked at this height and a FS was found to be 2.2.
9 Considers critical result from flexure, shear resistance from shear friction and concrete, and bearing.
10 TOW elevation for M145 is 759.9 ft.

Setting aside remediation measures that may be borne out of the results from the feasibility analysis, the following recommendations are made in light of the site visit and analysis:

1. Most closures need maintenance at the abutments, including cleaning and painting embedded steel.

2. If possible, inspect pile caps for cracks and corrosion of rebar, and repair if necessary.

3. Inspect timber stop logs and repair/replace if needed.

4. Seal cracks and joints with appropriate sealant for exposed conditions and movement of concrete.

5. Should a finite element model, or other more detailed analysis, be performed, it is recommended that a 5% to 15% reduction in strength capacity be applied given the observations made during the site visit. Site visit observations can be found in the appendix with closure structure calculations.

Additional structural feasibility analysis information and summary data can be found in Exhibit A-12.1 in the Supplemental Exhibits section at the end of this chapter. The exhibit is an excerpt of the complete analysis; therefore, additional information can be obtained upon request.

A-12.5 RCBS AND PIPES ASSOCIATED WITH GATEWELLS

There was no analysis done on the CID-MO pipes associated with the gatewells because there is no future conditions analysis (raise) for this unit. Existing pipes are assumed to be functioning at current state. Inspections and maintenance will be required for upkeep to system. The levee sponsor is responsible for maintaining all structures per the O&M (Operation and Maintenance) Manual.

A-12.6 REFERENCES

1. CID-MO Plans for Construction of Broadway and Santa Fe Pumping Plants (October 1947)
2. Record Drawings; Appendix I Record Drawings; Operations and Maintenance Manual; Kansas Citys Flood Control Project; Missouri and Kansas Rivers; Central Industrial Unit – Missouri Section; Sheet dates vary
3. Computations; Central Industrial District – MO Section Design Computations, dated 1946.

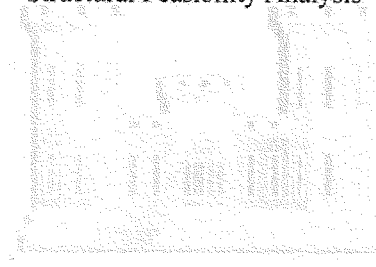
A-12.7 SUPPLEMENTAL EXHIBITS

EXHIBIT A-12.1

CID-MO Closures Structural Feasibility Analysis

KC Levees
CID MO Closures

Structural Feasibility Analysis



Prepared By:

Frederick R. Sheffield, P.E.

US Army Corps of Engineers—Kansas City District

August 8, 2008

REVISED AUG 2011

Order of Appearance

	Title	Page
1.	Summary	3
2.	Site Visit	8
3.	Pile Cap Analysis	12
4.	Spread Footer Analysis	176
5.	Load and Capacity Analysis for other Structural Components	280
6.	Geotechnical Referenced Data	379
7.	Incomplete GTSTRUDL Analysis	401

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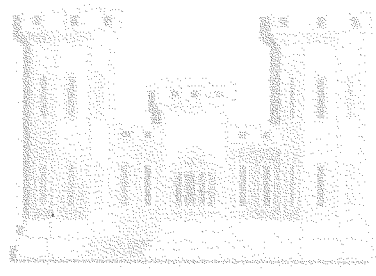
SUMMARY



4A1c /437

KC Levees Feasibility Study

CID MO – Closure Analysis Summary

This document is a summary of findings, assumptions, and recommendations for the KC Levees, CID MO Closures.

The work contained herein is to provide Feasibility Analysis results only; it does not replace a deterministic analysis, or answer the questions that only a deterministic analysis can. The closures were analyzed, without factors of safety and with consistent assumptions, in order to evaluate the relative risk and consequences for economic and risk-informed decision-making purposes. Risk and Reliability studies do not replace deterministic analyses, nor do such studies confirm the satisfaction of any design criteria, past or present. They simply provide additional information for the decision-maker with respect to the possible performance of the structure for the loads under consideration. This provides a risk-informed decision with respect to project repairs or improvements.

Assumptions to Analysis.

The following lists major assumptions in the feasibility study.

1. The closures were analyzed based on dimensions, quantities, and conditions represented by record drawings. Deviations from plans cannot be verified per scope and budget. This is a consistent assumption for relative risk and reliability assessment for KC Levees feasibility study. A site visit was conducted, but its findings are limited to observable deficiencies above ground. In general, the condition of the closures is fair. Some significant cracking, separation, and rotation have been observed which may impact the closure's ability to perform in a peak flood.
2. The risk analysis reflects a structure in the condition as shown on plans, or during an inspection, at the time of the extreme (water to top of flood protection) flood. Secondary and incremental load effects are not considered for the feasibility analysis. Permanent deformation or damages from any less than extreme floods that occur prior to the extreme flood are expected to have been repaired.
3. Construction practices are considered good, and all specifications noted on plans satisfied.
4. Materials, such as reinforcing bar, are in good condition.
5. Soil profiles are represented accurately by drawings. Soil is adequately compacted and fill type and strength parameters supplied by Geotechnical Section is correct.
6. Concrete compression zone is without voids.
7. Some structural components were not analyzed. Only components judged to be critical based upon engineering experience were analyzed for feasibility. Components that were not analyzed include, but may not be limited to, minimum rebar embedment lengths, structure capacity at rebar cutoff locations, various cut-off wall efficiencies, and various soil resistances.

8. Cut-off walls are 50% efficient, and are of adequate strength and good condition. The upstream face of structural wedges was analyzed with a Line of Creep, reduction beginning at top of ground. This is the default method in CTWALL.
9. The Sand Bag Closure (STA 1+53.14) requires no Structural Analysis.
10. To assess flexible behavior of a closure's pile cap and localized load effects on piles, a simplified torsional analysis is considered sufficient for the current scope and budget. A finite element model is required for more precision, and confidence in decisions regarding the reliability of the structures. A maximum width of 4 ft, centered about each abutment, was selected due to pile spacing, concentrated loads at the abutment, minimum horizontal steel, and the absence of torsion reinforcement. Using a 4 ft width assumes rotation so that torsion is minimized (note, torsion reliability, with respect to ¼ τ_{cr}, was less than 99.8% for water at top of the wall). The abutment width is only 2.5 ft wide, and placement of primary reinforcement was limited to within the abutment width. Considering sill heights, footer thicknesses, and that torsional reliability was near 99%; however, distribution of stresses beyond the 2.5 ft width is reasonable.
11. For Risk and Reliability analysis, the strength of structural materials has an expected value of 125% of their design strength, based upon FEMA 310. Structural materials are assumed to have a 14% standard deviation. Geotechnical materials are assumed to have an 18% standard deviation.
12. Due to smaller geotechnical strengths, only the long-term load case of the seemingly critical closure was analyzed with consideration of pile strengths.
13. The footer's heel and toe maximum moments occur at the stem.
14. Loads are capable of distributing throughout the pilecap for a width of 4 ft, centered about the abutments. It is noted that the rebar detailing has primary steel only under abutment (typically 2 ft 6 in). USACE program CPGA was used to analyze the pile forces, which assumes a rigid cap.
15. The pile cap under the closure width is adequate for strength and in good condition.

Results of Risk & Reliability.

The method of analysis begins with a deterministic analysis (without any load or resistant factors) of critical components to establish an expected Factor of Safety (FS). A Reliability analysis was performed for the critical structural components to establish a probability of failure (POF). The critical structural components were chosen based upon lowest FS and largest COV. Sometimes more than one component can control for development of a POF curve at different water surface elevations. The Reliability was determined based upon the "Reliability Index," β, which assumes a lognormal distribution, and relative to FS of 1.0. For structural components whose factors of safety met or exceeded 1.5, or for geotechnical components whose FS met or exceeded 1.7, Reliability was assigned 99.8%.

The FS results from the deterministic analysis of key structural components are summarized in the attached spreadsheet. Results of any Reliability Analyses are summarized below:

4 C/K/457

Sheetpile Pullout. Pertaining to pile-founded closures (M145, M158, M162, and the Levee Closure), considering a simplified torsional analysis, the geotechnical pullout strength of the tension piles (sheetpile) was critical. Reliability is 36% with water elevation at Tow of Wall (TOW); 85% with water elevation at 1' below TOW; and 99.5% with water elevation at 2' below TOW. With water elevation below 2 ft from TOW, the reliability is considered 99.8%. M145 was the most critical of all the pile-founded monoliths, and its results were used to establish POF plots vs water elevation.

Comparison with Original Design.

Since the closures were not found 99% reliable, without adjustment for structure degradation, a comparison in design assumptions was performed. Like the present analysis, it seems that the original designers included water level to top of the wall. Unlike the present analysis, the original designers apparently only used undrained geotechnical strengths for the pile capacities (whereas the feasibility study included lower drained strengths), considered the closure's pile caps as rigid (whereas torsion was qualified in the feasibility study), but very conservatively ignored any efficiency of the cutoff wall to mitigate seepage and uplift pressures (whereas the cutoff wall efficiency was assumed 50% in the feasibility study). A few pages of original design comps were copied and attached for reference.

Recommendations.

1. Most closures need maintenance at the abutments, including cleaning and painting embedded steel.
2. If possible, inspect pile caps for cracks and corrosion of rebar.
3. Inspect timber stop logs and repair/replace if needed.
4. Seal cracks and joints with appropriate sealant for exposed conditions and movement of concrete.
5. Should a finite element analysis be performed, it is recommended that a 5% to 15% reduction in strength capacity be applied given the observations made during the site visit.

By Frederick R. Sheffield, P.E.
08 August 2008 (REV SEP 2011)
✓ KSM 11/16/11

(ORIGINAL CALCS)

Control Approval Symbol KE-6 5A/D/

CORPS OF ENGINEERS - U. S. ARMY
Office of the District Engineer
Section Kansas City 8, Missouri

Sheet 1 of 3
By JHT date 1-24-46
Ckd. RRD date 1-29-46

Return to Struc. Design

COMPUTATIONS

Project CID Mo. Design for General - 12'-13'-14'-15'-16' Pile Brg Floodw

Design 12', 13', 14', 15' & 16' pile bearing flood walls with a concrete sheet pile cut-off wall at the heel and a drain at the toe.

Assume:

- Earth scoured away to top of sheet pile on river side
- Sat. Earth on land side
- No horizontal load from earth on land side
- Uplift: A straight line reduction in uplift from full head at the sheet pile cut-off wall to the static head in the toe drain at the toe, with no percolation loss thru the sheet piles
- Four conditions of loading have been considered on all walls.
  1. Water to top of wall - uplift - moments in piles due to fixity in base included
  2. Same as 1 but piles not considered fixed in base hence no moments from piles
  3. Water to top of wall - no uplift - moments in piles due to fixity in base included
  4. Same as 3 but piles not considered fixed in base hence no moments from piles.

In addition the 15' wall was checked for the above cases with R.R. surcharge

Control Approval Symbol KE-6

ORIGINAL CALCS

5B/

CORPS OF ENGINEERS - U. S. ARMY Sheet 2 of 3

urn to Office of the District Engineer By JHT date 1-29-46

uc. Design Section Kansas City 8, Missouri Ckd. CCD date 1-29-46

COMPUTATIONS

ject C.I.D. Mo. Design for General - Pile Bearing Floodwalls

and for the cases of RR. surcharge - no water on riverside of wall and consequently no uplift - with and - without moments in base from footn piles. These additional cases of loading showed that case 2 above with surcharge was the critical case for the steel in the top of the base at the riverside of the stem but that the increase in steel reqd. by this case over that required in the 4 cases of loading for all the walls was only 2.5 %. For this reason RR. surcharge was not included in the design loadings.

- Conc. sheet piles between consecutive bearing piles are considered to be equivalent to one bearing pile for vertical load carrying capacities.
- Horizontal thrust of water is resisted by bearing piles and sheet piles, the load being divided between them in proportion to their stiffnesses.
- Where moments and deflections in the footn. piles have been included in the computations they have been taken from data (attached).

ORIGINAL CALCS

Control Approval Symbol KE-6 5C/

CORPS OF ENGINEERS - U. S. ARMY Sheet 3 of 3
Return to Office of the District Engineer By HT date 1-24-46
Attn: Design Section Kansas City 8, Missouri Ckd. _____ date _____

COMPUTATIONS

Project C.I.D. Mo. Design for General Pile Bearing Floodwalls

Compiled by HAN. from formulas presented by Cummings in A.S.C.E. Transactions of 1937.

— Allowable loads.

Vertical loads on piling.

16" □ Conc. pile — 25 Tons.

18" □ " " — 30 "

10" Conc. Sh. pile — 6 " per lin ft.

10" " " — " uplift per lin. ft.

Horizontal loads

16" □ Conc. pile — 5 Tons.

18" □ " " — 5 Tons.

10" " Sh. pile — 2 Tons Lin. Ft.

— Design stresses

Steel — 18000#/□" Tension.

Conc. 1050#/□" Compr.

n = 10.

V = 60#/□" (No anchorage of long. reinf).

V = 90#/□" (Spec. " " " "

(ORIGINAL CALCS) Control Approval Symbol KE-6 5D/

CORPS OF ENGINEERS - U. S. ARMY Sheet 2 of ____
Office of the District Engineer By RRD date 1-22-46
Design Section Kansas City 8, Missouri Ckd. JPC date 4-1-46

COMPUTATIONS

Project C.I.D. Mo. Design for 15' FLOOD WALL - FULL UPLIFT - NO SURCHARGE

	↓	↑	→	←	ARM	↻	↺
Earth Forces							
Concrete 150 x 3 x 1	450				11.5	5200	
" 150 x .5 x 1	80				9.67	800	
" 150 x 13 x 2	3900				6.5	25400	
" 150 x 1 x 13	1950				4.0	7800	
" 150 x .33 x 13	650				4.72	3100	
Water 62.5 x 7.83 x 13	6360				9.08	57700	
62.5 x .33 x 13	270				4.85	1300	
Earth 125 x 3.5 x 2	880				1.75	1500	
	14540				7.09	102800	
Lift		8880					68400
ΣV	5660					33400	
Hor. Forces							
Water 1000 x 8				8000	4.33		34600
Uplift 864 x 1			860		0.5		400
" 68 x .5			30		0.67		
" 250 x 2			500		1.33	700	
Piles ① & ②			3400		0.90		
Pile ③			3210		1.00		3200
ΣH			0				37500

Defl. per 3.0K load 18" conc. brg. pile = .005556 x 3 = .016668"
" " 1.0K " 10' " Sh. " = .017637"

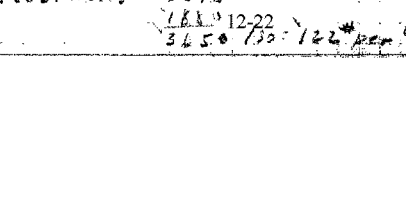
Hor. Shear in Brg. Piles = $\frac{6610 \times .017637}{.017637 + .016668}$ = 3400 51.44 %

" " " Sh. " = $\frac{6610 \times .016668}{0.034305}$ = 3210 48.56 %

Moment in Brg. " = 4865 x 3.4 = 16500'#
" " Sh. " = 3111 x 3.21 = 10000'# / 26500'#

Stability $a = \frac{33400 - 37500 - 26500}{5660} = -5.41'$ = Dist. outside the Resultant cuts Base.

Pile Loads



cg = (2+6+11.5)/3 = 6.5 from Toe
Σd² = 4.5² + 0.5² + 5² = 45.5

① $\frac{5660}{3}\left(1 + \frac{3 \times 11.91}{45.5} \times 4.5\right)$ = 8550# per L.F. or 51300# per pile (3.53)

② 1887(1 + .78527 x 0.5) = 2620# " " (.39)

③ 1887(1 - .78527 x 5.0) = -5530 (3.93)

1887 / 12.22 = 154# per '

Case I

Comp. By F.R.S. Date ____ Project ____ CID MO Flood Protection ____ Sheet 7 of 437
Chkd. By KSM Date 9/2011 Subject Summary of Closures ____ Sheet ____ of ____

	Closure Type	Wall STA Begin	Wall STA End	Closes	Width (ft)	Height above ground	Monolith	On Piles?	Comments
1	Sand Bag	1+38.14	1+68.14	Pavement	30	4' 6"	NA	No	Requires No Structural Analysis
2	Stop Log	5+11.95	5+35.97	Pavement	24.02	6' 7"	10	No	
3	Stop Log	8+57.90	8+77.90	Railroad	22.5	8' 7 3/4"	18	No	"Z" Monolith
4	Stop Log	14+70.09	14+90.09	Railroad	22.5	11' 11"	32	No	"Z" Monolith
5	Stop Log	63+05.28	63+25.28	Railroad	22.5	11' 0"	145	Yes	"Z" Monolith
6	Stop Log	68+80.14	69+00.14	Railroad	22.5	9' 9 1/2"	158	Yes	"Z" Monolith
7	Stop Log	70+53.14	70+88.14	Street	35	9' 1 1/2"	162	Yes	Has Center Guide Post
8	Stop Log	80+04.12	80+33.14	Street	29.04	6' 10 1/2"	NA	Yes	Through Levee

	Strength Checks							
	Guide Posts Meet FS?	Center Post Meet FS?	Postwell Meet FS?	Stoplogs Meet FS?	Can torsion be ignored? ACI 11.6.1	Key Meet FS?	Sill Meet FS?	12" Wide Sill Meet FS?
M10	Y	NA	NA	Y	Y	Y	Y	NA
M18	Y	NA	NA	Y	Y	Y	Y	Y
M32	Y	NA	NA	Y	Y	NA	Y	Y
M145	Y	NA	NA	Y	NO	NA	NA	Y
M158	Y	NA	NA	Y	NO	NA	Y	Y
M162	Y	Y	Y	Y	NO	NA	Y	NA
Levee	Y	NA	NA	Y	Y	NA	Y	NA

	Stability		Bearing
	Sliding FS	OT Base in Compression	Meet FS?
M10	1.35	79%	Y
M18	2.84	100%	Y
M32	1.59	100%	Y
M145	Stability and other strength parameters for pile founded		
M158	closures were performed for M145 Abutment (3' pile cap		
M162	width). A POF was performed for sheetpile pullout.		
Levee			

Notes:

Width is measured from out to out of abutments. Each abutment is 2' 6" wide, except for Levee Stop Log, width = 2' 6 1/4", and Monolith 10, width DS = 2' 0" and width US = 2' 0 1/4"

STA is measured from outside of abutments, or to CL of PI at Intersections.

"Z" Monolith refers to the Z shape the monolith makes in plan.

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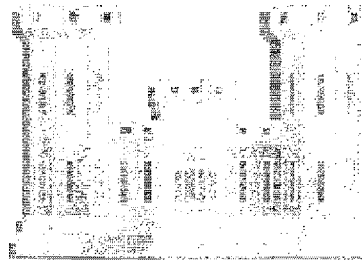
SITE VISIT



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Subject: 05 May 2008 Site Visit to CID MO Closures
Prepared By: Frederick Sheffield
Date: 06 May 2008

The following notes were made:

M145. Height above ground = 10'-7".
The North Abutment appeared to have some mechanical damage. One inch deep gashes were observed. The N. Abutment was skew to the floodwall, as if partially rotated about its primary axis. The maximum relative deflection from the abutment and the wall was about 4 ½" at the top. The base of the abutment was not deflected away from the wall. The top 12" of the abutment appears to have been repaired in the past.
The South Abutment has minor cracking from mid-height and down. Minor map cracking perceived at base. Embedded steel rusting and pitting observed at base.

M32. Height above ground = 11"-10". Stoplogs were placed; possibly abandoned closure.
Minor horizontal and vertical cracks observed at abutments, measuring 0.01" to 0.016" in width. There is a possible hollow spot near the ground elevation of N. Abutment.

M18. Height above ground = 8'-2"; N. Abutment has 11'-2" exposure on side of abutment (possible exposure on protected side). Stoplogs were placed; possibly abandoned closure.
Vertical cracks observed at abutments, measuring 0.013" wide. N. Abutment observed leaning toward river (base is flush with floodwall at base, top deflection approximately 6" to 7"). Direction of lean is towards abutment side with 11'-2" exposed height.

M10. Height above ground = 5'-7'. Clear width between abutments = 20'. Stoplogs were placed; possibly abandoned closure.
Top of abutments had "C" shape crack that measured 0.06" wide. It seems possible that these cracks could allow for degradation of vertical reinforcement. Each abutment had noticeable spalls on floodside. Vertical cracking typical of both abutments, each face. Embedded steel is rusting at base for first 18" from sill.

M158. Height above ground = 9'-1". Gated for vehicular use.
N. Abutment has Horizontal rebar visible and rusting.
Many horizontal and vertical cracks visible. Several patches (approx 2" dia) on each abutment.

M162. Height above ground = 8'-9". Gated for vehicular use.
E. Abutment has cracking on protected side. Metal trap for embedded steel is visible and rusting.
W. Abutment has fewer cracks than E. Abutment. 4 grout patches (approx 6" dia) on flood side. Embedded steel at base is delaminating from rust.

MLevee. Height above ground = 5'-11 ½"

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Levee slopes on wall and abutment on floodside (increasing load), more so than landside.
W. Abutment is separated and tilting away from wall – 2" at base, 3" at top. The gap between the abutment and wall has been filled at one time with grout, but the grout is mostly disintegrated and missing.
Reference M10 "C" shape cracking at abutment. No crack in plan view of abutments noticed at this monolith. The top of the abutments at other closures could not be viewed for inspection of cracking.

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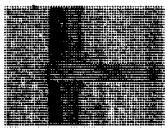
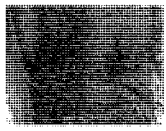
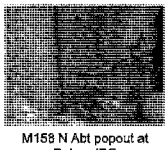

M130 REL DEF.JPG

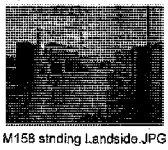

M145 Emb STL Rust at Base.JPG

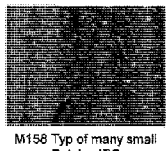
M145 N Abt REL DEF.JPG

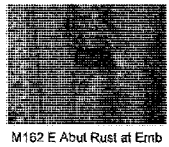
M145 S Abt Crcking typ Flood side.JPG

M145 stnding dry side.JPG

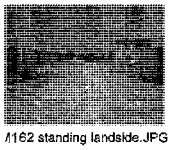

M158 N Abt popout at Rebar.JPG

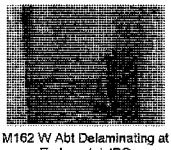

M158 stnding Landside.JPG

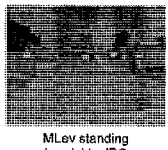

M158 Typ of many small Patchs.JPG


M162 E Abut Rust at Emb metal 2.JPG

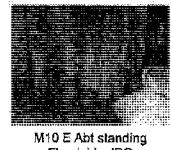
M162 E Abut Rust at Emb metal.JPG

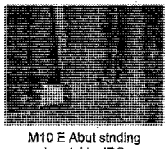

M162 standing landside.JPG

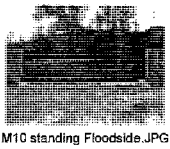

M162 W Abt Delaminating at Emb metal.JPG

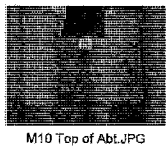

MLev standing Landside.JPG

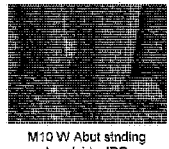
MLev W Abt leaning away from wall.JPG


M10 E Abt standing Floodside.JPG


M10 E Abut stnding Landside.JPG


M10 standing Floodside.JPG


M10 Top of Abt.JPG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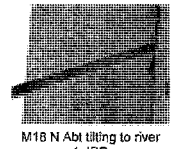
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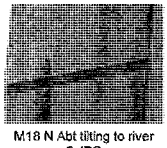
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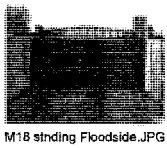
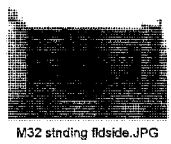

M18 stnding Floodside.JPG

M32 Possible delamination N Abt.JPG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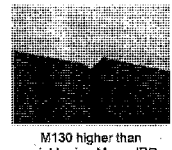

M130 higher than neighboring Mono.JPG

Chapter A-13

PUMP STATION ANALYSIS ARMOURDALE

CHAPTER A-13
PUMP STATION ANALYSIS - ARMOURDALE

A-13.1 SOURCES OF INFORMATION

Corps staff visually inspected every pump station studied, including outlet works when accessible. Printed information such as pump curves or as-built drawings was unavailable or limited for several of the facilities. Table A-13.1 summarizes whether the information was available or not.

TABLE A-13.1
Summary of Pump Station Available Information

Name	Station	As-Builts?	Pump Curves?
Osage (KCK #14)	76+83	Yes	Yes
12th St (KCK #13)	129+20	Partial	No
Mill St (KCK #12)	156+75	Yes	No
5th St (KCK #11)	185+70	Yes	No
MW Cold Storage	194+60	No	No
Shawnee Ave (KCK #10)	230+78	Yes	Yes
KCS RR	276+79	Yes	Yes
PBI Gordon	286+59	No	Yes
National Beef (KCK #8)	295+52	No	No
Central Ave	299+21	Yes	Yes

A-13.2 SUPPORTING INFORMATION AND CALCULATIONS

Supporting information for the pump station analysis is presented following the pump station write-up:

Exhibit A-13.1 – Pump Station Information and Photographs
Exhibit A-13.2 – Pump Station Calculations
Exhibit A-13.3 – Geotechnical Data (Hydraulic Grade Line Calculations)

Structural analysis calculations are available upon request.

A-13.3 METHODS OF ANALYSIS

Pump stations were evaluated both for structural reliability and, in the case of stations handling seepage flow from relief wells discharging below grade, hydraulic capacity. Detailed analyses were conducted for existing conditions in an effort to define the risk of failure under current project conditions. Detailed analyses were also conducted for the N500+3 design loading to determine the required modifications to accommodate the higher loading associated with a prospective levee or floodwall raise, consistent with the recommended levee raise across the Kansas River on the Armourdale Unit (Phase 1). Two other design conditions, N500+0 and N500+5 were also considered, but primarily

by engineering judgment rather than detailed analysis. The purposed of the two additional design points was to better define where the various design conditions fall on the net benefits curve, relative to the NED plan.

Structural analysis considered both strength and uplift (also called "stability" or "flotation" throughout the report) with water to the top of the existing line of protection. These analyses were applied to the pump station structures as well as outlet works (pipes, RCBs and gatewells). In some cases, as-built drawings and specifications were available, few assumptions had to be made, and the analysis was rather clear. In other cases, however, details of the original construction were not available and assumptions had to be made based on standard practice at the time of construction (sometimes as early as pre-1914) and previous findings on similar structures. Conclusions were drawn based on those assumptions, which were in some cases tagged for verification during PED. Initial strength and uplift analyses were based upon geotechnical parameters applied across the levee unit and hydraulic grade lines (HGLs) with water to the top of the existing line of protection. Initial calculations neglected the affects of skin friction, water that would be present in the pump station under normal operating conditions, and the affect of existing relief wells (where present) to lower HGLs. When such analysis showed "marginal" results, the analysis was refined, considering all of the available forces (i.e. skin friction, wells, water weight) to resist failure. In those cases where pump stations have separate gatewells within the line of protection, the gatewell analysis is presented in the General Structural Analysis chapter of the feasibility report appendix (along with drainage structures not associated with pump plants).

Strength analysis entailed examination of walls and slabs for shear and bending moment capacity. If the resulting FS for flexure and shear was 1.5 or greater, the member was considered to perform to 99.8% reliability when exposed to the existing condition event loading. If the resulting FS was less than 1.5, a reliability analysis was conducted.

Uplift analysis entailed examination of structures for flotation with water to the top of the existing line of protection. Several stations had subcomponents such as gatewells and discharge chambers that were constructed at different times. In those cases, the uplift analysis considered the subcomponents separately, and the recommended modification considered the ability of the subcomponents to be tied together to transfer forces. If the resulting FS was 1.1 or greater, the structure was considered to perform reliably.

Screening level hydrologic analyses were conducted at the beginning of the feasibility phase for interior drainage. Records and drainage studies by others were reviewed, and local Sponsors consulted. Since the focus of this study is an existing system and there were no interior drainage problems identified or reported by Sponsors, more detailed interior flood hydrology studies were not undertaken. It is also worth noting that there are several interior storm drainage systems that handle upland and lowland flow, ultimately draining to pump stations. In some cases, the total flow generated by the storm drainage system exceeds the capacity of the lowland system, including the pump station. In other words, if the drainage systems were improved such that the flow could travel unimpeded, the pump stations would be overloaded. Research and discussion with

the Sponsor indicates that this situation has existed for many years, and no improvements are planned. Therefore, no pump station improvements were considered based solely upon interior flood hydrology.

Several pump stations in the system, however, handle seepage flow from well systems (relief wells / header systems discharging below grade). If this flow is not pumped, the factor of safety of the line of protection will be compromised. In these cases, existing condition analysis verified the relief well flow/coincident storm and pump station capacity to handle the flow, with the required redundancy. Alternatives analysis was tied closely to geotechnical requirements for underseepage control (i.e. berms, cutoff trench, or relief wells). Care was taken to ensure that alternatives calling for additional relief wells considered all resultant costs, including pumping costs.

A-13.4 FINDINGS AND RECOMMENDATIONS

The analysis of the pump stations along the Armourdale line of protection resulted in the findings and recommendations below. Existing condition results and N500+3 design condition recommendations are also presented in Exhibits A-13.4 and A-13.5. The results are summarized in Table A-13.2 and Table A-13.3 following Section A-13.6. Required modifications to Osage, 12th St Sewer, Mill St, 5th St, KCS RR, National Beef and Central Ave are cost shared as part of the proposed Federal project. Required modifications to private pump stations with no formal easement (Midwest Cold Storage, PBI Gordon) are the responsibility of the owners.

A-13.4.1 Osage Pump Station (KCK #14)

The Osage Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). The Sponsor reported no problems with the hydraulic capacity of this pump plant. Hydraulic analyses were conducted for the purposes of evaluating alternatives involving additional relief well flow discharging below grade. These alternatives included a replacement pump station, adding a new pump station, and including a slurry cutoff wall. The hydraulic calculations are included for all these alternatives in Exhibit A-13.2, for reference and comparison. The final recommended levee section in this area is the slurry cutoff wall, negating the need for relief wells and consideration of handling additional flows at this via a replacement or additional pump station.

The structural evaluation indicates that the Osage Pump Station is marginally acceptable for flotation with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface hydrostatic pressure is required to alleviate flotation issues. A slurry cutoff wall is proposed in this area as part of the recommended N500+3 design and provides sufficient reduction in hydrostatic pressure for the pump station to meet flotation criteria. Strength of the pump station is adequate, both for existing conditions and for levee raises up to and including N500+3, given the planned slurry cutoff wall.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the

gatewell is within the pump station; not a separate structure in the levee crest). Strength of the discharge pipe was found to be marginal under existing conditions, and therefore inadequate for additional loading. N500+3 design conditions require minor work to address erosion in the invert, plus a structural liner (i.e. CIPP or steel slip liner). For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The Osage Pump Station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required. The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

N500+0 design conditions would include a cutoff wall to address underseepage concerns in the area, alleviating any flotation issues. No structural modifications of the pump plant structure would be required. Strength of the discharge pipe would be inadequate and still require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new N500+0 level of protection.

N500+5 design conditions would include a cutoff wall to address underseepage concerns in the area, alleviating any flotation issues. No structural modifications of the pump plant structure would be required. Strength of the discharge pipe would be inadequate and still require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new N500+0 level of protection.

A-13.4.2 12th Street Pump Station

The 12th St Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the 12th St Pump Station is unacceptable for both flotation and strength with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface pressure is required to alleviate flotation issues. Structural modifications are required to meet strength criteria. Calculations show that this pump station may be strengthened with a brace/strut system, and additional weight added at key locations to meet criteria for strength and flotation. To ensure adequate transfer of uplift forces, the gatewell may require anchoring to the pump station.

The existing pump discharge configuration is twin ductile iron pump discharge lines "up and over" the levee, discharging into a gatewell on the riverside levee crest. Strength of

the pump discharge lines was not evaluated due to the "up and over" configuration. The existing gravity discharge is a reinforced concrete box. Strength of the RCB is adequate, both for existing conditions and for the N500+3 design condition. Strength and stability of the gatewell is adequate, both for existing conditions and for the N500+3 design condition. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Raising the gatewell will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of a new stem. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The 12th Street pump station was designed and constructed prior to Corps of Engineers involvement in the flood control project. Though drawings were limited, the recommended remedy entails extensive structural re-work, including extensive brace and strut systems and addition of weight to achieve an adequate factor of safety against uplift forces. While such testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of relative uplift forces acting on the pump station structural components.

The recommended section in this area is a T-wall on existing levee, resulting in 3 to 5-ft of additional fill against the existing pump station structure. Preliminary consideration indicates that the pump station can safely withstand the increased lateral earth load. A short retaining wall may be considered during PED if required.

N500+0 design conditions would result in loadings exceeding the existing condition. Since strength and uplift were unacceptable for existing conditions, major modifications similar to those required for a N500+3 raise would also be required for the N500+0 condition. Strength of the existing RCB is acceptable; no modifications would be required. Strength and uplift for the gatewell are acceptable for the N500+0 condition, however the top of the gatewell would need to be raised to coincide with the new N500+0 level of protection. The twin "up and over" discharge lines would need to be relocated up and over the new, higher level of protection.

N500+5 design conditions would result in loadings in excess of what the existing structure and the separate gatewell could withstand with any level of modification. Complete replacement of the pump station and gatewell would be required. Though the strength of the existing RCB is acceptable at the N500+5 condition, the required new pump station would likely be reconfigured in conformance with current recommended design practices (i.e. integral with the line of protection), and with a completely different discharge configuration. Therefore, it is likely that the existing RCB would also be replaced.

A-13.4.3 Mill Street Pump Station (KCK #12)

The Mill St. Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the Mill St. Pump Station is unacceptable for both flotation and strength with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface pressure is required to alleviate flotation issues. Structural modifications are required to meet strength criteria. Calculations show that this pump station may be strengthened with a brace / strut system, and additional weight added at key locations to meet criteria for strength and flotation. To ensure adequate transfer of uplift forces, the gatewell may require anchoring to the pump station.

The existing pump discharge configuration is twin ductile iron pump discharge lines "up and over" the levee, discharging into a gatewell on the riverside levee crest. Strength of the pump discharge lines was not evaluated due to the "up and over" configuration. The existing gravity discharge is a reinforced concrete box. Strength of the RCB is adequate, both for existing conditions and for the N500+3 design condition. Strength and stability of the gatewell is adequate, both for existing conditions and for the N500+3 design condition. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Raising the gatewell will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem. One pump will also need to be upgraded to maintain original design discharges, but at the new, higher head that will result from any proposed levee raise. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection.

The pump station receives inflow from a brick sewer. There is no information regarding the age of the brick sewer, construction documents, or inspection documents. A flotation or strength analysis was not performed on the brick sewer. Inspection of the sewer by local entities to determine the condition of the sewer is recommended.

The Mill Street Pump Station was designed and constructed prior to Corps of Engineers involvement in the flood control project. Though drawings were limited, the recommended remedy entails extensive structural re-work, including extensive brace and strut systems and addition of weight to achieve an adequate factor of safety against uplift forces. While such testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of relative uplift forces acting on the pump station structural components.

The recommended section in this area is a T-wall on existing levee, resulting in 3 to 5-ft of additional feel against the existing pump station structure. Preliminary consideration indicates that the pump station can safely withstand the increased lateral earth load. A short retaining wall may be considered during PED if required.

N500+0 design conditions would result in loadings exceeding the existing condition. Since strength and uplift were unacceptable for existing conditions, major modifications similar to those required for a N500+3 raise would also be required for the N500+0 condition. Strength of the existing RCB is acceptable; no modifications would be

required. Strength and uplift for the gatewell are acceptable for the N500+0 condition, however the top of the gatewell would need to be raised to coincide with the new N500+0 level of protection. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection.

N500+5 design conditions would result in loadings in excess of what the existing structure and the separate gatewell could withstand with any level of modification. Complete replacement of the pump station and gatewell would be required. Though the strength of the existing RCB is acceptable at the N500+5 condition, the required new pump station would likely be reconfigured in conformance with current recommended design practices (i.e. integral with the line of protection), and with a completely different discharge configuration. Therefore, it is likely that the existing RCB would also be replaced.

A-13.4.4 5th Street Pump Station

The 5th St Pump Station handles interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade). Structural evaluations indicate that the 5th Street Pump Station is unacceptable for flotation, both for water to the top of the existing levee and for the N500+3 design condition. For a line of protection raise, control of or resistance to subsurface hydrostatic pressure is required to alleviate flotation issues. Features such as a slurry cutoff wall or relief wells may be used, or structural modifications such as base slab modifications or work anchorage may be considered. Rock anchorage is considered to be a more feasible alternative, given the large base slab extensions that would be required to stabilize the structure against uplift forces. The rock anchorage alternative may require micropiles if the rock anchors prompt settlement below the structure. Strength of the pump station is acceptable for existing conditions. Minor modifications to the base slab are required for the N500+3 design condition.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). The discharge line was not accessible for inspection. As-built drawings show the discharge line to be a reinforced concrete pressure pipe, however the type, class, bedding and associated load bearing capacity are unknown. Based on information supplied by Price Brothers, it is assumed that the pressure pipe is AWWA C302. This assumption is based upon records of pipe shipments to the area during construction of the 5th St. Pump Station. For purposes of adequately capturing potential costs, a structural liner is considered for feasibility-level analysis. Further investigations will be conducted during PED to more accurately determine the load carrying capacity of the pipe. Further inspection consists of performing an annual visual inspection of the pipe. This annual inspection should occur over a consecutive ten year period. This is necessary to monitor how the pipe behaves after addition of the levee fill material. If pipe distress is found, a structural liner should be installed. Any line of protection raise will require raising the top of the discharge chamber to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The as-built drawings show the pump station inlet pipe as being constructed of the same material as the aforementioned outlet pipe. The recommendations made for the outlet pipe also apply to the inlet pipe.

The 5th Street Pump Station was designed and constructed by local interests to Corps criteria. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended, except as noted above for the outlet and inlet pipes.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

N500+0 design conditions would result in loadings exceeding the existing condition. Since uplift was unacceptable for existing conditions, modifications similar to those required for a N500+3 raise would also be required for the N500+0 condition. Strength of the existing outlet pipe may be acceptable, but would need to be verified during PED since engineers were unable to determine the exact type of pipe used. The top of the discharge chamber would need to be raised to coincide with the new N500+0 level of protection.

N500+5 design conditions would also result in loadings exceeding the existing condition. Since uplift was unacceptable for existing conditions, modifications similar to those required for a N500+3 raise but designed for higher loading would also be required for the N500+5 condition. Strength of the structure was unacceptable for the floor slab under N500+3 conditions, requiring minor modification, and will be unacceptable under the higher loading condition of N500+5. Wall strengths were marginally acceptable for N500+3. Strength of the existing outlet pipe may be acceptable, but would need to be verified during PED since engineers were unable to determine the exact type of pipe used. The top of the discharge chamber would need to be raised to coincide with the new N500+5 level of protection. Since strength of the main pump station structure under less stringent N500+3 conditions is either inadequate (base slab) or marginal (walls), since the uplift problem is severe even under existing conditions, and since solving uplift problems requires anchorage to the structure (intimately related to strength), one may conclude that this pump plant and related outlet works would require complete replacement for N500+5 loading.

A-13.4.5 Midwest Cold Storage

The Midwest Storage Pump Station is a small privately owned sump and abandoned mechanical equipment located inside the Midwest Cold Storage warehouse, adjacent to the levee. The pump station was previously used to pump out condensate which accumulated as a result of the refrigeration equipment. The refrigeration equipment was upgraded some time ago, and the pump station was not used since. Preliminary plans considered the abandonment of the sump in place; however the proposed floodwall section in this area (which includes a line of relief wells) addresses uplift concerns at this location, negating the need for further analysis.

N500+0 and N500+5 design conditions would likely result in a floodwall section with wells in this area, and similar recommendations regarding Midwest Cold Storage (i.e. no modifications required – proposed section addresses potential uplift concerns).

A-13.4.6 Shawnee Avenue Pump Station (KCK #10)

The Shawnee Avenue Pump Station handles both interior drainage and relief well flows discharging below grade, through a collector system to the pump plant. *Under current conditions, pumping of the relief well flow is required during high river stage events to maintain the integrity of the line of protection along the relief well system.*

Hydraulic analysis for the pump station under existing conditions with water to the top of the existing line of protection (i.e. maximum relief well flow of 35cfs with 15.5cfs small coincident storm) shows excess capacity.

Under higher head conditions of N500+3, the existing line of wells will produce approximately 47cfs. Additional surface discharging relief wells producing a total of 40cfs are also recommended for the N500+3 design condition. Of this 40cfs which would be generated at maximum flow conditions (water to top of N500+3 levee), roughly half of the flow, or 20cfs, would be discharged within the service area of the Shawnee Avenue Pump Plant. The coincident storm produces 28cfs, so the total flow that must be pumped in order to maintain the below-grade discharge relief wells (thereby maintaining the factor of safety) is 47+20+28=95cfs. The pump station capacity at maximum N500+3 river stage ranges from 95 to 55 cfs, depending on water surface elevation at the intake, and is therefore inadequate. There is ample room within the existing pump station structure to install larger pumps to accommodate the N500+3 design flows.

The structural evaluation indicates that the Shawnee Avenue Pump Station is adequate for strength and flotation with water to the top of the existing line of protection, as well as for levee raises up to and including N500+3.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). Strength of the discharge RCB was found to be marginal under existing conditions, and therefore inadequate for additional loading. N500+3 design conditions require a structural liner. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

The inlet pipe consists of a 96" diameter RCP. The inlet pipe, upstream junction box and pump station design are shown on the same record drawings. The class of RCP used is not specified in the drawings. Given that pipe bedding conditions tend to improve over time, the project team believes this segment of pipe is adequate for the N500+3 raise. It is recommended, however, that local interests perform a visual inspection of the pipe

prior to construction and annually for the following ten years upon completion of the project.

The Shawnee Avenue Pump Station was designed and constructed by local interests to Corps criteria. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

The recommended section in this area is a T-wall on existing levee. Since the Shawnee Avenue pump plant is essentially constructed integral to the line of protection, the T-Wall would be constructed around the pump plant.

The pump station structure is adequate for the N500+0 and N500+5 design conditions. No structural modifications of the pump plant structure would be required. Strength of the discharge RCB would be inadequate for either condition would require a structural liner. The top of the discharge chamber would need to be raised to coincide with the new N500+0 or N500+5 level of protection.

A-13.4.7 KC Southern Railroad Pump Station

The KC Southern RR Pump Station handles both interior drainage and relief well flow which discharges at grade and flows overland to the pump station. Under existing conditions, the pump station may not be operated (i.e. ponded water from the relief well discharge may not be pumped) when the river is rising and passes a certain stage in an effort to add weight, thus maintaining the factor of safety for the line of protection in this area.

The N500+3 recommendation for this area entails filling of the "slot", the low lying area to which the relief wells flow, and abandonment of the wells. Although the pump station capacity is very low and will no longer be required for seepage flows, the pump station is recommended to remain in place to handle the remaining interior drainage flow.

The structural evaluation indicates that the KCS RR Pump Station is adequate for strength and flotation with water to the top of the existing line of protection, as well as for levee raises up to and including N500+3.

The existing discharge configuration is a chamber within the pump station that receives pumped flow, pushing it through the gravity discharge line and to the river (i.e. the gatewell is within the pump station; not a separate structure in the levee crest). The discharge line is cast iron, reportedly with leadite joints. Although detailed inspections of the discharge line were not possible, leadite has a history of poor performance, and unprotected ferrous pipe would likely experience corrosion. For these reasons, the discharge line replacement of the discharge line is recommended for any raise. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the new line of protection. Raising the discharge chamber will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of new stem.

According to available drawings, the pump station receives inflow from a 24" diameter corrugated metal pipe (CMP). A flotation or strength analysis was not performed on the CMP. Inspection by local entities to determine the condition of the CMP is recommended.

The KCS RR Pump Station was designed and constructed by the Corps as part of the original Federal project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not recommended.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

The pump station structure is adequate for both N500+0 and N500+5 design conditions. No structural modifications of the pump plant structure would be required. The discharge pipe would be inadequate for either condition would require replacement. The top of the discharge chamber would need to be raised to coincide with the new N500+0 or N500+5 level of protection.

A-13.4.8 PBI Gordon

PBI Gordon is a private business that routes all of its parking lot runoff to a small pump station, which then pumps directly to a gravity drainage pipe riverward of a gatewell structure, where it drains to the river. The sluice gate in this gatewell is permanently closed, allowing all runoff to be routed to the sump and sampled prior to pumping to the river.

The pump station is inadequate for flotation with water to the top of the existing line of protection, as well as for any raise. Based on the available information, the PBI Gordon pump plant can remain in place and perform reliably for the N500+3 event by adding weight inside the structure. To ensure adequate transfer of the uplift loads, anchoring of the gatewell to the pump pit may be necessary. Strength evaluation of the structure was not possible since drawings were not available. However addition of fill material will inherently strengthen the structure, and bracing / struts may be added as necessary to achieve the required strength.

The gatewell and gravity drainage pipe is recommended for abandonment, since its condition is unknown and all flow is pumped. The pump station discharge line will be routed up and over the new line of protection.

The pump station receives inflow from a 24" diameter vitrified clay pipe (VCP – file A-10-1994). There is no information regarding the material, age of the pipe, construction documents, or inspection documents. A flotation or strength analysis was not performed on the pipe. Inspection of the sewer by local entities to determine the condition of the pipe is recommended.

The age of the pump station is unknown. Though drawings were unavailable, the recommended remedy entails addition of significant amounts of weight. While such

testing as concrete coring is not required during the next project phase, further examination will be required during PED to assess the transfer of uplift load between the adjacent gatewell (to be abandoned) and the pump station structure.

The recommended section in this area is a flood wall. There are no earth fill impacts on the pump station structure (i.e. no additional earth fill against the pump station).

The pump station structure could be modified for either N500+0 or N500+5 design conditions by adjusting the weight and bracing commensurate with the design loading. The discharge pipe would be routed "up & over" the new line of protection.

A-13.4.9 National Beef (KCK #8)

The National Beef Pump Station was originally designed to handle interior drainage and ground-discharging seepage only (i.e. no flow from relief wells discharging below grade) from a small drainage area. The businesses for which the pump station was needed have since disappeared. While the pump does run and the station is officially considered to be still in service, preliminary investigations show that it is no longer needed, and that the pump station may be abandoned in place for the N500+3 raise, as well as for N500+0 or N500+5.

A-13.4.10 Central Avenue Pump Station

The Central Avenue Pump Station handles only relief well flows discharging below grade, through a collector system to the pump plant. *Under current conditions, pumping of the relief well flow is required during high river stage events to maintain the integrity of the line of protection along the relief well system.*

Structural evaluations indicate that the Central Avenue Pump Station is acceptable for flotation with water to the top of the existing line of protection. For any line of protection raise requiring continued service of the pump plant, control of subsurface hydrostatic pressure is required to alleviate flotation issues. Features such as a slurry cutoff wall or relief wells may be used, or alternately a structural modification such as base slab extensions or micropiles may me utilized to meet flotation criteria. Strength of the pump station is adequate for existing conditions, though modifications are required for levee raises up to and including N500+3.

This station services an existing relief well system (no interior drainage) and has no excess capacity. Geotechnical investigations in the area show that, under N500+3 design conditions, underseepage may be controlled with relief wells discharging to the surface (i.e. relief well flow would *not* need to be pumped) and draining to a nearby low area, as shown in Exhibit A-13.2. Portable pumping could be provided at the discretion of the Sponsor, and the Central Avenue pump plant could then be abandoned. The proposed ponding area, shown in Exhibit A-13.6 at the end of this chapter, is currently occupied by a local business and used as a storage area. Real estate considerations are currently underway, however it is anticipated that the business could continue to use the ponding area under non-flooding conditions. Under flooding conditions, the wells would start flowing at a very low rate once the river stage reached the elevation of the well

discharges (several feet above bank full). As river stages reach the top of the line of protection, the wells would reach a maximum combined flow rate of roughly 22 cfs.

N500+0 or N500+5 design conditions would be accommodated by a similar section and line of new surface-discharging relief wells / ponding area. Therefore, the pump station would not be required under either design condition.

A-13.5 SUMMARY OF RESULTS

See the following pages for a tabulated summary of results for both the existing and with project conditions.

A-13.6 RELIABILITIES

Probability of failure curves for 12th Street Pump Station, Mill Street Pump Station, and 5th Street Pump Station were used in the economic model. Refer to Exhibit A-13.7 at the end of this chapter for the results.

Calculations for the 12th Street Pump Station and Mill Street Pump Station revealed a 100% probability of failure with water at top of levee, driven primarily by inadequate strength of key structural members. Though both pump stations also show a very high probability of failure with water at much lower levels, they performed adequately during the 1993 flood event. Therefore, a 25% probability of failure is considered reasonable for the 1993 flood elevations at these two pump stations.

Uplift calculations of the 5th Street Pump Station reveal a 100% probability of failure with water several feet below the existing top of levee. Probabilities of failure for lower water levels were calculated, and defined the curve which was used in the economic model.

TABLE A-13.2
Existing Conditions Pump Station Analysis Results

Pump Station	Station	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments
Osage (KCK #14)	76+83	ok	ok	ok	ok	marginal	marginal	
12th St (KCK #13)	129+20	0.54	0.36	1.29	0.60	1.07	RCB is reliable	
Mill St (KCK #12)	156+75	0.71	0.43	1.4	0.63	1.1	RCB is reliable	
5th St (KCK #11)	185+70	1.5	1.5	2.17	1.5	0.88	marginal	
MW Cold Storage	194+60	privately owned sump and abandoned mechanical equipment, see write-up						
Shawnee Ave (KCK #10)	230+78	ok	ok	ok	ok	ok	marginal	
KCS RR	276+79	ok	ok	ok	ok	ok	no good	
PB1 Gordon	286+59	no drawings available - no analysis performed				0.60	no good	
National Beef (KCK #8)	295+52	can be abandoned, see write-up						
Central Ave	299+21	ok	ok	ok	ok	ok	ok	

Green - FS equal to or greater than 1.5 for strength and greater than or equal to 1.10 for stability
Yellow - FS between 1.3 and 1.5 for strength and/or between 0.96 and 1.09 for stability
Red - FS <1.3 and/or FS < 0.95 for stability
Strength: 1.5 FS required per established criteria
Stability: 1.5 FS required per established criteria

TABLE A-13.3
Summary of N500+3 Conditions Results and Recommendations

Pump Station	Sta.	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments
Osage (KCK #14)	76+83	1.15	1.27	1.63	2.74	0.84	likely to require rehab due to loading	structural modifications necessary for uplift - cut off wall, raise discharge chamber
12th St (KCK #13)	129+20	0.5	0.34	1.18	0.55	1.07	RCB is reliable	Major structural modifications. Brace and shore for strength, add weight for stability
Mill St (KCK #12)	156+75	0.67	0.41	1.29	0.58	1.1	RCB is reliable	Major structural modifications. Brace and shore for strength, add weight for stability
5th St (KCK #11)	185+70	1.49	1.5	1.99	1.4	0.74	RCP will require rehab	Minor strength mod required, major stability mod required
MW Cold Storage	194+60	privately owned sump and abandoned mechanical equipment, remains and no need for structural modification						
Shawnee Ave (KCK #10)	230+78	1.5	1.5	1.44	1.5	1.08	RCB undergo rehab prior to placing additional fill over it	No major structural modification required.
KCS RR	276+79	1.54	1.52	4.68	2.01	1.208	42-inch CIP will likely require rehab.	No major structural modification required.
PBI Gordon	286+59	no drawings available - no analysis performed				0.67	all flow is pumped over line of protection-recommend abandonment	To remain. Bracing/struts may need to be added to achieve required strength
National Beef (KCK #8)	295+52	Abandon in place, allow ponding, temporary pumping						
Central Ave	299+21	1.61	1.5	1.5	1.34	0.908		Abandon in place, install new line of relief wells, allow ponding, temporary pumping

Red=Rehabilitation Required

Green - FS equal to or greater than 1.5 for strength and greater than or equal to 1.10 for stability

Yellow - FS between 1.3 and 1.5 for strength and/or between 0.96 and 1.09 for stability

Strength: 1.5 FS required per established criteria for shear and moment

Stability: 1.1 FS required per established criteria

A-13.7 REFERENCES

1. US Army Corps of Engineers (Feb. 28, 1995), *EM 1110-2-3102 CECW-EE General Principles of Pumping Station Design and Layout*, Kansas City District.

2. US Army Corps of Engineers (June 30, 1989), *EM 1110-2-3104 CECW-ED Structural and Architectural Design of Pumping Stations*, Kansas City District.

3. US Army Corps of Engineers (Nov. 30, 1999), *EM 1110-2-3105 CECW-EE Mechanical and Electrical Design of Pumping Stations (Change 2)*, Kansas City District.

4. US Army Corps of Engineers (Apr. 30, 2000), *EM 1110-2-1913 CECW-EG Design and Construction of Levees*, Kansas City District.

5. US Army Corps of Engineers (Mar. 31, 1998), *EM 1110-2-2902 CECW-ED Conduits, Culverts, and Pipes*, Kansas City District.

A-13.8 SUPPLEMENTAL EXHIBITS

EXHIBIT A-13.1

Pump Station Information and Photographs

1. Osage (KCK FPS 14)

a. Year Constructed: 1953[12]
b. Owner/Operator: KCK
c. Levee Stationing: 76+83[1]
d. Distance From Flood Protection Centerline: 10ft[7]
e. Station Capacity: 11,650gpm[4]
f. Pumps:
   i. 16" 4,900gpm @40hp and 590 RPM[4]
   ii. 16" 4,900gpm @40hp and 590 RPM[4]
   iii. 6" 1,850gpm@15hp and 704 RPM[4]
g. Contributing Flows:
   i. Storm Sewer Flow from Prospect Park and the West Armourdale Sewer System, 344ac
h. River Discharge: 72" RCP
i. Discharge Arrangement: Gatewell at Pumping Plant & pressurized pipe under levee[7]
j. Drawing Availability:
   i. Date Unknown – Gatewell drawing[5]
   ii. Dec 1979 – General Plan & Profile[7]
   iii. Aug 1954 – Record Drawings (18 sheets)[7]
k. General
   i. Manhole #42 (see 1948 "Supplement on Interior Drainage", plate 14) separates the lowlands from the uplands.[3]
   ii. The excess runoff that bypasses the upland sewer system is stored in a large depression south of Muncie Boulevard.[3]
   iii. The flowline out at MH #42 is two feet higher than the critically low spots at the intersections of 18th Street and West Kansas Avenue and at 18th Street and Osage Avenue. Because of the 2 ft. of available head, the trunk sewer can discharge the hillside runoff through the 30-inch lateral sewers onto the intersections mentioned above.[3]
   iv. From field observations performed in 1948, flooding seemed to be more extreme at 18th Street and West Kansas Avenue.[3]
   v. Development has occurred in lower areas adjacent to the levee, increasing percent impervious area. This, in turn, has decreased the ponding area for seepage to collect. While the seepage flow was not designed to be collected and pumped, it may reach the pumping plant and decrease its pumping capacity.[3]
   vi. A pump was originally proposed to limit ponding in the area south of MH #42 so that, at a stage of 26.2 ft., the ponding will not be in excess of 0.158 in. This is the same ponding experienced at a stage of 14.0 ft. under gravity conditions.[3]
   vii. Pump plant located on/in earthen levee[4]
   viii. River Discharge shows degradation (scour hole) at conduit outlet. Hole approximately 1ft in depth and 1ft in width at largest extents.[9]

2. 12th Street Sewer (KCK FPS 13)

a. Year Constructed: N/A
b. Owner/Operator: KCK
c. Levee Stationing: 129+20[1]
d. Distance From Flood Protection Centerline: 80ft[5]
e. Station Capacity: 16,000gpm[1]
f. Pumps:
   i. 18" 8,000gpm @ 39.5TDH 125hp[1] 1185rpm[9]
   ii. 18" 8,000gpm @ 39.5TDH 125hp[1] 1185rpm[9]
   iii. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow 298ac of drainage[1]
   ii. Seepage Flow from Sta. 97+00 to Sta.129+20[1]
h. River Discharge: 5'x6' RCB[3]
i. Discharge Arrangement: Pressure pipe passing through levee (centerline @ 767.24) discharging to gatewell riverward side of levee.[5]
j. Drawing Availability:
   i. Jul 1995 – Mechanical Renovation [5]
   ii. Dec 1979 – General Plan & Profile [7]
k. General
   i. Seepage will collect in the low areas between the levee and the railroad track between stations 103+001 and 135+001, covering about 10 acres. Seepage will begin at about stage 32.8 and will reach a max flow of 3.2 cfs at design-flood stage (40.8).[3]
   ii. The ponding that is caused by the river tailwater which limits the capacity of the outfall is less then the ponding due to the insufficient capacity of the sewer during low river stages. In other words, the temporary ponding due to the higher river stages between 14 ft and 21.9 ft is less than the unavoidable ponding that occurs during lower stages from 14 ft and below. [3]
   iii. Pumping is started at stage 21.9', which corresponds to the ground elev. of the lowest manhole in the system. It is shown that the gravity flow capacity of the system is greater then the pump capacity at stage 21.9', but due to the high river stage resulting in backwater requires the gate to be close which effectively makes the sewer capacity zero. [3]
   iv. The COE, in the 1948 "Supplement on Interior Drainage", proposed no additional pumping capacity because the ponding due to the lack of pumping capacity is less then the ponding due to the sewer capacity. [3]
   v. The new pumps were installed in 1996. These two pumps were cited to have a capacity of 8000 gpm @ TDH 39.5 each. This capacity was compared to the original pumps at the same head and HNTB found that the original pump only had 6000 gpm @ TDH 39.5. This shows that the pump plant capacity has increased at the stages stated in the table. However, it is unknown how much the capacity has increased. [3]
   vi. Areas were originally determined to be undrained have been developed, thereby increasing the percent impervious. This increase could cause the peak flow to make the design pump capacity insufficient. [3]
   vii. Pump Station in close proximity to 12th street bridge, private auto salvage yard and overhead lines. [9]

3. Mill Street (KCK FPS 12)

a. Year Constructed: <1952[3]
b. Owner/Operator: KCK
c. Levee Stationing: 156+75[9]
d. Distance From Flood Protection Centerline: 30ft[7]
e. Station Capacity: 16,800gpm[4]
f. Pumps:
   i. 8,000gpm @ 39.5ft TDH[3]; 125hp @ 1185rpm[9]
   ii. 8,000gpm @ 39.5ft TDH[3]; 125hp @ 1185rpm[9]
   iii. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow 580ac[1]
   ii. Seepage Flow from Sta.129+20 to Sta. 173+00[1]
h. River Discharge: 6'x8' RCB[3]
i. Discharge Arrangement: Pressure pipe passing through levee (centerline @ 766.66) discharging to gatewell riverward side of levee.[5]
j. Drawing Availability:
   i. Jul 1995 – Mechanical Renovation[5]
   ii. Dec 1979 – General Plan & Profile[7]
   iii. Mar 1940 – Repair of pump house and floodgate[5]
k. General
   i. Seepage between stations 135+401 and 180+001 will begin at a stage of 32.8 ft. and reach a maximum flow of 4.5 cfs at the design flood stage (40.8 ft.). Seepage will pond in the depressed areas between the levee and the railroad tracks to the north. This area is drained by evaporation and infiltration.[3]
   ii. The present pump starting stage as indicated by the City of Kansas City, Kansas in 1948, is 14.9 ft. on the Hannibal Bridge gage. The pump operator actually uses that gage as a warning and the pumps are started when backflow in the sewer is about to pond in the intersection of Mill Street and Shawnee Avenue, which is equivalent to a stage of 19.4 ft. on the Hannibal Bridge gage. At this stage the sewer capacity and pump capacity are equal, therefore no additional pumping capacity was proposed.[3]
   iii. The Design Memorandum No. 3 written by the COE for the 1962 Modification states that the existing pump plant is outdated and is scheduled to be replaced by the city of Kansas City, Kansas.[3]
   iv. Overland flow cannot supplement sewer delivery to this outlet, due to a recently placed landfill adjacent to the levee. The landfill encroaches heavily on former ponding areas.[3]
   v. Areas that were originally undrained are now serviced by the pump plant due to the development in the area. The outlet now drains more then the original 304 acres, possibly the total 580 acres, and has an increased percent impervious.[3]
   vi. Two new pumps were installed in 1996. These pumps were cited to have a capacity of 8000 gpm @ TDH 39.5. This capacity was compared to the original pumps at the same head and HNTB found that the original configuration had one pump with 6000 gpm @ TDH 39.5 and another pump with 5750 gpm @ TDH 39.5. This shows that the pump plant capacity has increased at the stages stated in the table. However, it is unknown how much the capacity has increased.[3]
   vii. Pump station located under large overhead transmission lines.[9]

4. 5th Street (KCK FPS 11 / AID)

a. Year Constructed: 1963[12]
b. Owner/Operator: KCK
c. Levee Stationing: 185+70[1]
d. Distance From Flood Protection Centerline: 35ft[7]
e. Station Capacity: 39,000gpm[3]
f. Pumps:
   i. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   ii. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   iii. Fairbanks Morse 20" 13,000gpm @75hp and 580 RPM[4]
   iv. Sump Pump[4]
g. Contributing Flows:
   i. Storm Sewer Flow ~80ac[1]
   ii. Seepage Flow from Sta. 173+00 to Sta. 190+75 [1]
h. River Discharge: 72" RCP[3]
i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
k. General
   i. It is believed that when the packing industry dissolved and left this area, it was decided by either the KVDD or other local interests to improve the drainage system so that the land would be more appealing for development. For this reason, the 5th Street Pump Plant was installed. This changed the drainage characteristics that existed during the composition of the 1948 "Supplement on Interior Drainage".[3]
   ii. Originally, water that was collected in the area of the 5th Street pump plant services was allowed to pond and would have been removed by infiltration and evaporation. It is the area described in the 1948 "Supplement on Interior Drainage", paragraph 24g, as the non-draining area. This non-draining area would have ponded water and then overflowed to the Shawnee Avenue Sewer System if the ponded water reached a certain control elevation. Now the 5th Street Pump Plant removes the water from this area.[3]
   iii. The typical design condition variables do not apply, as it appears that the pump plant was just designed to handle certain flow rates at certain stages. No problems were reported with this pump plant.[3]

5. MidWest Cold Storage

a. Year Constructed: N/A
b. Owner/Operator: Mid West Cold Storage
c. Levee Stationing: 194+60 [1]
d. Distance From Flood Protection Centerline:20ft
e. Station Capacity: 1,800gpm
f. Pumps: N/A
g. Contributing Flows:
   i. Storm Sewer Flow [1]
h. River Discharge: 24" CIP[3]
i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
j. Drawing Availability:
k. General
   i. The 1948 "Supplement on Interior Drainage" explains that the pump was for the Midwest Cold Storage property. The outfall for the pump was a 6" removable pipe and pumped over the levee. [3]
   ii. It is believed that the Armourdale Pump Plant has made the function of this pump plant unnecessary. The KVDD is unaware of its use. However, the status has not been confirmed and the pump plant may be used periodically for the Midwest Cold Storage property. [3]
   iii. Pump Station has been abandoned. Discharge pipe passing through the levee has been filled with concrete. Gatewell on outside of building has been filled with rock and pump pit has not been filled.[10]

6. Shawnee Ave. (KCK FPS 10)

a. Year Constructed: 1954[3]
b. Owner/Operator: KCK
c. Levee Stationing: 230+77[1]
d. Distance From Flood Protection Centerline: 10ft[5]
e. Station Capacity: 63,000gpm[1]
f. Pumps:
   i. 30" 125hp[1] 710rpm[9]
   ii. 30" 125hp[1] 710rpm[9]
   iii. 30" 125hp[1] 710rpm[9]
   iv. 20" 100hp[1] 880rpm[9]
g. Contributing Flows:
   i. Storm Sewer Flow from Shawnee Sewer System, 140ac[1]
   ii. Seepage Flow from Sta. 190+75 to Sta. 246+35[1]
      1. Headerwell System 1 (1 to 24)[8]
h. River Discharge: 7.5'x7.5' RCB[3]
i. Discharge Arrangement:
j. Drawing Availability:
   i. Aug 1996 – Automatic lubrication system [5]
   ii. Dec 1979 – General Plan & Profile [7]
   iii. Sep 1969 – Record Drawings [5]
k. General
   i. The average elevation of the low lying contributing area is 14 feet below the design flood elevation. [3]
   ii. It was shown that this pump plant was replaced with a new pump plant in the plans for the 1962 revision. It was necessary to increase pump capacity because of the ponding that occurred during design flood stage. [3]
   iii. Ponding will occur under gravity flow due to inadequate sewer capacity. [3]
   iv. The main purpose of this plant is to discharge seepage from the Header Well System I. The current design needed to handle more seepage because the Design Flood elevation increased from original pump plant construction. [3]
   v. The recent (late 1970s) storm sewer system along Shawnee Ave., in conjunction with urban renewal, increased the delivery of storm water.[3]
   vi. Because of the seepage at stage 40.81, 88 acres of once pervious area is now computed to be impervious which increases runoff. Note that the area between 180+001 and 235+001 is storage. [3]
   vii. Railroad fill across Shawnee Avenue near the levee denies overland flow to the outlet. [3]
   viii. River discharge at outfall has been recently replaced.[9]

7. East Outfall Sewer Pump Plant

a. Year Constructed: N/A
b. Owner/Operator: KCK
c. Levee Stationing: 246+53[3]
d. Station Capacity: >12,000 gpm
e. Pumps: N/A
f. Contributing Flows:
   i. Storm Sewer System Flow (water from the railyards area)
g. River Discharge: 30" DIP[3]
h. Discharge Arrangement:
i. Drawing Availability:
j. General
   i. This pump is located in the railyards 650 feet west of 4th Street and Berger Avenue.[3]
   ii. A 24" DIP sewer line intercepts drainage from a 33" RCP culvert located under the tracks just west of the pump plant.[3]
   iii. The plant pumps water through the 24" sewer line to the 30" DIP outfall at station 246+53.[3]
   iv. The static head was determined as the difference between the centerline of the pipe at the sluice gate for the 30" DIP outfall and the minimum water level in the station. The elevation at the sluice gate is 748.1 (approx. stage 29.0 ft.). The impact of higher river stages on the static head was disregarded since the probability of simultaneous occurrence with the peak discharge of the pump plant was very remote.[3]
   v. Conditions in the area contributing to the pump plant have not changed significantly since the design of the plant.[3]
   vi. Pump plant is located approximately ¼ mile from flood protection and not station strength or floatation not considered to be an issue.[10]
   vii. Outlet conduit will be considered as utility.[10]

8. KCS RailRoad

a. Year Constructed: 1949[12]
b. Owner/Operator: KVDD
c. Levee Stationing: 276+79[1]
d. Station Capacity: N/A
e. Distance From Flood Protection Centerline: 40ft[7]
f. Pumps:
   i. 14" Fairbanks Morse submersible, 703RPM 37.9HP[11]
g. Contributing Flows:
   i. Seepage Flow from Sta. 268+59 to Sta. 282+29[1]
      1. Headerwell System 2 (1 to 14)[8]
h. River Discharge: 42"CIP[1]
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile[7]
   ii. Apr 1951 – Structural Details[7]
k. General
   i. No performance curves were found for the pump, so the capacity was assumed to be 2.23 cfs, which is the pump rating.[3]
   ii. The pump outfall is through a 6 inch pipe 8 inches below the top of wall. Gravity flow is through a 10" CIP. Because of the low capacity of the pump, it was proposed (in the "Supplement on Interior Drainage") to construct a pump plant with a capacity of 13.5 cfs. This did not happen.[3]
   iii. Wells #1 through #6 no longer discharge to the KCS Pump because of a stability berm that now covers the discharge openings at the top of the structure.[3]
   iv. This pump is to be operated only when the Kansas River is below elevation 747.4 and falling. If the Kansas River is rising, the plant sluice gate must be closed which causes ponding in the railroad area.[3]
   v. An open ditch conveys seepage water to the plant. Due to the open ditch, there is a runoff flow in an uncontrolled condition. The worst case is at stage 17.7 when the runoff is 4.6 cfs.[3]
   vi. A new pump was added in 1995.[3]
   vii. Pump plant is located approximately 40ft from floodwall.
   viii. Some drawings found in Vol 1 O&M manual on the E:drive.
   ix. The main purpose of this pump is to discharge seepage from the Header Well System II.[1]
   x. Wells, #1 through #6, no longer discharge to the KCS Pump because of a stability berm that now covers the discharge openings at the top of the structure. However, the pump capacity is likely still inadequate.[1]
   xi. Old vertical pump was removes and replaced with submersible pump in 1995. Pump information including curves was supplied by KVDD.[10]

9. PBI Gordon

a. Year Constructed:
b. Owner/Operator: PBI Gordon
c. Levee Stationing: 286+59[6]
d. Distance From Flood Protection Centerline: 20ft
e. Station Capacity: 600gpm
f. Pumps:
   i. Aurora Pump 4x5x9B 600gpm at 36ft of head, 1750rpm, powered by a 7.5hp motor
g. Contributing Flows:
   i. Storm Runoff[6] Pumps drainage from employee parking[10]
h. River Discharge:
i. Discharge Arrangement: Gatewell at pump station and pressurized pipe through levee[10]
j. Drawing Availability:
k. General: Aurora pump noted above was new and not yet installed at Aug2005 site visit. PBI contact said that the new pump was approx. the same size / capacity as old one. Normally, gate is closed (i.e. ALL drainage flow from small parking area is pumped). This is to ensure that any chemical leaks are contained.

10. National Beef (KCK FPS 8)

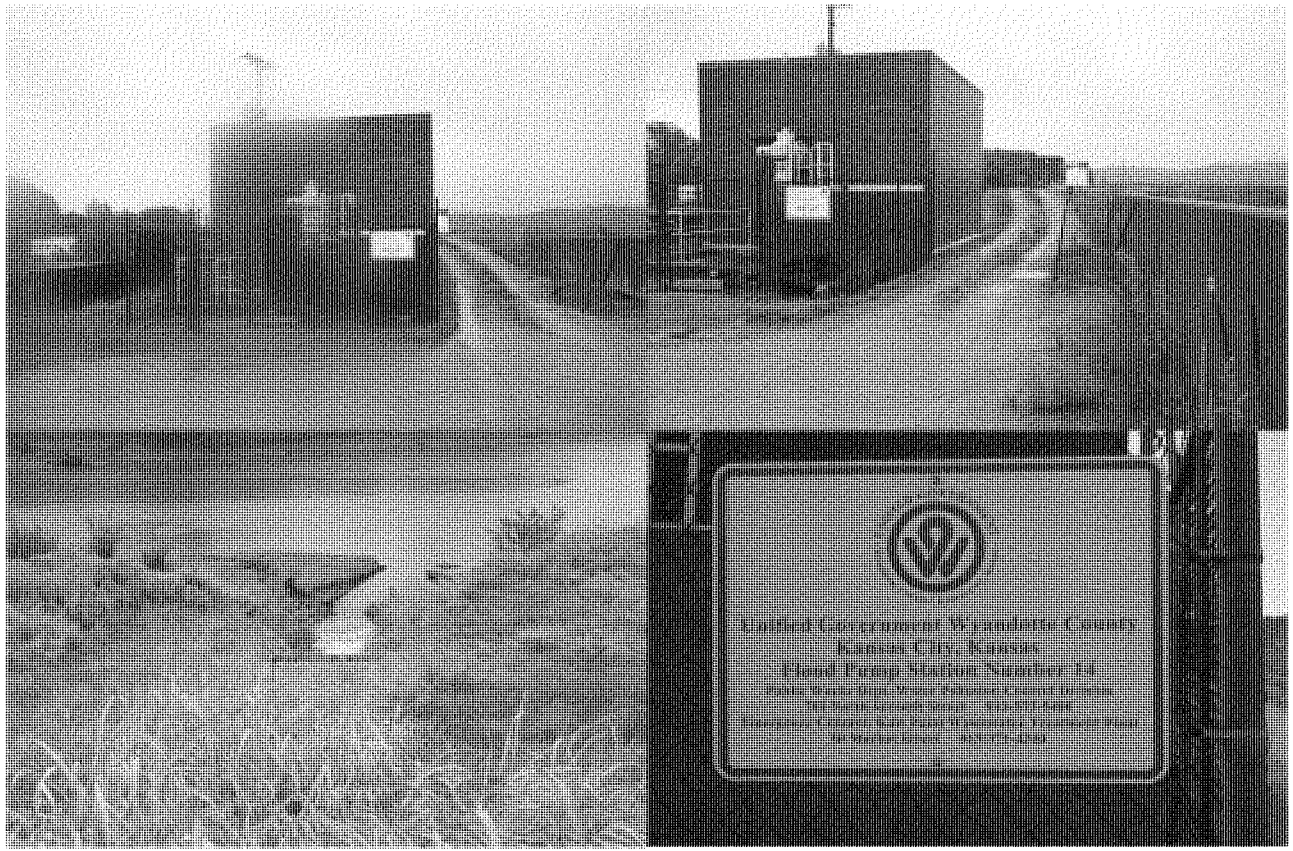
a. Year Constructed: 1928[5]
b. Owner/Operator: KCK
c. Levee Stationing: 295+52 [1]
d. Distance From Flood Protection Centerline: 40ft[7]
e. Station Capacity: 3,000gpm [5]
f. Pumps:
   i. Vertical Pump 12" suction and 10" discharge 3,000gpm[5]
   ii. Sump Pump 3" discharge
g. Contributing Flows:
   i. Storm Sewer System Flow from the Riverside Sewer System, 8 acres. [1]
h. River Discharge: 10"CIP[5]
i. Discharge Arrangement: Pressurized pipe exiting at floodwall.[5]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile [7]
   ii. March 1928 – General layout, plans & elevations, and structural steel [5]
   iii. March 1918 – Proposed Riverside Pump Station [5]
k. General
   i. The plant was originally built to service the National Beef Packing Plant, which included surface runoff, seepage, and negligible sanitary flow on the National Beef property. This industry has dissolved, but the drainage characteristics have not changed. Therefore, the pump is still in service. [3]
   ii. Seepage flow has been removed because the Header Well System III was built and now collects the seepage flow. [3]
   iii. The KVDD refers to this pump plant as the Central Avenue pump plant, yet the operational drawings label this pump plant as the National Beef pump plant.[3]
   iv. The pump capacity is remains inadequate at the Design River Stage, but ponding should not cause damage.[3]

11. Central Avenue

a. Year Constructed: 1976[3]
b. Owner/Operator: KVDD
c. Levee Stationing: 299+20[1]
d. Distance From Flood Protection Centerline: 16ft[7]
e. Station Capacity: N/A
f. Pumps:
   i. 3200gpm at 32.5ft of head at 1760rpm powered by a 40hp motor[10]
   ii. 3200gpm at 32.5ft of head at 1760rpm powered by a 40hp motor[10]
   iii. Sump Pump[9]
g. Contributing Flows:
   i. Seepage Flow from Sta. 296+23 to Sta. 302+40[1]
      1. Headerwell System 3 (1 to 7a)[8]
h. River Discharge: 2 – 14" CIP, 3" CIP
i. Discharge Arrangement: Pressurized pipes, above levee, terminating riverward of floodwall[7]
j. Drawing Availability:
   i. Dec 1979 – General Plan & Profile[7]
   ii. Sep 1978 – Structural Plans[7]
k. General
   i. The well system consists of seven artesian wells.[1]
   ii. Conditions have not changed since the pump plant design.[1]
   iii. Since its construction it has operated in 1993 only.[10]

References:

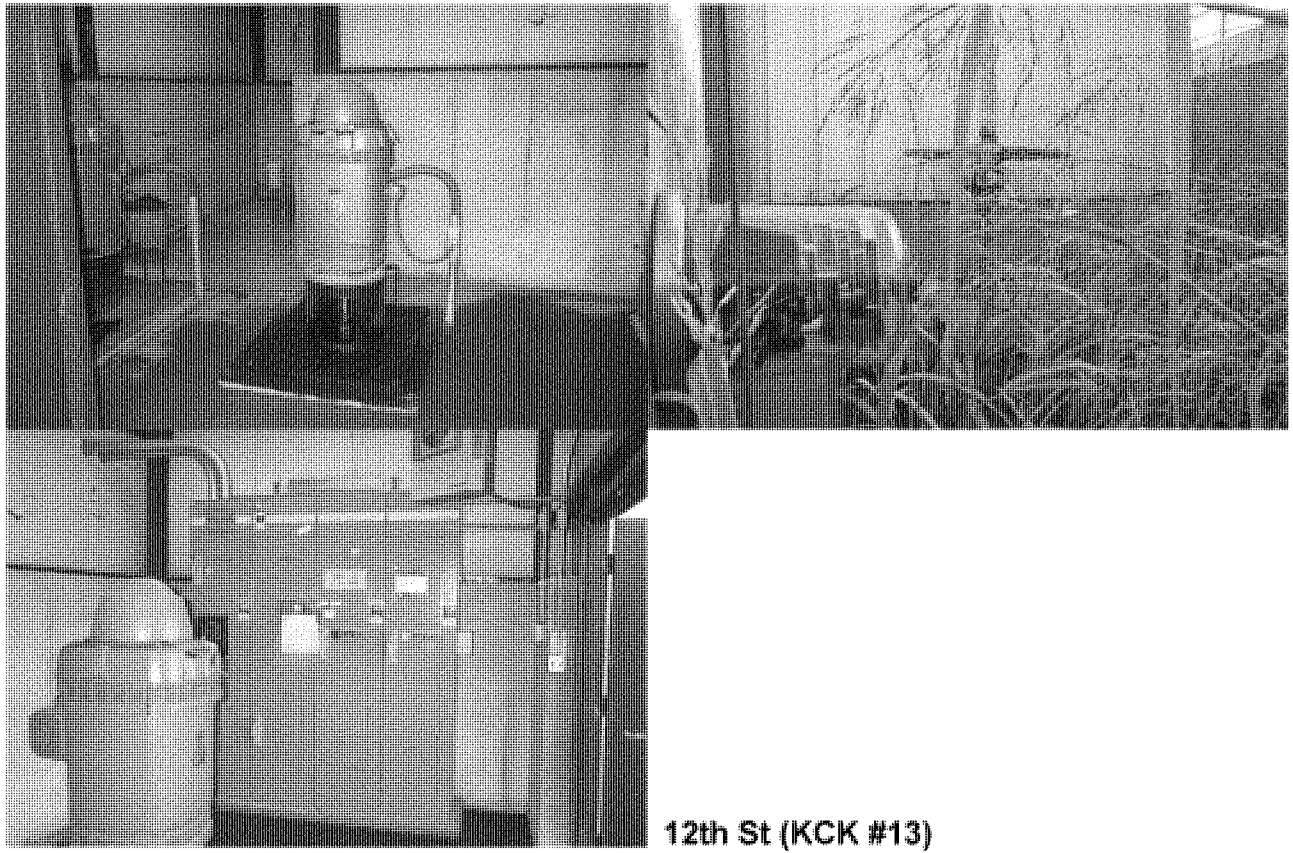
1. HNTB Drainage Feature Inventory 2001 with updates
2. HNTB Ownership Listing
3. HNTB Latest
4. KCK O&M Manuals from KCK Maintenance Library (Cary Houchins)
5. KCK As-builts from KCK Sewer Department (Larry Henak)
6. Information provided verbally by Ernie Quinto
7. O&M Drawings located at E:\sec\LocalProtectionProjects\KansasCity
8. Per Scott Loehr, EC-GD
9. Per 25Jul05 site visit
10. Site visit with KVDD (Ernie Quinto & Larry Brennen)
11. Asbuilts obtained from KVDD
12. 1962 Mod Design Memorandum #3



Osage Ave (KCK #14)

Osage Ave (KCK #14)

12th St (KCK #13)



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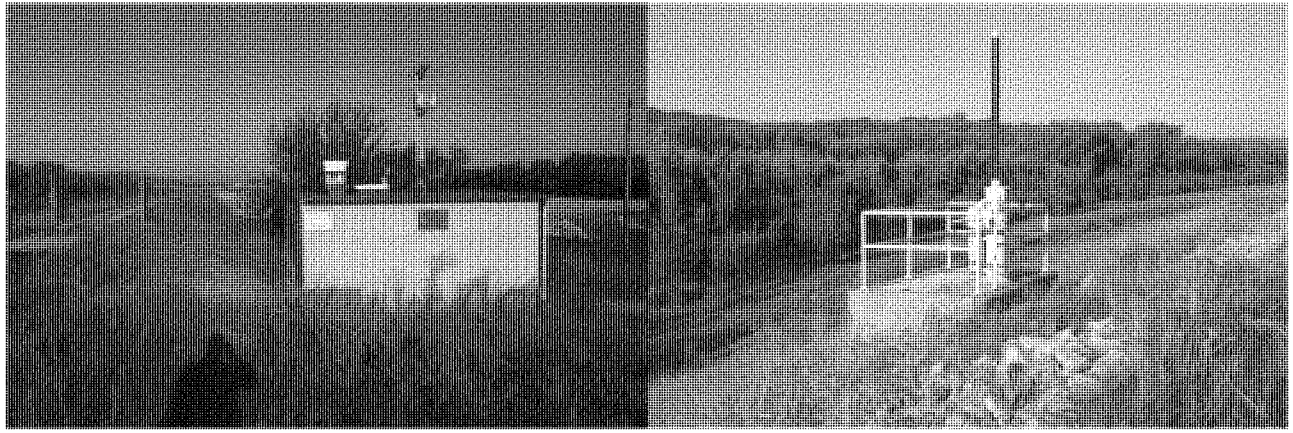

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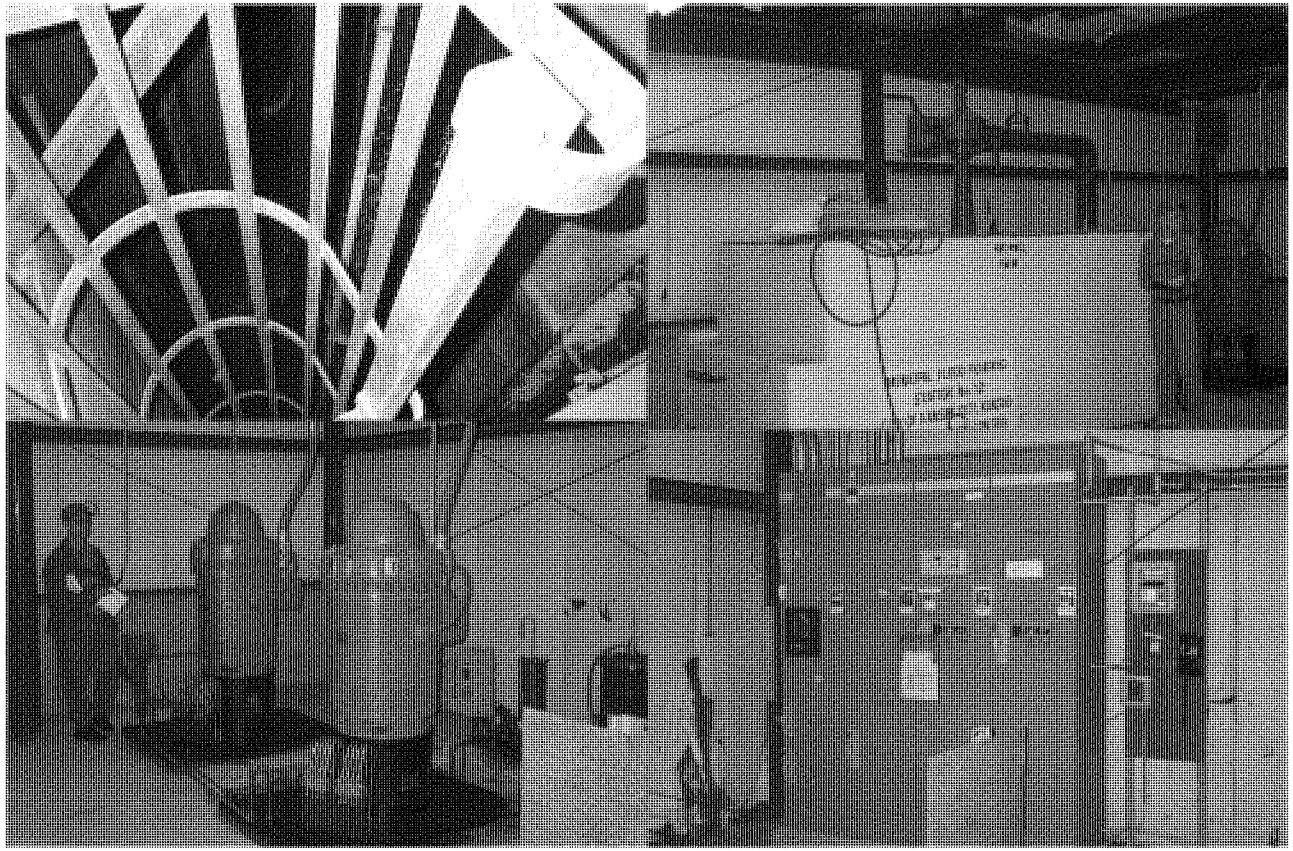
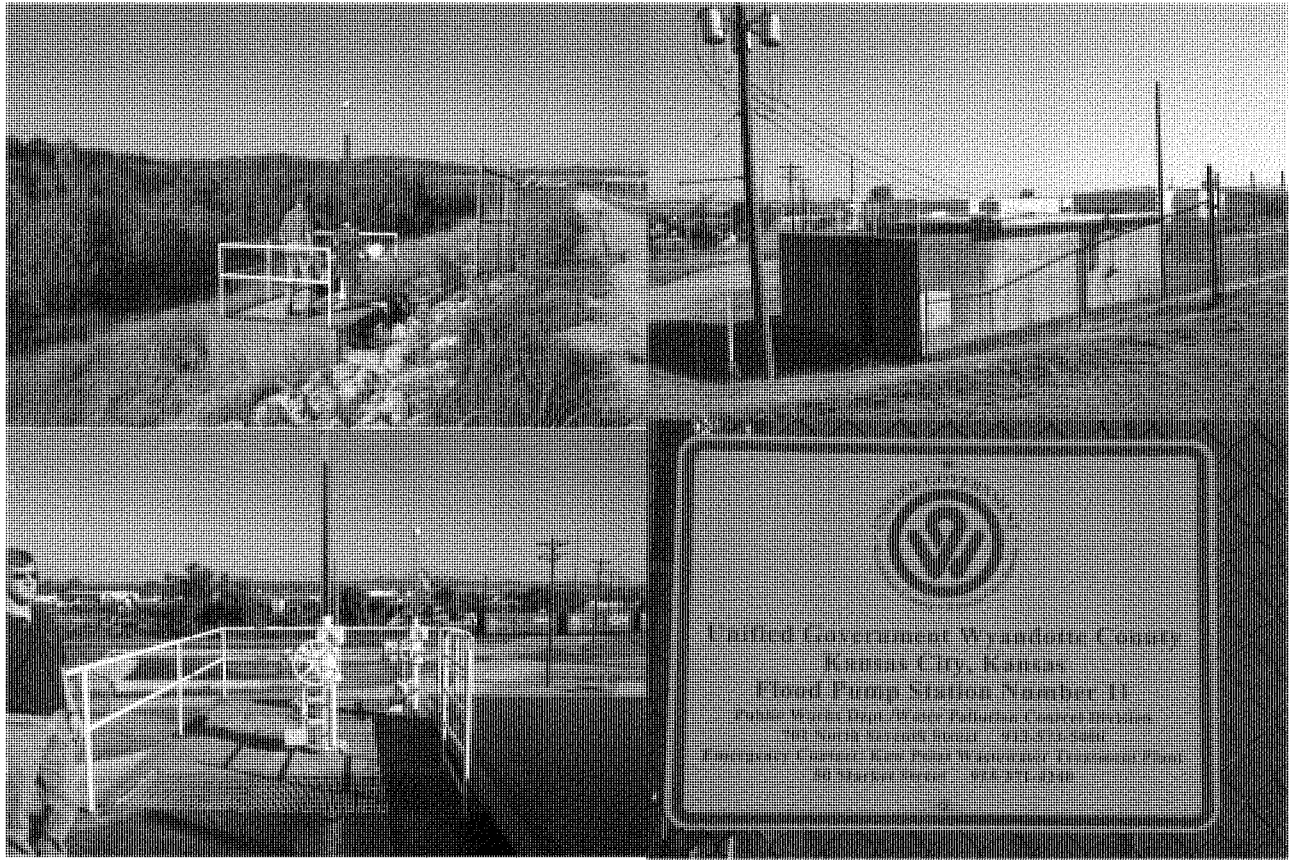
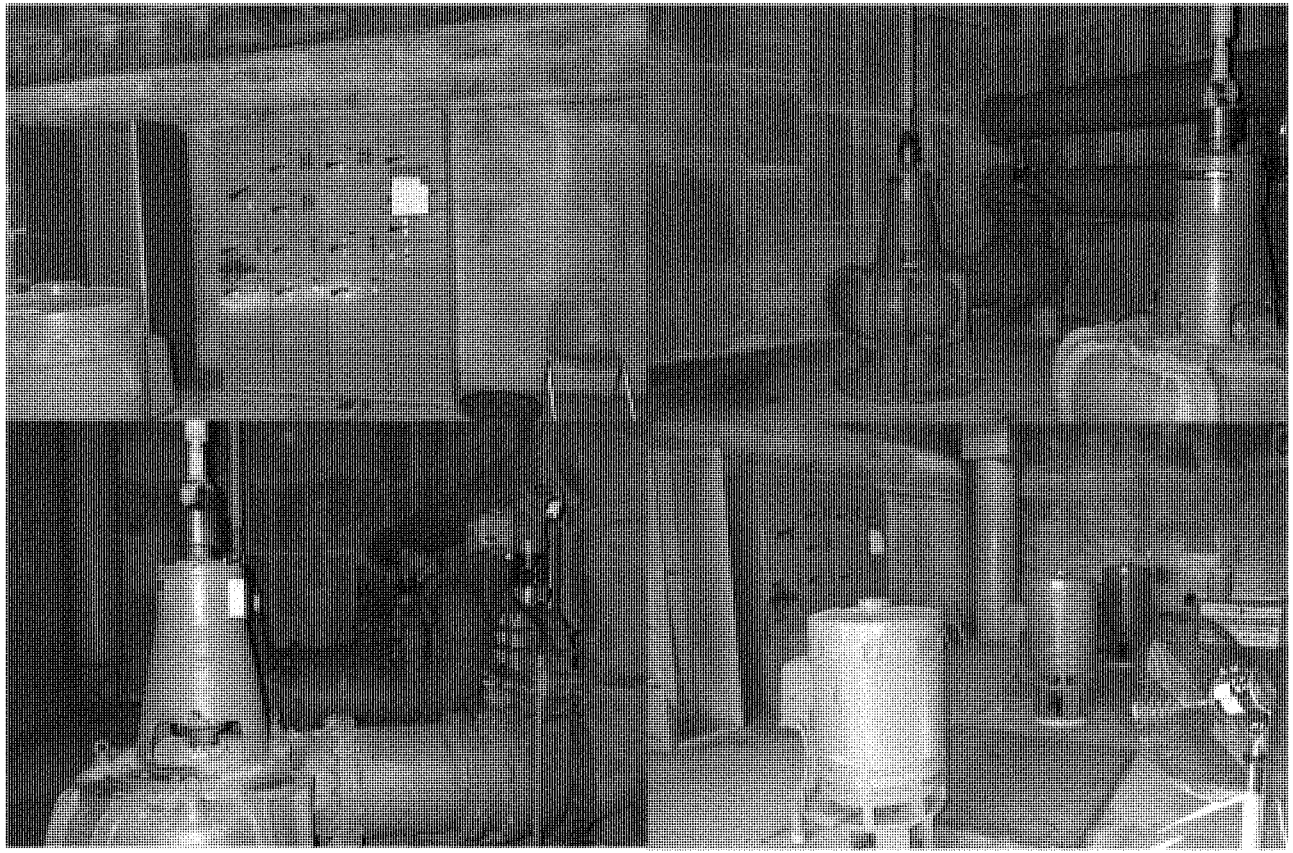


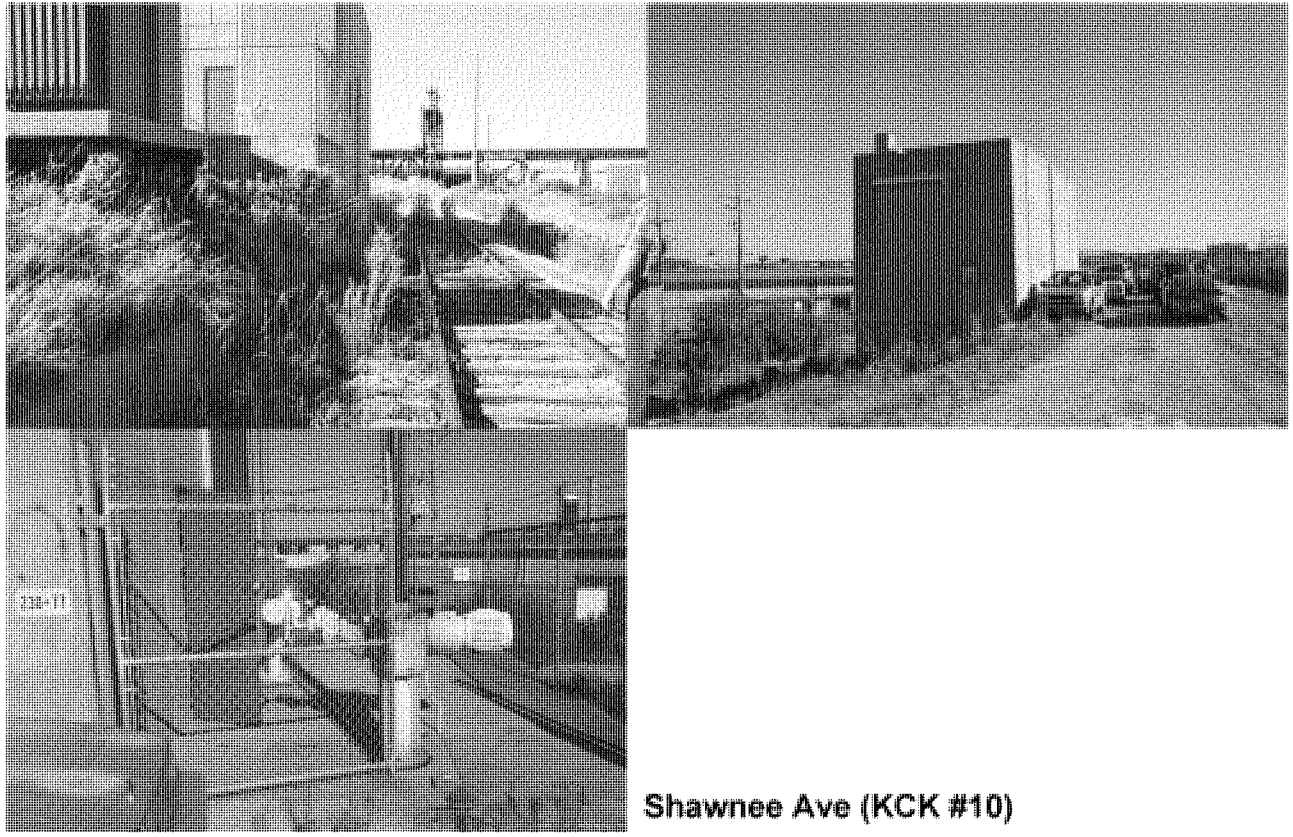
Mill St (KCK #12)


Unified Government Wyandotte County
Kansas City, Kansas
Flood Pump Station Number 11

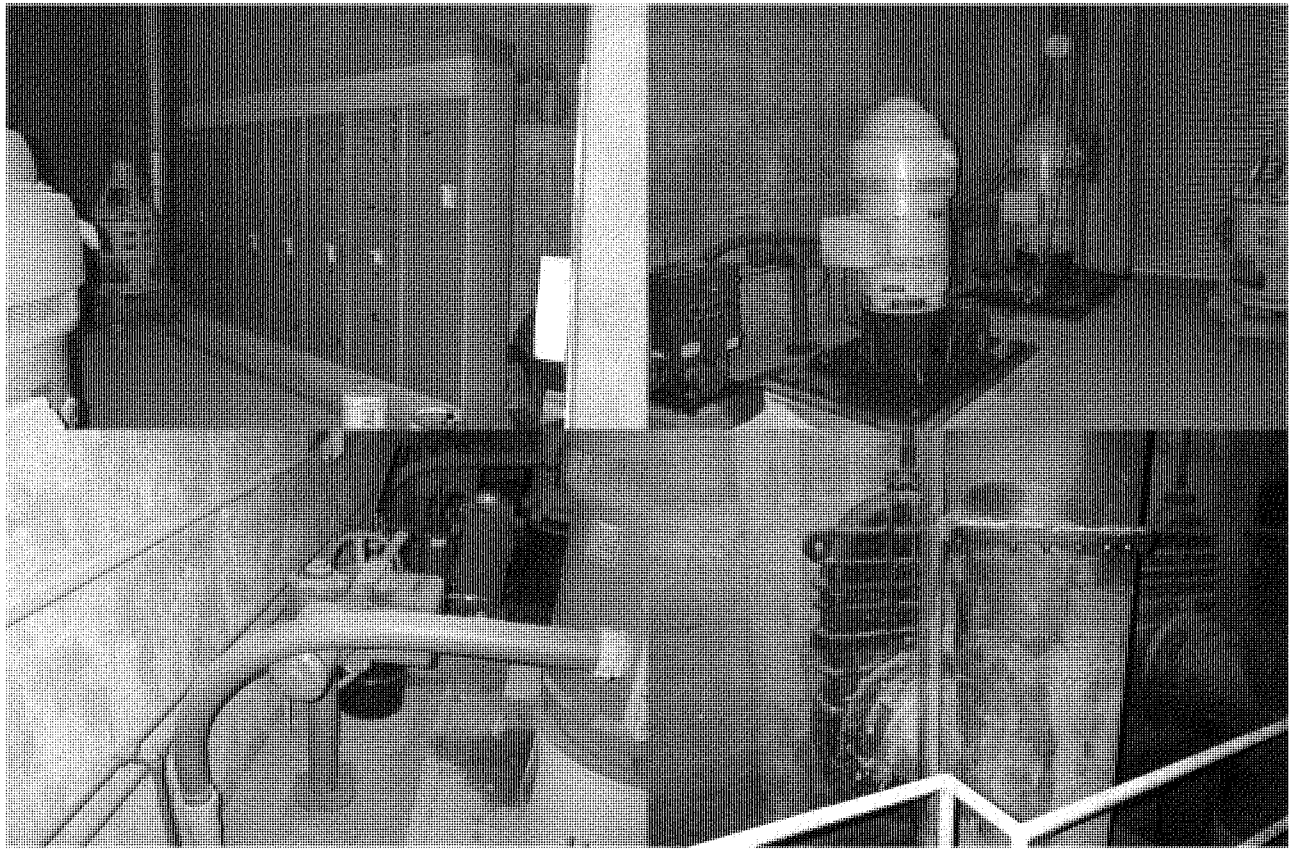
5th St (KCK #11)



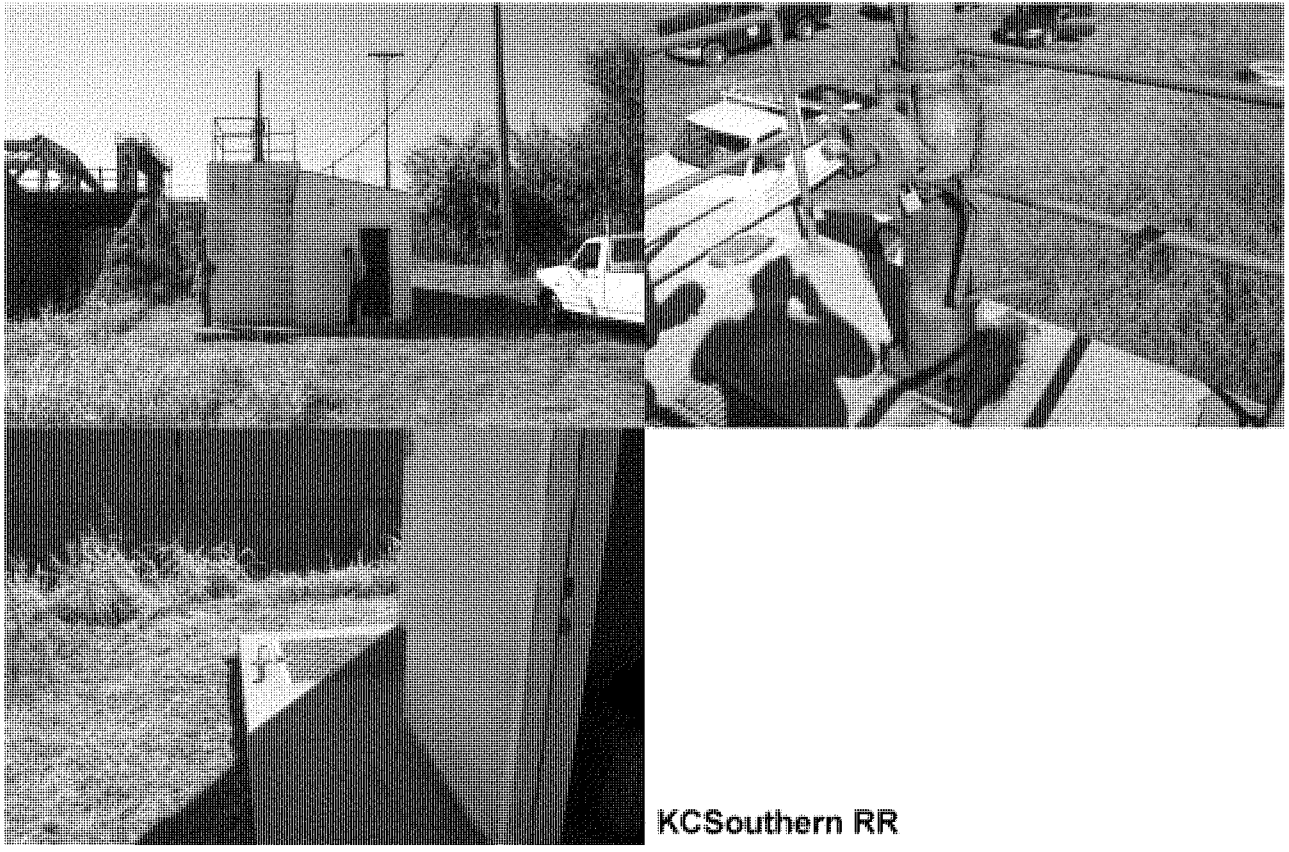
5th St (KCK #11)



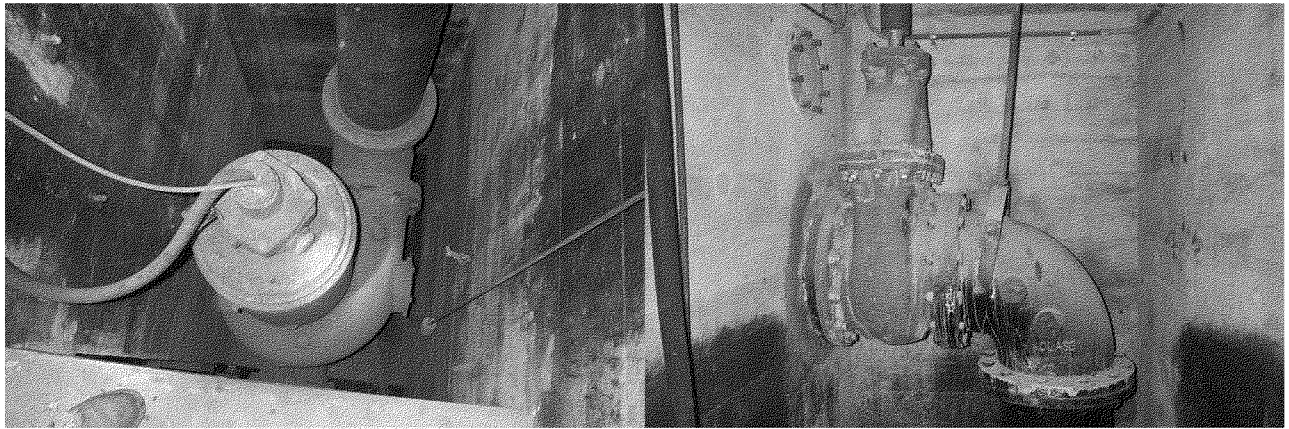
Shawnee Ave (KCK #10)



Shawnee Ave (KCK #10)



KCSouthern RR



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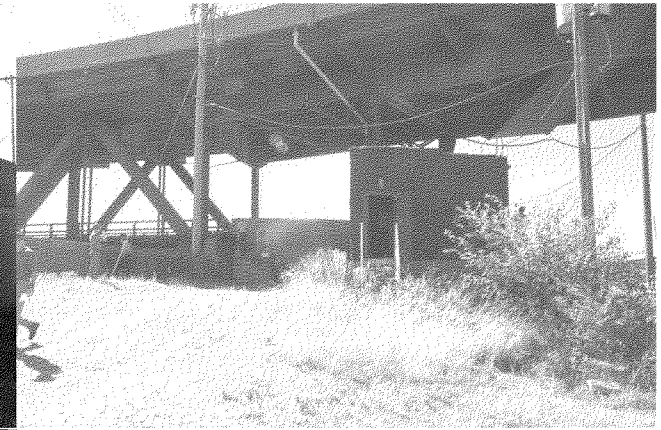
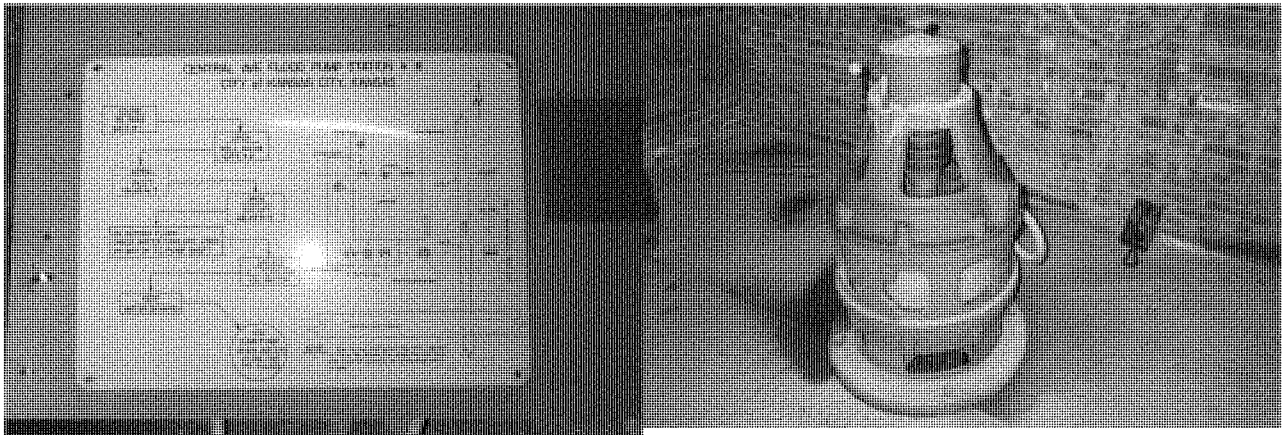
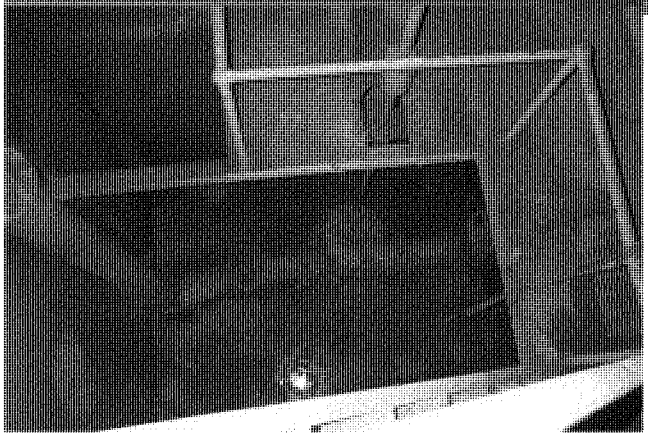
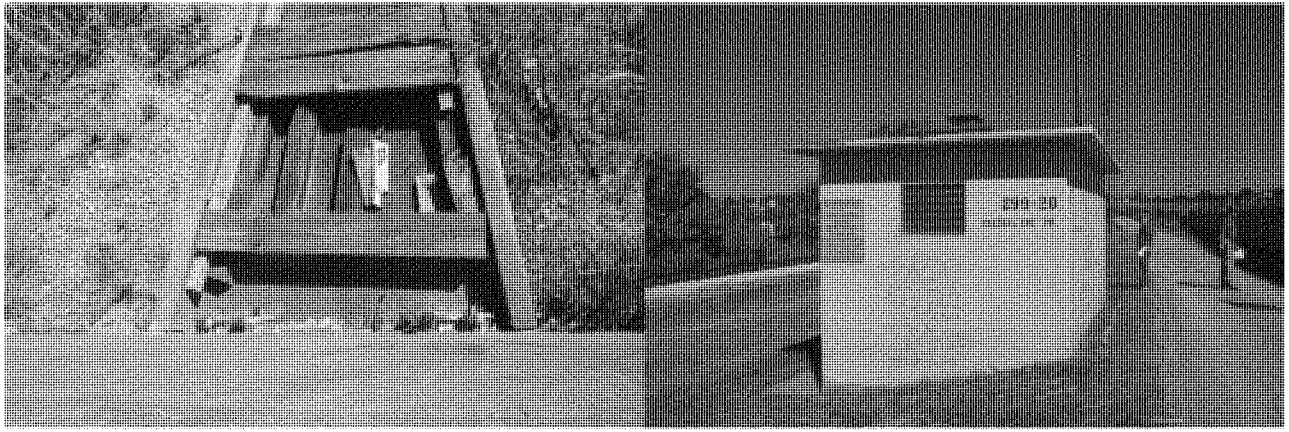


National Beef (KCK #8)



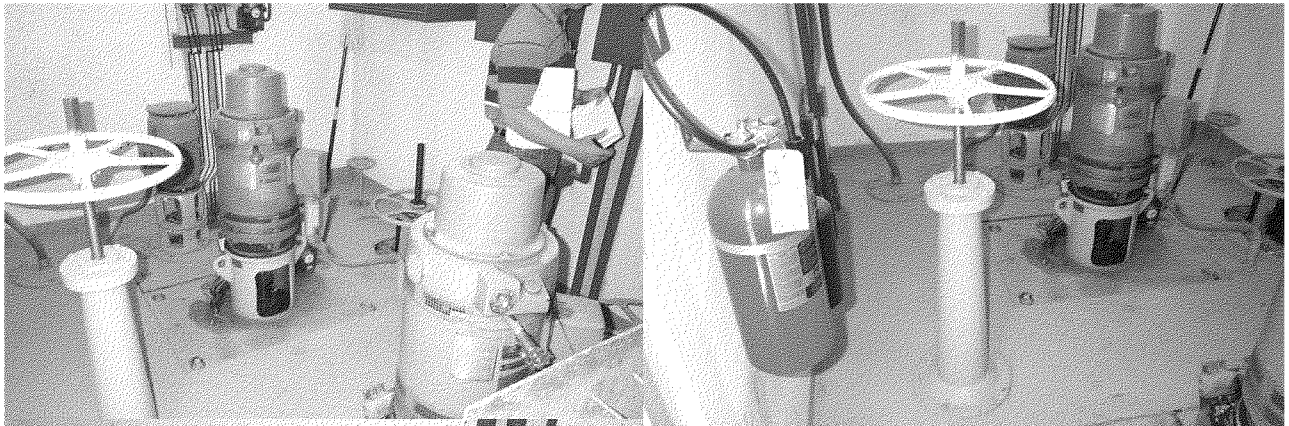


National Beef (KCK #8)



Central Avenue

512



Central Avenue

EXHIBIT A-13.2

Pump Station Calculations

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station
file=Osage Avenue EC-FC Hyd.xls

Existing conditions calculations for Osage Avenue Pump Station.

Step 1 - Determine pump operating capacity at the following conditions

Pump 1

River Elevations - 753.00 and 772.37
Sump Elevations - 747.75 and 753.75

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753 ft	753.75 ft	-0.75 ft
2	753 ft	747.75 ft	5.25 ft
3	772.37 ft	753.75 ft	18.62 ft
4	772.37 ft	747.75 ft	24.62 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avenue Pump Station
Head Losses
Pump 1

Flow	System Losses	Static Head			
GPM	ft	-0.75 ft	5.25 ft	18.62 ft	24.62 ft
1,000	0.20	-0.55 ft	5.45 ft	18.82 ft	24.82 ft
2,000	0.40	-0.35 ft	5.65 ft	19.02 ft	25.02 ft
3,000	0.80	0.05 ft	6.05 ft	19.42 ft	25.42 ft
4,000	1.50	0.75 ft	6.75 ft	20.12 ft	26.12 ft
5,000	2.20	1.45 ft	7.45 ft	20.82 ft	26.82 ft
6,000	3.20	2.45 ft	8.45 ft	21.82 ft	27.82 ft
7,000	4.25	3.50 ft	9.50 ft	22.87 ft	28.87 ft
8,000	5.60	4.85 ft	10.85 ft	24.22 ft	30.22 ft

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Existing conditions calculations for Osage Avenue Pump Station.

Step 1 (cont.) - Determine pump operating capacity at the following conditions

Pump 2

River Elevations - 753.00 and 772.37
Sump Elevations - 747.00 and 753.00

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753.00 ft	753.00 ft	0.00 ft
2	753.00 ft	747.00 ft	6.00 ft
3	772.37 ft	753.00 ft	19.37 ft
4	772.37 ft	747.00 ft	25.37 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avenue Pump Station Head Losses Pump 2					
Flow	System Losses	Static Head			
GPM	ft	0.00 ft	6.00 ft	19.37 ft	25.37 ft
1,000	0.20	0.20 ft	6.20 ft	19.57 ft	25.57 ft
2,000	0.40	0.40 ft	6.40 ft	19.77 ft	25.77 ft
3,000	0.80	0.80 ft	6.80 ft	20.17 ft	26.17 ft
4,000	1.50	1.50 ft	7.50 ft	20.87 ft	26.87 ft
5,000	2.20	2.20 ft	8.20 ft	21.57 ft	27.57 ft
6,000	3.20	3.20 ft	9.20 ft	22.57 ft	28.57 ft
7,000	4.25	4.25 ft	10.25 ft	23.62 ft	29.62 ft
8,000	5.60	5.60 ft	11.60 ft	24.97 ft	30.97 ft

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Existing conditions calculations for Osage Avenue Pump Station.

Step 2 - Determine pump station operating capacity at the following conditions

Pump Station Capacity

Case 1

River Elevation -	753.00		
Sump Elevations -	747.75		
Pump 1	7,250	gpm	@ 9.9ft head
Pump 2	7,250	gpm	@ 9.9ft head
	14,500	gpm	
	32.3	cfs	

Case 2

River Elevation -	772.37		
Sump Elevations -	747.75		
Pump 1	4,700	gpm	@ 26.32ft head
Pump 2	4,700	gpm	@ 26.32ft head
	9,400	gpm	
	20.9	cfs	

Case 3

River Elevation -	753.00		
Sump Elevations -	753.00		
Pump 1	7,700	gpm	@ 5ft head (slightly off chart)
Pump 2	7,700	gpm	@ 5ft head (slightly off chart)
	15,400	gpm	
	34.3	cfs	

Case 4

River Elevation -	772.37		
Sump Elevations -	753.00		
Pump 1	5,550	gpm	@ 22ft head
Pump 2	5,550	gpm	@ 22ft head
	11,100	gpm	
	24.7	cfs	

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Future conditions calculations for Osage Avenue Pump Station.

Step 1 - Determine pump operating capacity at the following conditions

Pump 1

River Elevations -	753.00	and	774.00
Sump Elevations -	747.75	and	753.75

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753 ft	753.75 ft	-0.75 ft
2	753 ft	747.75 ft	5.25 ft
3	774.00 ft	753.75 ft	20.25 ft
4	774.00 ft	747.75 ft	26.25 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avanue Pump Station Head Losses Pump 1					
Flow	System Losses	Static Head			
GPM	ft	-0.75 ft	5.25 ft	20.25 ft	26.25 ft
1,000	0.20	-0.55 ft	5.45 ft	20.45 ft	26.45 ft
2,000	0.40	-0.35 ft	5.65 ft	20.65 ft	26.65 ft
3,000	0.80	0.05 ft	6.05 ft	21.05 ft	27.05 ft
4,000	1.50	0.75 ft	6.75 ft	21.75 ft	27.75 ft
5,000	2.20	1.45 ft	7.45 ft	22.45 ft	28.45 ft
6,000	3.20	2.45 ft	8.45 ft	23.45 ft	29.45 ft
7,000	4.25	3.50 ft	9.50 ft	24.50 ft	30.50 ft
8,000	5.60	4.85 ft	10.85 ft	25.85 ft	31.85 ft

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Existing conditions calculations for Osage Avenue Pump Station.

Future conditions calculations for Osage Avenue Pump Station.

Pump 2

River Elevations - 753.00 and 774.00
Sump Elevations - 747.00 and 753.00

Static Heads

	River Elevation	Sump Elevation	Static Head
1	753.00 ft	753.00 ft	0.00 ft
2	753.00 ft	747.00 ft	6.00 ft
3	774.00 ft	753.00 ft	21.00 ft
4	774.00 ft	747.00 ft	27.00 ft

System loss curves for both pumps were developed and included in the design plans. This system curve is attached to the calculations.

Osage Avanue Pump Station Head Losses Pump 2					
Flow	System Losses	Static Head			
GPM	ft	0.00 ft	6.00 ft	21.00 ft	27.00 ft
1,000	0.20	0.20 ft	6.20 ft	21.20 ft	27.20 ft
2,000	0.40	0.40 ft	6.40 ft	21.40 ft	27.40 ft
3,000	0.80	0.80 ft	6.80 ft	21.80 ft	27.80 ft
4,000	1.50	1.50 ft	7.50 ft	22.50 ft	28.50 ft
5,000	2.20	2.20 ft	8.20 ft	23.20 ft	29.20 ft
6,000	3.20	3.20 ft	9.20 ft	24.20 ft	30.20 ft
7,000	4.25	4.25 ft	10.25 ft	25.25 ft	31.25 ft
8,000	5.60	5.60 ft	11.60 ft	26.60 ft	32.60 ft

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Future Conditions Hydraulics

Existing conditions calculations for Osage Avenue Pump Station.

Future conditions calculations for Osage Avenue Pump Station.

Pump Station Capacity

Case 1

River Elevation -	753.00	
Sump Elevations -	747.75	
Pump 1	7,250 gpm	@ 9.9ft head
Pump 2	7,250 gpm	@ 9.9ft head
	14,500 gpm	
	32.3 cfs	

Case 2

River Elevation -	774.00	
Sump Elevations -	747.75	
Pump 1	4,350 gpm	@ 28 ft head
Pump 2	4,350 gpm	@ 28 ft head
	8,700 gpm	
	19.4 cfs	

Case 3

River Elevation -	753.00	
Sump Elevations -	753.00	
Pump 1	7,700 gpm	@ 5ft head (slightly off chart)
Pump 2	7,700 gpm	@ 5ft head (slightly off chart)
	15,400 gpm	
	34.3 cfs	

Case 4

River Elevation -	774.00	
Sump Elevations -	753.00	
Pump 1	5,250 gpm	@ 23.7 ft head
Pump 2	5,250 gpm	@ 23.7 ft head
	10,500 gpm	
	23.4 cfs	

8000 PROPELLER PUMPS
PUMP PERFORMANCE

16″
8312

705 RPM

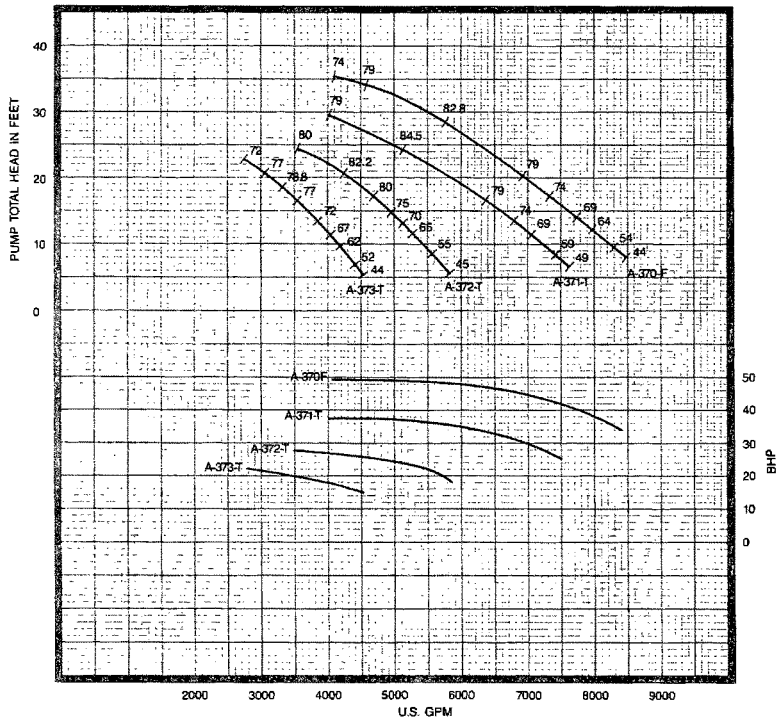
2 STAGE

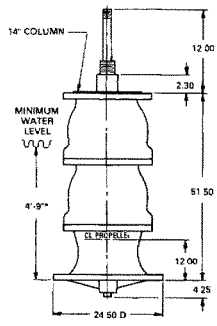
16″ COLUMN

16″ FABRICATED STEEL ELBOW

1-11/16″ LINESHAFT

2-1/2″ ENCLOSING TUBE





*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	1.6875 IN.	
MAXIMUM SPHERE SIZE	2.63 IN.	
Kt (THRUST FACTOR)	62 LBS./FT.	
Ka (TOTAL ROTOR WEIGHT)	132 LBS.	
Ks (SETTING CONSTANT)	7.6 LBS./FT.	
WK^2	28 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	1025 LBS.	
EYE AREA: PROPELLER NO. A-372-T	124.0 SQ. IN.	3 VANE
PROPELLER NO. A-373-T	124.0 SQ. IN.	3 VANE
PROPELLER NO. A-370-F	124.0 SQ. IN.	4 VANE
PROPELLER NO. A-371-T	124.0 SQ. IN.	3 VANE
PROPELLER NO.		
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

20" 8312

705 RPM

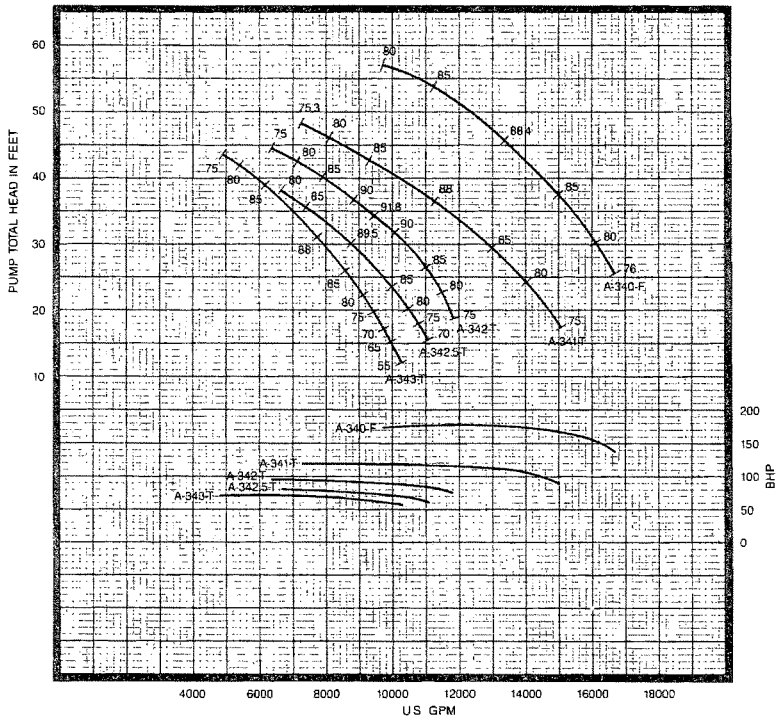
2 STAGE

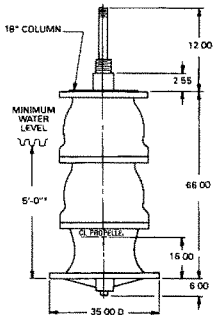
24" COLUMN

24" FABRICATED STEEL ELBOW

1-15/16" LINESHAFT

3" ENCLOSING TUBE





*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	1.9375 IN.	
MAXIMUM SPHERE SIZE	3.38 IN.	
Kt (THRUST FACTOR)	101 LBS./FT.	
Ka (TOTAL ROTOR WEIGHT)	250 LBS.	
Ks (SETTING CONSTANT)	10.0 LBS./FT.	
WK^2	86 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	2150 LBS.	
EYE AREA: PROPELLER NO. A-340-F	203.0 SQ. IN.	4 VANE
PROPELLER NO. A-341-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-342-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-342.5-T	203.0 SQ. IN.	3 VANE
PROPELLER NO. A-343-T	203.0 SQ. IN.	3 VANE
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

8000 PROPELLER PUMPS
PUMP PERFORMANCE

418

30″
8312

580 RPM

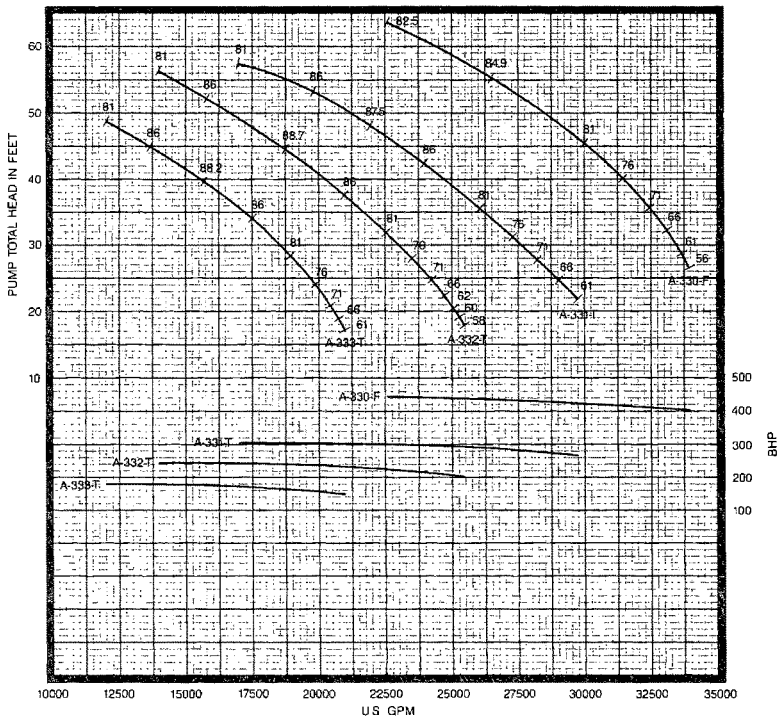
2 STAGE

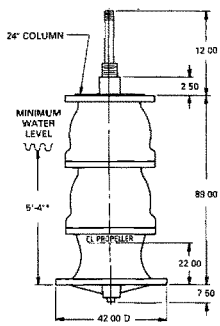
30″ COLUMN

30″ FABRICATED STEEL ELBOW

2-15/16″ LINESHAFT

5″ ENCLOSING TUBE





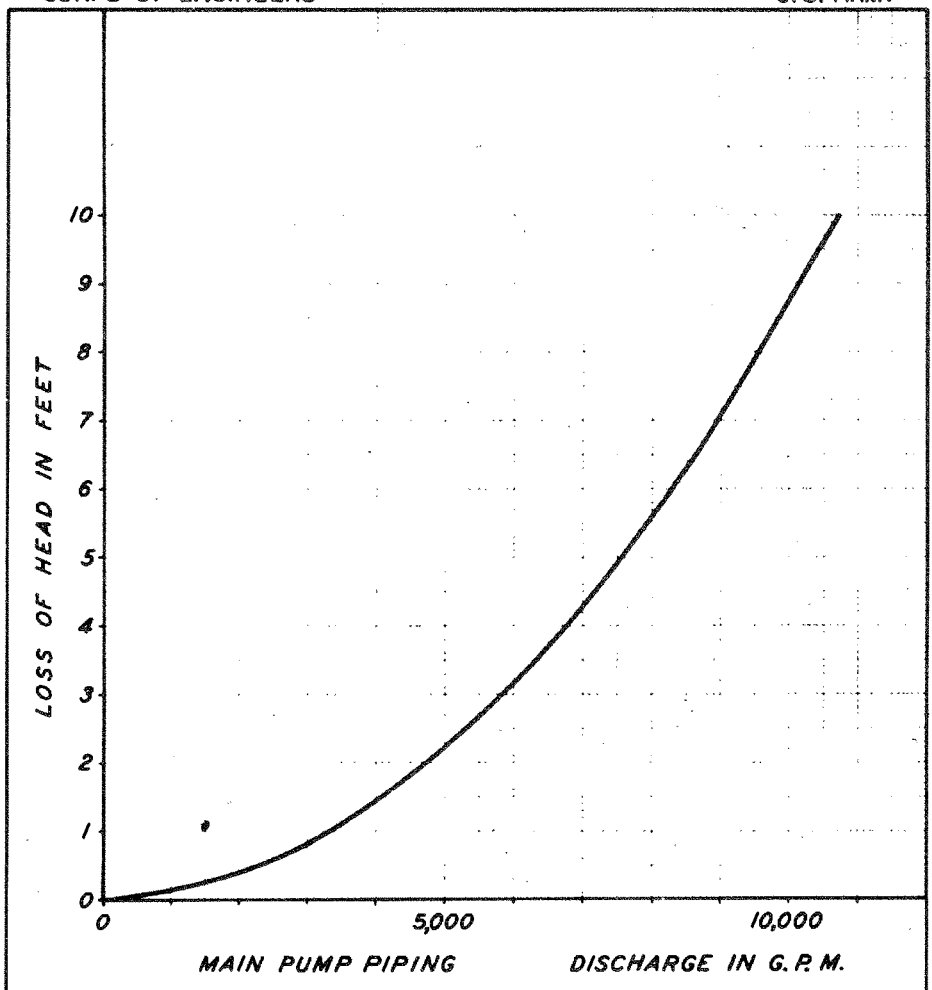
*This value is the minimum submergence required to prevent vortexing only. This value may need to be increased to provide adequate NPSHA.

DATA	VALUE	
PUMP SHAFT DIAMETER	2.9375 IN.	
MAXIMUM SPHERE SIZE	4.50 IN.	
K_t (THRUST FACTOR)	185 LBS./FT.	
K_a (TOTAL ROTOR WEIGHT)	620 LBS.	
K_s (SETTING CONSTANT)	23.0 LBS./FT.	
WK^2	390 LBS.-FT.^{2}	
BOWL ASSEMBLY WEIGHT	4300 LBS.	
EYE AREA: PROPELLER NO. A-330-F	367.8 SQ. IN.	4 VANE
PROPELLER NO. A-331-T	367.8 SQ. IN.	3 VANE
PROPELLER NO. A-332-T	367.8 SQ. IN.	3 VANE
PROPELLER NO. A-333-T	367.8 SQ. IN.	3 VANE
PROPELLER NO.		
PROPELLER NO.		

HYDRAULIC PERFORMANCE IS CONTINGENT ON FURNISHING THE PUMP WITH SPECIFIED AMOUNT OF CLEAR, FRESH, NON-AERATED WATER NOT TO EXCEED 85° F.

PUMP PERFORMANCE SHOWN IS BOWL ASSEMBLY WITH 10 FEET OF COLUMN INCLUDING A STANDARD ABOVE GROUND DISCHARGE ELBOW. ADDITIONAL COLUMN LOSSES SHOULD BE ADDED WHEN SETTINGS ARE DEEPER THAN 10 FEET AND/OR FOR OTHER DISCHARGE ARRANGEMENTS.

CORPS OF ENGINEERS U. S. ARMY



KANSAS CITYS FLOOD CONTROL PROJECT

ARMOURDALE UNIT

OSAGE AVENUE PUMPING PLANT

LOSS OF HEAD CURVE
MAIN PUMP PIPING

OFFICE, DISTRICT ENGINEER KANSAS CITY, MO.
KANSAS CITY DISTRICT APRIL 1952
FILE NO. A-10-3131

To accompany specifications dated: Sept. 1952

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station
file=Replacement Pump Station.xls

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells

Flow from each well	1.25	cfs
Number of wells	26	
Total system flow	32.5	cfs
Discharge depth below grade	1.5	ft

HEADER

Assumptions/Notes

Pump station is located at one end of the area of concern. Therforeone header systems will be installed.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Header Pipe Diameter	36	in
Flow Depth (Q_{max})	28.8	in
FL EL in	-1.5	ft
FL EL out	-6	ft
Length	1800	ft
Slope	0.0025	
Manning's "N" (RCP)	0.012	
Flow Area	873.0	in^2
Wetted Perimeter	79.7	in
Flow	35.31	cfs
Velocity	5.83	ft/s

MANHOLES

Assumptions/Notes
In order to account of entrance and exit losses through manholes each invert out will be set 3" lower than inlet invert.

El drop at each manhole	3	inches
Number of manholes	26	
Elevation Drop Through Manholes	6.5	ft

ELEVATION OF HEADER PIPE INVERT INTO PUMP STATION

Elevation below grade = (Pipe Length * Slope) + El drop through Manholes + Discharge depth + Pipe Diameter

Elevation below grade = 15.5 ft below grade

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

General information regarding project design.

Length of discharge line	150	ft
Design Low Water	740	ft (msl)
Highest Point in System	776	ft (msl)
Static Head	36	ft
Runoff Flow	16	cfs
Seepage Flow	32.5	cfs
Desired System Output	21,768	gal/min
Desired Pump Output	10,884	gal/min

Friction loss in discharge pipe can be found with the Hazen Williams Formula.

$h_f = (4.72*Q^{1.852}*L)/(C^{1.852}*D^{4.87})$ (AWWA M11 Eq. 3-2)

h_f	Headloss due to friction		
Q	Flow	24.25	ft^3/s
L	Length of discharge pipe	150	ft
C	Hazen-Williams Coefficient	100	
D	Inside pipe diameter		ft

Velocity head loss can be figured by the following.

$h_v = V^2/2g$ (AWWA M11 Eq. 3-8)

h_v	Headloss in feet		
V	Velocity		
g	acceleration due to gravity	32.2	ft/s^2

Minor losses were then found using the following assumptions. From AWWA M 11 Figure 3.5

Description	K	Number	$K_{subtotal}$
60 degree miter bend	0.6	4	2.4

$h_m = K(V^2/2g)$

The following table presents head required for various pipe sizes

Diameter	Velocity	Head Loss (ft)				
in	*ft/s*	Static	Friction	Velocity	Minor	Total
20	11.12	36	4.27	1.92	4.60	46.8
24	7.72	36	1.76	0.93	2.22	40.9
30	4.94	36	0.59	0.38	0.91	37.9
36	3.43	36	0.24	0.18	0.44	36.9
	Used in Calculations for system curve					

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

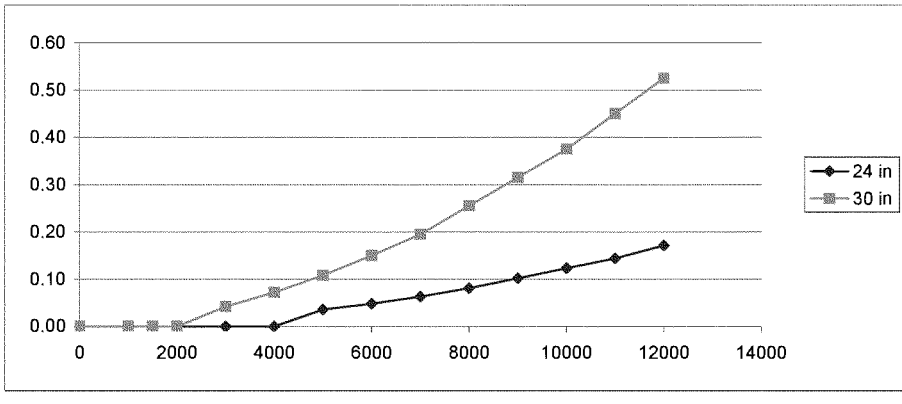
TASK **Osage Avenue Pump Station**
Replacement Pump Station

Column losses for this pump were determined using pg 276 of the Fairbanks Morse 8000 Propeller Pumps Application & Reference Data

The following table presents column losses in ft/100 of column for 20 and 24 inch columns.

	24 in		30 in	
GPM	per 100ft	**Total**	per 100ft	**Total**
0	0.00	**0.00**	0.00	**0.00**
1,000	0.00	**0.00**	0.00	**0.00**
1,500	0.00	**0.00**	0.00	**0.00**
2,000	0.00	**0.00**	0.00	**0.00**
3,000	0.14	**0.04**	0.00	**0.00**
4,000	0.24	**0.07**	0.00	**0.00**
5,000	0.36	**0.11**	0.12	**0.04**
6,000	0.50	**0.15**	0.16	**0.05**
7,000	0.65	**0.20**	0.21	**0.06**
8,000	0.85	**0.26**	0.27	**0.08**
9,000	1.05	**0.32**	0.34	**0.10**
10,000	1.25	**0.38**	0.41	**0.12**
11,000	1.50	**0.45**	0.48	**0.14**
12,000	1.75	**0.53**	0.57	**0.17**

* 30 feet of Column



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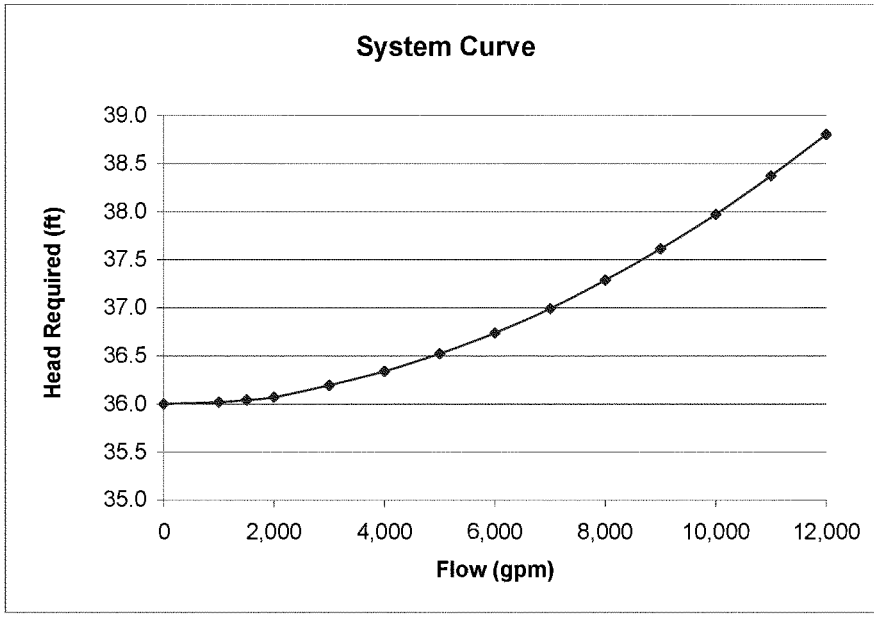
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

PRELIMINARY SYSTEM CURVE

Flow		Head			
GPM	ft^3/s	Static	Dynamic	Column	Total
0	0.00	36.00	0.00	0.00	36.00
1,000	2.23	36.00	0.02	0.00	36.02
1,500	3.34	36.00	0.04	0.00	36.04
2,000	4.46	36.00	0.07	0.00	36.07
3,000	6.68	36.00	0.15	0.04	36.19
4,000	8.91	36.00	0.27	0.07	36.34
5,000	11.14	36.00	0.41	0.11	36.52
6,000	13.37	36.00	0.59	0.15	36.74
7,000	15.60	36.00	0.79	0.20	36.99
8,000	17.82	36.00	1.03	0.26	37.29
9,000	20.05	36.00	1.30	0.32	37.61
10,000	22.28	36.00	1.59	0.38	37.97
11,000	24.51	36.00	1.92	0.45	38.37
12,000	26.74	36.00	2.28	0.53	38.80

*System Curve, given: 30 in discharge pipe



US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

Horsepower Required at top of shaft.

Brake Horsepower = [Total head * Flow (GPM)]/[3960 * Pump efficiency]

Total Head	38.37	ft
Flow	10,884	gpm
Pump Eff.	0.82	

Brake Horsepower = 129 hp

From pg 277 of Fairbanks Morse Pumps line shaft horse power loss per 100ft of 2-7/16" shaft is 0.72 at 580 RPM

Hp line shaft loss = (loss/100ft)*setting depth/100

Loss/100ft	0.96	hp
setting depth	30	ft

Hp line shaft loss = 0.29 hp

Pump thrust = head * Kt + Ka + Setting in ft * Ks

Total Head	38.37	ft
Kt	127	lb/ft
Ka	350	lb
setting depth	30	
Ks	15.9	lb/ft

Pump thrust = 5,700 lb

Loss through thrust bearing = .0075hp per 1,000lb at 100rpm

RPM	580	
Thrust	5,700	lbs

Loss through thrust bearing = 0.25 hp

Motor HP = (HP_{brake} + $HP_{lineloss}$ + $HP_{lossthrustbearing}$)

Motor HP = 129

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

Pump station discharge when inflow equal to 1/2 the flow rate of first pump.

Pump Discharge	24.25	cfs
	10,884	gpm
Max starts per hr	2	
V=tQ/4	10,912	ft^3

Pump Station Discharge inflow equal to 1/2 the difference of flow rate of first pump and second pump.

Pump Discharge	24.25	cfs
	10,884	gpm
Max starts per hr	2	
V=tQ/4	10,912	ft^3

In order to obtain required storage volume the following storage dimensions were used.

Storage Configuration		
Width	34	ft
Length	34	ft
Depth	10.0	ft
Volume	11,560	ft^3

Sump Depth

Sump Depth = Storage Depth + Pump Submergence + Pump Hieght off Floor + Inflow Pipe Invert

Storage Depth =	10.0	ft	
Pump Submergence =	5	ft	taken from pump literature
Pump Hieght off Floor =	20	in	assumed to be 1/2 bell diameter
Inflow Pipe Invert =	15.50	ft	below grade
Sump Depth =	32.2	ft	below grade
Assumed grade elevation =	766.5	ft	
Sump Floor Elevation =	734.3	ft	

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
Replacement Pump Station

EXCAVATION

Assumptions

Mudsill will be placed to work from.
Walls will be 1in thick for every foot of depth
Heel extensions will be provided to prevent uplift.
Floor thickness will be assumed

Calculations

Heel Extension Width	1.5	ft
Wall Thickness	33	in
Max Sump Depth	32	ft
Sump Floor Thickness	3	ft
Mudsill Thickness	1	ft
Max Excavation Depth	36	ft

Excavation Length 43 ft
Sump Length + 2*Wall Thickness + 2*Heel Width

Excavation Width 43 ft
Sump Width + 2*Wall Thickness + 2*Heel Width

Assume Excavation Slope of 1.5 (H):1(V)

Excavation Area at max depth 1,806 ft^2

Excavation Length at grade 139 ft
Excavation Width at grade 139 ft

Excavation Area at grade 19,321 ft^2

Excavation Volume 382,051 ft^3
14,150 yd^3
(Area at depth + Area at Grade)/Depth

Volume of pump station/Spoil 2,152 yd^3
(Area at Depth * Depth)

Volume to be Compacted 11,998 yd^3
(Excavation - Pump station Vol)

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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station
file=New Pump Station.xls

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells

Flow from each well	1.25	cfs
Number of wells	26	
Total system flow	32.5	cfs
Discharge depth below grade	1.5	ft

HEADER

Assumptions/Notes

Pump station will be located in center of area of concern. Therfore two header systems will be installed. Each system will handle 1/2 of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Header Pipe Diameter	24	in
Flow Depth (Q_{max})	19.2	in
FL EL in	-1.5	ft
FL EL out	-6	ft
Length	900	ft
Slope	0.005	
Manning's "N" (RCP)	0.012	
Flow Area	388.0	in^2
Wetted Perimeter	53.1	in
Flow	16.94	cfs
Velocity	6.29	ft/s

MANHOLES

Assumptions/Notes
In order to account of entrance and exit losses through manholes each invert out will be set 3" lower than inlet invert.

El drop at each manhole	3	inches
Number of manholes	13	
Elevation Drop Through Manholes	3.25	ft

ELEVATION OF HEADER PIPE INVERT INTO PUMP STATION

Elevation below grade = (Pipe Length * Slope) + El drop through Manholes + Discharge depth + Pipe Diameter

Elevation below grade = 11.25 ft below grade

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

General information regarding project design.

Length of discharge line	150	ft
Design Low Water	745	ft (msl)
Highest Point in System	776	ft (msl)
Static Head	31	ft
Desired System Output	14,585	gal/min
Desired Pump Output	7292.5	gal/min

Friction loss in discharge pipe can be found with the Hazen Williams Formula.

$h_f = (4.72*Q^{1.852}*L)/(C^{1.852}*D^{4.87})$ (AWWA M11 Eq. 3-2)

h_f	Headloss due to friction		
Q	Flow	16.25	ft^3/s
L	Length of discharge pipe	150	ft
C	Hazen-Williams Coefficient	100	
D	Inside pipe diameter		ft

Velocity head loss can be figured by the following.

$h_v = V^2/2g$ (AWWA M11 Eq. 3-8)

h_v	Headloss in feet		
V	Velocity		
g	acceleration due to gravity	32.2	ft/s^2

Minor losses were then found using the following assumptions. From AWWA M 11 Figure 3.5

Description	K	Number	$K_{subtotal}$
60 degree miter bend	0.6	4	2.4

$h_m = K(V^2/2g)$

The following table presents head required for various pipe sizes

Diameter	Velocity	Head Loss (ft)				
in	*ft/s*	Static	Friction	Velocity	Minor	Total
20	7.45	31	2.03	0.86	2.07	36.0
24	5.17	31	0.84	0.42	1.00	33.2
30	3.31	31	0.28	0.17	0.41	31.9
36	2.30	31	0.12	0.08	0.20	31.4

Used in Calculations for system curve

US Army Corps of Engineers
Kansas City District

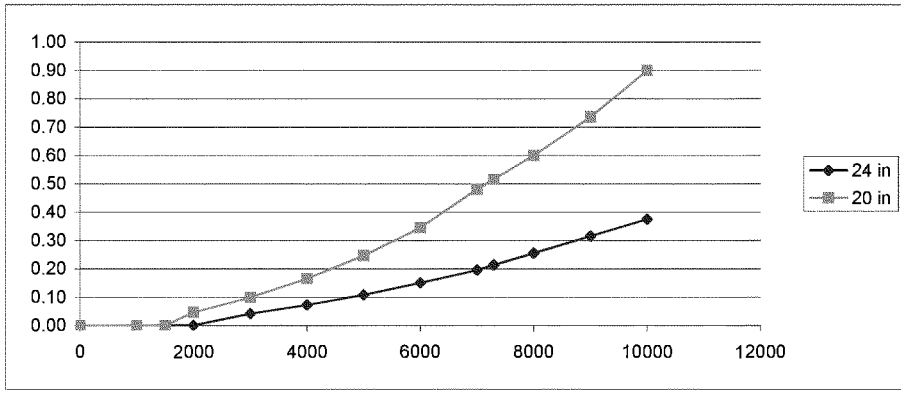
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Column losses for this pump were determined using pg 276 of the Fairbanks Morse 8000 Propeller Pumps Application & Reference Data

The following table presents column losses in ft/100 of column for 20 and 24 inch columns.

	20 in		24 in	
GPM	per 100ft	**Total**	per 100ft	**Total**
0	0.00	**0.00**	0.00	**0.00**
1,000	0.00	**0.00**	0.00	**0.00**
1,500	0.00	**0.00**	0.00	**0.00**
2,000	0.16	**0.05**	0.00	**0.00**
3,000	0.33	**0.10**	0.14	**0.04**
4,000	0.55	**0.17**	0.24	**0.07**
5,000	0.82	**0.25**	0.36	**0.11**
6,000	1.15	**0.35**	0.50	**0.15**
7,000	1.60	**0.48**	0.65	**0.20**
7,293	1.72	**0.52**	0.71	**0.21**
8,000	2.00	**0.60**	0.85	**0.26**
9,000	2.45	**0.74**	1.05	**0.32**
10,000	3.00	**0.90**	1.25	**0.38**
11,000	0.00	**0.00**	0.00	**0.00**
12,000	0.00	**0.00**	0.00	**0.00**
*	30 feet of Column			



US Army Corps of Engineers
Kansas City District

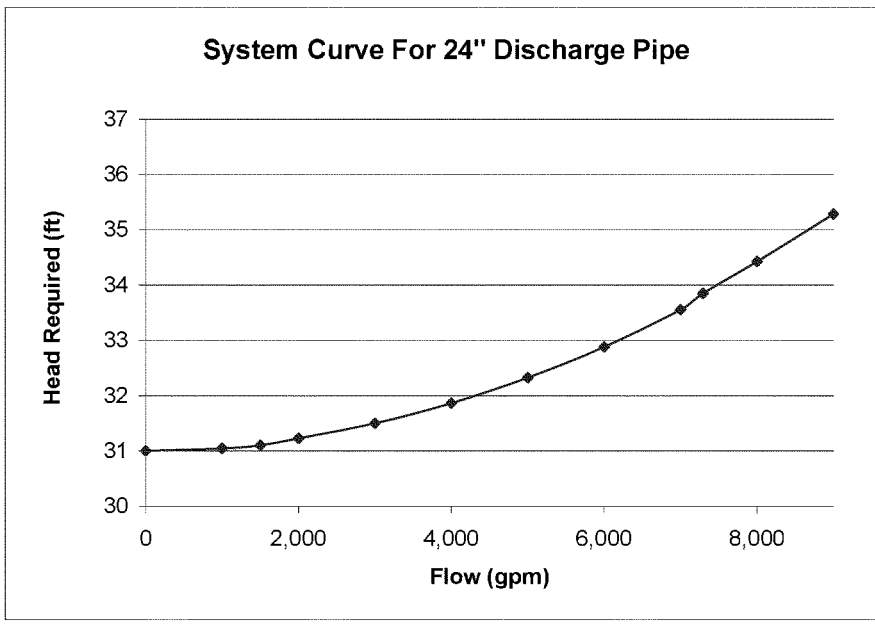
PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

PRELIMINARY SYSTEM CURVE

Flow		Head			
GPM	ft^3/s	Static	Dynamic	Column	Total
0	0.00	31.00	0.00	0.00	31.00
1,000	2.23	31.00	0.05	0.00	31.05
1,500	3.34	31.00	0.10	0.00	31.10
2,000	4.46	31.00	0.18	0.05	31.23
3,000	6.68	31.00	0.40	0.10	31.50
4,000	8.91	31.00	0.70	0.17	31.86
5,000	11.14	31.00	1.08	0.25	32.33
6,000	13.37	31.00	1.54	0.35	32.88
7,000	15.60	31.00	2.08	0.48	33.56
7,293	16.25	31.00	2.25	0.60	33.85
8,000	17.82	31.00	2.69	0.74	34.43
9,000	20.05	31.00	3.39	0.90	35.29

*System Curve, given: 24 in discharge pipe



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Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Horsepower Required at top of shaft.

Brake Horsepower = [Total head * Flow (GPM)]/[3960 * Pump efficiency]

Total Head	33.85	ft
Flow	7,293	gpm
Pump Eff.	0.80	

Brake Horsepower = 77.9 hp

From pg 277 of Fairbanks Morse Pumps line shaft horse power loss per 100ft of 1-15/16" shaft is 0.72 at 720 RPM

Hp line shaft loss = (loss/100ft)*setting depth/100

Loss/100ft	0.72	hp
setting depth	30	ft

Hp line shaft loss = 0.22 hp

Pump thrust = head * Kt + Ka + Setting in ft * Ks

Total Head	33.85	
Kt	101	lb/ft
Ka	250	lb
setting depth	30	
Ks	10	lb/ft

Pump thrust = 3,969 lb

Loss through thrust bearing = .0075hp per 1,000lb at 100rpm

RPM	705	
Thrust	3,969	lbs

Loss through thrust bearing = 0.21 hp

Motor HP = (HP_{brake} + $HP_{lineloss}$ + $HP_{lossthrustbearing}$)

Motor HP = 78.3

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

Pump station discharge when inflow equal to 1/2 the flow rate of first pump.

Pump Discharge	16.25	cfs
	7,293	gpm
Max starts per hr	3	
V=tQ/4	4,874	ft^3

Pump Station Discharge inflow equal to 1/2 the difference of flow rate of first pump and second pump.

Pump Discharge	16.25	cfs
	7,293	gpm
Max starts per hr	3	
V=tQ/4	4,874	ft^3

In order to obtain required storage volume the following storage dimensions were used.

Storage Configuration		
Width	25	ft
Length	25	ft
Depth	8.0	ft
Volume	5,000	ft^3

Sump Depth

Sump Depth = Storage Depth + Pump Submergence + Pump Hieght off Floor + Inflow Pipe Invert

Storage Depth =	8.0	ft	
Pump Submergence =	5	ft	taken from pump literature
Pump Hieght off Floor =	20	in	assumed to be 1/2 bell diameter
Inflow Pipe Invert =	11.25	ft	below grade
Sump Depth =	25.9	ft	below grade
Assumed grade elevation =	766.5	ft	
Sump Floor Elevation =	740.6	ft	

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Osage Avenue Pump Station**
New Pump Station

EXCAVATION

Assumptions

Mudsill will be placed to work from.
Walls will be 1in thick for every foot of depth
Heel extensions will be provided to prevent uplift.
Floor thickness will be assumed

Calculations

Heel Extension Width	1.5	ft
Wall Thickness	26	in
Max Sump Depth	26	ft
Sump Floor Thickness	3	ft
Mudsill Thickness	1	ft
Max Excavation Depth	30	ft

Excavation Length 32 ft
Sump Length + 2*Wall Thickness + 2*Heel Width

Excavation Width 32 ft
Sump Width + 2*Wall Thickness + 2*Heel Width

Assume Excavation Slope of 1.5 (H):1(V)

Excavation Area at max depth 1,045 ft^2

Excavation Length at grade 110 ft
Excavation Width at grade 110 ft

Excavation Area at grade 12,118 ft^2

Excavation Volume 196,908 ft^3
7,293 yd^3
(Area at depth + Area at Grade)/Depth

Volume of pump station/Spoil 1,003 yd^3
(Area at Depth * Depth)

Volume to be Compacted 6,289 yd^3
(Excavation - Pump station Vol)

TABLE 12.--SHAWNEE AVENUE SEWER SYSTEM

Summary of run-offs at various stages

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
													P O N D A G E[a]			
													Without pumping		With pumping present facilities	
Stage		Equiv. water surface elevation	Rainfall		Unit run-off		Peak run-off		Avg. supply rate	Mass run-off	Max.cap. sewer (gravity flow)	Pump cap.	Avg. depth	Duration	Avg. depth	Duration
Argentine Bridge gage	Kansas City gage		60-min. i.p.h.	15-min. i.p.h.	Imp. i.p.h.	Perv. i.p.h.	c.f.s.	i.p.h.	i.p.h.	inches	c.f.s.	c.f.s.	inches	min.	inches	min.
7.2	12.2	732.6(b)	2.50	4.98	4.42	1.50	399.0	2.84	3.52	0.94	145	-	0.39	55	-	-
12.5	16.0(c)	737.2	2.26	4.54	3.98	1.05	336.0	2.40	3.09	0.82	111	27.4	0.38	63	0.70	272
15.0	18.0	739.6	2.01	4.00	3.44	0.73	277.0	1.98	2.55	0.68	88	27.0	0.32	66	0.56	228
17.4	20.0	741.8	1.52	3.02	2.46	0.18	172.0	1.23	1.57	0.42	62	26.4	0.18	59	0.31	144
18.5	21.0	743.0	1.43	2.84	2.28	0.11	155.0	1.12	1.39	0.37	41	26.2	0.21	79	0.26	128
19.6	22.0	744.0	1.36	2.71	2.15	0.07	144.0	1.03	1.26	0.34	0	26.0	0.34		0.23	119
22.4	25.0	747.1	1.07	2.13	1.57	0	101.0	0.72	0.72	0.19	0	25.4	0.19		0.11	68
23.2	26.8	748.8	0.78	1.52	0.96	0	61.9	0.44	0.44	0.12	0	25.2	0.12		0.05	43
26.3	30.0	751.3	0.50	1.00	0.44	0	28.4	0.20	0.20	0.05	0	24.7	0.05		Negligible	10
27.6	32.0	752.8	0.40	0.80	0.24	0	15.5	0.11	0.11	0.03	0	24.2	0.03		0.0	0
31.8	40.8(d)	758.8	0.40	0.80	0.24	0	39.4	0.17	0.17	0.05	0	22.0	0.05		0.02	41

Total tributary area is 140.03 acres of which 64.44 acres are impervious.

(a) Pondages were computed by reducing the pump and sewer capacities by the amount of sanitary sewage (1.9 c.f.s.)
(b) Intrados of the 57-inch outlet at the pumping plant
(c) Present pump starting stage
(d) Refer to paragraph 24g

NOTE: There is no longer sanitary flow, and the seepage flow is intercepted at the levee toe by relief well system. Extent of drainage basin is essentially unchanged from 1947. Therefore, use coincident storm flow of 15.5 cfs.

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Table 12
Page 45

f. Shawnee Avenue Pumping Plant.

(1) The Shawnee Avenue Pumping Plant at station 231+39 is deteriorated and obsolete. A new outlet and pumping plant just upstream to this location is being built by Kansas City, Kansas, and engineered by Schlup, Becker & Brennan, Consulting Engineers, at an estimated cost of $650,000. Construction is expected during 1971. The proposed outlet plans indicate a 7-1/2- by 7-1/2-foot RCB with integral sluice gate and pumping plant within the levee fill. A newly proposed storm sewer system along Shawnee Avenue, in conjunction with urban renewal, will increase the delivery of stormflow to the outlet. A railroad fill across Shawnee Avenue near the levee denies overland flow to the outlet structure. The outlet will discharge a relatively high frequency which cannot be pinpointed due to lack of data. Four vertical, mixed flow propeller pumps; three 30-inch pumps driven by 200 hp. motors; ~~and one 20-inch pump driven by a 100 hp. motor will be installed.~~ Using the proposed pumps, the plant will have a capacity of 141 c.f.s. when pumping with river at top of levee elevation 765.2 feet from a pit level of 734.0 feet, m.s.l., the average of ~~pump starting elevations. The pumps will discharge 185 c.f.s.~~ with river stage at elevation 740. The elevation 740 becomes a significant river stage because at that stage the pumping plant capability is equal to the gravity outlet capability. Therefore, with the river above this stage more stormflow can be discharged by pumping, and below this stage more can be discharged by gravity (assuming the temporary ponding within the protected area is comparable). The maximum estimated pressure relief well seepage of 35 c.f.s. with river stage at 765.2 will decrease to zero at stage 745. The proposed capability of the storm pumping plant when gaged by the 1948 Supplement approaches a flood recurrence interval of 100 years.

(2) The existing Shawnee Avenue Pumping Plant is located landward of the levee at station 231+39. The pumping plant is equipped with two pumps, one 16-inch vertical centrifugal sewage pump driven by a 100 hp. electric motor, and one 10-inch horizontal sewage pump driven by a 40 hp. motor. Both pumps are of the dry pit type. The plant has a capacity of 21.5 c.f.s. with river at top of levee elevation 765.2 feet, m.s.l., when pumping from the pump starting elevation of 737.2 feet, m.s.l. At a river stage of 747.2 feet, m.s.l., and the above pit elevation, the pumping capacity is 26.2 c.f.s.

NOTE: The pump curves provided indicate slightly less capacity than indicated here. Need to verify exactly which pumps we have.....OK for existing condition either way, but if the pumps are as described on this page, may be adequate capacity for n500+3 design condition. As analyzed (i.e. with the smaller pumps, per pump curves provided), there is insufficient capacity at n500+3 max river stage condition requiring pump upgrades.

Colt Industries Fairbanks Morse Pump Division

Shawnee Ave Pumps

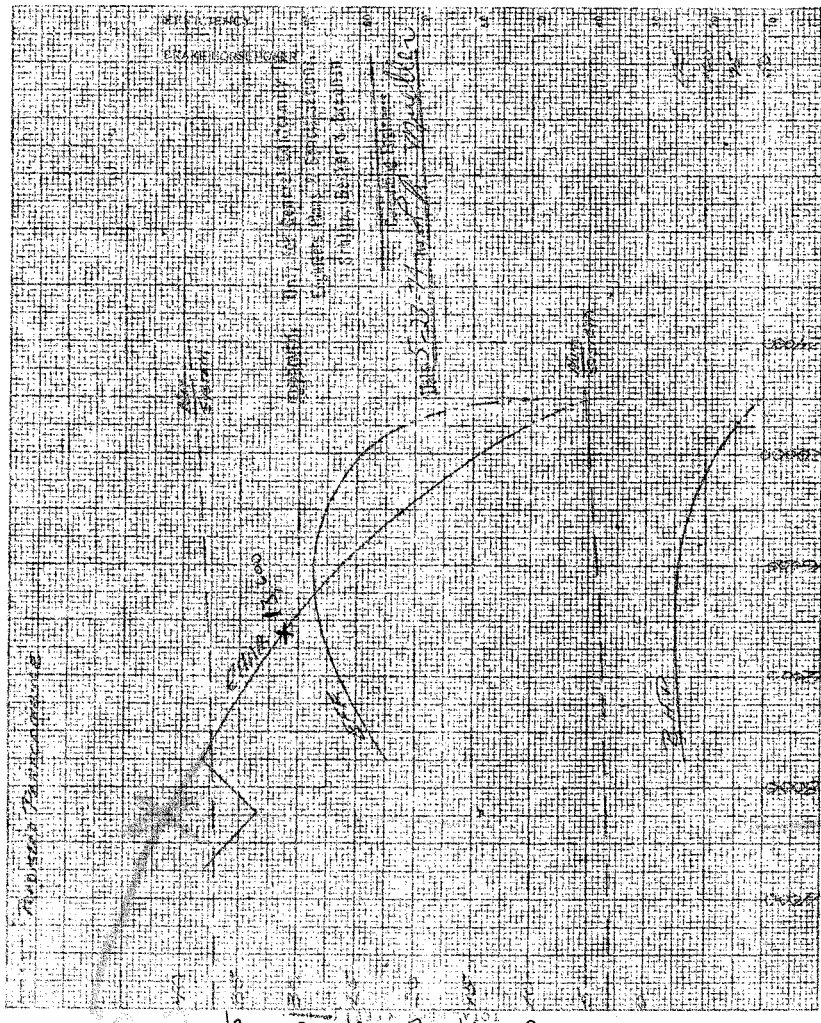
CERTIFIED CURVE

PUMP NO. CWD-071097-1 STAGES 1 SIZE-FIGURE #20-6360

TESTED 5/14/74 DRIVER Test Mtr. IMPELLER F2F9W

Certified Correct by [signature] ENGR. 5/74 DATE IMP. DIA. 22 3/32" R.P.M. 705

CERTIFIED PUMP PERFORMANCE CURVE



13-74

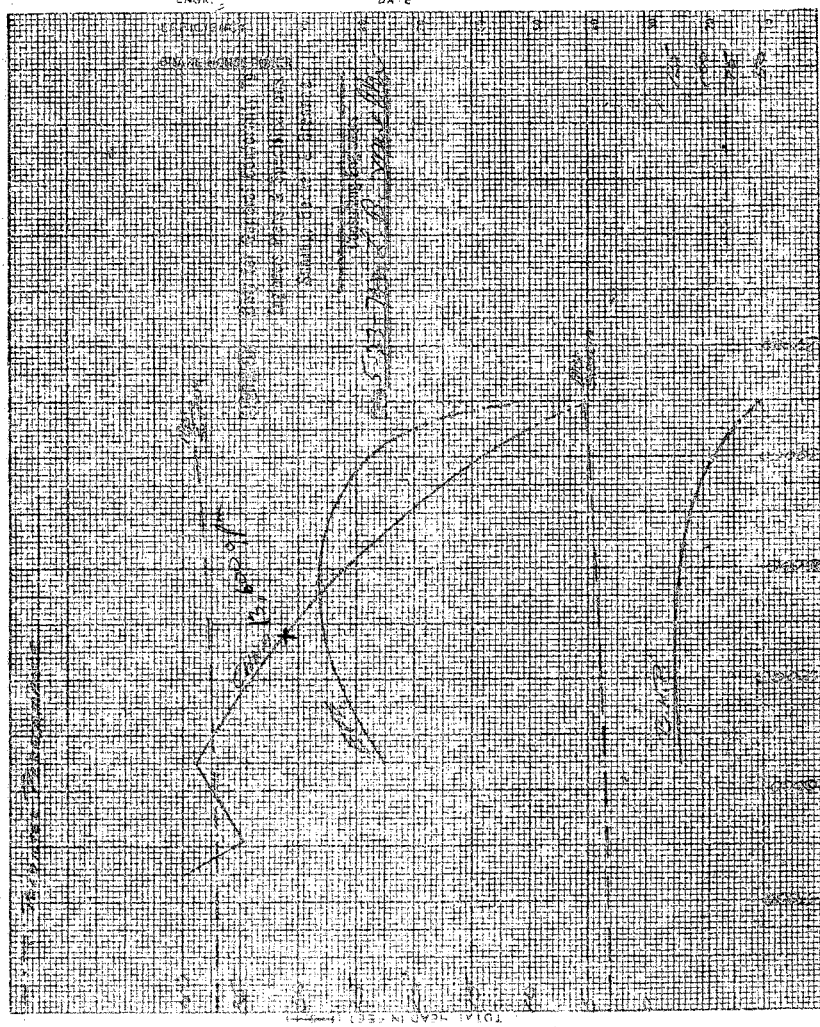
Gulf Industries

Fairbanks Morse
Pump Division

CERTIFIED CURVE

PUMP NO. F242-071099-0 STAGES 1 SIZE-FIGURE #20-6560
TESTED 5/13/74 DRIVER Test Mtr. IMPELLER F2F9W
IMP. DIA. 22 3/32
Certified Correct by [signature] ENGR. 5/74 DATE R.P.M. 705

CERTIFIED PUMP PERFORMANCE CURVE

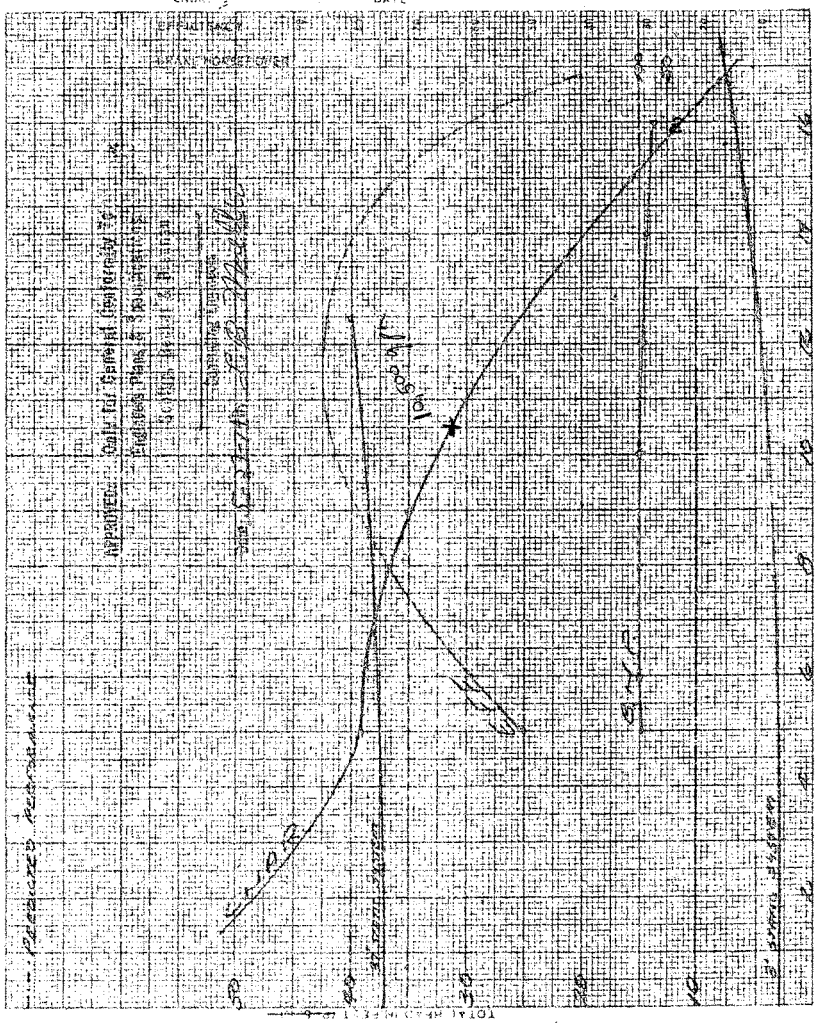


13-75

Colt Industries — Fairbanks Morse Pump Division

CERTIFIED CURVE

PUMP NO. KZVZ-071150 STAGES 1 SIZE-FIGURE #18 - 6360
TESTED 5/16/74 DRIVER TMAE - 150 IMPELLER F2E9C
IMP. DIA. 19 1/2
Certified Correct by C.R. Bould (ENGR.) 5/20/74 (DATE) R.P.M. 880



CERTIFIED PUMP PERFORMANCE CURVE

U. S. GALLONS PER MINUTE X 1000

13-76

Colt Industries — Fairbanks Morse Pump Division

CERTIFIED CURVE

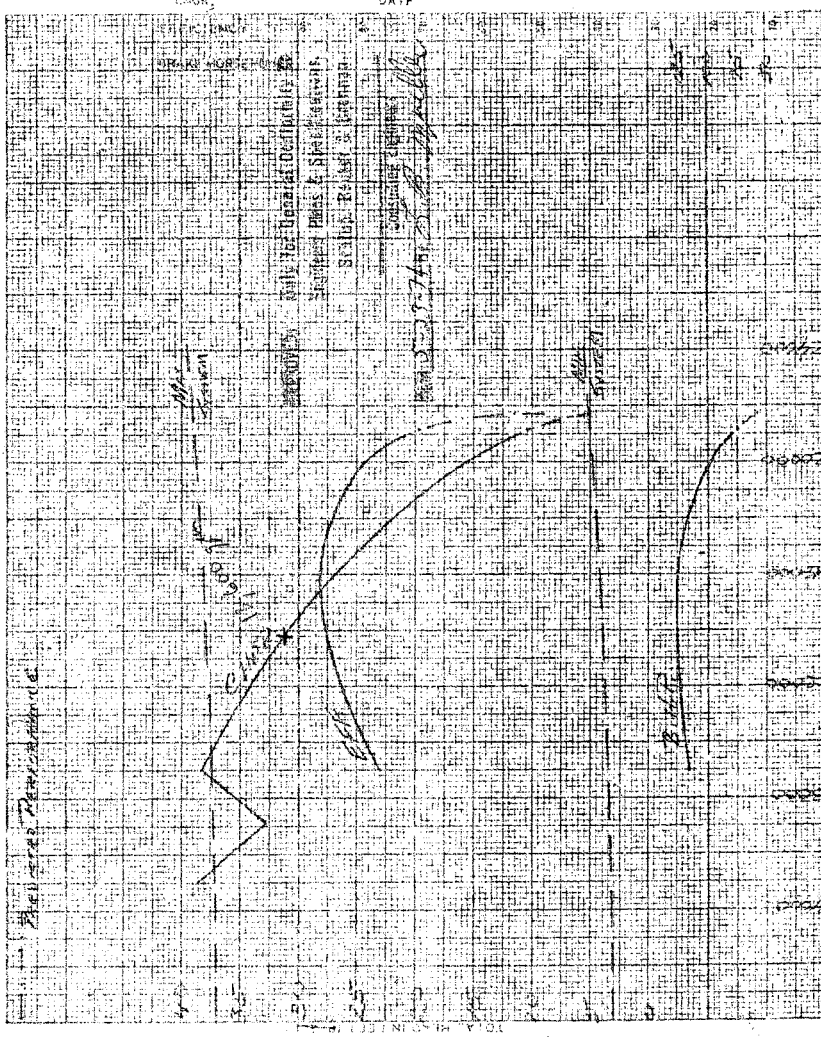
PUMP NO. K2V2-C7[illegible]-2 STAGES 1 SIZE-FIGURE #20-6360

TESTED [illegible] DRIVER [illegible] IMPELLER E2E9W

IMP. DIA. 22 3/32

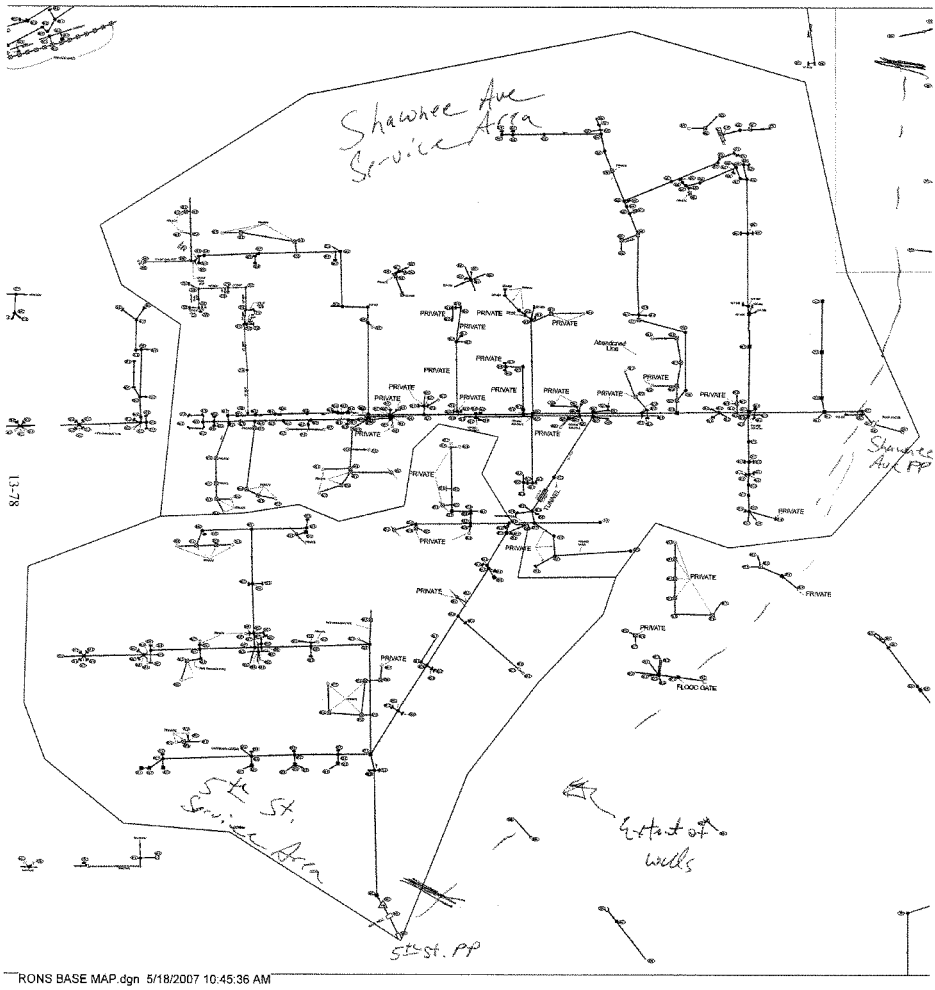
Certified Correct by [illegible] DATE R.P.M. 765

CERTIFIED PUMP PERFORMANCE CURVE



U.S. GALLONS PER MINUTE

13-77



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13-78

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Pump Curve Table
Pump Number 1
(Test Pump # K2V2-071100)

Head	Capacity gpm
6.5	17,100
10.0	16,300
12.0	15,800
20.0	
25.0	
30.0	11,000
35.0	8,800
37.0	8,000
40.0	
42.0	3,700
50.0	

Pump Curve Table
Pump Number 2-4

Head	K2V2-071099-0 Capacity gpm	K2V2-071099-1 Capacity gpm	K2V2-071099-2 Capacity gpm	Average Capacity gpm
5.0	21,000	21,000	20,700	20,900
10.0	1			1
12.0	20,600	20,600	20,800	20,667
20.0	1			1
25.0	1			1
30.0	14,200	14,200	14,400	14,267
35.0	11,400	11,200	11,400	11,333
37.0	10,200	9,800	9,900	9,967
40.0	1			1
45.0	1			1
50.0	1			1

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
New Pump Station

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding relief wells
This is the n500+3 design condition for the EXISTING line of relief wells

Flow from each well	0.75 cfs
Number of wells	47
Total system flow	35 cfs

HEADER

Assumptions/Notes

Pump station is located within the of area of concern. Therfore two header systems service the pump station. Each system will handle a portion of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Reach Description	Number of wells	Flow
From sta 190+75 to pump station	27	20.1 cfs
From pump station to sta 246+35	20	14.9 cfs

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 1 - Determine pump operating Ranges

Pump Number	Start Elevation	Stop Elevation	Description
1	732	724	$19^{1}/_{2}$" Impeller @ 880 RPM
2	733	725	$22^{3}/_{32}$" Impeller @ 705 RPM
3	734	726	$22^{3}/_{32}$" Impeller @ 705 RPM
4	735	727	$22^{3}/_{32}$" Impeller @ 705 RPM

Assumptions
Since pump outlet piping is short (<10') dynamic heads were assumed to be negligible
Average pump curve developed for pumps 2-4 based on supplied pump curves.

Step 2 - Determine head pumps will operate at

Case 1A - Low river stage with large interior event (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	735	5 *	17,100	20,900	79,800	178

* Off chart for pump 1 used 6.5' head

Case 1B - Low river stage with large interior event (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
740	728	12	15,800	20,650	77,750	173

Case 2A - High river stage (max capacity)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
765.2	735	30.2	11,000	14,250	53,750	120

Case 2B - High river stage (min capacity with all pumps running)

River EL. ft	Sump El ft	Head ft	Discharge gpm Pump 1	Pumps 2-4 (ea)	Total gpm	cfs
765.2	728	37.2	8,000	9,950	37,850	84

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Existing Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 3 - Determine pumping capacity for interior Drainage

Case Number	Total Flow cfs	Seepage Flow cfs	Pumped Interior Drainage cfs
1	178	0	178
2	173	0	173
3	120	35 *	85
4	84	35 *	49

Step 4 - Determine if sufficient redundancy for seepage flow exists.

Redundancy requirements state 100% capacity is required in the event of any one pump failure. Critical pump failure would be pump 2, 3 or 4.

Interior drainage flow for case 4	15.5 cfs*
Total inflow to station for case4	50.5 cfs
Pump station capacity at case 4	84 cfs
Capcity of pump 2 at case 4	22 cfs
Pump station capacity with pump 2 failure	62 cfs

REDUNDANCY REQUIREMENT MET

Assumptions
* 15.5cfs value obtained from 1949 Supplement on Interior Drainage, and considers coincident storm only. Any seepage flow from original design is now intercepted by exist. wells.

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
New Pump Station

Relief Wells and Collector Pipe

WELLS

The following information was supplied by EC-GD regarding EXISTING relief wells but now flowing under higher head associated with n500+3

Flow from each well	1	cfs
Number of wells	47	
Total system flow	47	cfs

THERE IS ALSO a proposed line of surface discharging wells flowing at a total of 40cfs.
Roughly HALF of this flow will be in the basin flowing back to the Shawnee Ave pump plant.

HEADER

Assumptions/Notes

Pump station is located within the of area of concern. Therfore two header systems service the pump station. Each system will handle a portion of the flow.
Per UHC 3-240-07FA header pipe is designed to flow at 80% depth of pipe diameter.

Reach Description	Number of wells	Flow	
From sta 190+75 to pump station	27	27	cfs
From pump station to sta 246+35	20	20	cfs

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Future Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 1 - Determine pump operating Ranges

Pump Number	Start Elevation	Stop Elevation	Description
1	732	724	$19^{1}/_{2}$" Impeller @ 880 RPM
2	733	725	$22^{3}/_{32}$" Impeller @ 705 RPM
3	734	726	$22^{3}/_{32}$" Impeller @ 705 RPM
4	735	727	$22^{3}/_{32}$" Impeller @ 705 RPM

Assumptions
Since pump outlet piping is short (<10') dynamic heads were assumed to be negligible
Average pump curve developed for pumps 2-4 based on supplied pump curves.

Step 2 - Determine head pumps will operate at

Case 3A - Low river stage with large interior event (max capacity)

River EL.	Sump El	Head	Discharge gpm		Total	
ft	ft	ft	Pump 1	Pumps 2-4 (ea)	gpm	cfs
740	735	5 *	17,100	20,900	79,800	178

* Off chart for pump 1 used 6.5' head

Case 3B - Low river stage with large interior event (min capacity with all pumps running)

River EL.	Sump El	Head	Discharge gpm		Total	
ft	ft	ft	Pump 1	Pumps 2-4 (ea)	gpm	cfs
740	728	12	15,800	20,650	77,750	173

Case 4A - High river stage (max capacity)

River EL.	Sump El	Head	Discharge gpm		Total	
ft	ft	ft	Pump 1	Pumps 2-4 (ea)	gpm	cfs
770.0	735	35	8,800	11,300	42,700	95

Case 4B - High river stage (min capacity with all pumps running)

River EL.	Sump El	Head	Discharge gpm		Total	
ft	ft	ft	Pump 1	Pumps 2-4 (ea)	gpm	cfs
770.0	728	42	3,700	7,000	24,700	55

off chart - values are extrapolated
NOTE that this is less than optimum efficiency, and outside of desired range of pumps

US Army Corps of Engineers
Kansas City District

PROJECT Kansas City Seven Levees

TASK **Shawnee Avenue Pump Station**
Future Conditions

Existing conditions calculations for Shawnee Avenue Pump Station.

Step 3 - Determine pumping capacity for interior Drainage

Case Number	Total Cap'y cfs	Below-Grade Seepage Flow cfs	Above-grade Seepage Flow cfs	Capacity for Interior Drainage Flow cfs
1	178	0	0	178
2	173	0	0	173
3	95	47	20	28
4	55	47	20	-12

Step 4 - Determine if sufficient redundancy for seepage flow exists.

Redundancy requirements state 100% capacity is required in the event of any one pump failure. Critical pump failure would be pump 2, 3 or 4.

Interior drainage flow for case 4	15.5 cfs*
Surface discharge relief well flow	20 cfs
Total inflow to station for case4	83 cfs
Pump station capacity at case 4	55 cfs
Capcity of pump 2 at case 4	22 cfs
Pump station capacity with pump 2 failure	33 cfs

REDUNDANCY REQUIREMENT NOT MET

Step 5 - Select new pumps to meet redundancy requirements.

Pump 1 - remain unchanged
Pump 2 - Select FM 30" 580RPM 2 stage with A-331-T impellers.
Pump 3 - Select FM 30" 580RPM 2 stage with A-331-T impellers.
Pump 4 - Select FM 30" 580RPM 2 stage with A-331-T impellers.

Check for clear space between pump bowls. 1 bowl diameter required.
Pump centerline spacing 96" is adquate for spacing of 42" bowls.

Check for sumbergance 5'-4" required plus 1/2 bowl diameter off floor

Floor El	716 ft	
Distance from floor	40 in	
Submergance depth	64 in	
Submergance EL	724.7 ft	Borderline Pump 2 stop at 725ft Consider raising operational levels

Assumptions
* 15.5cfs value obtained from 1949 Supplement on Interior Drainage, and considers coincident storm only. Any seepage flow from original design is now intercepted by exist. wells.

EXHIBIT A-13.3

Geotechnical Data (Hydraulic Gradeline Calculations)

Geotechnical Data

Hydraulic Grade Lines for Pump Plants

In addition to the overall underseepage analysis performed for the Armourdale Unit, the underseepage conditions were also evaluated specifically for all pump plants using the underseepage criteria discussed previously. The underseepage was evaluated for existing conditions, the n500+0, n500+3 and n500+5 river levels. For feasibility level detail, the cross section of the protection was not changed to reflect changes required for higher raises. The results of this analysis should still reasonably reflect the underseepage conditions that should be expected for river levels higher than existing conditions, and should be conservative.

The purpose of the analysis was to provide the Structural Engineers with an excess head value at the base of the natural blanket for the different river levels at each pump plant location. To accomplish this, hydraulic grade lines were computed at each pump plant location for each river level.

Pump Plants Analyzed for Underseepage

Pump Plant	Approximate Station
Osage	76+80
12th Street	129+20
Mill Street	156+75
5th Street	185+70
Midwest Cold Storage	194+50
Shawnee	230+80
Kansas City Southern	276+80
PBI Gordon	289+60
National Beef	295+80
Central Avenue	299+20

USAGE PUMP PLANT

FEB 2007

Armourdale Kansas - Hydraulic Grade Line for the Osage Pump Station Sta 76+80

ARM Osage PP 76+00.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniskowski EC-GD
Date : 2/14/2007

Checked by : Bellew EG-GD
Date : 02/26/2007 GMB

NOTES:
1. Replace floodwall with floodwall proposed
2. L_3 not revised for minor change due to floodwall raise
3. Design includes fully penetrating cutoff wall from station 62+00 to 83+00

Elevation Used

Top of Existing Levee:	772.4 (From O&M Manual)
500 year + 0':	774.0
500 year + 3':	777.0
500 year + 5':	779.0
Ground Surface Land Side:	760.0
Ground Surface River Side:	750.0 (Average blanket thickness over slope)
Bottom of Blanket:	736.0
Top of Bedrock:	668.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K0/Kb)
SM	100
ML	200 to 600
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K0/Kb) * Df * Db]^{1/2}$
D_{bc}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat.	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_1	= length of landside blanket	where L_1	$= C * (e^{(X/C)} - 1) / (e^{(L/C)} - 1)$
H	= max head or levee height	L_2	= landside effective length
L_2	= levee base width	L_T	= total effective length
		L_T'	= Total Effective Length + 1/2 of Berm

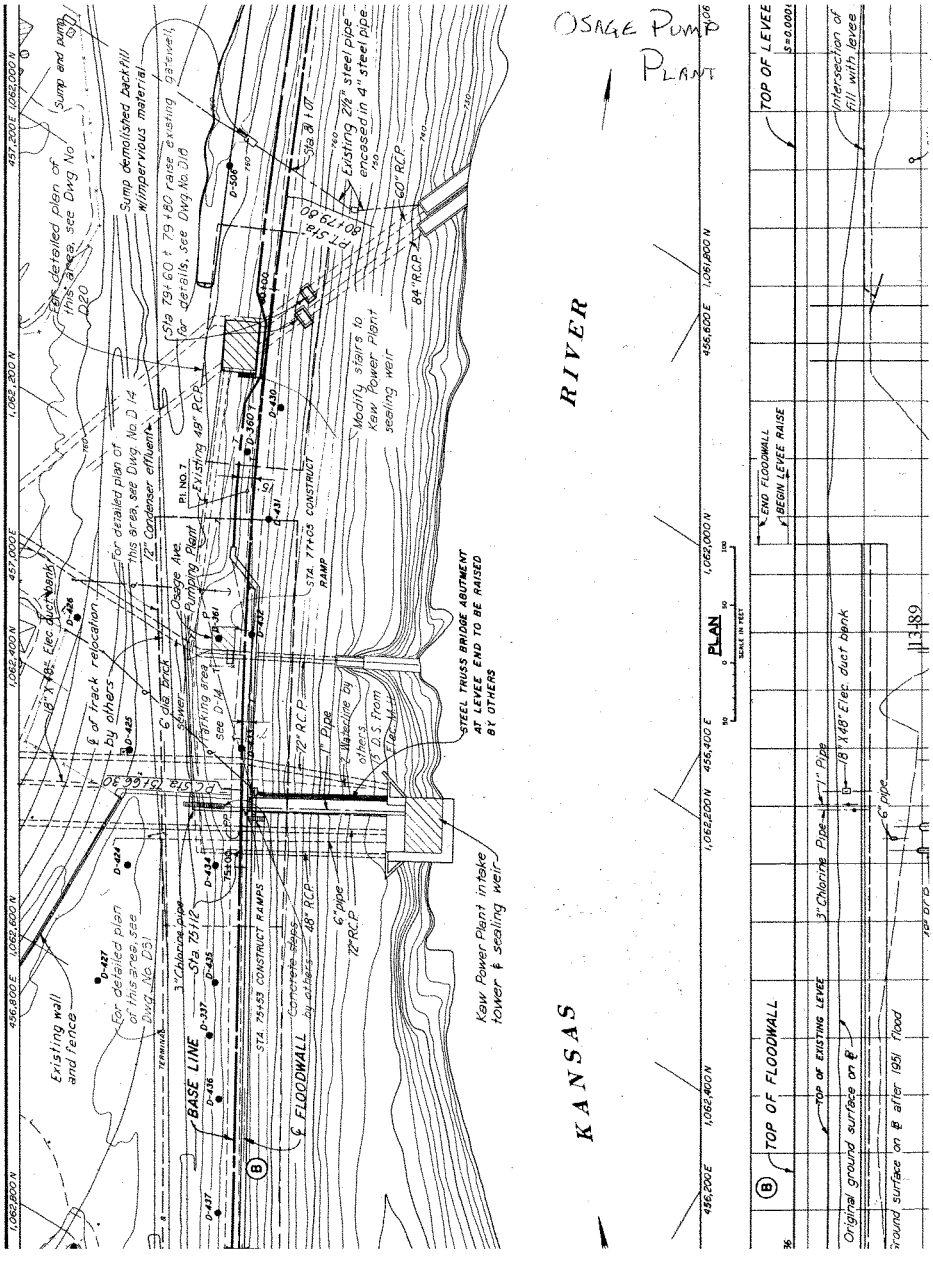
Input Parameters

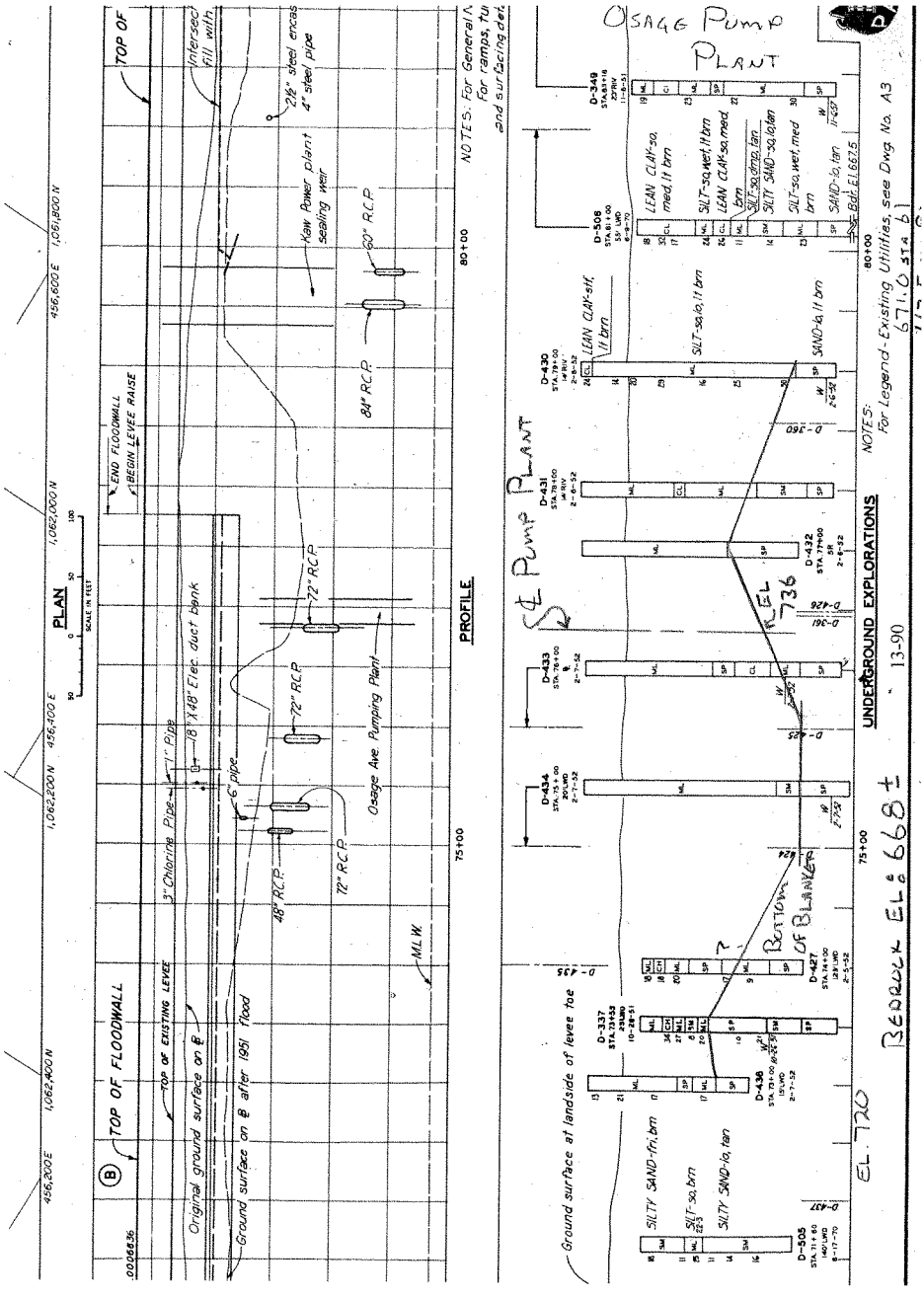
Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bc} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
76+80	Existing Conditions		300	300	24.0	19.0	14.0	68.0	100	12.4	EC
76+80	n500 + 0		300	300	24.0	19.0	14.0	68.0	100	14.0	n500
76+80	n500 +3		300	300	24.0	19.0	14.0	68.0	100	17.0	n500+3
76+80	n500 + 5		300	300	24.0	19.0	14.0	68.0	100	19.0	n500+5

Analysis

C_R Feet	C_L Feet	L_{T1} Feet	L_{T2} Feet	L_R Feet	L_T Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ratio		
534	700	99	53	700	849	10.22	0.43	0.84	1.97		
534	700	99	680	700	1479	6.03	0.28	0.84	3.04		
534	700	99	680	700	1479	8.05	0.34	0.84	2.51		
534	700	99	680	700	1479	8.99	0.37	0.84	2.24		

555

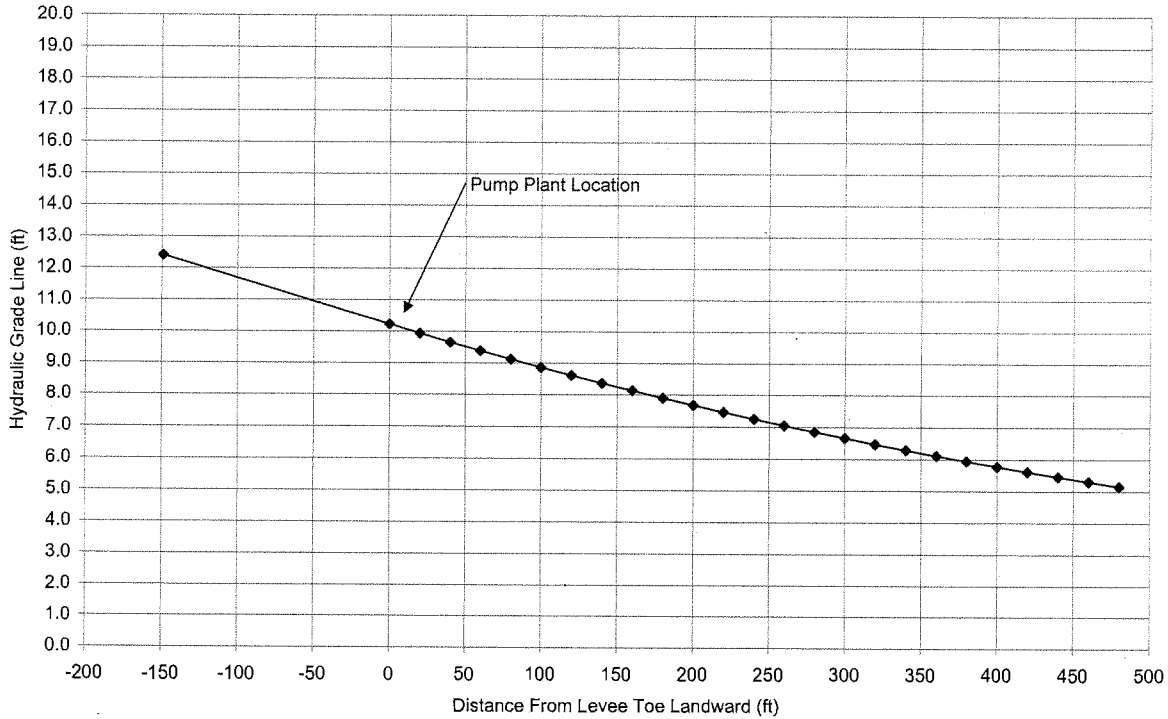

OSAGE PUMP PLANT
KANSAS RIVER
PLAN
SCALE IN FEET
BASE LINE
℄ FLOODWALL
STA. 75+53 CONSTRUCT RAMPS
STA. 77+05 CONSTRUCT RAMP
STEEL TRUSS BRIDGE ABUTMENT AT LEVEE END TO BE RAISED BY OTHERS
Kaw Power Plant intake tower & sealing weir
Existing wall and fence
Sump demolished backfill w/impervious material
Sta 79+60 ± 79+80 raise existing gatewell, for details, see Dwg. No. D18
Existing 2½" steel pipe encased in 4" steel pipe
Modify stairs to Kaw Power Plant sealing weir
Osage Ave. Pumping Plant
Existing 48" R.C.P.
12" Condenser effluent
For detailed plan of this area, see Dwg. No. D 14
For detailed plan of this area, see Dwg. No. D51
For detailed plan of this area, see Dwg. No. D20
Sump and pump
℄ of track relocation by others
18" X 48" Elec. duct bank
6" dia. brick sewer
Parking area see D-14
72" R.C.P.
1" Pipe
2" Waterline by others
3" D.S. from Elec. H.H.
3" Chlorine pipe
Sta. 75+12
Concrete steps by others
48" R.C.P.
6" pipe
72" R.C.P.
60" R.C.P.
84" R.C.P.
Sta. 81+07
PT. Sta. 80+79.80
PI. NO. 7
TERMINAL
TOP OF FLOODWALL
TOP OF EXISTING LEVEE
Original ground surface on ℄
Ground surface on ℄ after 1951 flood
3" Chlorine Pipe
1" Pipe
18" X 48" Elec. duct bank
6" pipe
END FLOODWALL
BEGIN LEVEE RAISE
TOP OF LEVEE
Intersection of fill with levee
13-89


PLAN
SCALE IN FEET
TOP OF FLOODWALL
TOP OF EXISTING LEVEE
Original ground surface on ℄
Ground surface on ℄ after 1951 flood
3" Chlorine Pipe
1" Pipe
18" X 48" Elec. duct bank
6" pipe
48" R.C.P.
72" R.C.P.
Osage Ave. Pumping Plant
END FLOODWALL
BEGIN LEVEE RAISE
Kaw Power plant sealing weir
2½" steel encas
4" steel pipe
Intersect fill with
60" R.C.P.
84" R.C.P.
M.L.W.
PROFILE
NOTES: For General N
For ramps, tu
and surfacing det
OSAGE PUMP PLANT
℄ PUMP PLANT
EL 736
BOTTOM OF BLANKET
Ground surface at landside of levee toe
SILTY SAND-fri, brn
SILT-so, brn
SILTY SAND-lo, tan
LEAN CLAY-sa, med, lt brn
SILT-so, wet, lt brn
LEAN CLAY-so, med, brn
SILT-so, dmp, tan
SILTY SAND-so, lo, tan
SILT-so, wet, med brn
SAND-lo, tan
Bdr. El. 667.5
LEAN CLAY-stf, lt brn
SILT-so, lo, lt brn
SAND-lo, lt brn
EL. 720
BEDROCK EL. 668±
UNDERGROUND EXPLORATIONS
13-90
NOTES:
For Legend-Existing Utilities, see Dwg. No. A3
671.0 STA 61

Hydraulic Grade Line for the Osage Pump Plant
Existing Conditions

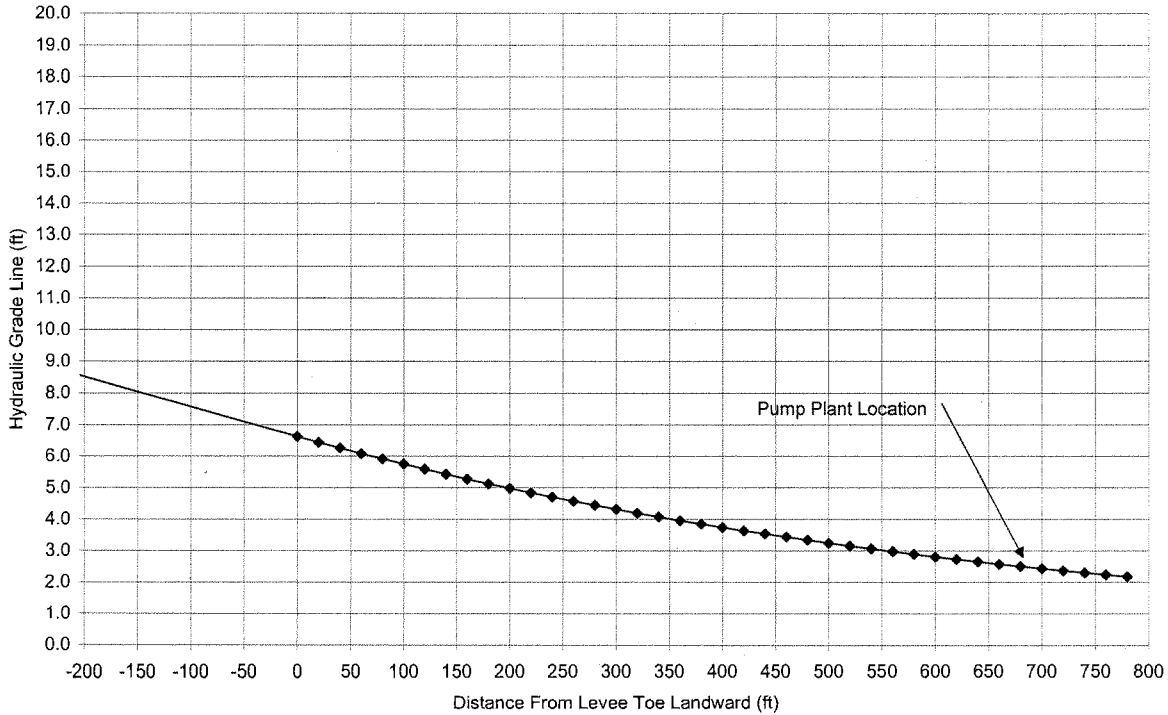
Elevation Top of levee (MSL)	772.4	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	12.4		
L1 (ft)	99		
L2 (ft)	50		
Le (ft)	700		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-149	12.40	772.40
Pump Plant Location	0	10.22	770.22
	20	9.94	769.94
	40	9.66	769.66
	60	9.38	769.38
	80	9.12	769.12
	100	8.86	768.86
	120	8.61	768.61
	140	8.37	768.37
	160	8.13	768.13
	180	7.91	767.91
	200	7.68	767.68
	220	7.47	767.47
	240	7.26	767.26
	260	7.05	767.05
	280	6.85	766.85
	300	6.66	766.66
	320	6.47	766.47
	340	6.29	766.29
	360	6.11	766.11
	380	5.94	765.94
	400	5.77	765.77
	420	5.61	765.61
	440	5.45	765.45
	460	5.30	765.30
	480	5.15	765.15

Existing Conditions Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



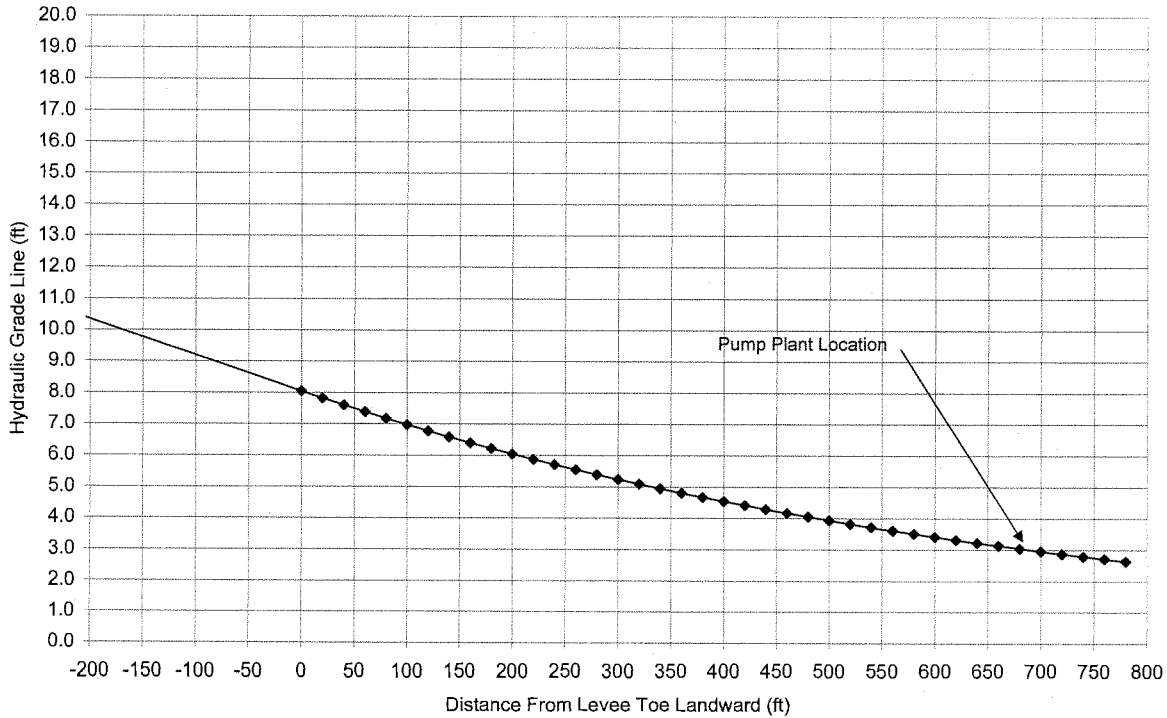
Hydraulic Grade Line for the Osage Pump Plant Design Conditions n500+0 with cutoff wall			
Elevation Top of levee (MSL)	774	n500+0	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	14		
L1 (ft)	99		
L2 (ft)	680		
Le (ft)	700		
ho	6.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-779	14.00	774.00
	0	6.63	766.63
	20	6.44	766.44
	40	6.26	766.26
	60	6.08	766.08
	80	5.91	765.91
	100	5.74	765.74
	120	5.58	765.58
	140	5.42	765.42
	160	5.27	765.27
	180	5.12	765.12
	200	4.98	764.98
	220	4.84	764.84
	240	4.70	764.70
	260	4.57	764.57
	280	4.44	764.44
	300	4.32	764.32
	320	4.19	764.19
	340	4.08	764.08
	360	3.96	763.96
	380	3.85	763.85
	400	3.74	763.74
	420	3.64	763.64
	440	3.53	763.53
	460	3.43	763.43
	480	3.34	763.34
	500	3.24	763.24
	520	3.15	763.15
	540	3.06	763.06
	560	2.98	762.98
	580	2.89	762.89
	600	2.81	762.81
	620	2.73	762.73
	640	2.65	762.65
	660	2.58	762.58
Pump Plant Location	680	2.51	762.51
	700	2.44	762.44
	720	2.37	762.37
	740	2.30	762.30
	760	2.24	762.24
	780	2.17	762.17

Design Condition n500+0 Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



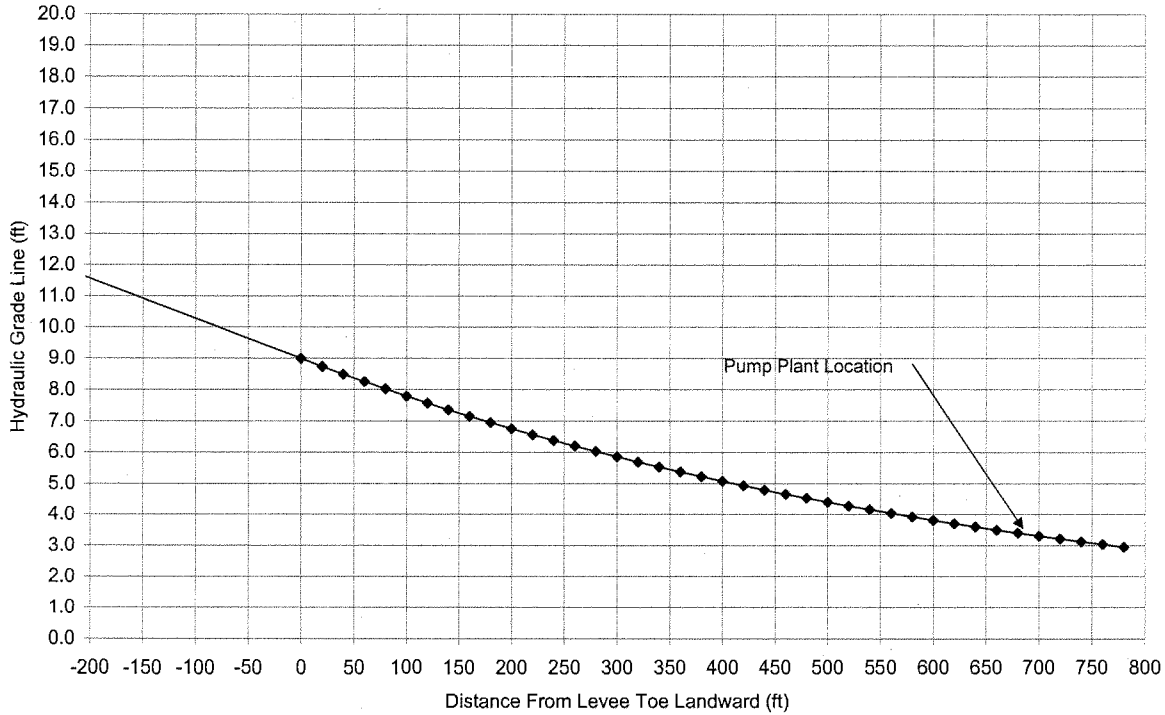
Hydraulic Grade Line for the Osage Pump Plant Design Conditions n500+3 with cutoff wall			
Elevation New Top of levee (MSL)	777	n500 +3	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	17		
L1 (ft)	99		
L2 (ft)	680		
Le (ft)	700		
ho	8.05		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-779	17.00	777.00
	0	8.05	768.05
	20	7.82	767.82
	40	7.60	767.60
	60	7.38	767.38
	80	7.18	767.18
	100	6.97	766.97
	120	6.78	766.78
	140	6.59	766.59
	160	6.40	766.40
	180	6.22	766.22
	200	6.04	766.04
	220	5.87	765.87
	240	5.71	765.71
	260	5.55	765.55
	280	5.39	765.39
	300	5.24	765.24
	320	5.09	765.09
	340	4.95	764.95
	360	4.81	764.81
	380	4.67	764.67
	400	4.54	764.54
	420	4.41	764.41
	440	4.29	764.29
	460	4.17	764.17
	480	4.05	764.05
	500	3.94	763.94
	520	3.83	763.83
	540	3.72	763.72
	560	3.61	763.61
	580	3.51	763.51
	600	3.41	763.41
	620	3.32	763.32
	640	3.22	763.22
	660	3.13	763.13
Pump Plant Location	680	3.04	763.04
	700	2.96	762.96
	720	2.88	762.88
	740	2.79	762.79
	760	2.72	762.72
	780	2.64	762.64

Design Condition n500+3 Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



Hydraulic Grade Line for the Osage Pump Plant Design Conditions n500+5 with cutoff wall			
Elevation New Top of levee (MSL)	779	n500 +5	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	736		
Driving Head (ft)	19		
L1 (ft)	99		
L2 (ft)	680		
Le (ft)	700		
ho	8.99		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-779	19.00	779.00
	0	8.99	768.99
	20	8.74	768.74
	40	8.49	768.49
	60	8.25	768.25
	80	8.02	768.02
	100	7.79	767.79
	120	7.57	767.57
	140	7.36	767.36
	160	7.15	767.15
	180	6.95	766.95
	200	6.76	766.76
	220	6.57	766.57
	240	6.38	766.38
	260	6.20	766.20
	280	6.03	766.03
	300	5.86	765.86
	320	5.69	765.69
	340	5.53	765.53
	360	5.38	765.38
	380	5.22	765.22
	400	5.08	765.08
	420	4.93	764.93
	440	4.79	764.79
	460	4.66	764.66
	480	4.53	764.53
	500	4.40	764.40
	520	4.28	764.28
	540	4.16	764.16
	560	4.04	764.04
	580	3.93	763.93
	600	3.81	763.81
	620	3.71	763.71
	640	3.60	763.60
	660	3.50	763.50
Pump Plant Location	680	3.40	763.40
	700	3.31	763.31
	720	3.21	763.21
	740	3.12	763.12
	760	3.03	763.03
	780	2.95	762.95

Design Condition n500+5 Hydraulic Grade Line for the Osage Pump Plant Sta 76+80 Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the 12th Street Pump Station Sta 129+20

ARM 12th Street PP 129+20.xls

NOMENCLATURE 12TH ST. PUMP PLANT

FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kaczniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD GMB
Date : 02/12/07

NOTES:
1. Landside levee raise proposed
2. L_2 not revised for minor change due to levee raise

Elevation Used

Top of Existing Levee:	769.5 (From O&M Manual)
500 year + 0':	771.4
500 year + 3':	774.4
500 year + 5':	776.4
Ground Surface Land Side:	760.0
Ground Surface River Side:	742.0
Bottom of Blanket:	734.0
Top of Bedrock:	671.5

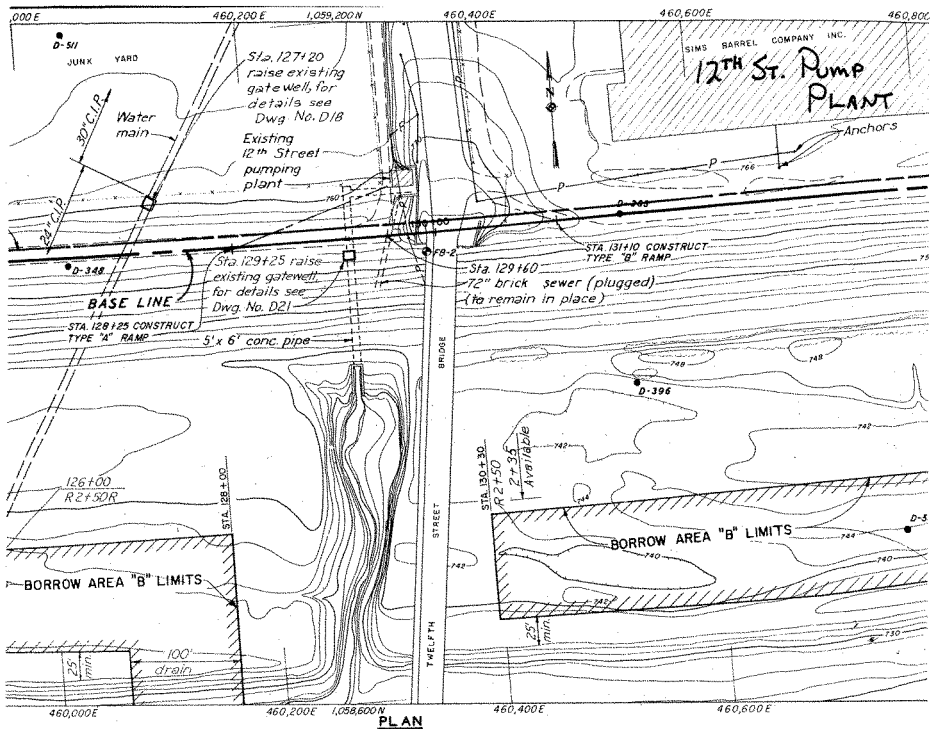
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(f/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundati	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	= $C * (e^{(2.8)(C-1)}) / (e^{(2.8)(C-1)})$
H	= max head or levee height	L_e	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_1	= Total Effective Length + 1/2 of Berm

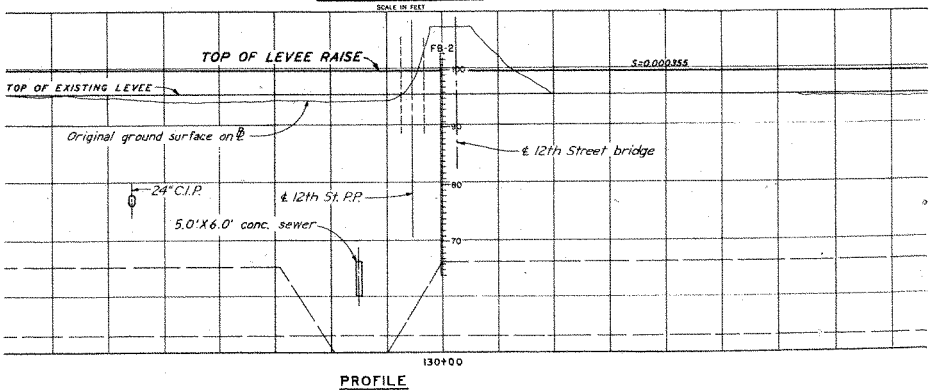
Station			Inpot Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
129+20	Existing Conditions		300	300	26.0	17.0	8.0	62.5	20	9.5	EC
129+20	n500 + 0		300	300	26.0	17.0	8.0	62.5	20	11.4	n500
129+20	n500 +3		300	300	26.0	17.0	8.0	62.5	20	14.4	n500+3
129+20	n500 + 5		300	300	26.0	17.0	8.0	62.5	20	16.4	n500+5

Analysis												
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io			
387	698	20	120	698	838	7.91	0.30	0.84	2.76			
387	698	20	120	698	838	9.50	0.37	0.84	2.30			
387	698	20	120	698	838	12.00	0.46	0.84	1.83			
387	698	20	120	698	838	13.66	0.53	0.84	1.60			

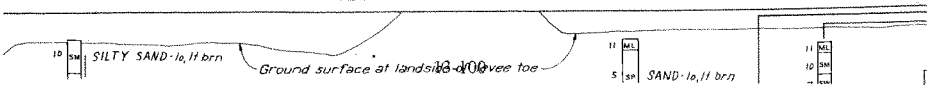


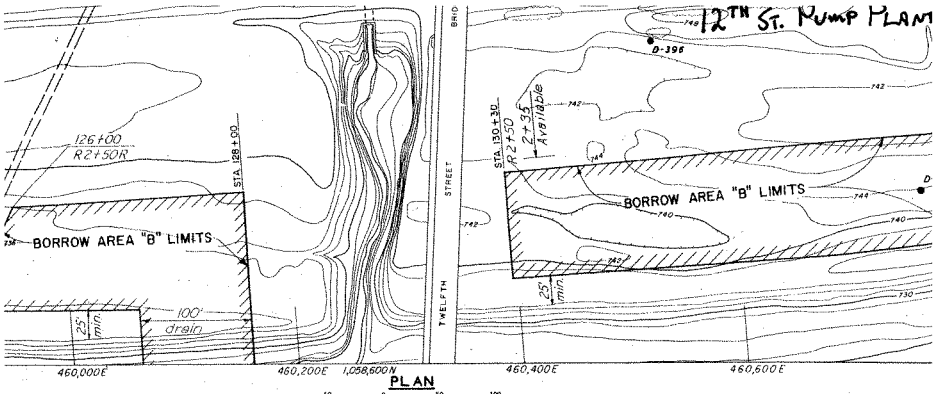
PLAN

SCALE IN FEET



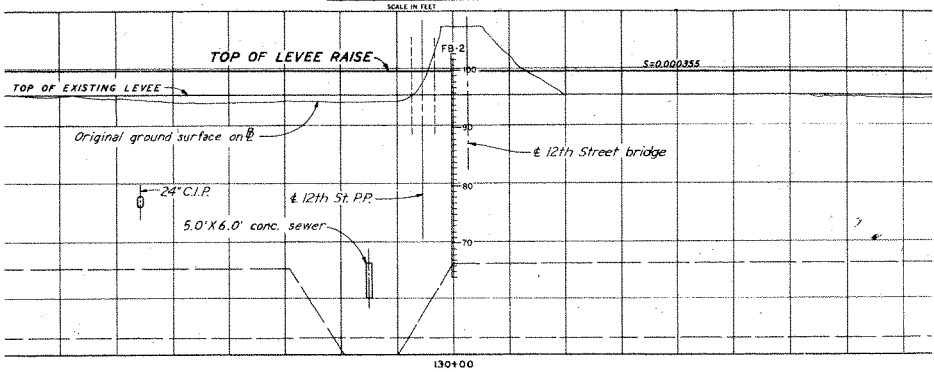
PROFILE



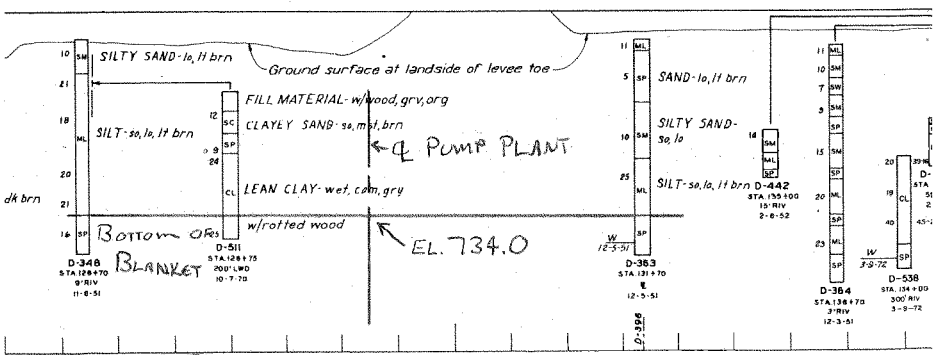


PLAN

SCALE IN FEET



PROFILE



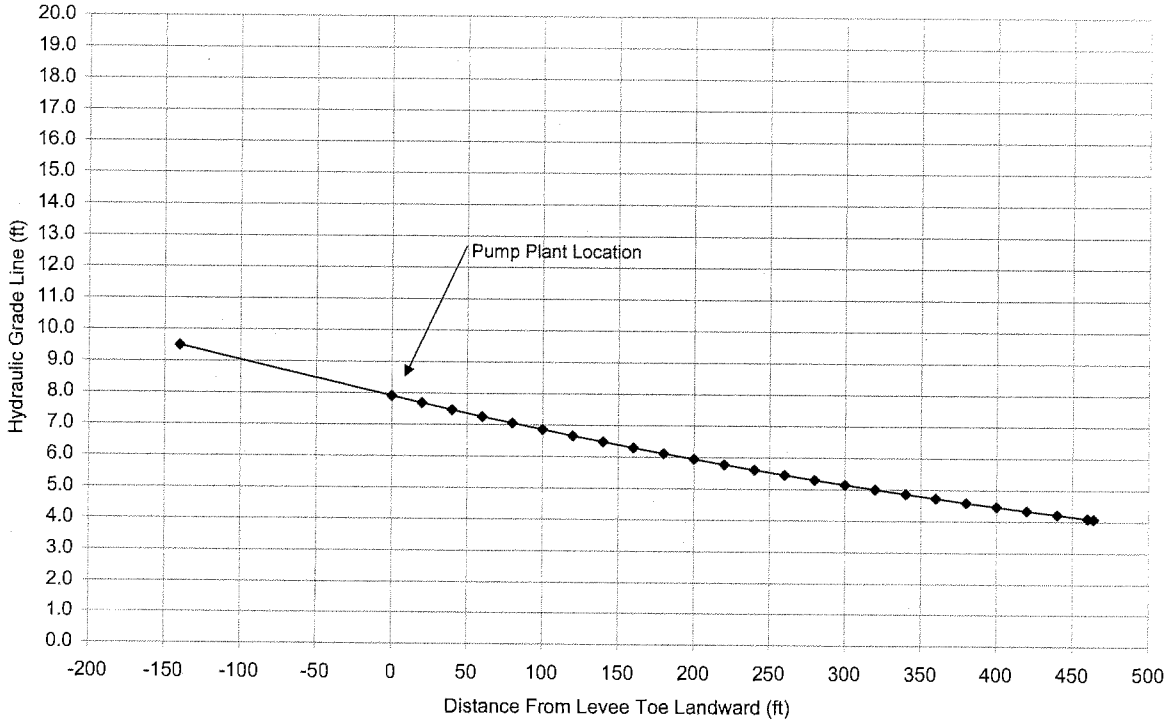
UNDERGROUND EXPLORATIONS

NOTES:
For ramps, turnouts, turnaround and surfacing details see Dwg. No C7

TOP OF BEDROCK ≈ 671.5 ± 13-101 667.5 STA 80 672.6 STA 140

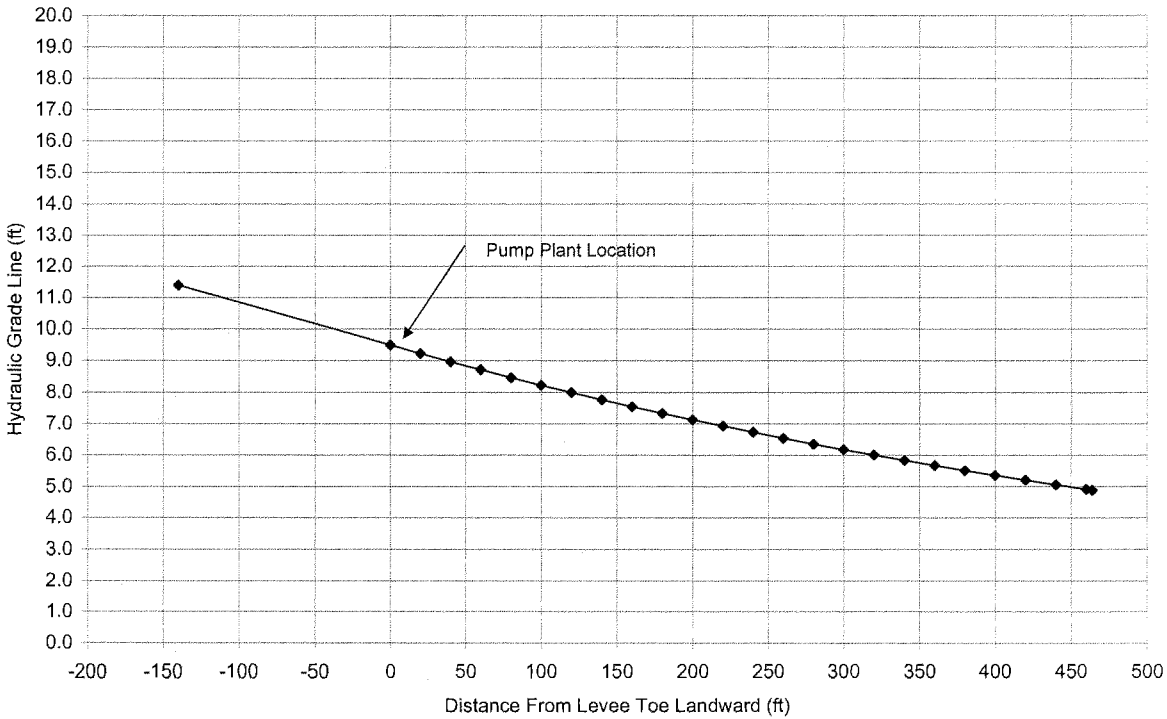
Hydraulic Grade Line for the 12th Street Pump Plant Existing Conditions			
Elevation Top of levee (MSL)	769.5	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	9.5		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	7.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-140	9.50	769.50
Pump Plant Location	0	7.91	767.91
	20	7.69	767.69
	40	7.47	767.47
	60	7.26	767.26
	80	7.06	767.06
	100	6.86	766.86
	120	6.66	766.66
	140	6.48	766.48
	160	6.29	766.29
	180	6.12	766.12
	200	5.94	765.94
	220	5.77	765.77
	240	5.61	765.61
	260	5.45	765.45
	280	5.30	765.30
	300	5.15	765.15
	320	5.00	765.00
	340	4.86	764.86
	360	4.73	764.73
	380	4.59	764.59
	400	4.46	764.46
	420	4.34	764.34
	440	4.21	764.21
	460	4.09	764.09
	464	4.07	764.07

Existing Conditions Hydraulic Grade Line for the 12th Street Pump Plant Sta 129+20 Armourdale Levee Unit



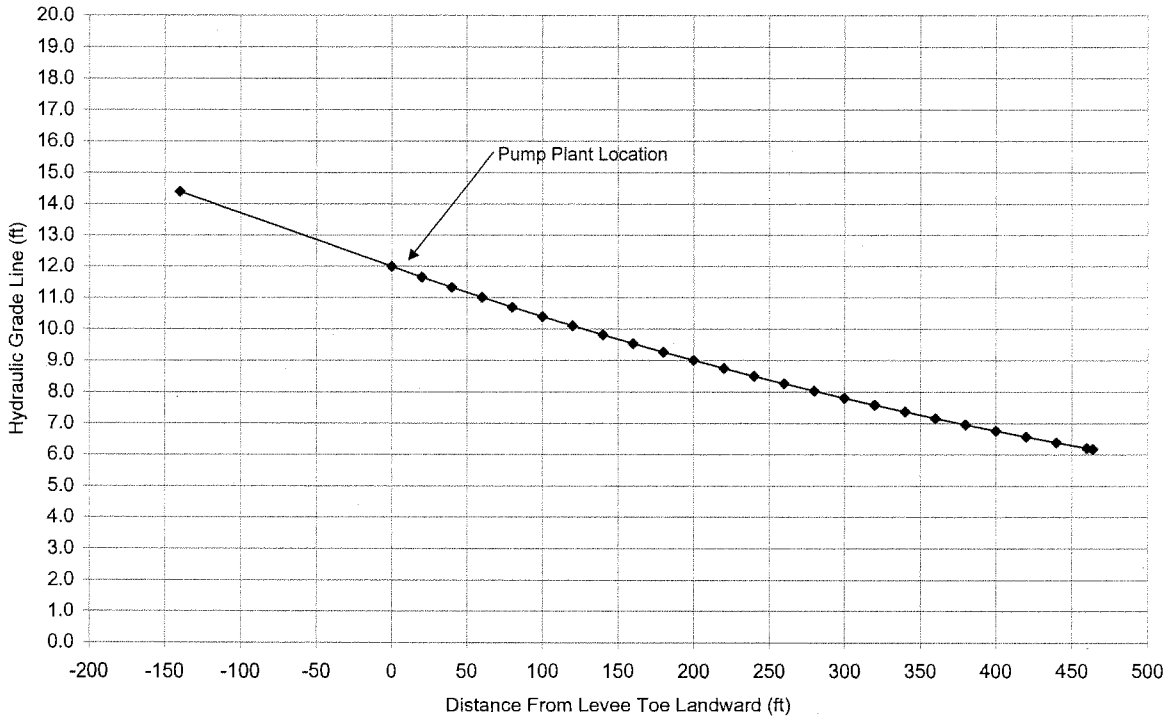
Hydraulic Grade Line for the 12th Street Pump Plant Design Conditions n500+0			
Elevation Top of levee (MSL)	771.35	n500+0	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	11.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	9.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-140	11.40	771.40
Pump Plant Location	0	9.50	769.50
	20	9.23	769.23
	40	8.97	768.97
	60	8.71	768.71
	80	8.47	768.47
	100	8.23	768.23
	120	8.00	768.00
	140	7.77	767.77
	160	7.55	767.55
	180	7.34	767.34
	200	7.13	767.13
	220	6.93	766.93
	240	6.73	766.73
	260	6.54	766.54
	280	6.36	766.36
	300	6.18	766.18
	320	6.00	766.00
	340	5.84	765.84
	360	5.67	765.67
	380	5.51	765.51
	400	5.35	765.35
	420	5.20	765.20
	440	5.06	765.06
	460	4.91	764.91
	464	4.89	764.89

Design Condition n500+0 Hydraulic Grade Line for the 12th Street Pump Plant Sta 129+20 Armourdale Levee Unit

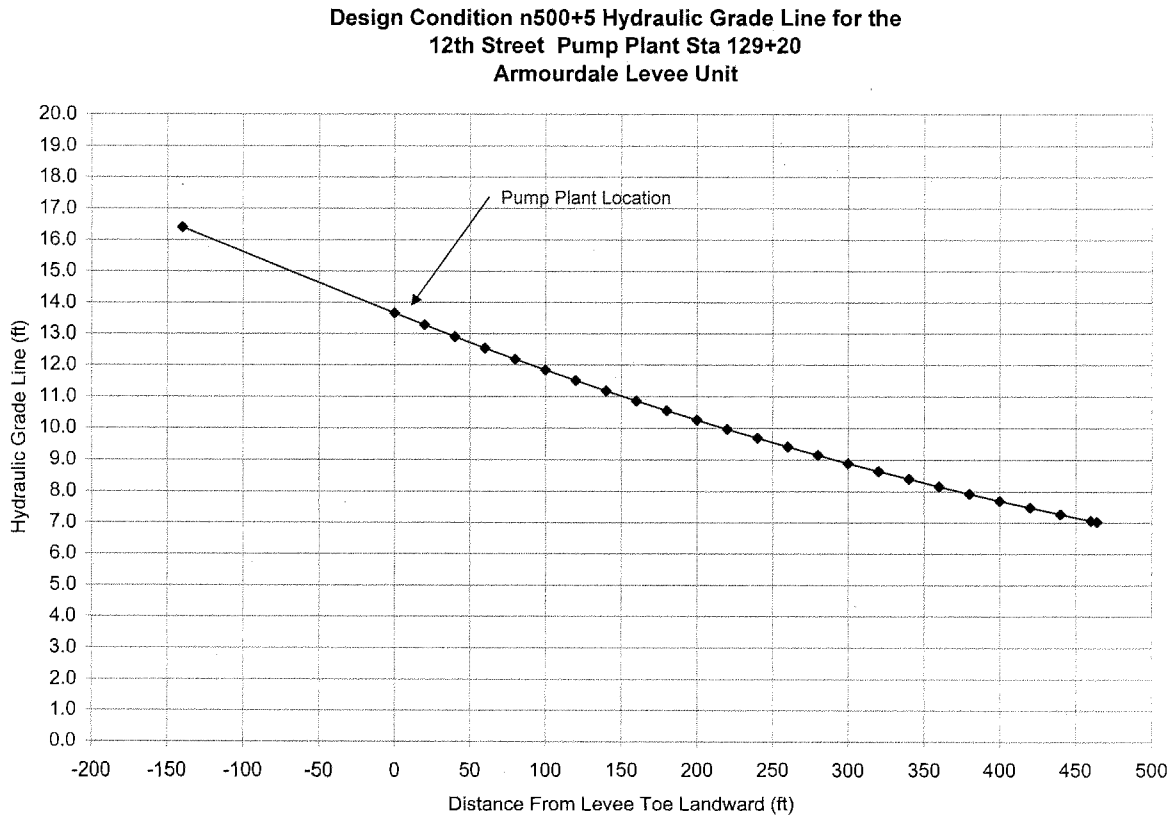


Hydraulic Grade Line for the 12th Street Pump Plant Design Conditions n500+3			
Elevation New Top of levee (MSL)	774.35	n500 +3	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	14.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	12.00		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-140	14.40	774.40
	0	12.00	772.00
	20	11.66	771.66
	40	11.33	771.33
	60	11.01	771.01
	80	10.70	770.70
	100	10.39	770.39
	120	10.10	770.10
	140	9.82	769.82
	160	9.54	769.54
	180	9.27	769.27
	200	9.01	769.01
	220	8.75	768.75
	240	8.51	768.51
	260	8.27	768.27
	280	8.03	768.03
	300	7.81	767.81
	320	7.59	767.59
	340	7.37	767.37
	360	7.16	767.16
	380	6.96	766.96
	400	6.76	766.76
	420	6.57	766.57
	440	6.39	766.39
	460	6.21	766.21
	464	6.17	766.17

Design Condition n500+3 Hydraulic Grade Line for the 12th Street Pump Plant Sta 129+20 Armourdale Levee Unit



Hydraulic Grade Line for the 12th Street Pump Plant Design Conditions n500+5			
Elevation New Top of levee (MSL)	776.35	n500 +5	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	734		
Driving Head (ft)	16.4		
L1 (ft)	20		
L2 (ft)	120		
Le (ft)	698		
ho	13.66		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-140	16.40	776.40
Pump Plant Location	0	13.66	773.66
	20	13.28	773.28
	40	12.90	772.90
	60	12.54	772.54
	80	12.18	772.18
	100	11.84	771.84
	120	11.50	771.50
	140	11.18	771.18
	160	10.86	770.86
	180	10.56	770.56
	200	10.26	770.26
	220	9.97	769.97
	240	9.69	769.69
	260	9.41	769.41
	280	9.15	769.15
	300	8.89	768.89
	320	8.64	768.64
	340	8.39	768.39
	360	8.16	768.16
	380	7.93	767.93
	400	7.70	767.70
	420	7.49	767.49
	440	7.27	767.27
	460	7.07	767.07
	464	7.03	767.03


Design Condition n500+5 Hydraulic Grade Line for the
12th Street Pump Plant Sta 129+20
Armourdale Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Armourdale Kansas - Hydraulic Grade Line for the Mill Street Pump Station Sta 156+75

ARM Mill Street PP 156+75.xls

NOMENCLATURE Mill St Pump Plant

Feb 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD GMB
Date : 02/18/07

NOTES:
1. T-wall on existing levee raise proposed

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	= $C * (e^{(2.8 C/D)}) / (e^{(2.8 C/D)})$
H	= max head or levee height	L_2	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Elevation Used

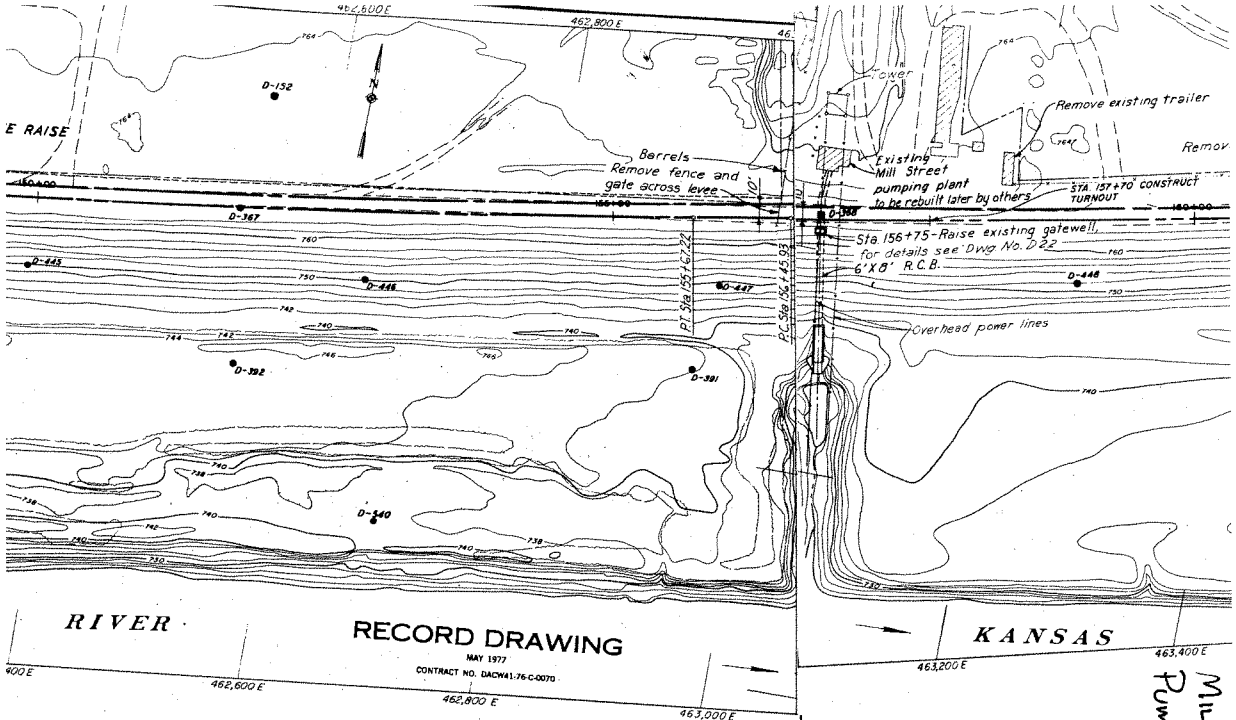
Top of Existing Levee:	768.7 (From O&M Manual)
500 year + 0':	770.3
500 year + 3':	773.3
500 year + 5':	775.3
Ground Surface Land Side:	760.0
Ground Surface River Side:	740.0
Bottom of Blanket:	725.0
Top of Bedrock:	667.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

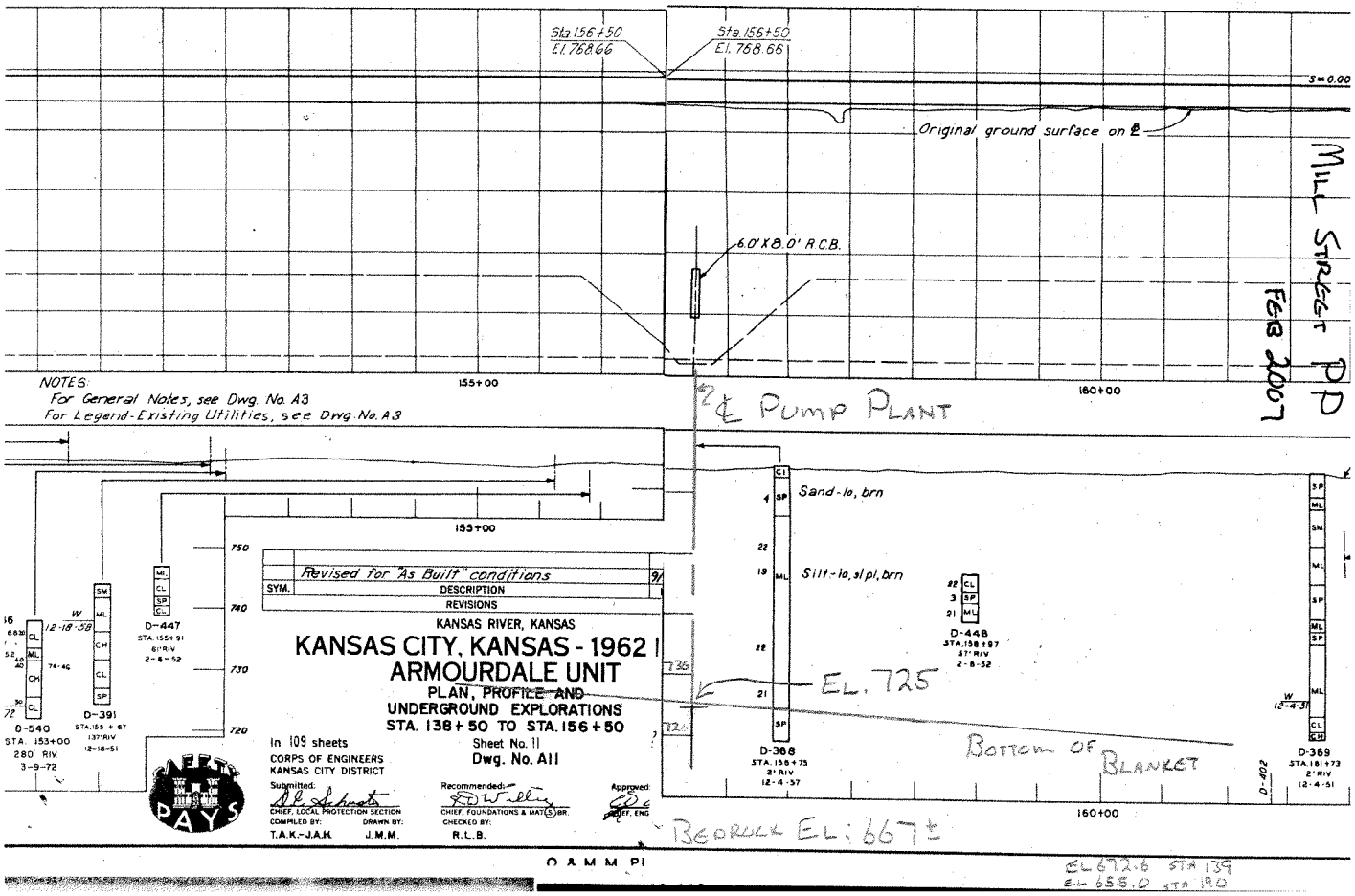
Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
156+75	Existing Conditions		300	300	35.0	25.0	15.0	58.0	75	8.7	EC
156+75	n500 + 0		300	300	35.0	25.0	15.0	58.0	75	10.3	n500
156+75	n500 +3		300	300	35.0	25.0	15.0	58.0	75	13.3	n500+3
156+75	n500 + 5		300	300	35.0	25.0	15.0	58.0	75	15.3	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_t Feet	L_t Feet	h_o Feet	i_o FtFt	i_c FtFt	FS ic/io		
511	780	74	100	780	955	7.11	0.20	0.84	4.13		
511	780	74	100	780	955	8.42	0.24	0.84	3.49		
511	780	74	100	780	955	10.87	0.31	0.84	2.70		
511	780	74	100	780	955	12.50	0.36	0.84	2.35		



Mill Street
Pump Plant
Feb 2007

578

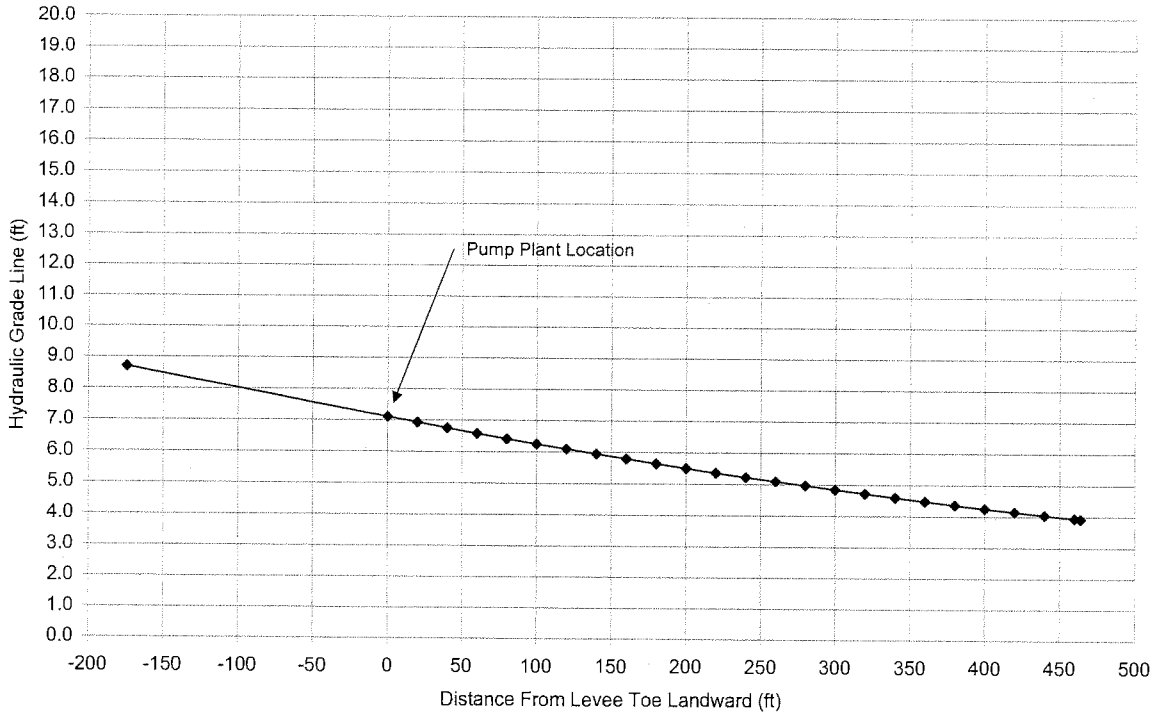


Mill Street PD
Feb 2007

Hydraulic Grade Line for the Mill Street Pump Plant
Existing Conditions

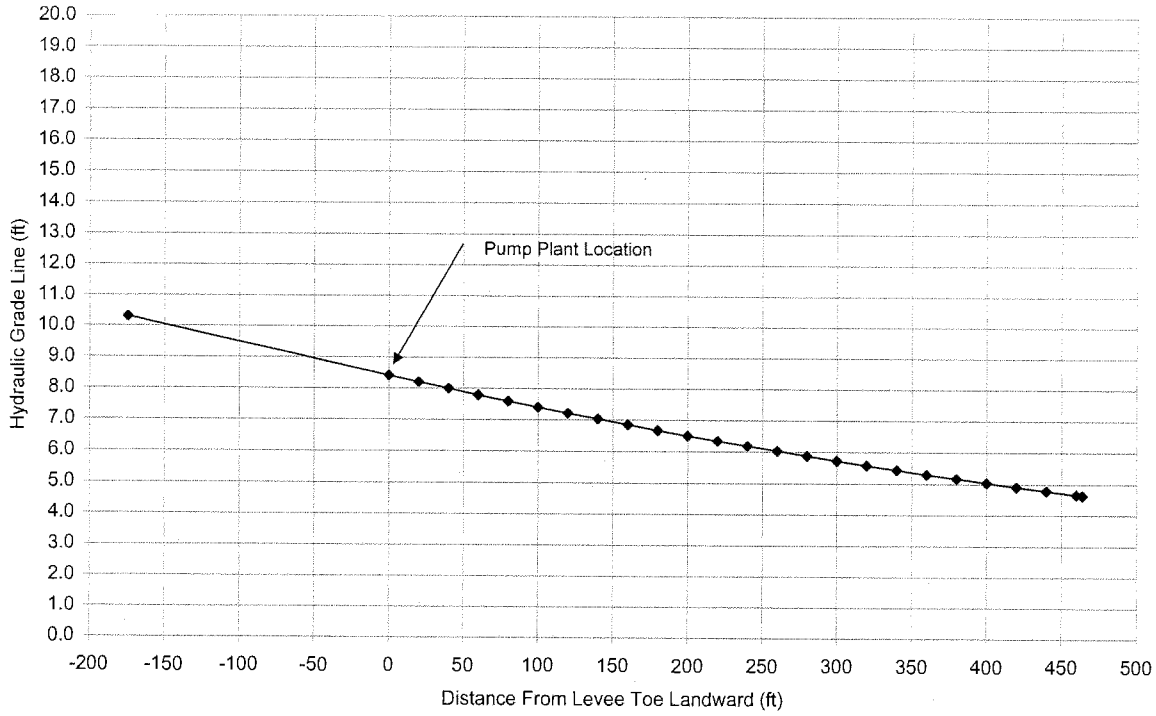
Elevation Top of levee (MSL)	768.65	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	8.7		
L1 (ft)	74		
L2 (ft)	100		
Le (ft)	780		
ho	7.1		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-174	8.70	768.70
Pump Plant Location	0	7.11	767.11
	20	6.93	766.93
	40	6.76	766.76
	60	6.58	766.58
	80	6.42	766.42
	100	6.26	766.26
	120	6.10	766.10
	140	5.94	765.94
	160	5.79	765.79
	180	5.65	765.65
	200	5.50	765.50
	220	5.36	765.36
	240	5.23	765.23
	260	5.10	765.10
	280	4.97	764.97
	300	4.84	764.84
	320	4.72	764.72
	340	4.60	764.60
	360	4.48	764.48
	380	4.37	764.37
	400	4.26	764.26
	420	4.15	764.15
	440	4.05	764.05
	460	3.94	763.94
	464	3.92	763.92

Existing Conditions Hydraulic Grade Line for the Mill Street Pump Plant Sta 156+75 Armourdale Levee Unit



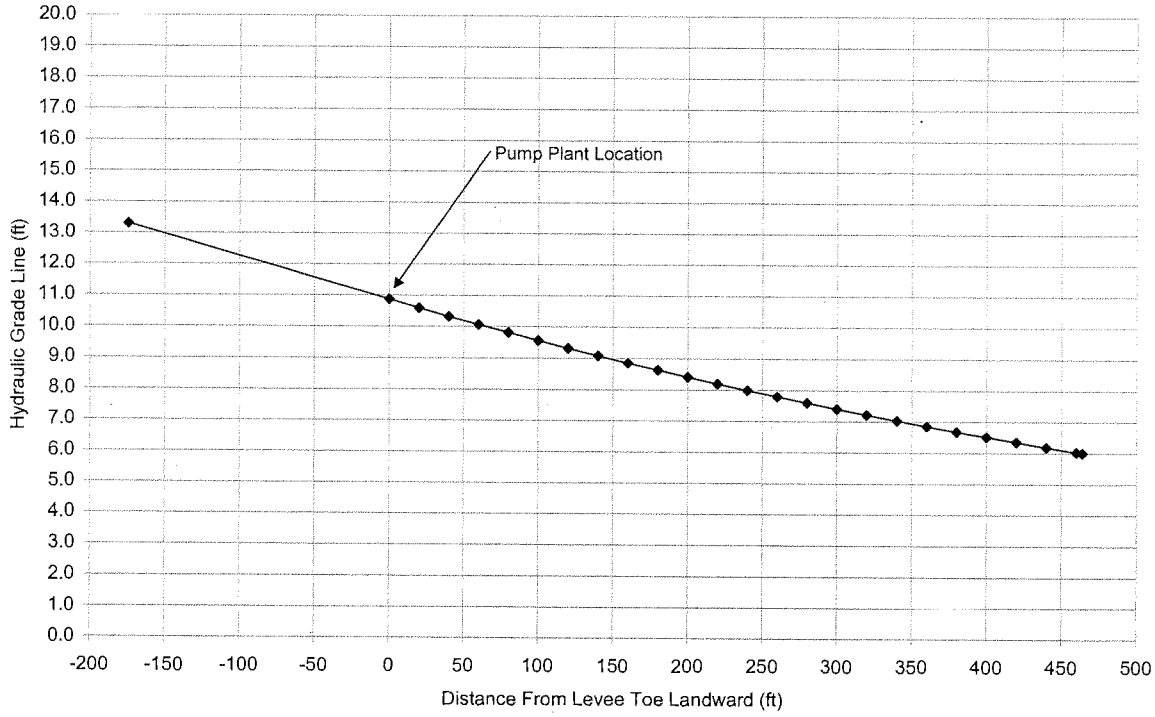
Hydraulic Grade Line for the Mill Street Pump Plant Design Conditions n500+0			
Elevation Top of levee (MSL)	770.3	n500+0	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	10.3		
L1 (ft)	74		
L2 (ft)	100		
Le (ft)	780		
ho	8.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-174	10.30	770.30
Pump Plant Location	0	8.42	768.42
	20	8.21	768.21
	40	8.00	768.00
	60	7.80	767.80
	80	7.60	767.60
	100	7.41	767.41
	120	7.22	767.22
	140	7.04	767.04
	160	6.86	766.86
	180	6.68	766.68
	200	6.51	766.51
	220	6.35	766.35
	240	6.19	766.19
	260	6.03	766.03
	280	5.88	765.88
	300	5.73	765.73
	320	5.59	765.59
	340	5.44	765.44
	360	5.31	765.31
	380	5.17	765.17
	400	5.04	765.04
	420	4.91	764.91
	440	4.79	764.79
	460	4.67	764.67
	464	4.65	764.65

Design Condition n500+0 Hydraulic Grade Line for the Mill Street Pump Plant Sta 156+75 Armourdale Levee Unit



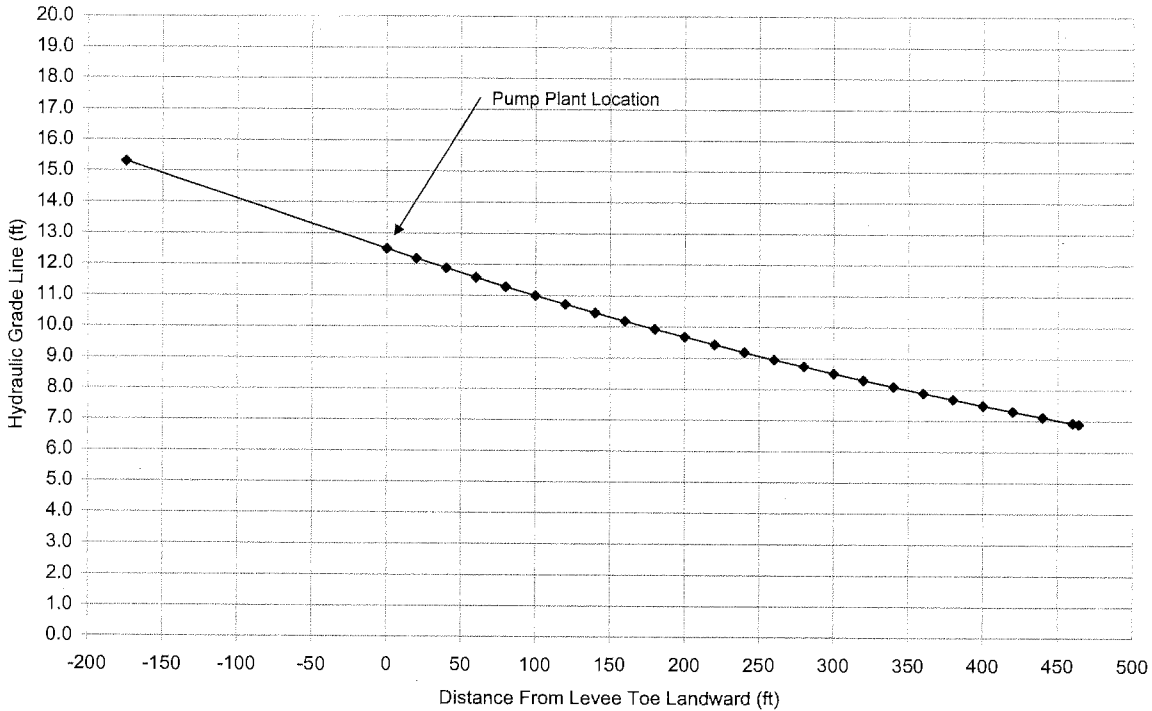
Hydraulic Grade Line for the Mill Street Pump Plant Design Conditions n500+3			
Elevation New Top of levee (MSL)	773.3	n500 +3	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	13.3		
L1 (ft)	74		
L2 (ft)	100		
Le (ft)	780		
ho	10.87		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-174	13.30	773.30
	0	10.87	770.87
	20	10.59	770.59
	40	10.33	770.33
	60	10.07	770.07
	80	9.81	769.81
	100	9.56	769.56
	120	9.32	769.32
	140	9.08	769.08
	160	8.85	768.85
	180	8.63	768.63
	200	8.41	768.41
	220	8.20	768.20
	240	7.99	767.99
	260	7.79	767.79
	280	7.59	767.59
	300	7.40	767.40
	320	7.21	767.21
	340	7.03	767.03
	360	6.85	766.85
	380	6.68	766.68
	400	6.51	766.51
	420	6.35	766.35
	440	6.19	766.19
	460	6.03	766.03
	464	6.00	766.00

Design Condition n500+3 Hydraulic Grade Line for the Mill Street Pump Plant Sta 156+75 Armourdale Levee Unit



Hydraulic Grade Line for the Mill Street Pump Plant Design Conditions n500+5			
Elevation New Top of levee (MSL)	775.3	n500 +5	
Elevation at Landside Levee Toe (MSL)	760		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	15.3		
L1 (ft)	74		
L2 (ft)	100		
Le (ft)	780		
ho	12.50		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-174	15.30	775.30
Pump Plant Location	0	12.50	772.50
	20	12.19	772.19
	40	11.88	771.88
	60	11.58	771.58
	80	11.29	771.29
	100	11.00	771.00
	120	10.72	770.72
	140	10.45	770.45
	160	10.19	770.19
	180	9.93	769.93
	200	9.68	769.68
	220	9.43	769.43
	240	9.19	769.19
	260	8.96	768.96
	280	8.73	768.73
	300	8.51	768.51
	320	8.30	768.30
	340	8.09	768.09
	360	7.88	767.88
	380	7.68	767.68
	400	7.49	767.49
	420	7.30	767.30
	440	7.12	767.12
	460	6.94	766.94
	464	6.90	766.90

Design Condition n500+5 Hydraulic Grade Line for the Mill Street Pump Plant Sta 156+75 Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the 5th Street Pump Station Sta 185+70

ARM 5th Street 185+70.xls

NOMENCLATURE 5TH ST. PUMP PLANT FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kazniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD
Date : 2/26/2007

NOTES:
1. Landside levee raise proposed
2. L_2 not revised for minor change due to levee raise

Elevation Used

Top of Existing Levee:	767.6 (From O&M Manual)
500 year + 0':	769.0
500 year + 3':	772.0
500 year + 5':	775.0
Ground Surface Land Side:	755.0
Ground Surface River Side:	742.0
Bottom of Blanket:	720.0
Top of Bedrock:	655.0

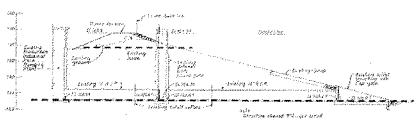
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

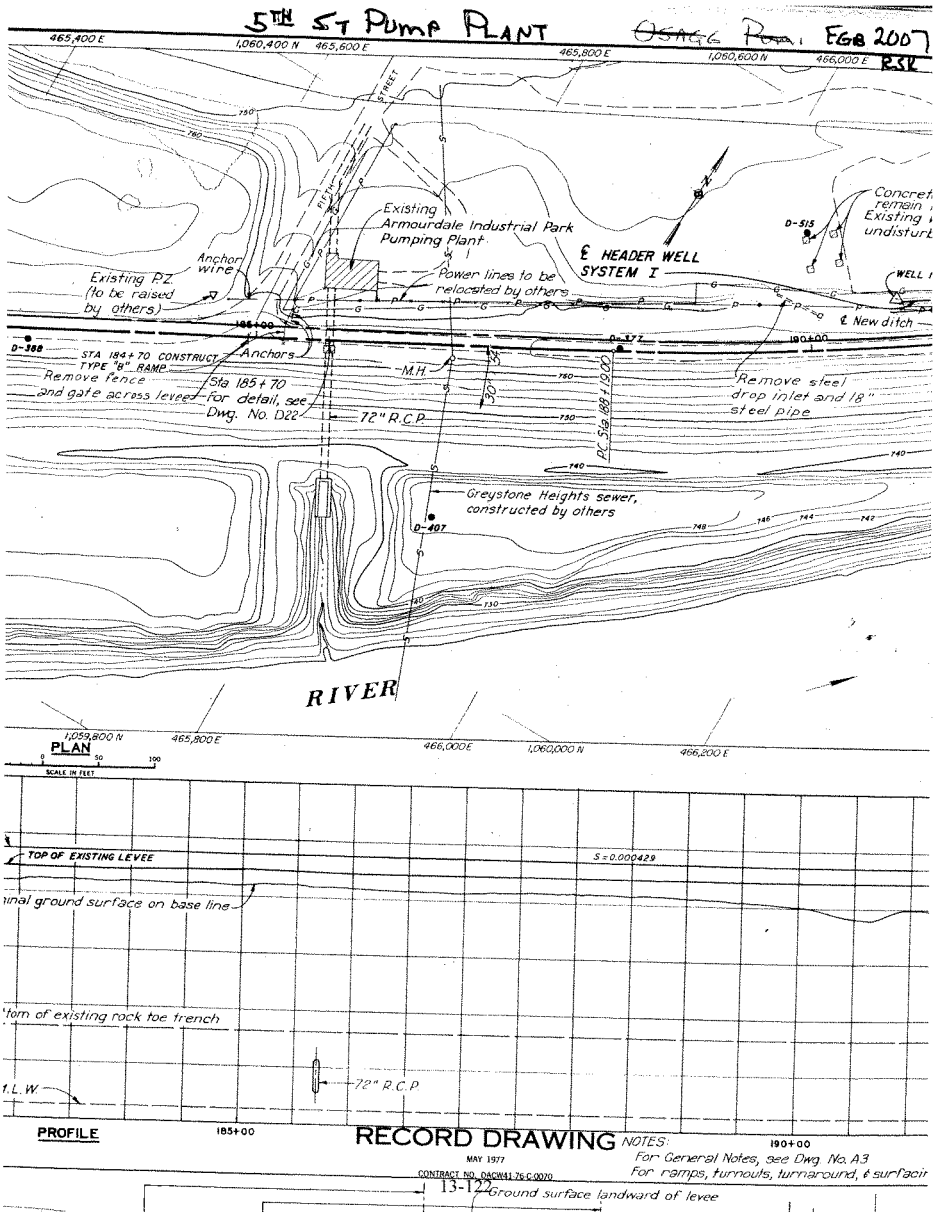
Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

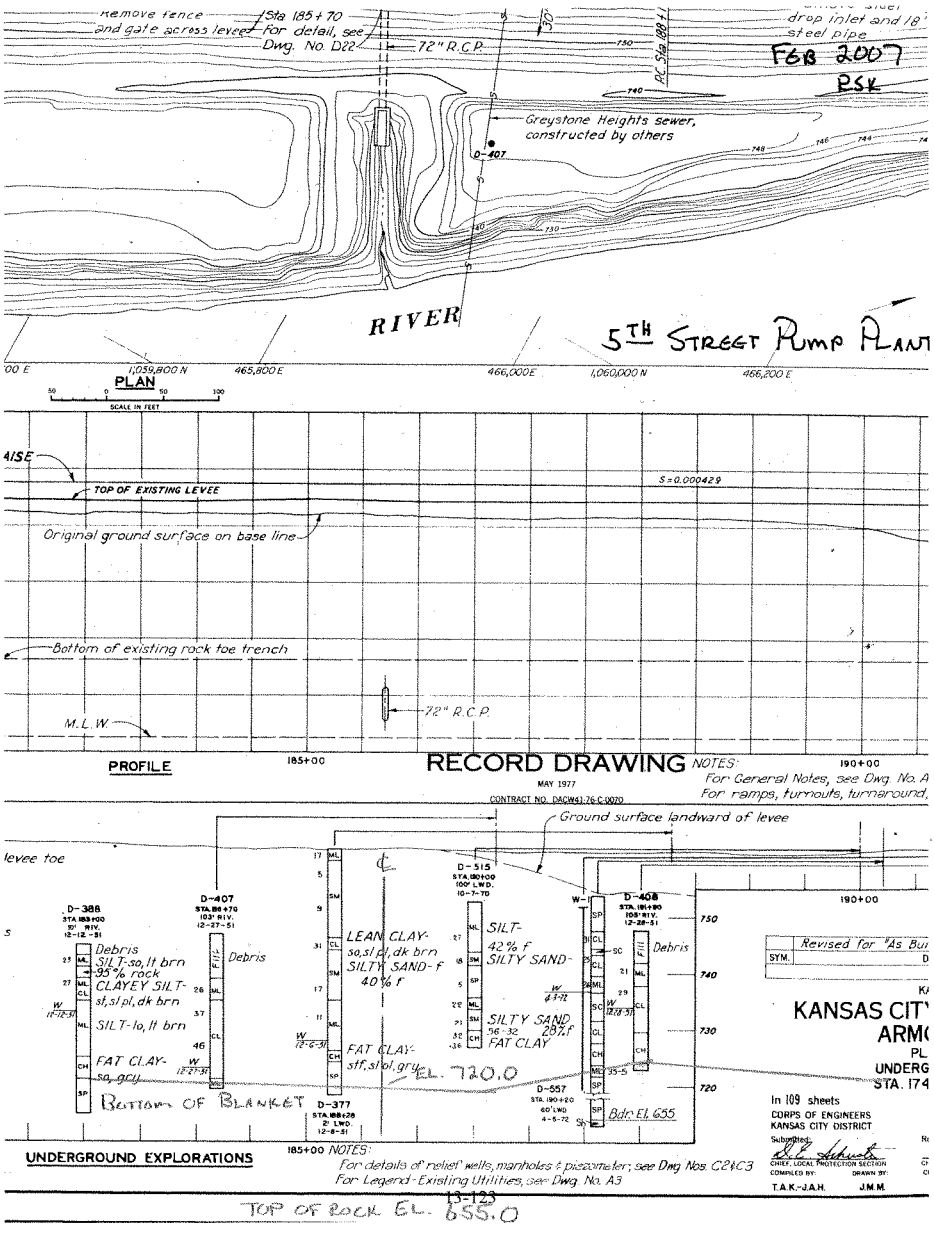
Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundati	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	= $C * (e^{(R/(C+1))}) / (e^{(R/(C+1))})$
H	= max head or levee height	L_2	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
185+70	Existing Conditions		300	300	35.0	30.0	22.0	65.0	45	12.6	EC
185+70	n500 + 0		300	300	35.0	30.0	22.0	65.0	45	14.0	n500
185+70	n500 +3		300	300	35.0	30.0	22.0	65.0	45	17.0	n500+3
185+70	n500 + 5		300	300	35.0	30.0	22.0	65.0	45	19.0	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_t Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
655	826	45	100	826	971	10.72	0.31	0.84	2.74		
655	826	45	100	826	971	11.91	0.34	0.84	2.47		
655	826	45	100	826	971	14.46	0.41	0.84	2.03		
655	826	45	100	826	971	16.16	0.54	0.84	1.56		

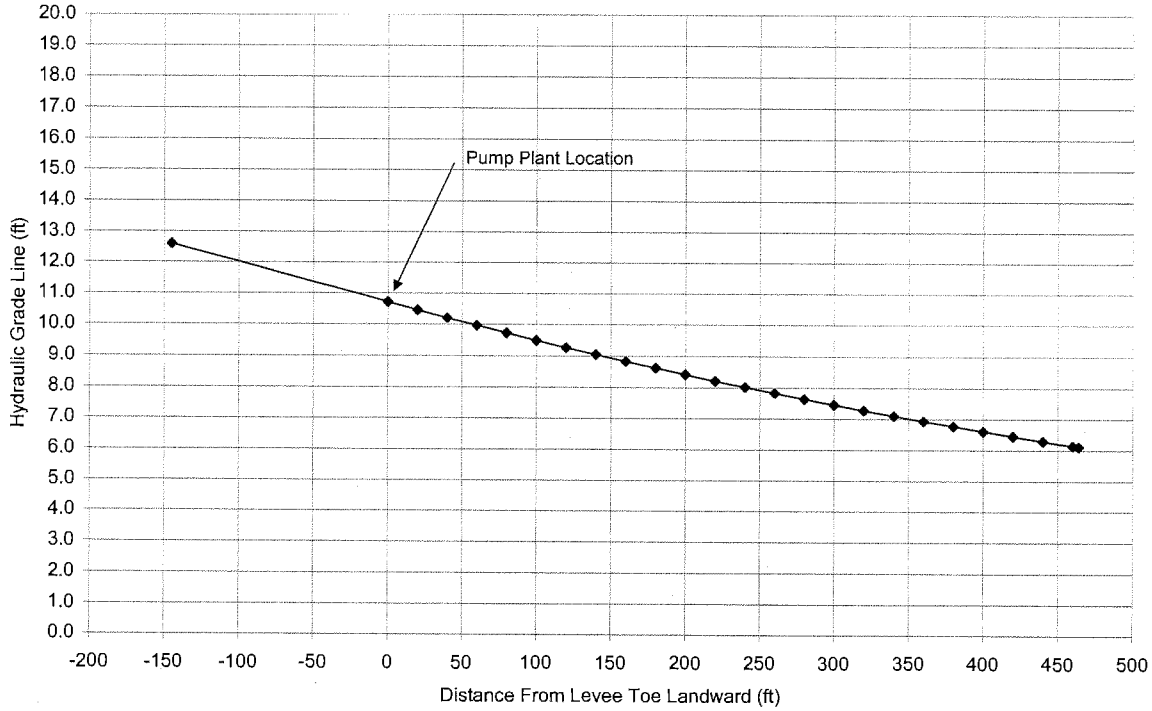



5TH ST PUMP PLANT
EGB 2007
465,400 E
1,060,400 N
465,600 E
465,800 E
1,060,600 N
466,000 E
Existing Armourdale Industrial Park Pumping Plant
Anchor wire
Existing P.Z. (to be raised by others)
Power lines to be relocated by others
℄ HEADER WELL SYSTEM I
℄ New ditch
D-388
D-515
D-407
STA 184+70 CONSTRUCT TYPE "B" RAMP
Anchors
Remove fence and gate across levee
Sta 185+70 For detail, see Dwg. No. D22
M.H.
72" R.C.P.
P.C. Sta 188+19.00
Remove steel drop inlet and 18" steel pipe
Greystone Heights sewer, constructed by others
RIVER
1,059,800 N
465,800 E
466,000 E
1,060,000 N
466,200 E
PLAN
SCALE IN FEET
TOP OF EXISTING LEVEE
S = 0.000429
inal ground surface on base line
'tom of existing rock toe trench
T.L.W.
72" R.C.P.
PROFILE
185+00
190+00
RECORD DRAWING
MAY 1977
CONTRACT NO. DACW41-76-C-0070
NOTES:
For General Notes, see Dwg. No. A3
For ramps, turnouts, turnaround, & surfacin
Ground surface landward of levee


remove fence and gate across levee
Sta. 185+70 for detail, see Dwg. No. D22
72" R.C.P.
drop inlet and 18" steel pipe
FEB 2007
PSK
Greystone Heights sewer, constructed by others
D-407
RIVER
5TH STREET PUMP PLANT
1,059,800 N
465,800 E
466,000 E
1,060,000 N
466,200 E
PLAN
SCALE IN FEET
TOP OF EXISTING LEVEE
S=0.000429
Original ground surface on base line
Bottom of existing rock toe trench
72" R.C.P.
M.L.W.
PROFILE
185+00
190+00
RECORD DRAWING
MAY 1977
CONTRACT NO. DACW41-76-C-0070
NOTES:
For General Notes, see Dwg. No. A
For ramps, turnouts, turnaround,
Ground surface landward of levee
levee toe
D-388
D-407
D-377
D-515
D-408
D-557
Debris
SILT-so, lt brn
35% rock
CLAYEY SILT-st, sl pl, dk brn
SILT-lo, lt brn
FAT CLAY-so, gry
BOTTOM OF BLANKET
LEAN CLAY-so, sl pl, dk brn
SILTY SAND-f 40% f
FAT CLAY-stf, sl pl, gry
SILT-42% f
SILTY SAND-f
SILTY SAND 56-32 28% f
FAT CLAY
EL. 720.0
Bdr. EL 655
750
740
730
720
Revised for "As Bui
KANSAS CITY ARMO
PL
UNDERG
STA. 174
In 109 sheets
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
CHIEF, LOCAL PROTECTION SECTION
COMPILED BY: T.A.K.-J.A.H.
DRAWN BY: J.M.M.
UNDERGROUND EXPLORATIONS
185+00
NOTES:
For details of relief wells, manholes & piezometer; see Dwg. Nos. C2 & C3
For Legend-Existing Utilities, see Dwg. No. A3
13-123
TOP OF ROCK EL. 655.0

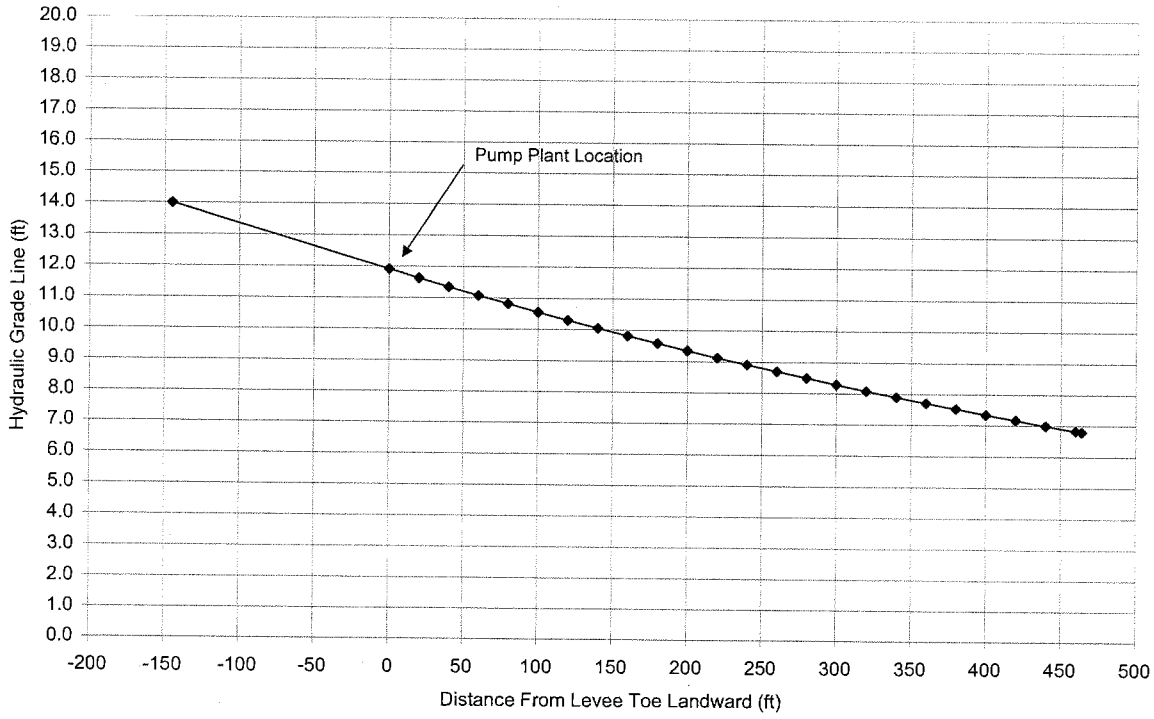
Hydraulic Grade Line for the 5th Street Pump Plant Existing Conditions			
Elevation Top of levee (MSL)	767.6	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	755		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	12.6		
L1 (ft)	45		
L2 (ft)	100		
Le (ft)	826		
ho	10.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	12.60	767.60
Pump Plant Location	0	10.72	765.72
	20	10.46	765.46
	40	10.21	765.21
	60	9.97	764.97
	80	9.73	764.73
	100	9.50	764.50
	120	9.27	764.27
	140	9.05	764.05
	160	8.83	763.83
	180	8.62	763.62
	200	8.41	763.41
	220	8.21	763.21
	240	8.02	763.02
	260	7.83	762.83
	280	7.64	762.64
	300	7.46	762.46
	320	7.28	762.28
	340	7.10	762.10
	360	6.93	761.93
	380	6.77	761.77
	400	6.61	761.61
	420	6.45	761.45
	440	6.29	761.29
	460	6.14	761.14
	464	6.11	761.11

Existing Conditions Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit



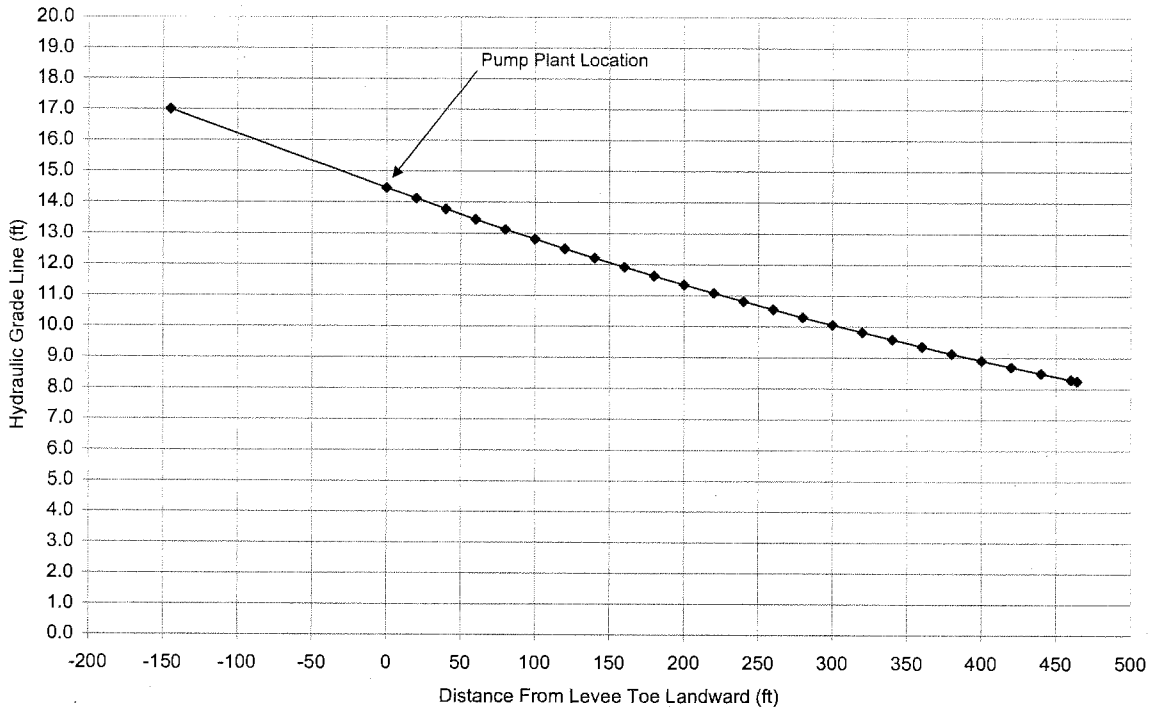
Hydraulic Grade Line for the 5th Street Pump Plant Design Conditions n500+0			
Elevation Top of levee (MSL)	769	n500+0	
Elevation at Landside Levee Toe (MSL)	755		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	14		
L1 (ft)	45		
L2 (ft)	100		
Le (ft)	826		
ho	11.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	14.00	769.00
Pump Plant Location	0	11.91	766.91
	20	11.63	766.63
	40	11.35	766.35
	60	11.08	766.08
	80	10.81	765.81
	100	10.55	765.55
	120	10.30	765.30
	140	10.05	765.05
	160	9.81	764.81
	180	9.58	764.58
	200	9.35	764.35
	220	9.13	764.13
	240	8.91	763.91
	260	8.69	763.69
	280	8.49	763.49
	300	8.28	763.28
	320	8.09	763.09
	340	7.89	762.89
	360	7.70	762.70
	380	7.52	762.52
	400	7.34	762.34
	420	7.16	762.16
	440	6.99	761.99
	460	6.83	761.83
	464	6.79	761.79

Design Condition n500+0 Hydraulic Grade Line for the 5th Street Pump Plant Sta 185+70 Armourdale Levee Unit



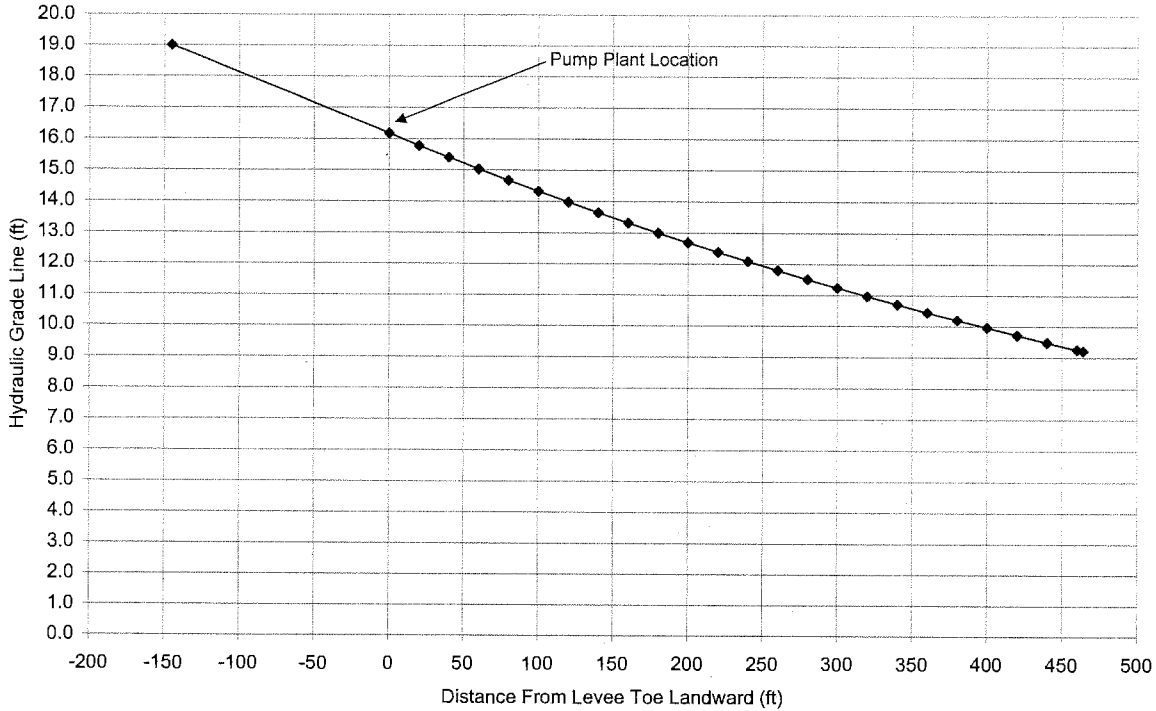
Hydraulic Grade Line for the 5th Street Pump Plant Design Conditions n500+3			
Elevation New Top of levee (MSL)	772	n500 +3	
Elevation at Landside Levee Toe (MSL)	755		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	17		
L1 (ft)	45		
L2 (ft)	100		
Le (ft)	826		
ho	14.46		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-145	17.00	772.00
Pump Plant Location	0	14.46	769.46
	20	14.12	769.12
	40	13.78	768.78
	60	13.45	768.45
	80	13.13	768.13
	100	12.81	767.81
	120	12.51	767.51
	140	12.21	767.21
	160	11.92	766.92
	180	11.63	766.63
	200	11.35	766.35
	220	11.08	766.08
	240	10.82	765.82
	260	10.56	765.56
	280	10.31	765.31
	300	10.06	765.06
	320	9.82	764.82
	340	9.58	764.58
	360	9.35	764.35
	380	9.13	764.13
	400	8.91	763.91
	420	8.70	763.70
	440	8.49	763.49
	460	8.29	763.29
	464	8.25	763.25

Design Condition n500+3 Hydraulic Grade Line for the 5th Street Pump Plant Sta 185+70 Armourdale Levee Unit



Hydraulic Grade Line for the 5th Street Pump Plant Design Conditions n500+5			
Elevation New Top of levee (MSL)	775	n500 +5	
Elevation at Landside Levee Toe (MSL)	755		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	19		
L1 (ft)	45		
L2 (ft)	100		
Le (ft)	826		
ho	16.16		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-145	19.00	774.00
Pump Plant Location	0	16.16	771.16
	20	15.78	770.78
	40	15.40	770.40
	60	15.03	770.03
	80	14.67	769.67
	100	14.32	769.32
	120	13.98	768.98
	140	13.64	768.64
	160	13.32	768.32
	180	13.00	768.00
	200	12.69	767.69
	220	12.39	767.39
	240	12.09	767.09
	260	11.80	766.80
	280	11.52	766.52
	300	11.24	766.24
	320	10.97	765.97
	340	10.71	765.71
	360	10.45	765.45
	380	10.20	765.20
	400	9.96	764.96
	420	9.72	764.72
	440	9.49	764.49
	460	9.26	764.26
	464	9.22	764.22

Design Condition n500+5 Hydraulic Grade Line for the
5th Street Pump Plant Sta 185+70
Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the Midwest Cold Storage Pump Station 194+60

ARM Midwest Cold Storage 194+60.xls

NOMENCLATURE Midwest Cold Storage
Feb 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kazniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD
Date : 2/12/2007

NOTES:
1. T-wall on levee raise proposed
2. L_2 not revised, levee footprint won't change

Elevation Used

Top of Existing Levee:	767.2 (From O&M Manual)
500 year + 0':	768.4
500 year + 3':	771.4
500 year + 5':	773.4
Ground Surface Land Side:	754.0 (Basement Elevation)
Ground Surface River Side:	740.0
Bottom of Blanket:	726.0
Top of Bedrock:	659.0

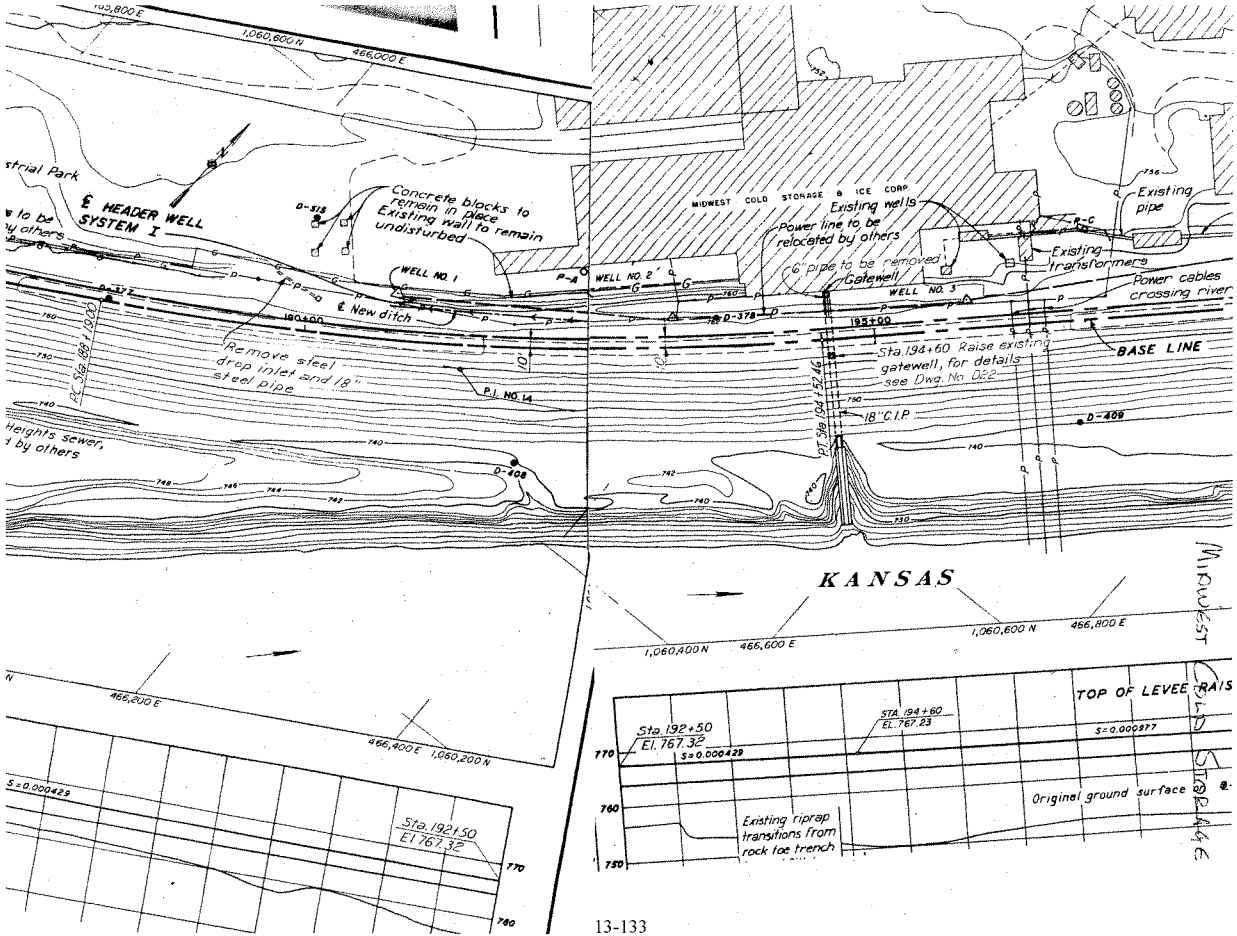
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

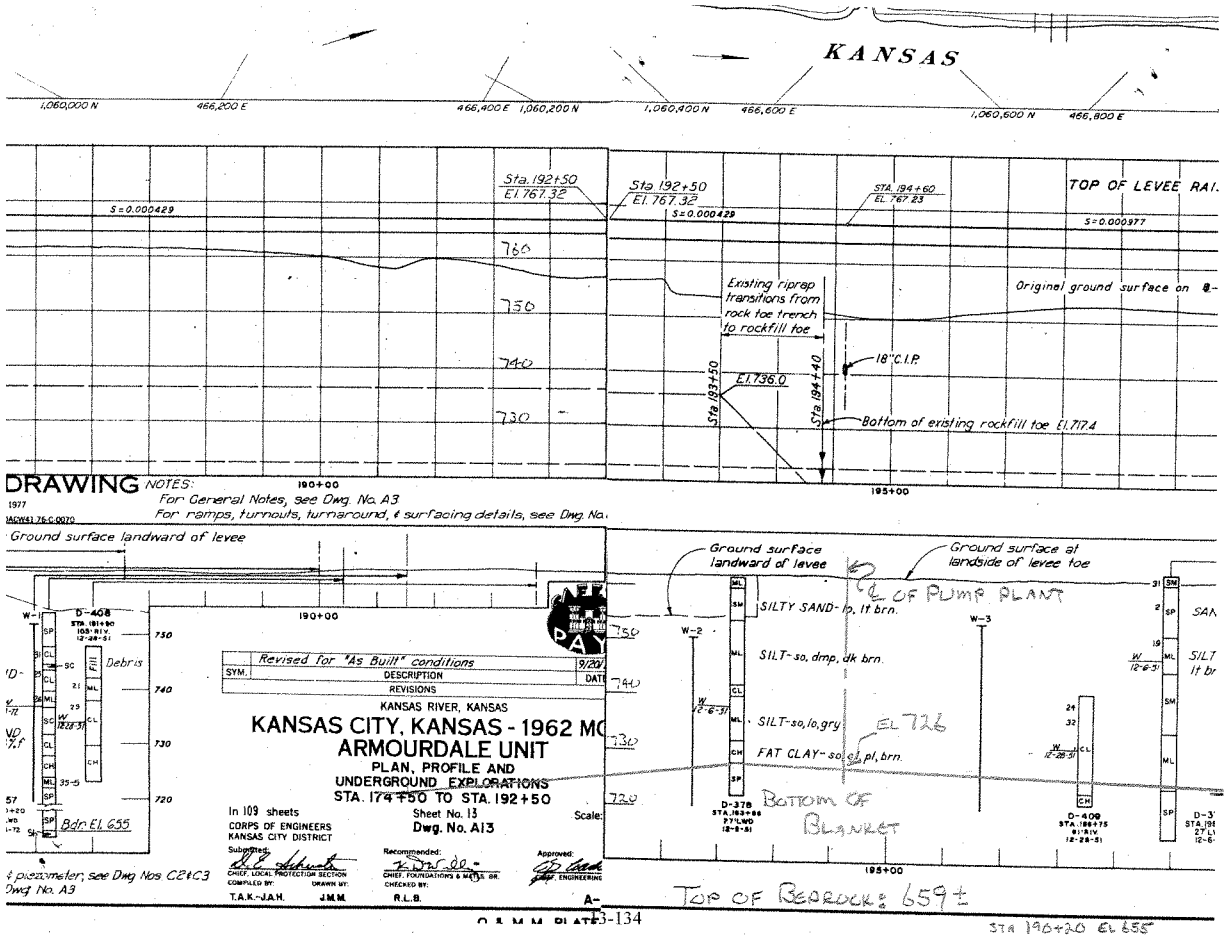
Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{bf}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundan	i_o	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2.8X/C-1)}) / (e^{(2.8X/C-1)})$
H	= max head or levee height	L_o	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{bf} Feet	D_f Feet	L_R Feet	H Feet	Remarks
194+60	Existing Conditions		300	300	28.0	21.0	14.0	67.0	80	13.2	BC
194+60	n500 + 0		300	300	28.0	21.0	14.0	67.0	80	14.4	n500
194+60	n500 +3		300	300	28.0	21.0	14.0	67.0	80	17.4	n500+3
194+60	n500 + 5		300	300	28.0	21.0	14.0	67.0	80	19.4	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_o Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS in/in		
530	750	79	100	750	930	10.65	0.38	0.84	2.21		
530	750	79	100	750	930	11.62	0.42	0.84	2.02		
530	750	79	100	750	930	14.04	0.50	0.84	1.67		
530	750	79	100	750	930	15.86	0.75	0.84	1.13		

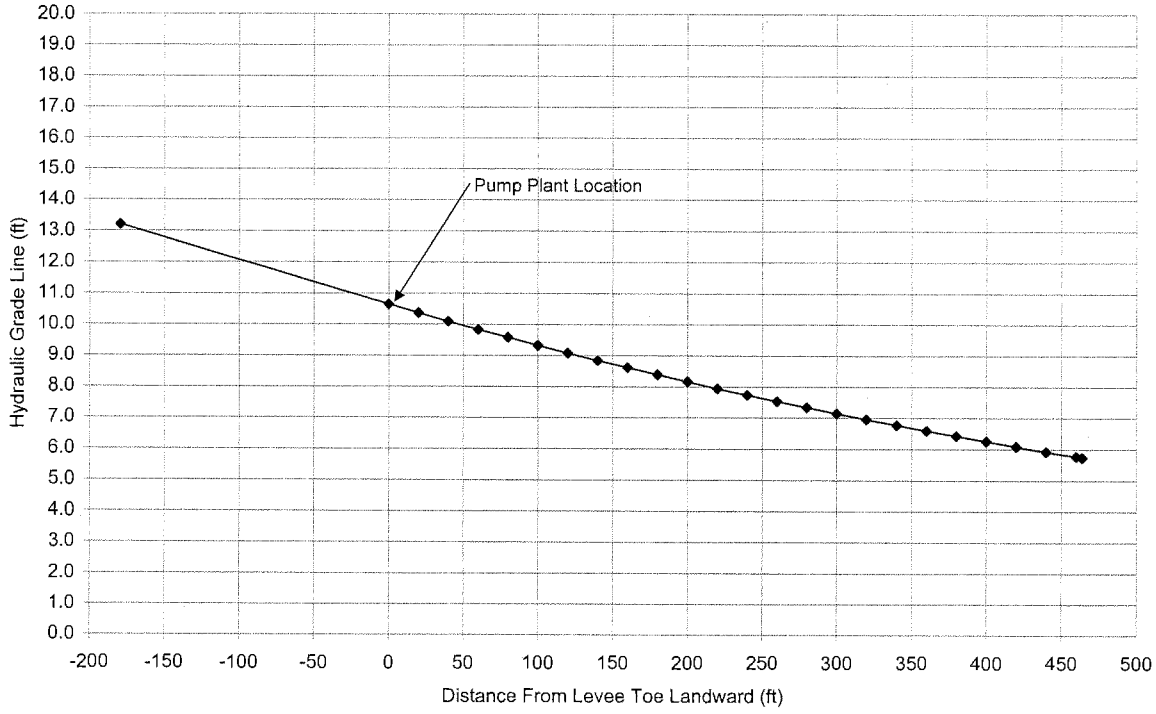

1,060,600 N
466,000 E
strial Park
℄ HEADER WELL SYSTEM I
D-515
Concrete blocks to remain in place Existing wall to remain undisturbed
WELL NO. 1
P-4
WELL NO. 2
MIDWEST COLD STORAGE & ICE CORP
Existing wells
Power line to be relocated by others
6" pipe to be removed
Gatewell
WELL NO. 3
Existing pipe
Existing transformers
Power cables crossing river
℄ New ditch
190+00
193+00
D-376
Remove steel drop inlet and 18" steel pipe
P.I. NO. 14
Sta. 194+60 Raise existing gatewell, for details see Dwg. No. 022
BASE LINE
18" C.I.P.
D-409
D-408
Heights sewer, by others
KANSAS
1,060,400 N
466,600 E
1,060,600 N
466,800 E
466,200 E
466,400 E
1,060,200 N
S=0.000429
Sta. 192+50
El. 767.32
770
760
Sta. 194+60
El. 767.23
S=0.000877
TOP OF LEVEE
Original ground surface
Existing riprap transitions from rock toe trench
750

600


KANSAS
1,060,000 N
466,200 E
466,400 E
1,060,200 N
1,060,400 N
466,600 E
1,060,600 N
466,800 E
Sta. 192+50
El. 767.32
S=0.000429
Sta 192+50
El. 767.32
S=0.000429
STA. 194+60
EL. 767.23
TOP OF LEVEE RAI
S=0.000977
760
750
740
730
Existing riprap transitions from rock toe trench to rockfill toe
Original ground surface on
18" C.I.P.
El. 736.0
Sta 193+50
Sta 194+40
Bottom of existing rockfill toe El. 717.4
190+00
195+00
DRAWING NOTES:
For General Notes, see Dwg. No. A3
For ramps, turnouts, turnaround, & surfacing details, see Dwg. No.
1977
Ground surface landward of levee
Ground surface landward of levee
Ground surface at landside of levee toe
℄ OF PUMP PLANT
D-408
STA. 191+90
103' RIV.
12-28-51
Debris
Fill
W-1
W-2
W-3
SILTY SAND-lo, lt brn.
SILT-so, dmp, dk brn.
SILT-so, lo, gry
EL 726
FAT CLAY-so, cl, pl, brn.
BOTTOM OF BLANKET
D-378
STA. 183+86
27' LWD
12-8-51
D-409
STA. 196+75
81' RIV.
12-28-51
D-3
STA. 191
27' L
12-6-
Bdn EL 655
& piezometer, see Dwg Nos C2 & C3
Dwg No. A3
Revised for "As Built" conditions
SYM.
DESCRIPTION
DATE
REVISIONS
KANSAS RIVER, KANSAS
KANSAS CITY, KANSAS - 1962 MC
ARMOURDALE UNIT
PLAN, PROFILE AND
UNDERGROUND EXPLORATIONS
STA. 174+50 TO STA. 192+50
In 109 sheets
Sheet No. 13
Dwg. No. A13
Scale:
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
Submitted:
CHIEF, LOCAL PROTECTION SECTION
Recommended:
CHIEF, FOUNDATIONS & MATLS. BR.
Approved:
CHIEF, ENGINEERING
COMPILED BY: T.A.K.-J.A.H.
DRAWN BY: J.M.M.
CHECKED BY: R.L.B.
TOP OF BEDROCK: 659±
STA 196+20 EL 655

Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Existing Conditions - Assume No Wells			
Elevation Top of levee (MSL)	767.2	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	13.2		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	10.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	13.20	767.20
Pump Plant Location	0	10.65	764.65
	20	10.37	764.37
	40	10.10	764.10
	60	9.83	763.83
	80	9.58	763.58
	100	9.32	763.32
	120	9.08	763.08
	140	8.84	762.84
	160	8.61	762.61
	180	8.38	762.38
	200	8.16	762.16
	220	7.95	761.95
	240	7.74	761.74
	260	7.53	761.53
	280	7.33	761.33
	300	7.14	761.14
	320	6.95	760.95
	340	6.77	760.77
	360	6.59	760.59
	380	6.42	760.42
	400	6.25	760.25
	420	6.09	760.09
	440	5.93	759.93
	460	5.77	759.77
	464	5.74	759.74

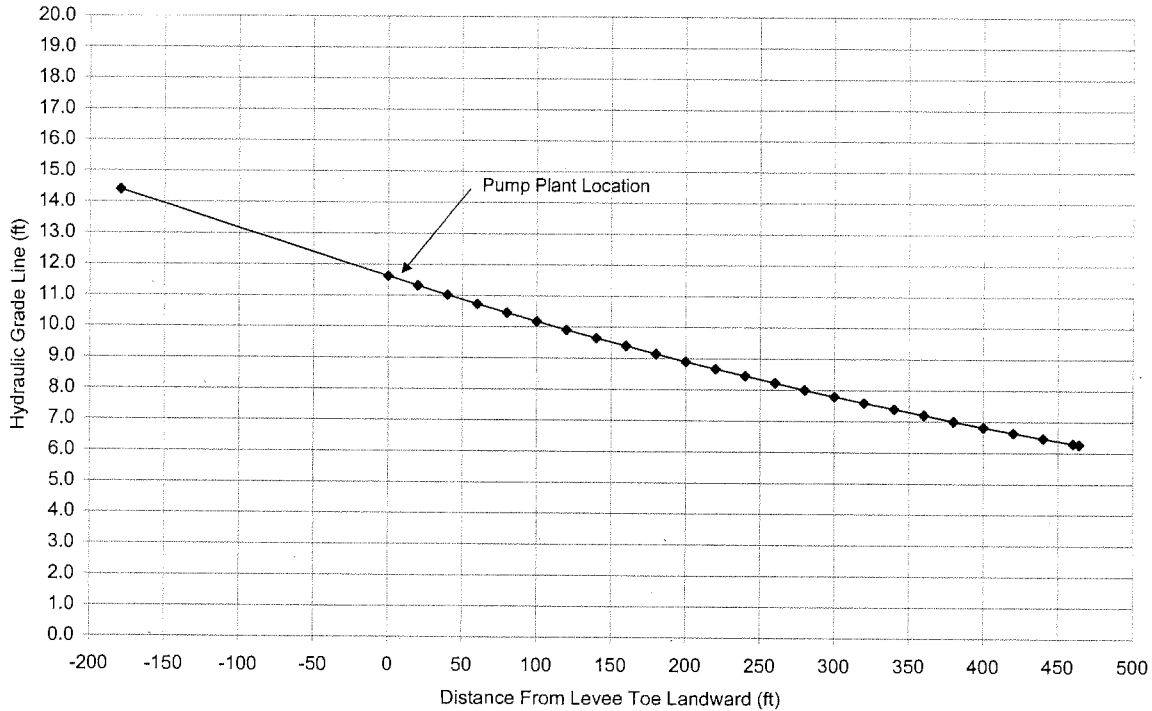
Existing Conditions Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



603

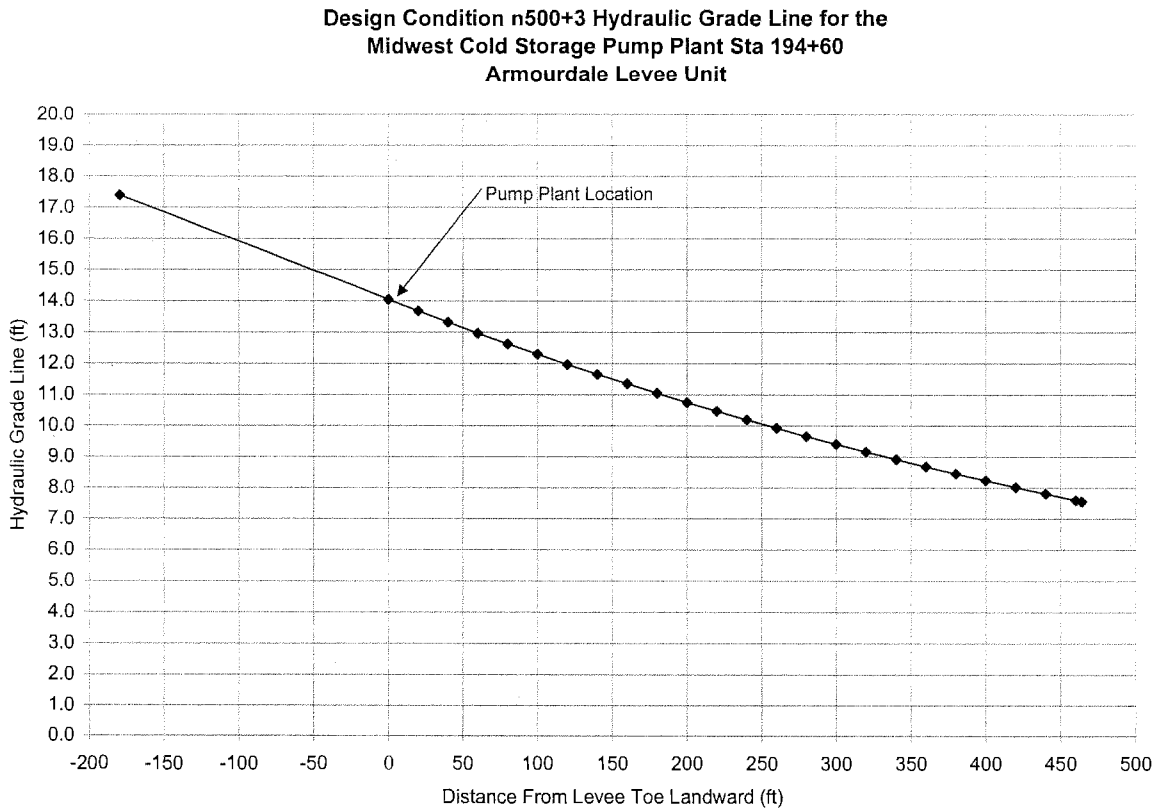
Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Design Conditions n500+0 - Assumes no Wells			
Elevation Top of levee (MSL)	768.4	n500+0	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	14.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	11.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	14.40	768.40
Pump Plant Location	0	11.62	765.62
	20	11.32	765.32
	40	11.02	765.02
	60	10.73	764.73
	80	10.45	764.45
	100	10.17	764.17
	120	9.90	763.90
	140	9.64	763.64
	160	9.39	763.39
	180	9.14	763.14
	200	8.90	762.90
	220	8.67	762.67
	240	8.44	762.44
	260	8.22	762.22
	280	8.00	762.00
	300	7.79	761.79
	320	7.59	761.59
	340	7.39	761.39
	360	7.19	761.19
	380	7.00	761.00
	400	6.82	760.82
	420	6.64	760.64
	440	6.46	760.46
	460	6.29	760.29
	464	6.26	760.26

Design Condition n500+0 Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



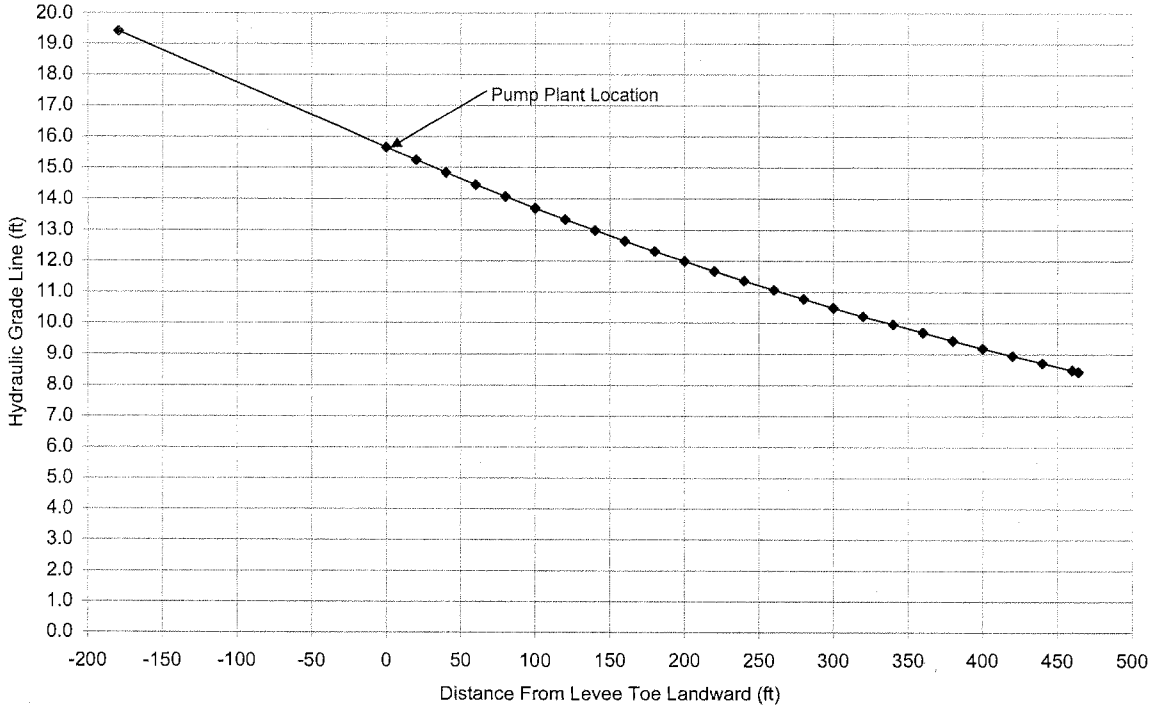
605

Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Design Conditions n500+3 - Assumes no Wells			
Elevation New Top of levee (MSL)	771.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	17.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	14.04		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-179	17.40	771.40
Pump Plant Location	0	14.04	768.04
	20	13.67	767.67
	40	13.31	767.31
	60	12.96	766.96
	80	12.62	766.62
	100	12.29	766.29
	120	11.97	765.97
	140	11.65	765.65
	160	11.35	765.35
	180	11.05	765.05
	200	10.76	764.76
	220	10.47	764.47
	240	10.20	764.20
	260	9.93	763.93
	280	9.67	763.67
	300	9.41	763.41
	320	9.17	763.17
	340	8.92	762.92
	360	8.69	762.69
	380	8.46	762.46
	400	8.24	762.24
	420	8.02	762.02
	440	7.81	761.81
	460	7.61	761.61
	464	7.57	761.57


Design Condition n500+3 Hydraulic Grade Line for the
Midwest Cold Storage Pump Plant Sta 194+60
Armourdale Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Design Conditions n500+5 - Assumes no Wells			
Elevation New Top of levee (MSL)	773.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	19.4		
L1 (ft)	79		
L2 (ft)	100		
Le (ft)	750		
ho	15.66		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-179	19.40	773.40
Pump Plant Location	0	15.66	769.66
	20	15.24	769.24
	40	14.84	768.84
	60	14.45	768.45
	80	14.07	768.07
	100	13.70	767.70
	120	13.34	767.34
	140	12.99	766.99
	160	12.65	766.65
	180	12.32	766.32
	200	11.99	765.99
	220	11.68	765.68
	240	11.37	765.37
	260	11.07	765.07
	280	10.78	764.78
	300	10.50	764.50
	320	10.22	764.22
	340	9.95	763.95
	360	9.69	763.69
	380	9.43	763.43
	400	9.19	763.19
	420	8.94	762.94
	440	8.71	762.71
	460	8.48	762.48
	464	8.43	762.43

Design Condition n500+5 Hydraulic Grade Line for the Midwest Cold Storage Pump Plant Sta 194+60 Armourdale Levee Unit



Armourdale Kansas - Hydraulic Grade Line for the Shawnee Pump Station Sta 230+80 no Wells **Assumes**

ARM Shawnee PP 230+80 No Wells.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/10/2007

Checked by : Bellew EC-GD GMB
Date : 02/12/07

NOTES:
1. T-wall on existing levee raise proposed
2. L_2 not revised for minor change due to raise

Elevation Used

Top of Existing Levee:	765.3 (From O&M Manual)
500 year + 0':	766.4
500 year + 3':	769.4
500 year + 5':	771.4
Ground Surface Land Side:	750.0
Ground Surface River Side:	738.0 (Average blanket thickness over slope)
Bottom of Blanket:	725.0
Top of Bedrock:	658.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

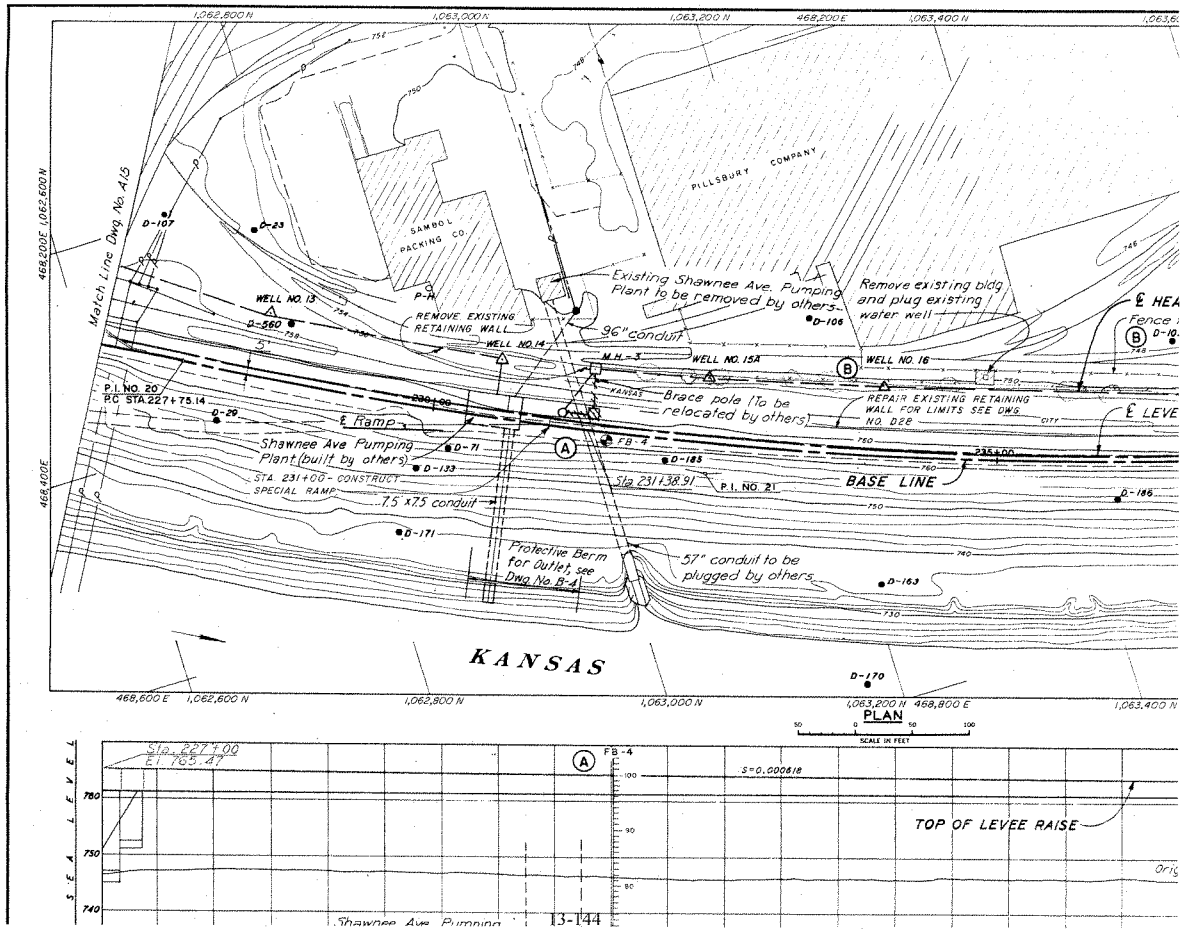
NOMENCLATURE

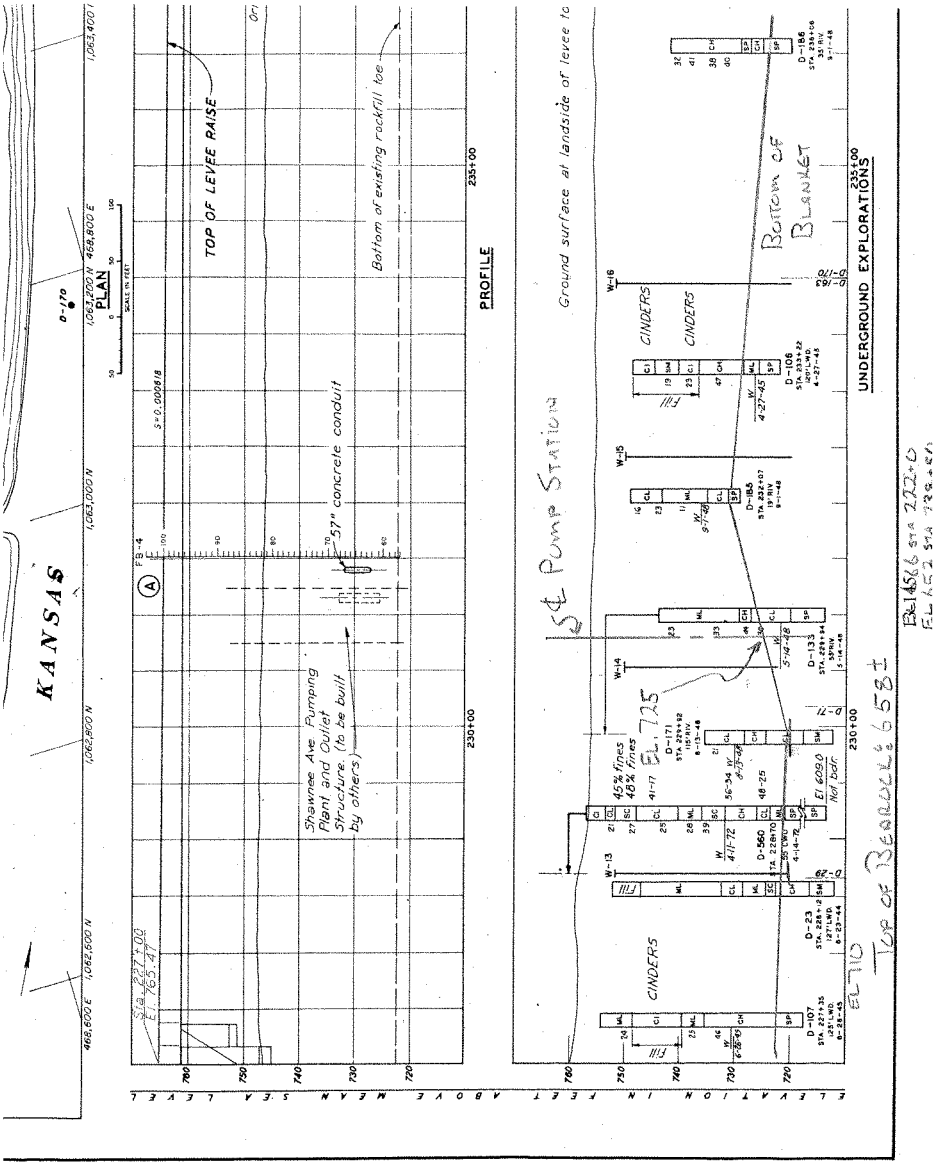
SHAWNEE PUMP PLANT
FEB 2007

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(Kf/Kb) * Df * Db]^{1/2}$
D_{bw}	= levee toe blanket thickness	h_o	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundation	i_e	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(2.8/C-1)}) / (e^{(2.8/C-1)})$
H	= max head or levee height	L_2	= landside effective length
L_2	= levee base width	L_3	= total effective length
		L'_3	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								Remarks
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bw} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	
230+80	Existing Conditions		400	400	25.0	19.0	13.0	67.0	120	15.3	EC
230+80	n500 + 0		400	400	25.0	19.0	13.0	67.0	120	16.4	n500
230+80	n500 +3		400	400	25.0	19.0	13.0	67.0	120	19.4	n500+3
230+80	n500 + 5		400	400	25.0	19.0	13.0	67.0	120	21.4	n500+5

Analysis											
C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_1 Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
590	819	118	100	819	1037	12.08	0.48	0.84	1.74		
590	819	118	100	819	1037	12.95	0.52	0.84	1.62		
590	819	118	100	819	1037	15.31	0.61	0.84	1.37		
590	819	118	100	819	1037	16.89	0.69	0.84	0.94		


Match Line Dwg. No. A15
PILLSBURY COMPANY
SAMBOL PACKING CO.
Existing Shawnee Ave. Pumping Plant to be removed by others
Remove existing bldg and plug existing water well
REMOVE EXISTING RETAINING WALL
96" conduit
WELL NO. 13
WELL NO. 14
WELL NO. 15A
WELL NO. 16
M.H. 3
P.I. NO. 20
P.C. STA. 227+75.14
℄ Ramp
Shawnee Ave Pumping Plant (built by others)
STA 231+00 - CONSTRUCT SPECIAL RAMP
7.5 X 7.5 conduit
Brace pole (to be relocated by others)
REPAIR EXISTING RETAINING WALL FOR LIMITS SEE DWG. NO. D22
KANSAS
CITY
Sta. 231+38.91
P.I. NO. 21
BASE LINE
Protective Berm for Outlet, see Dwg. No. B-4
57" conduit to be plugged by others
KANSAS
PLAN
SCALE IN FEET
SEA LEVEL
Sta. 227+00
El. 765.47
S=0.000618
TOP OF LEVEE RAISE
Shawnee Ave. Pumping

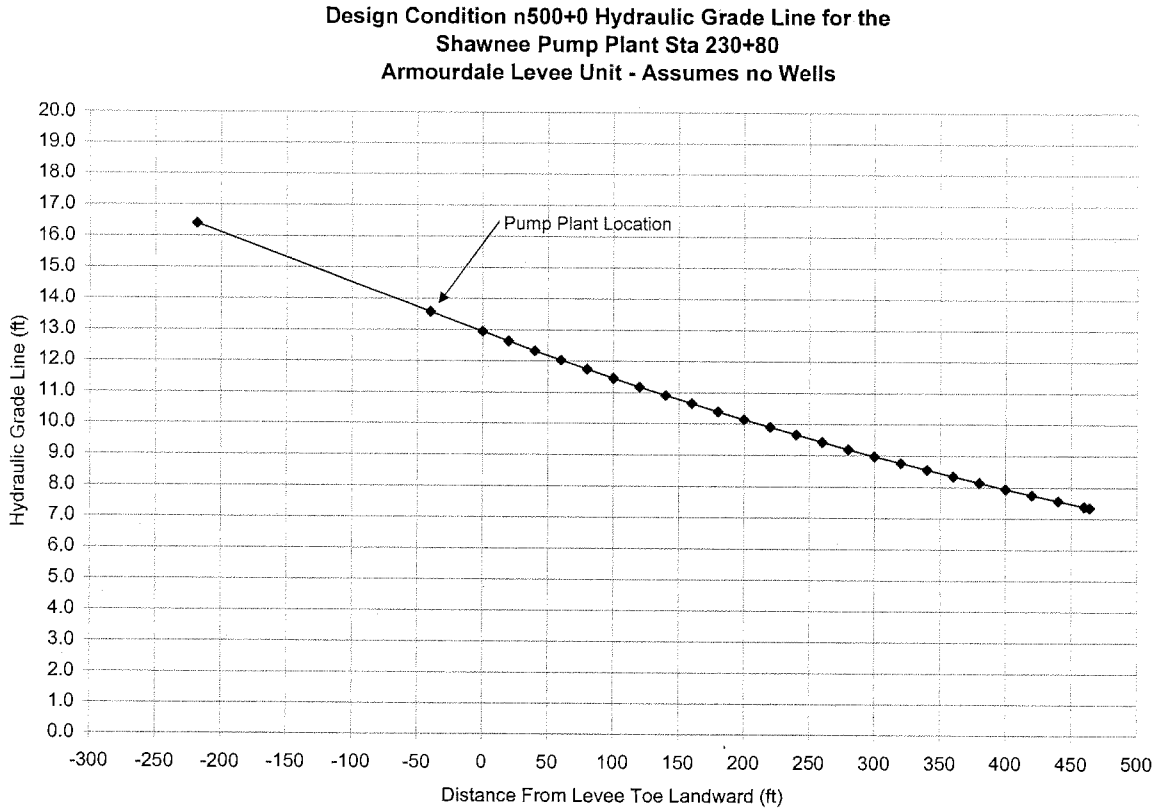

KANSAS
PLAN
PROFILE
UNDERGROUND EXPLORATIONS
TOP OF LEVEE RAISE
Bottom of existing rockfill toe
57" concrete conduit
Shawnee Ave. Pumping Plant and Outlet Structure. (to be built by others)
Ground surface at landside of levee to
S&L Pump Station
Bottom of Blanket
CINDERS
Top of Bedrock: 658±
EL 725
EL 710
45% fines
48% fines
Sta. 227+00
El. 765.47
El. 609.0
Not bdr.
230+00
235+00
468,600 E
468,800 E
1,062,600 N
1,062,800 N
1,063,000 N
1,063,200 N
1,063,400 N
S=0.000818
FB-4
D-170
D-107
D-23
D-560
D-171
D-133
D-185
D-106
D-186
W-13
W-14
W-15
W-16
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
700
720
730
740
750
760

Hydraulic Grade Line for the Shawnee Pump Plant
Existing Conditions - Assumes No Wells

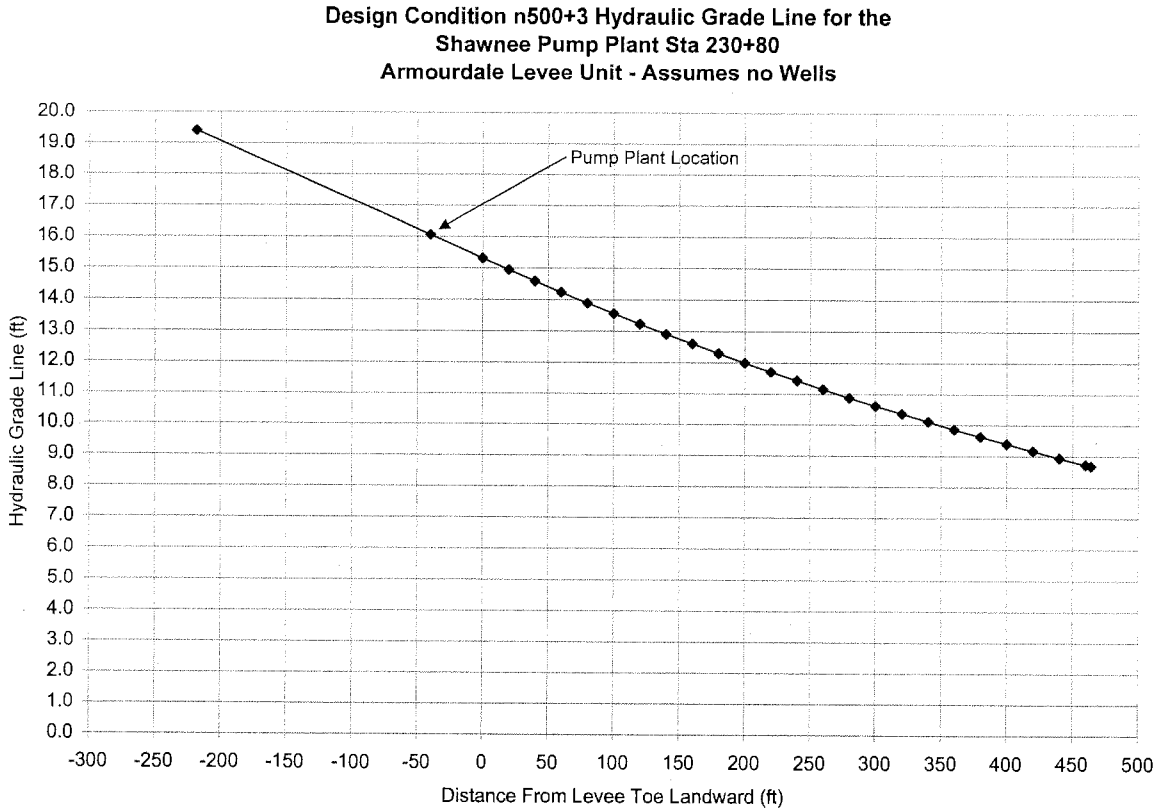
Elevation Top of levee (MSL)	765.3	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	15.3		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	12.1		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	15.30	765.30
Pump Plant Location	-40	12.67	762.67
	0	12.08	762.08
	20	11.79	761.79
	40	11.50	761.50
	60	11.22	761.22
	80	10.95	760.95
	100	10.69	760.69
	120	10.43	760.43
	140	10.18	760.18
	160	9.93	759.93
	180	9.69	759.69
	200	9.46	759.46
	220	9.23	759.23
	240	9.01	759.01
	260	8.79	758.79
	280	8.58	758.58
	300	8.37	758.37
	320	8.17	758.17
	340	7.97	757.97
	360	7.78	757.78
	380	7.59	757.59
	400	7.41	757.41
	420	7.23	757.23
	440	7.06	757.06
	460	6.89	756.89
	464	6.85	756.85

Hydraulic Grade Line for the Shawnee Pump Plant
Design Conditions n500+0 - Assumes no Wells

Elevation Top of levee (MSL)	766.4	n500+0	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	16.4		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	12.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	16.40	766.40
Pump Plant Location	-40	13.58	763.58
	0	12.95	762.95
	20	12.63	762.63
	40	12.33	762.33
	60	12.03	762.03
	80	11.74	761.74
	100	11.46	761.46
	120	11.18	761.18
	140	10.91	760.91
	160	10.65	760.65
	180	10.39	760.39
	200	10.14	760.14
	220	9.89	759.89
	240	9.66	759.66
	260	9.42	759.42
	280	9.20	759.20
	300	8.97	758.97
	320	8.76	758.76
	340	8.55	758.55
	360	8.34	758.34
	380	8.14	758.14
	400	7.94	757.94
	420	7.75	757.75
	440	7.56	757.56
	460	7.38	757.38
	464	7.34	757.34

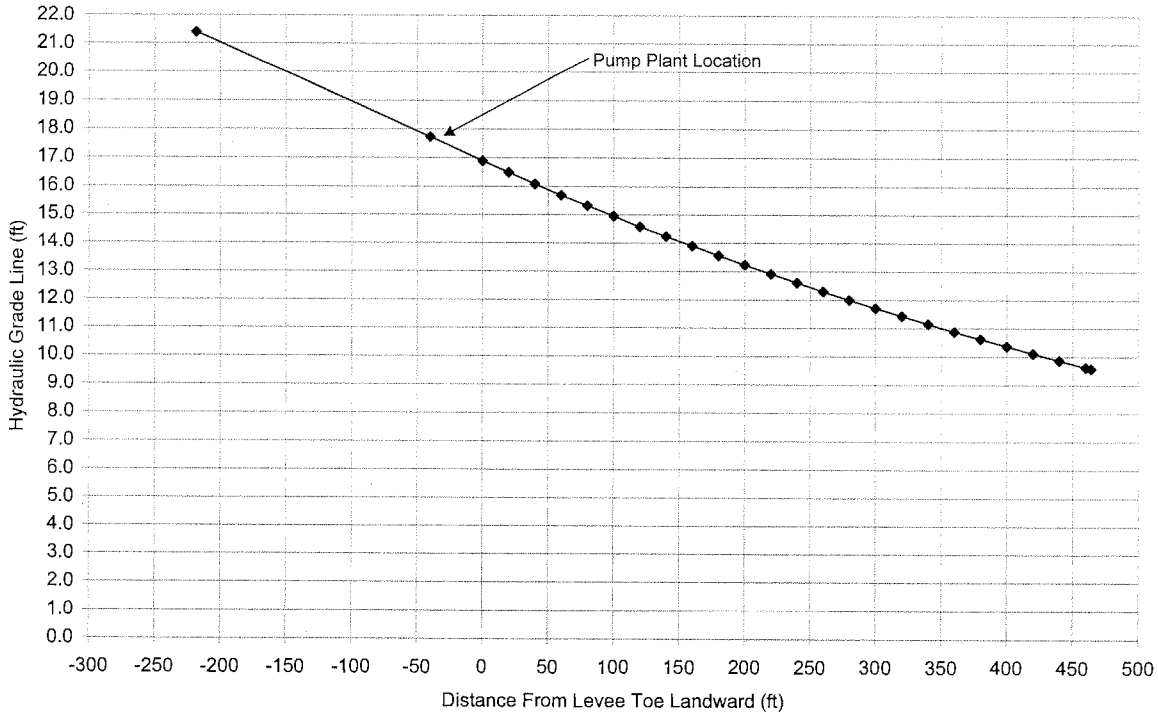

Design Condition n500+0 Hydraulic Grade Line for the
Shawnee Pump Plant Sta 230+80
Armourdale Levee Unit - Assumes no Wells
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Shawnee Pump Plant Design Conditions n500+3 - Assumes no Wells			
Elevation New Top of levee (MSL)	769.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	19.4		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	15.31		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-218	19.40	769.40
Pump Plant Location	-40	16.06	766.06
	0	15.31	765.31
	20	14.94	764.94
	40	14.58	764.58
	60	14.23	764.23
	80	13.89	763.89
	100	13.55	763.55
	120	13.23	763.23
	140	12.91	762.91
	160	12.60	762.60
	180	12.29	762.29
	200	11.99	761.99
	220	11.71	761.71
	240	11.42	761.42
	260	11.15	761.15
	280	10.88	760.88
	300	10.62	760.62
	320	10.36	760.36
	340	10.11	760.11
	360	9.86	759.86
	380	9.63	759.63
	400	9.39	759.39
	420	9.17	759.17
	440	8.95	758.95
	460	8.73	758.73
	464	8.69	758.69


Design Condition n500+3 Hydraulic Grade Line for the
Shawnee Pump Plant Sta 230+80
Armourdale Levee Unit - Assumes no Wells
Pump Plant Location
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Shawnee Pump Plant Design Conditions n500+5 - Assumes on Wells			
Elevation New Top of levee (MSL)	771.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	21.4		
L1 (ft)	118		
L2 (ft)	100		
Le (ft)	819		
ho	16.89		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-218	21.40	771.40
Pump Plant Location	-40	17.72	767.72
~~Pump Plant Location~~	0	16.89	766.89
	20	16.49	766.49
	40	16.09	766.09
	60	15.70	765.70
	80	15.32	765.32
	100	14.95	764.95
	120	14.59	764.59
	140	14.24	764.24
	160	13.89	763.89
	180	13.56	763.56
	200	13.23	763.23
	220	12.91	762.91
	240	12.60	762.60
	260	12.30	762.30
	280	12.00	762.00
	300	11.71	761.71
	320	11.43	761.43
	340	11.15	761.15
	360	10.88	760.88
	380	10.62	760.62
	400	10.36	760.36
	420	10.11	760.11
	440	9.87	759.87
	460	9.63	759.63
	464	9.58	759.58

Design Condition n500+5 Hydraulic Grade Line for the Shawnee Pump Plant Sta 230+80 Armourdale Levee Unit - Assumes no Wells



Armourdale Kansas - Hydraulic Grade Line for the KCS Railroad Pump Station Sta 276+80
Assume No Wells and Ignore the Slot

ARM KCS RR PP 276+80.xls

NOMENCLATURE KCS RAILROAD PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kuzniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD
Date : 02/12/07

NOTES:
1. New floodwall raise proposed
2. L_2 not revised for minor change due to floodwall raise

Elevation Used

Top of Existing Levee:	762.5 (From O&M Manual)
500 year + 0':	762.4 (Equal to Existing Conditions)
500 year + 3':	765.4
500 year + 5':	767.4
Ground Surface Land Side:	750.0 (Ignore the slot)
Ground Surface River Side:	737.0 (For average blanket thickness)
Bottom of Blanket:	722.0
Top of Bedrock:	672.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

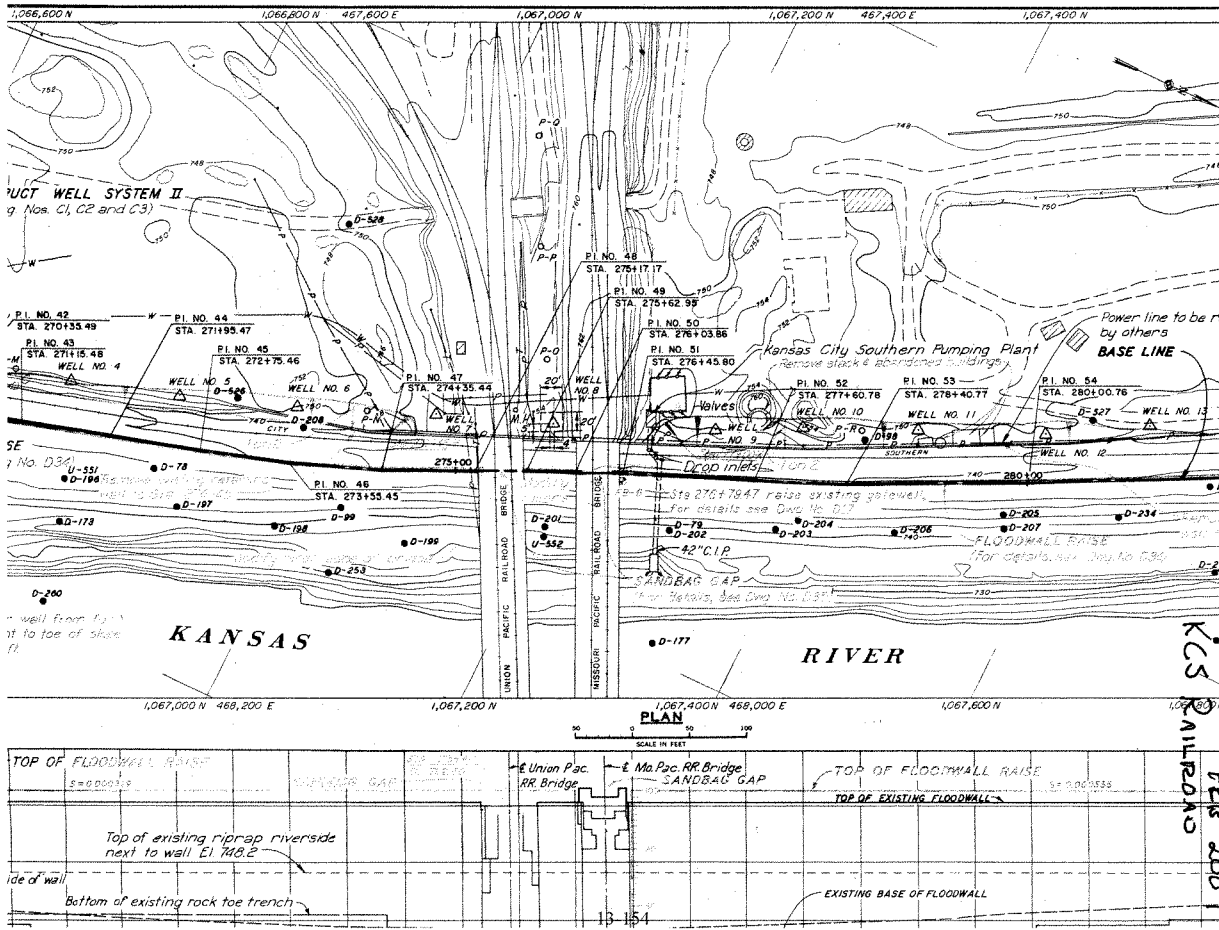
Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_r/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_r/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K/Kb) * Df * Db]^{1/2}$
D_{bw}	= levee toe blanket thickness	h_o	= head above tailwater at levee toe (w/ berm)
D_{bR}	= riverside blanket thickness	i_c	= critical seepage gradient
Df	= thickness of pervious foundat.	i_o	= seepage gradient
L_{iR}	= length of riverside blanket	L_e	= riverside effective length
L_{iL}	= length of landside blanket	where L_e	$= C * e^{(0.0693 H)} / (e^{(0.0693 H)} - 1)$
H	= max head or levee height	L_R	= landside effective length
L_2	= levee base width	L_1	= total effective length
		L_T	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								Remarks
			$(K_r/K_b)_R$	$(K_r/K_b)_L$	D_{bL} Feet	D_{bw} Feet	D_{bR} Feet	D_f Feet	L_{iR} Feet	H Feet	
276+80	Existing Conditions		400	400	28.0	22.0	15.0	50.0	120	12.5	EC
276+80	n500 + 0		400	400	28.0	22.0	15.0	50.0	120	12.4	n500
276+80	n500 +3		400	400	28.0	22.0	15.0	50.0	120	15.4	n500+3
276+80	n500 + 5		400	400	28.0	22.0	15.0	50.0	120	17.4	n500+5

Analysis											
C_R Feet	C_L Feet	L_{-1} Feet	L_2 Feet	L_e Feet	L_1 Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
548	748	118	20	748	886	10.55	0.38	0.84	2.23		
548	748	118	20	748	886	10.47	0.37	0.84	2.25		
548	748	118	20	748	886	13.00	0.48	0.84	1.81		
548	748	118	20	748	886	14.69	0.52	0.84	1.60		

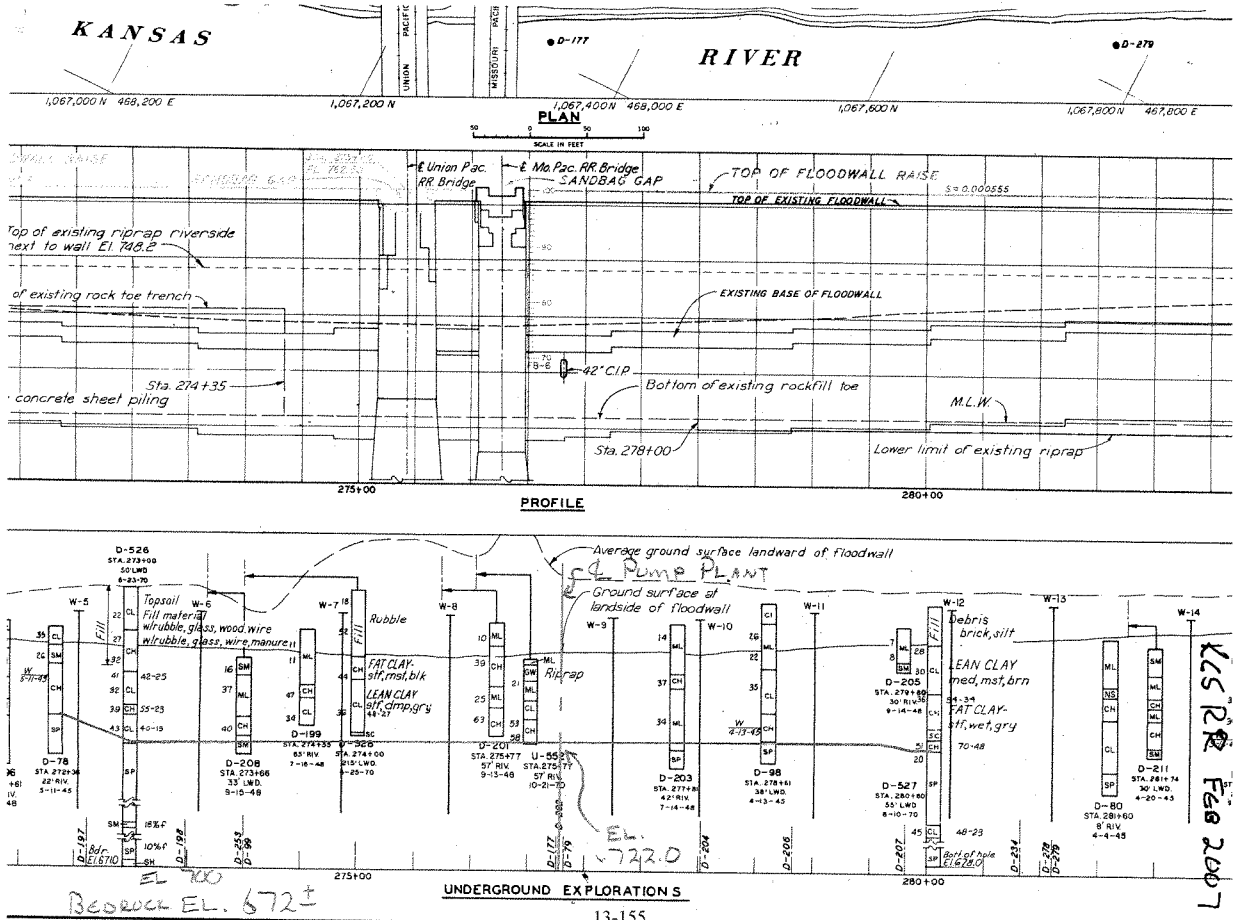
620



KCS RAILROAD

FEB 2007

621



13-155

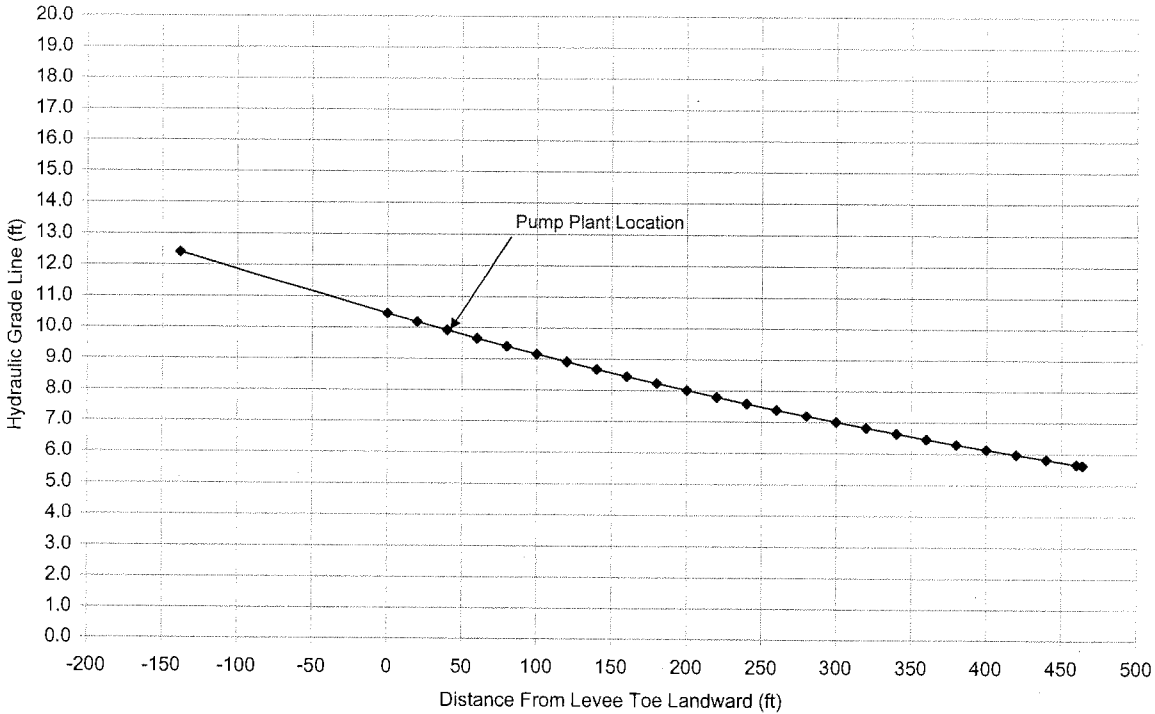
KCS RR Fig 2007

Hydraulic Grade Line for the KCS Railroad Pump Plant
Existing Conditions - Assumes No Wells and No Slot

Elevation Top of levee (MSL)	762.5	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	12.5		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	10.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	12.50	762.50
	0	10.55	760.55
	20	10.27	760.27
Pump Plant Location	40	10.00	760.00
	60	9.74	759.74
	80	9.48	759.48
	100	9.23	759.23
	120	8.99	758.99
	140	8.75	758.75
	160	8.52	758.52
	180	8.30	758.30
	200	8.08	758.08
	220	7.86	757.86
	240	7.66	757.66
	260	7.46	757.46
	280	7.26	757.26
	300	7.07	757.07
	320	6.88	756.88
	340	6.70	756.70
	360	6.52	756.52
	380	6.35	756.35
	400	6.18	756.18
	420	6.02	756.02
	440	5.86	755.86
	460	5.71	755.71
	464	5.68	755.68

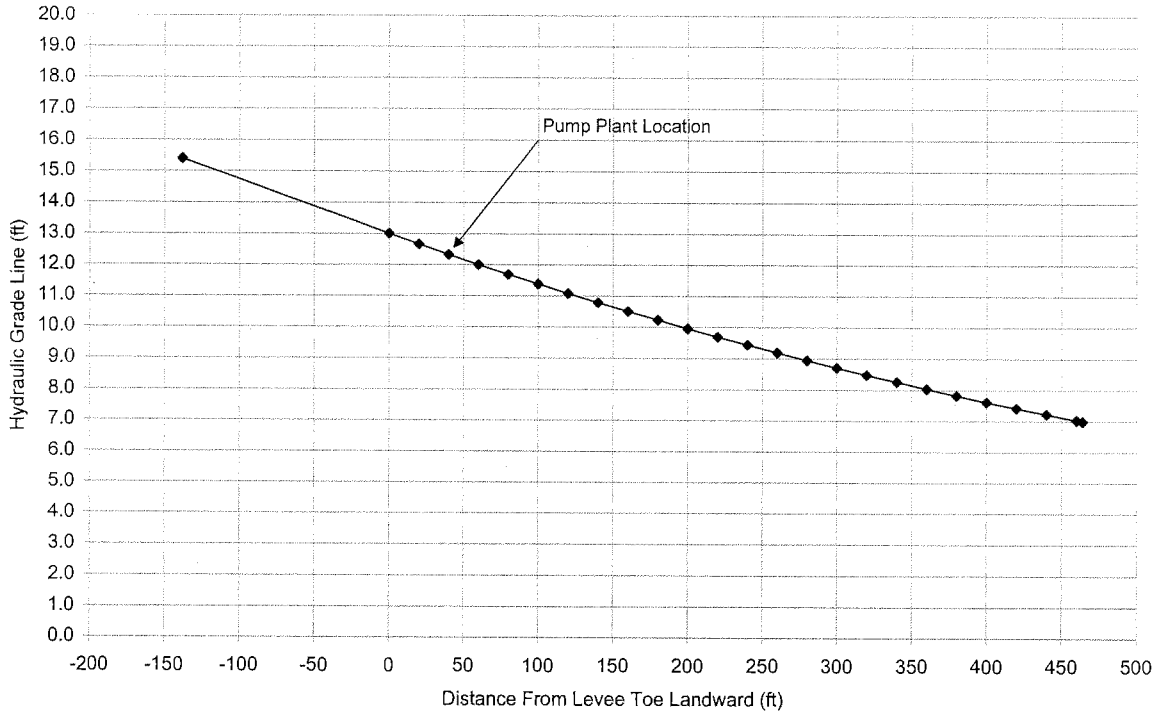
Hydraulic Grade Line for the KCS Railroad Pump Plant Design Conditions n500+0 - Assumes No Wells and No Slot			
Elevation Top of levee (MSL)	762.4	n500+0	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	12.4		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	10.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	12.40	762.40
	0	10.47	760.47
	20	10.19	760.19
Pump Plant Location	40	9.92	759.92
	60	9.66	759.66
	80	9.41	759.41
	100	9.16	759.16
	120	8.92	758.92
	140	8.68	758.68
	160	8.45	758.45
	180	8.23	758.23
	200	8.01	758.01
	220	7.80	757.80
	240	7.60	757.60
	260	7.40	757.40
	280	7.20	757.20
	300	7.01	757.01
	320	6.83	756.83
	340	6.65	756.65
	360	6.47	756.47
	380	6.30	756.30
	400	6.13	756.13
	420	5.97	755.97
	440	5.81	755.81
	460	5.66	755.66
	464	5.63	755.63

Design Condition n500+0 Hydraulic Grade Line for the KCS Railroad Pump Plant Sta 276+80 Armourdale Levee Unit - Assumes No Wells and No Slot



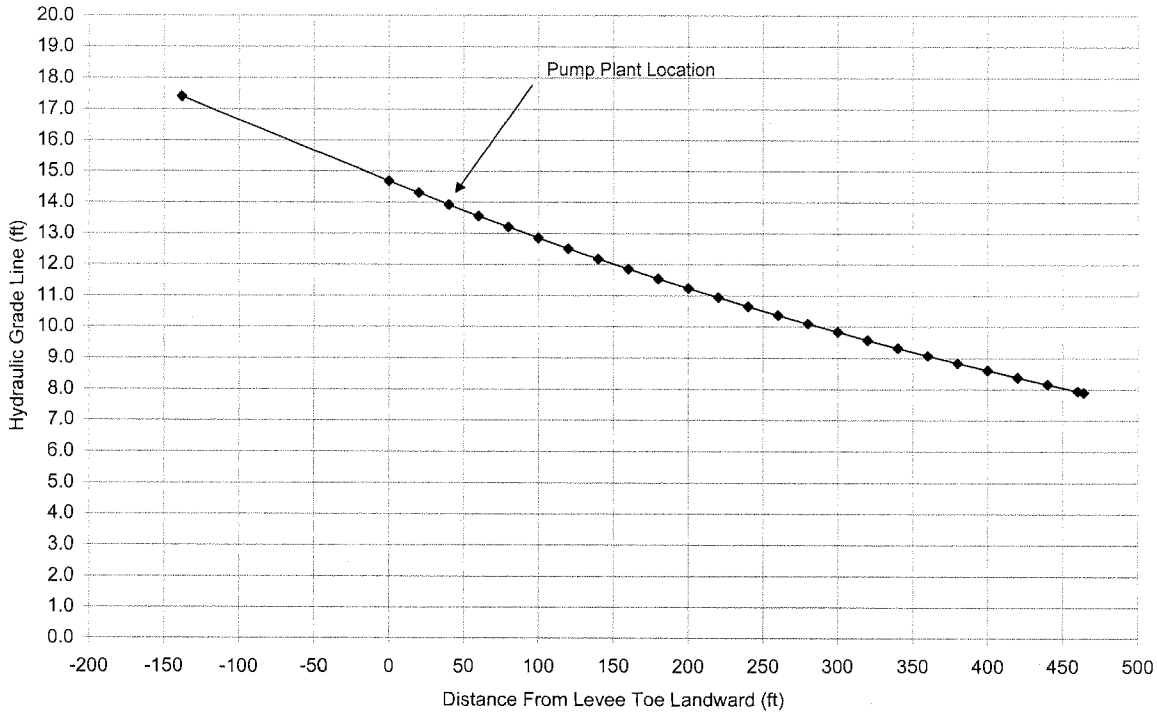
Hydraulic Grade Line for the KCS Railroad Pump Plant Design Conditions n500+3 - Assumes No Wells and No Slot			
Elevation New Top of levee (MSL)	765.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.4		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	13.00		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-138	15.40	765.40
	0	13.00	763.00
	20	12.66	762.66
Pump Plant Location	40	12.32	762.32
	60	12.00	762.00
	80	11.68	761.68
	100	11.37	761.37
	120	11.07	761.07
	140	10.78	760.78
	160	10.50	760.50
	180	10.22	760.22
	200	9.95	759.95
	220	9.69	759.69
	240	9.43	759.43
	260	9.18	759.18
	280	8.94	758.94
	300	8.71	758.71
	320	8.48	758.48
	340	8.25	758.25
	360	8.04	758.04
	380	7.82	757.82
	400	7.62	757.62
	420	7.42	757.42
	440	7.22	757.22
	460	7.03	757.03
	464	6.99	756.99

Design Condition n500+3 Hydraulic Grade Line for the KCS Railroad Pump Plant Sta 276+80 Armourdale Levee Unit - Assumes No Wells and No Slot



Hydraulic Grade Line for the KCS Railroad Pump Plant Design Conditions n500+5 - Assumes No Wells and No Slot			
Elevation New Top of levee (MSL)	767.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	17.4		
L1 (ft)	118		
L2 (ft)	20		
Le (ft)	748		
ho	14.69		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-138	17.40	767.40
	0	14.69	764.69
	20	14.30	764.30
Pump Plant Location	40	13.92	763.92
	60	13.56	763.56
	80	13.20	763.20
	100	12.85	762.85
	120	12.51	762.51
	140	12.18	762.18
	160	11.86	761.86
	180	11.55	761.55
	200	11.24	761.24
	220	10.95	760.95
	240	10.66	760.66
	260	10.38	760.38
	280	10.10	760.10
	300	9.84	759.84
	320	9.58	759.58
	340	9.33	759.33
	360	9.08	759.08
	380	8.84	758.84
	400	8.61	758.61
	420	8.38	758.38
	440	8.16	758.16
	460	7.94	757.94
	464	7.90	757.90

Design Condition n500+5 Hydraulic Grade Line for the KCS Railroad Pump Plant Sta 276+80 Armourdale Levee Unit - Assumes No Wells and No Slot



Armourdale Kansas - Hydraulic Grade Line for the PBI Gordon Pump Station Sta 289+60

ARM PBI Gordon PP 289+60.xls

NOMENCLATURE

PBI GORDON PUMP PLANT FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kozniakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD
Date : 07/28/07 GMB

NOTES:
1. Modify floodwall proposed

Input	Parameters	Output	Calculations
$(K_r/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_r/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K_f/K_b) * D_f * D_b]^{1/2}$
D_{bc}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{bc}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundate	i_e	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	= $C * (e^{(D_f/C)/2}) / (e^{(D_f/C)/2})$
H	= max head or levee height	L_o	= landside effective length
L_2	= levee base width	L_1	= total effective length
		L'_1	= Total Effective Length + 1/2 of Berm

Elevation Used

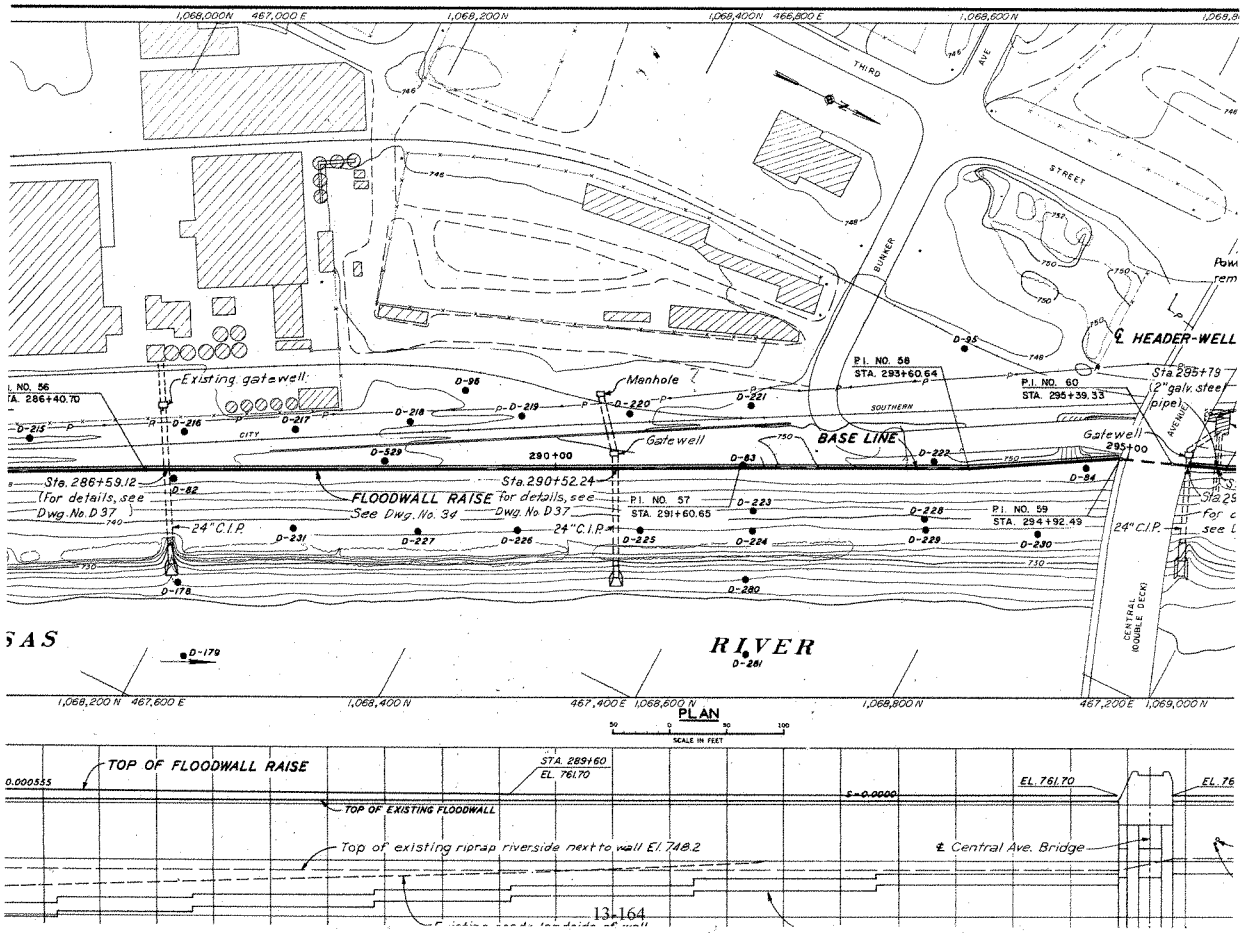
Top of Existing Levee:	761.7 (From O&M Manual)
500 year + 0':	761.0 (Lower than existing conditions)
500 year + 3':	764.0
500 year + 5':	766.0
Ground Surface Land Side:	747.0
Ground Surface River Side:	732.0 (To obtain average thickness of blanket due to slope)
Bottom of Blanket:	720.0
Top of Bedrock:	678.0

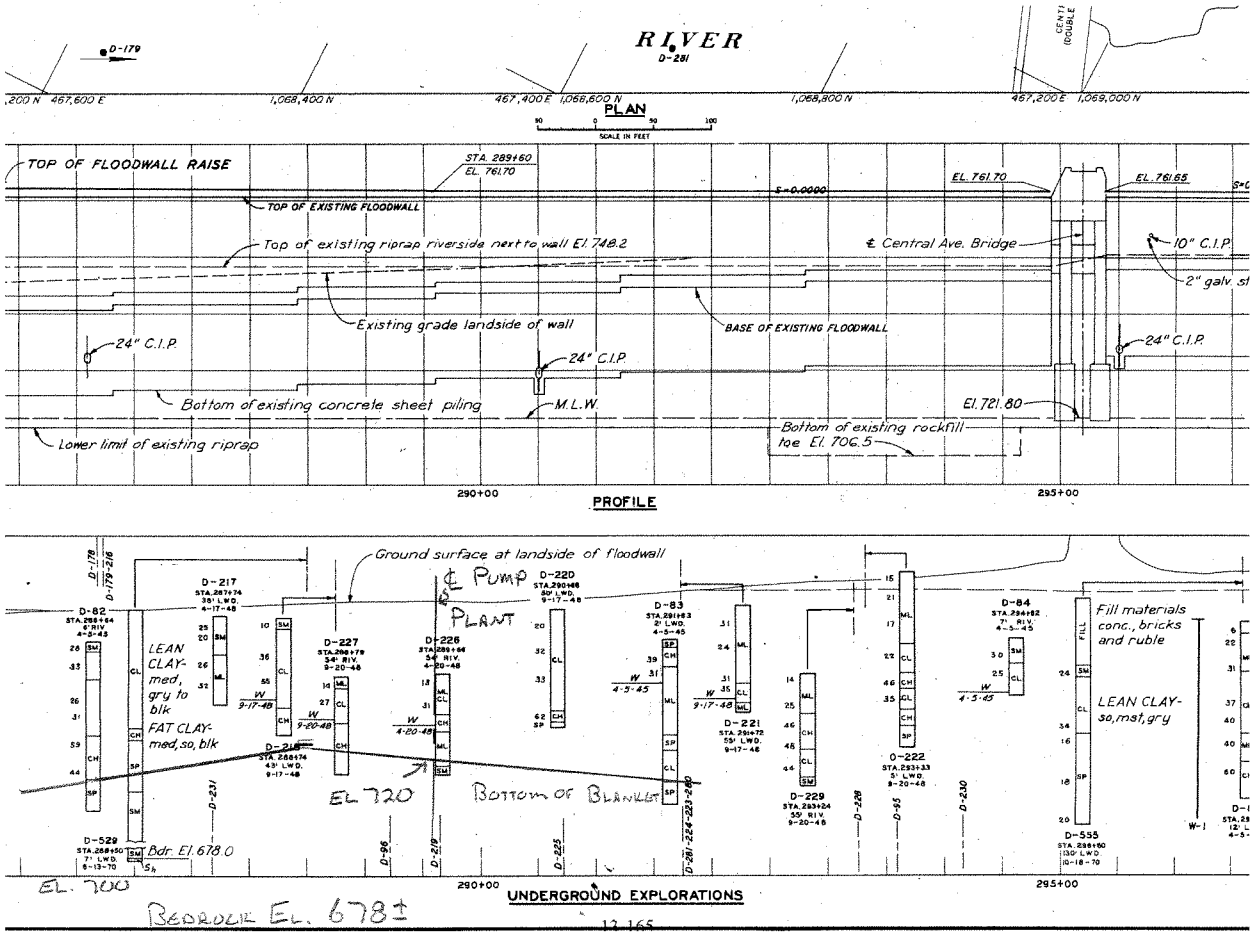
Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K/KB)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Station			Input Parameters								
			$(K_r/K_b)_R$	$(K_r/K_b)_L$	D_{bL} Feet	D_{bc} Feet	D_{bc} Feet	D_f Feet	L_R Feet	H Feet	Remarks
289+60	Existing Conditions		400	400	27.0	20.0	12.0	42.0	80	14.7	EC
289+60	n500 + 0		400	400	27.0	20.0	12.0	42.0	80	13.4	n500
289+60	n500 +3		400	400	27.0	20.0	12.0	42.0	80	16.4	n500+3
289+60	n500 + 5		400	400	27.0	20.0	12.0	42.0	80	18.4	n500+5

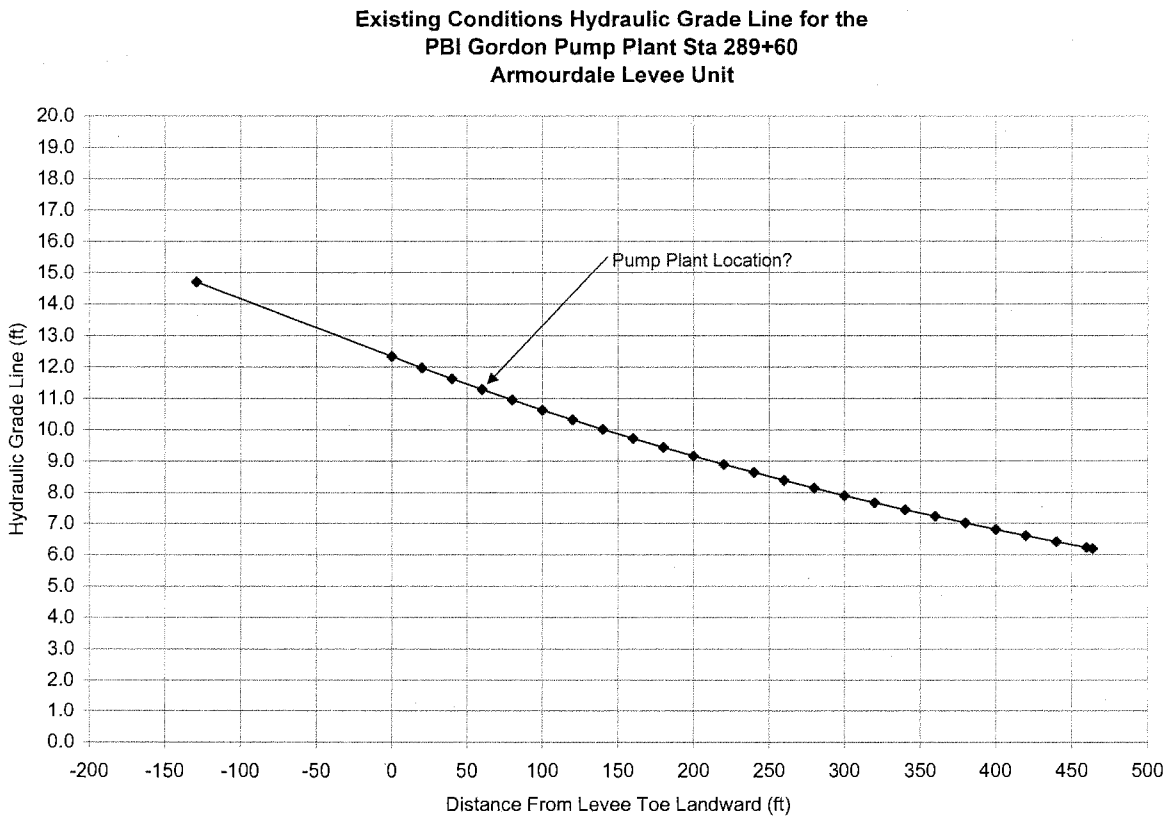
Station	Analysis											
	C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_f Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS ic/io		
289+60	449	673	79	50	673	803	12.33	0.46	0.84	1.84		
289+60	449	673	79	50	673	803	11.24	0.42	0.84	2.02		
289+60	449	673	79	50	673	803	13.78	0.51	0.84	1.65		
289+60	449	673	79	50	673	803	15.44	0.57	0.84	1.47		


1,068,000N 467,000 E
1,068,200N
1,068,400N 466,800 E
1,068,600N
THIRD
AVE
STREET
BUNKER
SOUTHERN
CITY
AVENUE
℄ HEADER-WELL
P.I. NO. 58
STA. 293+60.64
P.I. NO. 60
STA. 295+39.33
Sta 295+79
(2" galv. steel pipe)
Existing gatewell
Manhole
Gatewell
BASE LINE
Gatewell
295+00
290+00
Sta. 286+59.12
(For details, see Dwg. No. D 37)
FLOODWALL RAISE for details, see See Dwg. No. 34 Dwg. No. D 37
Sta. 290+52.24
P.I. NO. 57
STA. 291+60.65
P.I. NO. 59
STA. 294+92.49
24" C.I.P.
CENTRAL (DOUBLE DECK)
RIVER
1,068,200 N 467,600 E
1,068,400 N
467,400 E 1,068,600 N
1,068,800 N
467,200 E 1,069,000 N
PLAN
SCALE IN FEET
TOP OF FLOODWALL RAISE
STA. 289+60
EL. 761.70
TOP OF EXISTING FLOODWALL
S=0.0000
EL. 761.70
Top of existing riprap riverside next to wall El. 748.2
℄ Central Ave. Bridge


RIVER
D-281
D-179
1,068,400 N
467,400 E
1,068,600 N
1,068,800 N
467,200 E
1,069,000 N
PLAN
SCALE IN FEET
TOP OF FLOODWALL RAISE
STA. 289+60
EL. 761.70
TOP OF EXISTING FLOODWALL
S=0.0000
EL. 761.70
EL. 761.65
Top of existing riprap riverside next to wall El. 748.2
℄ Central Ave. Bridge
10" C.I.P.
2" galv. st
Existing grade landside of wall
BASE OF EXISTING FLOODWALL
24" C.I.P.
24" C.I.P.
24" C.I.P.
Bottom of existing concrete sheet piling
M.L.W.
El. 721.80
Bottom of existing rockfill toe El. 706.5
Lower limit of existing riprap
290+00
295+00
PROFILE
Ground surface at landside of floodwall
℄ PUMP PLANT
D-82
D-217
D-227
D-226
D-220
D-83
D-221
D-229
D-222
D-84
D-555
D-529
LEAN CLAY-med, gry to blk
FAT CLAY-med, so, blk
EL 720
BOTTOM OF BLANKET
Fill materials conc., bricks and ruble
LEAN CLAY-so, mst, gry
Bdr. El. 678.0
EL. 700
BEDROCK EL. 678±
290+00
295+00
UNDERGROUND EXPLORATIONS

Hydraulic Grade Line for the PBI Gordon Pump Plant
Existing Conditions

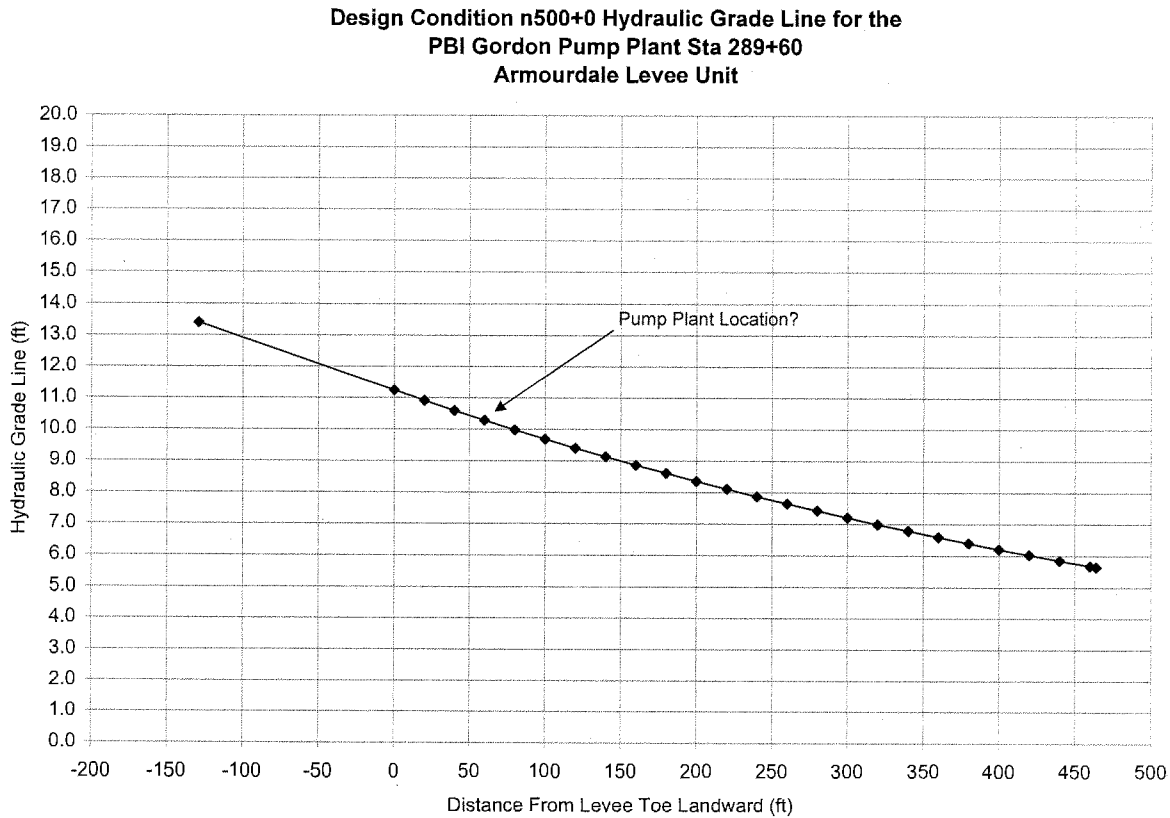
Elevation Top of levee (MSL)	761.7	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	14.7		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	12.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	14.70	761.70
	0	12.33	759.33
	20	11.97	758.97
	40	11.62	758.62
Pump Plant Location?	60	11.28	758.28
	80	10.95	757.95
	100	10.63	757.63
	120	10.32	757.32
	140	10.02	757.02
	160	9.73	756.73
	180	9.44	756.44
	200	9.17	756.17
	220	8.90	755.90
	240	8.64	755.64
	260	8.38	755.38
	280	8.14	755.14
	300	7.90	754.90
	320	7.67	754.67
	340	7.45	754.45
	360	7.23	754.23
	380	7.02	754.02
	400	6.81	753.81
	420	6.61	753.61
	440	6.42	753.42
	460	6.23	753.23
	464	6.19	753.19


Existing Conditions Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit
Pump Plant Location?
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

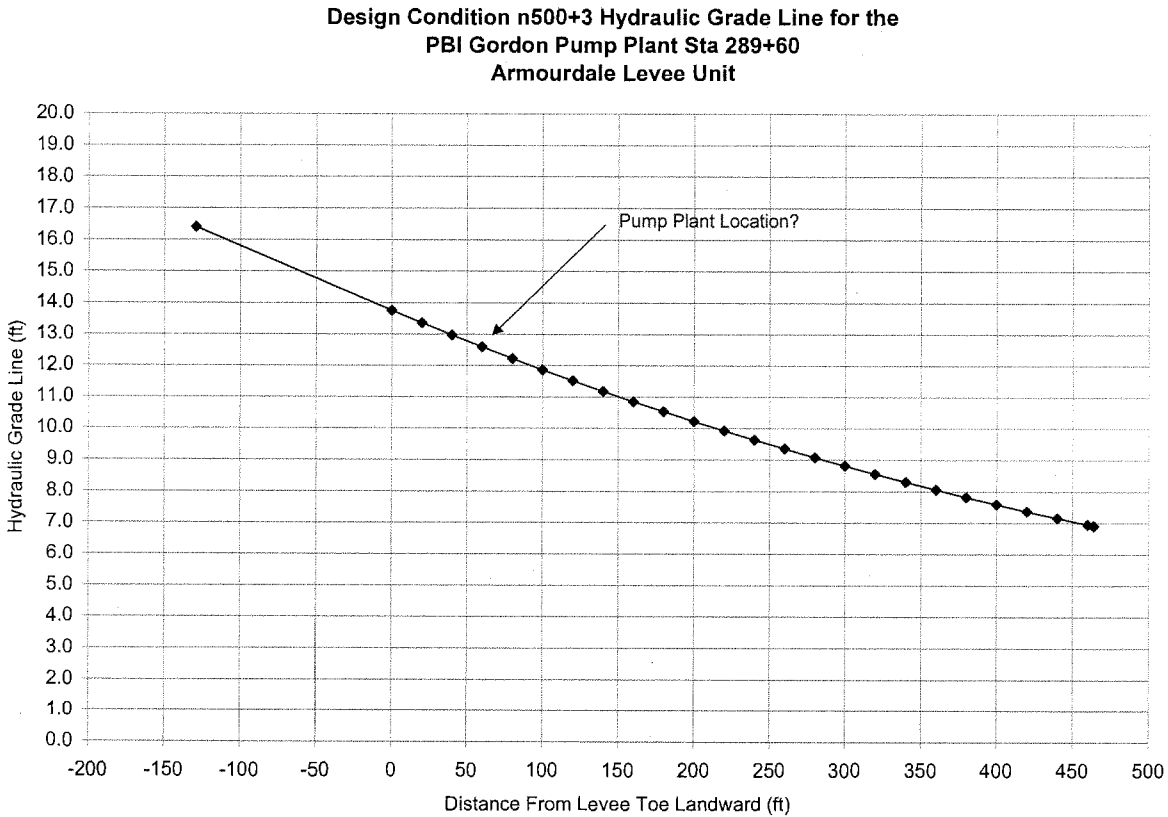
Hydraulic Grade Line for the PBI Gordon Pump Plant
Design Conditions n500+0

Elevation Top of levee (MSL)	761	n500+0
Elevation at Landside Levee Toe (MSL)	747	
Elevation Bottom of Blanket (MSL)	720	
Driving Head (ft)	13.4	
L1 (ft)	79	
L2 (ft)	50	
Le (ft)	673	
ho	11.2	

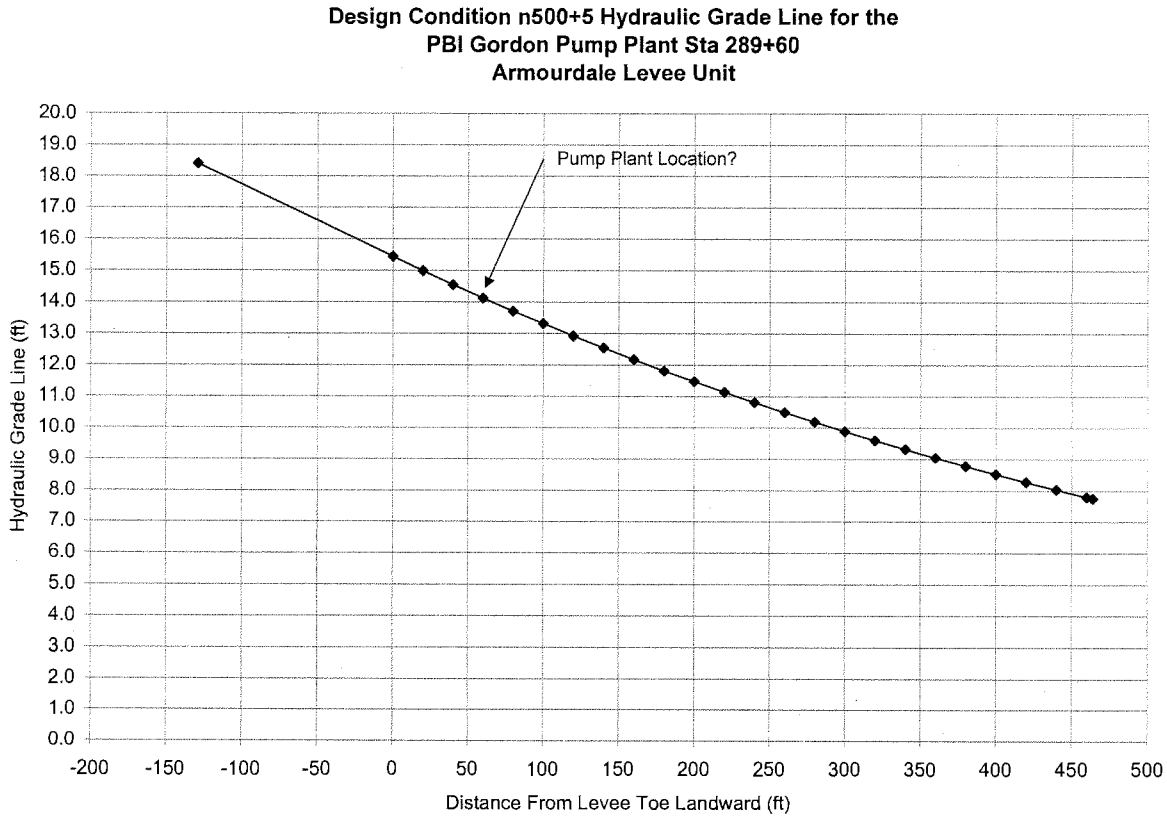
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	13.40	760.40
	0	11.24	758.24
	20	10.91	757.91
	40	10.60	757.60
Pump Plant Location?	60	10.29	757.29
	80	9.98	756.98
	100	9.69	756.69
	120	9.41	756.41
	140	9.13	756.13
	160	8.87	755.87
	180	8.61	755.61
	200	8.35	755.35
	220	8.11	755.11
	240	7.87	754.87
	260	7.64	754.64
	280	7.42	754.42
	300	7.20	754.20
	320	6.99	753.99
	340	6.79	753.79
	360	6.59	753.59
	380	6.40	753.40
	400	6.21	753.21
	420	6.03	753.03
	440	5.85	752.85
	460	5.68	752.68
	464	5.65	752.65


Design Condition n500+0 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit
Pump Plant Location?
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the PBI Gordon Pump Plant Design Conditions n500+3			
Elevation New Top of levee (MSL)	764	n500 +3	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	16.4		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	13.76		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-129	16.40	763.40
	0	13.76	760.76
	20	13.36	760.36
	40	12.97	759.97
Pump Plant Location?	60	12.59	759.59
	80	12.22	759.22
	100	11.86	758.86
	120	11.52	758.52
	140	11.18	758.18
	160	10.85	757.85
	180	10.53	757.53
	200	10.23	757.23
	220	9.93	756.93
	240	9.64	756.64
	260	9.35	756.35
	280	9.08	756.08
	300	8.81	755.81
	320	8.56	755.56
	340	8.31	755.31
	360	8.06	755.06
	380	7.83	754.83
	400	7.60	754.60
	420	7.38	754.38
	440	7.16	754.16
	460	6.95	753.95
	464	6.91	753.91


Design Condition n500+3 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit
Pump Plant Location?
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the PBI Gordon Pump Plant Design Conditions n500+5			
Elevation New Top of levee (MSL)	766	n500 +5	
Elevation at Landside Levee Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	18.4		
L1 (ft)	79		
L2 (ft)	50		
Le (ft)	673		
ho	15.44		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-129	18.40	765.40
	0	15.44	762.44
	20	14.99	761.99
	40	14.55	761.55
Pump Plant Location?	60	14.12	761.12
	80	13.71	760.71
	100	13.31	760.31
	120	12.92	759.92
	140	12.54	759.54
	160	12.17	759.17
	180	11.82	758.82
	200	11.47	758.47
	220	11.14	758.14
	240	10.81	757.81
	260	10.49	757.49
	280	10.19	757.19
	300	9.89	756.89
	320	9.60	756.60
	340	9.32	756.32
	360	9.05	756.05
	380	8.78	755.78
	400	8.52	755.52
	420	8.28	755.28
	440	8.03	755.03
	460	7.80	754.80
	464	7.75	754.75


Design Condition n500+5 Hydraulic Grade Line for the
PBI Gordon Pump Plant Sta 289+60
Armourdale Levee Unit
Pump Plant Location?
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Armourdale Kansas - Hydraulic Grade Line for the National Beef Pump Station Sta 295+80 Assuming No Wells

ARM National Beef PP 295+80 No Wells.xls

NOMENCLATURE

NATIONAL BEEF PUMP PLANT
FEB 2007

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS — Name : Kaczmiarowski EC-GD — Date : 2/11/2007 — Checked by : Hollow EC-GD — Date : 02/18/07 — GMB

NOTES:
1. Replace floodwall with landside levee proposed
2. L_3 revised for design conditions due to new levee

Elevation Used

Top of Existing Levee:	761.7 (From O&M Manual)
500 year + 0':	766.4 (Lower than existing conditions)
500 year + 3':	763.4
500 year + 5':	765.4
Ground Surface Land Side:	750.0 (For existing conditions)
Ground Surface River Side:	738.0 (To obtain average thickness of blanket due to slope)
Bottom of Blanket:	726.0
Top of Bedrock:	678.0

Recommended Permeability Ratio For Foundation Blanket Materials Chapter Underseepage Revision dated Oct 1998	
Blanket Material	Assumed Permeability Ratio, (K0/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K_f/K_b) * D] * D_b]^{1/2}$
D_{bw}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of previous foundation	i_e	= seepage gradient
L_R	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * \{e^{[2.8(C-1)]}\} / (e^{[2.8(C-1)]})$
H	= max head in levee height	L_2	= landside effective length
L_3	= levee base width	L_T	= total effective length
		L'_T	= Total Effective Length + 1/2 of Berm

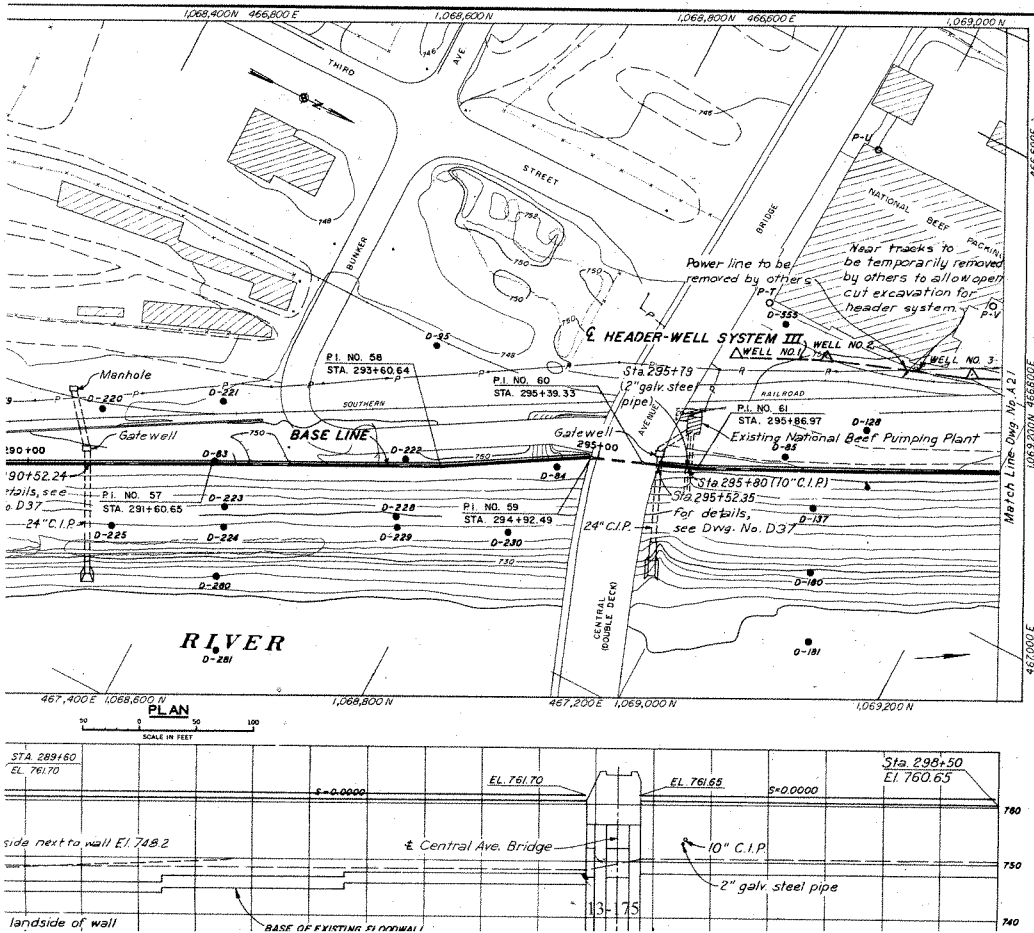
Input Parameters

Station			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bw} Feet	D_{br} Feet	D_f Feet	L_R Feet	H Feet	Remarks
295+80	Existing Conditions		400	400	24.0	18.0	12.0	48.0	80	11.7	EC
295+80	n500 + 0		400	400	24.0	18.0	12.0	48.0	50	10.4	n500
295+80	n500 +3		400	400	24.0	18.0	12.0	48.0	50	13.4	n500+3
295+80	n500 + 5		400	400	24.0	18.0	12.0	48.0	50	15.4	n500+5

Analysis

C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_3 Feet	L_T Feet	h_o Feet	i_o Ft/Ft	i_c Ft/Ft	FS to/in		
480	679	79	20	679	778	10.21	0.43	0.84	1.98		
480	679	50	100	679	829	8.52	0.35	0.84	2.37		
480	679	50	100	679	829	10.98	0.46	0.84	1.84		
480	679	50	100	679	829	12.62	0.53	0.84	1.60		

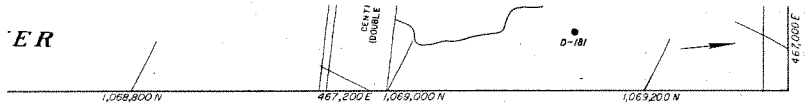
641

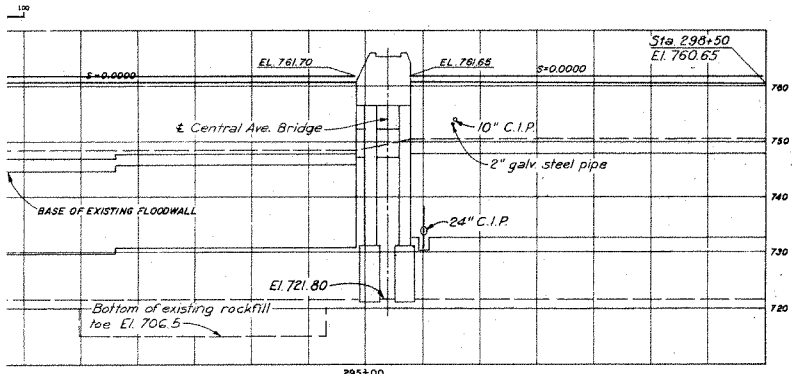


13-175

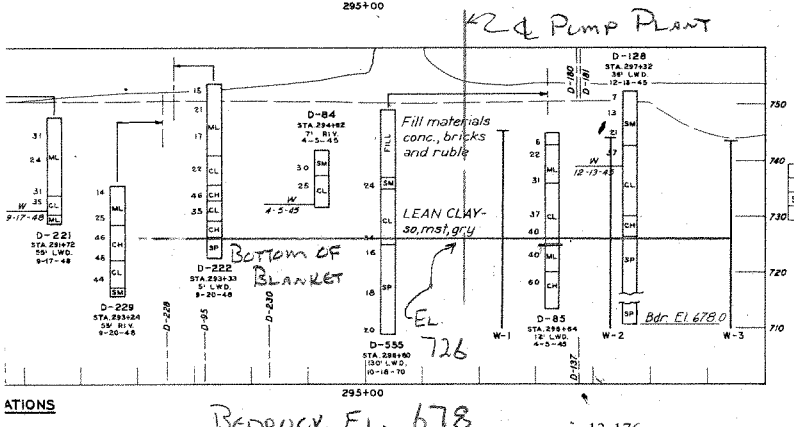
NOTES:
For General Notes, s
For details of relie
piezometers, see Dwg.
For Header Profile
Dwg. No. C1

National Beef Pump Plant.





NOTES:
For General Notes, see Dwg. No. A3
For details of relief wells, manholes & piezometers, see Dwg. Nos. C2 & C3
For Header Profile-Well System III, see Dwg. No. C1
For Legend - Existing Utilities, see Dwg. No. A3



ATIONS

13-176

RECORD DRAWING

MAY 1976

CONTRACT NO. DACW41-74-C-0093

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	9/20/78	

REVISIONS

KANSAS RIVER, KANSAS

KANSAS CITY, KANSAS - 1962 MOD. ARMOURDALE UNIT

PLAN, PROFILE AND UNDERGROUND EXPLORATIONS

STA. 284+00 TO STA. 298+50

In 109 sheets

Sheet No. 20

Dwg. No. A20

Scale: as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY
MAY 1973

Submitted: CHIEF, LOCAL PROTECTION SECTION

Recommended: CHIEF, FOUNDATIONS & MATERIALS BR.

Approved: CHIEF, ENGINEERING DIVISION

COMPILED BY: T.A.K.–J.A.H.

DRAWN BY: J.E.B.

CHECKED BY: R.L.B.

FILE NO. A-10-6020

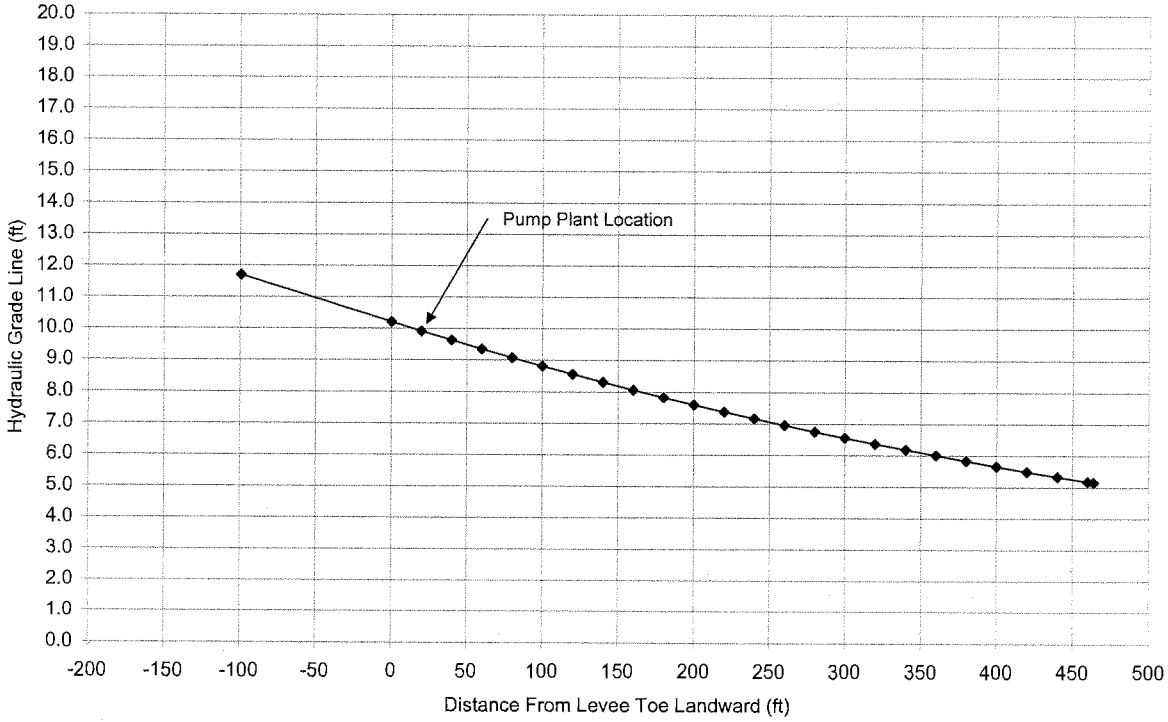
O & M M PLATE NO. 182

643

Hydraulic Grade Line for the National Beef Pump Plant
Existing Conditions - Assuming No Wells

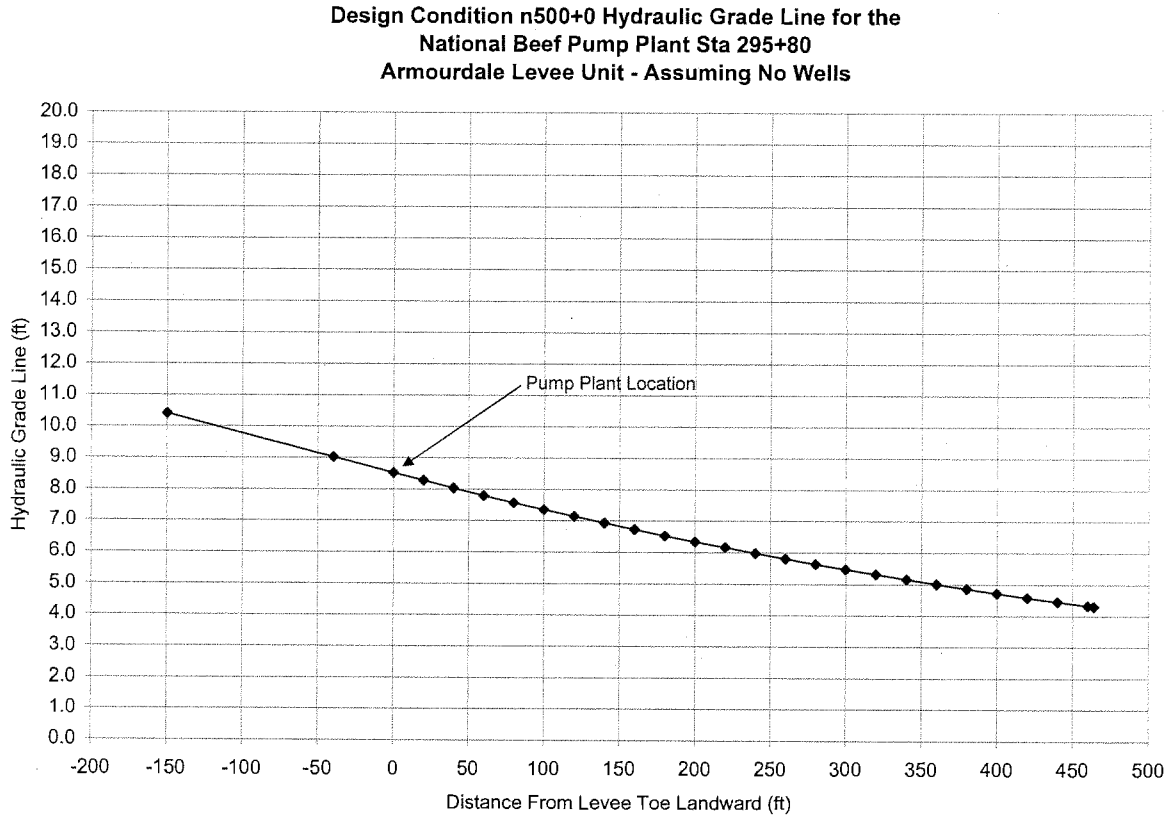
Elevation Top of levee (MSL)	761.65	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	11.7		
L1 (ft)	79		
L2 (ft)	20		
Le (ft)	679		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-99	11.70	761.70
	0	10.21	760.21
Pump Plant Location	20	9.91	759.91
	40	9.62	759.62
	60	9.34	759.34
	80	9.07	759.07
	100	8.81	758.81
	120	8.55	758.55
	140	8.31	758.31
	160	8.06	758.06
	180	7.83	757.83
	200	7.60	757.60
	220	7.38	757.38
	240	7.17	757.17
	260	6.96	756.96
	280	6.76	756.76
	300	6.56	756.56
	320	6.37	756.37
	340	6.19	756.19
	360	6.01	756.01
	380	5.83	755.83
	400	5.66	755.66
	420	5.50	755.50
	440	5.34	755.34
	460	5.18	755.18
	464	5.15	755.15

Existing Conditions Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells



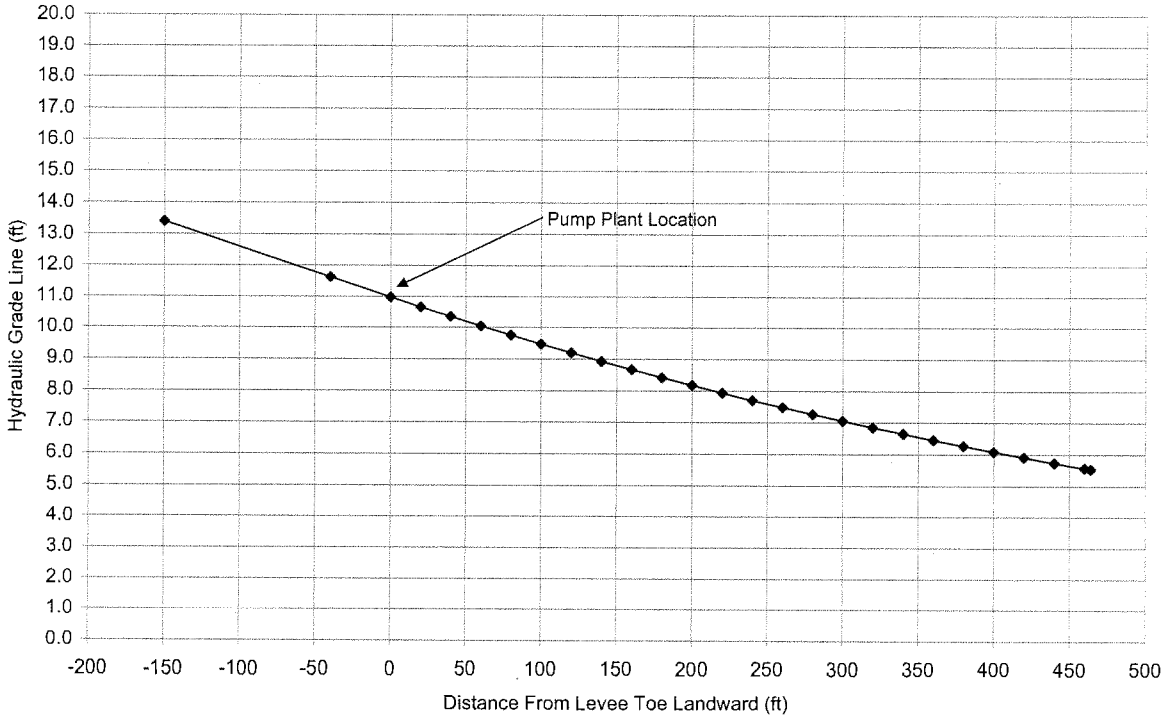
Hydraulic Grade Line for the National Beef Pump Plant
Design Conditions n500+0 - Assuming No Wells

Elevation Top of levee (MSL)	760.4	n500+0	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	10.4		
L1 (ft)	50		
L2 (ft)	100		
Le (ft)	679		
ho	8.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-150	10.40	760.40
	-40	9.02	759.02
Pump Plant Location	0	8.52	758.52
	20	8.27	758.27
	40	8.03	758.03
	60	7.80	757.80
	80	7.57	757.57
	100	7.35	757.35
	120	7.14	757.14
	140	6.93	756.93
	160	6.73	756.73
	180	6.54	756.54
	200	6.35	756.35
	220	6.16	756.16
	240	5.98	755.98
	260	5.81	755.81
	280	5.64	755.64
	300	5.48	755.48
	320	5.32	755.32
	340	5.16	755.16
	360	5.01	755.01
	380	4.87	754.87
	400	4.73	754.73
	420	4.59	754.59
	440	4.46	754.46
	460	4.33	754.33
	464	4.30	754.30


Design Condition n500+0 Hydraulic Grade Line for the
National Beef Pump Plant Sta 295+80
Armourdale Levee Unit - Assuming No Wells
Hydraulic Grade Line (ft)
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Pump Plant Location
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

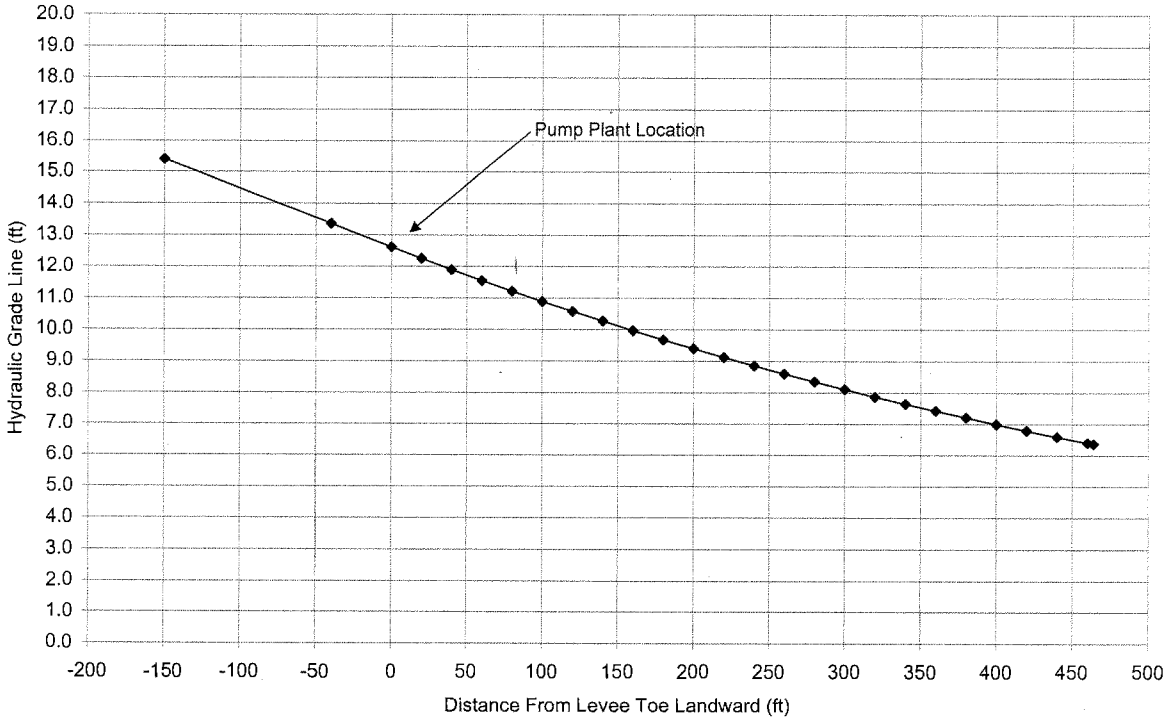
Hydraulic Grade Line for the National Beef Pump Plant Design Conditions n500+3 - Assuming No Wells			
Elevation New Top of levee (MSL)	763.4	n500 +3	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	13.4		
L1 (ft)	50		
L2 (ft)	100		
Le (ft)	679		
ho	10.98		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-150	13.40	763.40
	-40	11.62	761.62
Pump Plant Location	0	10.98	760.98
	20	10.66	760.66
	40	10.35	760.35
	60	10.05	760.05
	80	9.76	759.76
	100	9.47	759.47
	120	9.20	759.20
	140	8.93	758.93
	160	8.67	758.67
	180	8.42	758.42
	200	8.18	758.18
	220	7.94	757.94
	240	7.71	757.71
	260	7.48	757.48
	280	7.27	757.27
	300	7.06	757.06
	320	6.85	756.85
	340	6.65	756.65
	360	6.46	756.46
	380	6.27	756.27
	400	6.09	756.09
	420	5.91	755.91
	440	5.74	755.74
	460	5.57	755.57
	464	5.54	755.54

Design Condition n500+3 Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells



Hydraulic Grade Line for the National Beef Pump Plant Design Conditions n500+5 - Assuming No Wells			
Elevation New Top of levee (MSL)	765.4	n500 +5	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	726		
Driving Head (ft)	15.4		
L1 (ft)	50		
L2 (ft)	100		
Le (ft)	679		
ho	12.62		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-150	15.40	765.40
	-40	13.36	763.36
Pump Plant Location	0	12.62	762.62
	20	12.25	762.25
	40	11.89	761.89
	60	11.55	761.55
	80	11.21	761.21
	100	10.89	760.89
	120	10.57	760.57
	140	10.26	760.26
	160	9.97	759.97
	180	9.68	759.68
	200	9.40	759.40
	220	9.12	759.12
	240	8.86	758.86
	260	8.60	758.60
	280	8.35	758.35
	300	8.11	758.11
	320	7.87	757.87
	340	7.65	757.65
	360	7.42	757.42
	380	7.21	757.21
	400	7.00	757.00
	420	6.80	756.80
	440	6.60	756.60
	460	6.41	756.41
	464	6.37	756.37

Design Condition n500+5 Hydraulic Grade Line for the National Beef Pump Plant Sta 295+80 Armourdale Levee Unit - Assuming No Wells



Armourdale Kansas - Hydraulic Grade Line for the Central Ave. Pump Station Sta 299+20
Assuming No Wells

ARM Central Ave PP 299+21 No Wells.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

KCMOKS

Name : Kazmiakowski EC-GD
Date : 2/11/2007

Checked by : Bellew EC-GD
Date : 02/26/02 GMB

NOTES:
1. Replace floodwall with landside levee proposed
2. L_2 revised for design conditions due to new levee

Elevation Used

Top of Existing Levee:	761.7 (From O&M Manual)
500 year + 0':	760.1 (Lower than existing conditions)
500 year + 3':	763.1
500 year + 5':	765.1
Ground Surface Land Side:	750.0 (For existing conditions)
Ground Surface Land Side:	742.0 (For design conditions)
Ground Surface River Side:	735.0 (To obtain average thickness of blanket)
Bottom of Blanket:	728.0
Top of Bedrock:	675.0

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

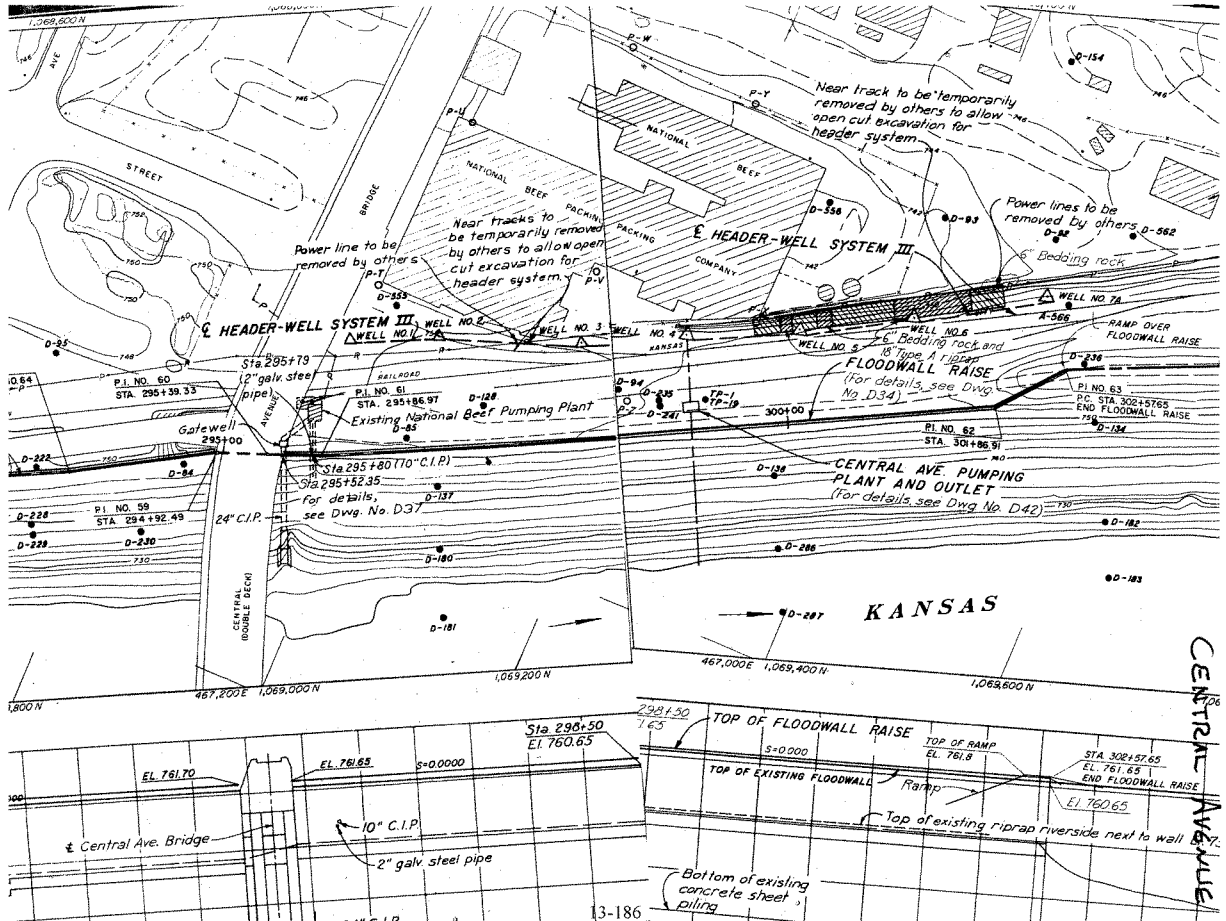
NOMENCLATURE

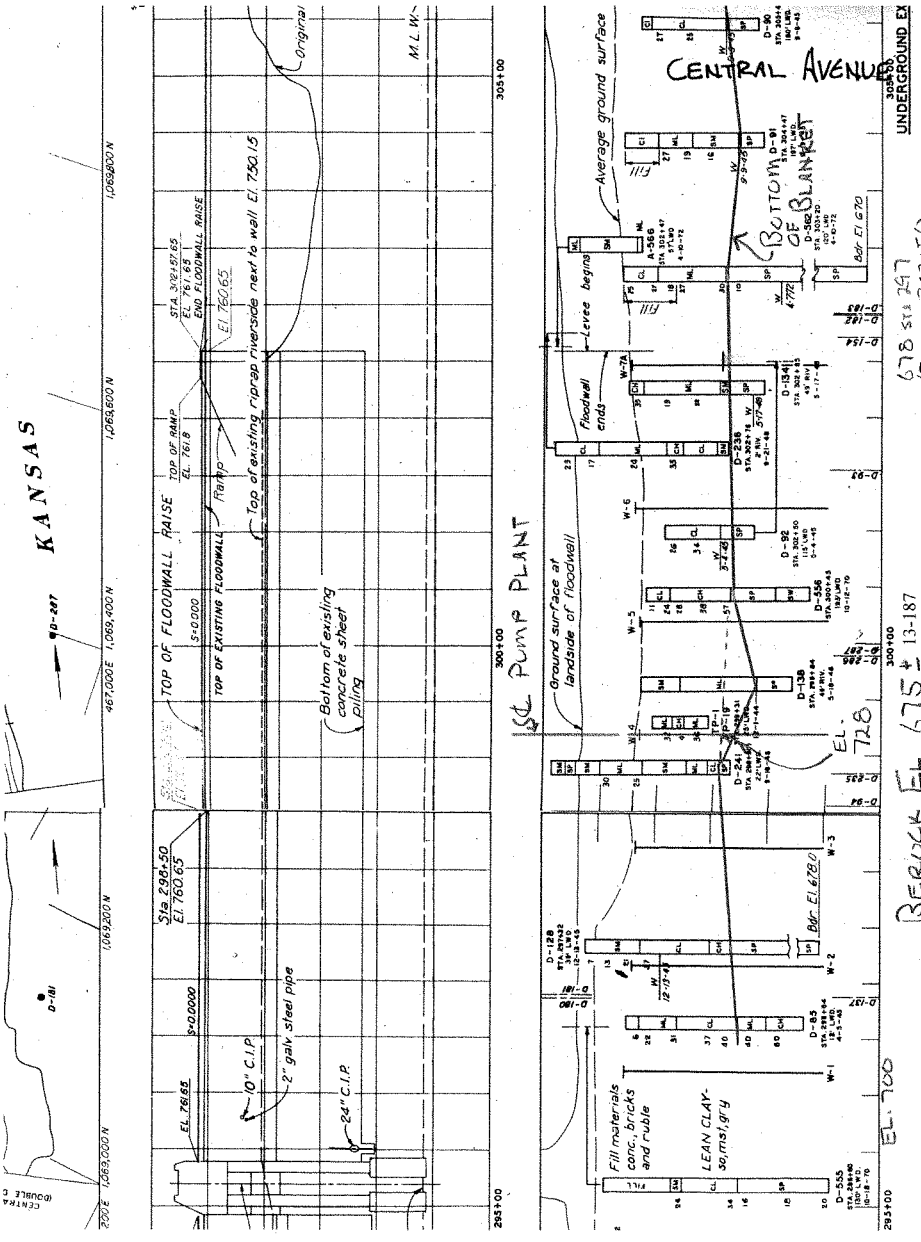
CENTRAL AVE PUMP PLANT
FEB 2007

Input	Parameters	Output	Calculations
$(K_f/K_b)_R$	= riverside permeability	C_R	= riverside effective length coefficient
$(K_f/K_b)_L$	= landside permeability	C_L	= landside effective length coefficient
D_{bL}	= landside blanket thickness	where C =	$[(K/Kb) * Df * Db]^{1/2}$
D_{bo}	= levee toe blanket thickness	ho	= head above tailwater at levee toe (w/ berm)
D_{br}	= riverside blanket thickness	i_c	= critical seepage gradient
D_f	= thickness of pervious foundat	i_o	= seepage gradient
L_{bL}	= length of riverside blanket	L_1	= riverside effective length
L_L	= length of landside blanket	where L_1	$= C * (e^{(3.8/C+1)}) / (e^{(3.8/C+1)})$
H	= max head or levee height	L_2	= landside effective length
L_2	= levee base width	L_t	= total effective length
		L'_t	= Total Effective Length + 1/2 of Berm

Station			Input Parameters								
			$(K_f/K_b)_R$	$(K_f/K_b)_L$	D_{bL} Feet	D_{bo} Feet	D_{br} Feet	D_f Feet	L_{bL} Feet	H Feet	Remarks
299+20	Existing Conditions		300	300	24.0	17.0	7.0	53.0	70	11.7	EC
299+20	n500 + 0		300	300	14.0	10.5	7.0	53.0	40	18.1	n500
299+20	n500 +3		300	300	14.0	10.5	7.0	53.0	40	21.1	n500+3
299+20	n500 + 5		300	300	14.0	10.5	7.0	53.0	40	23.1	n500+5

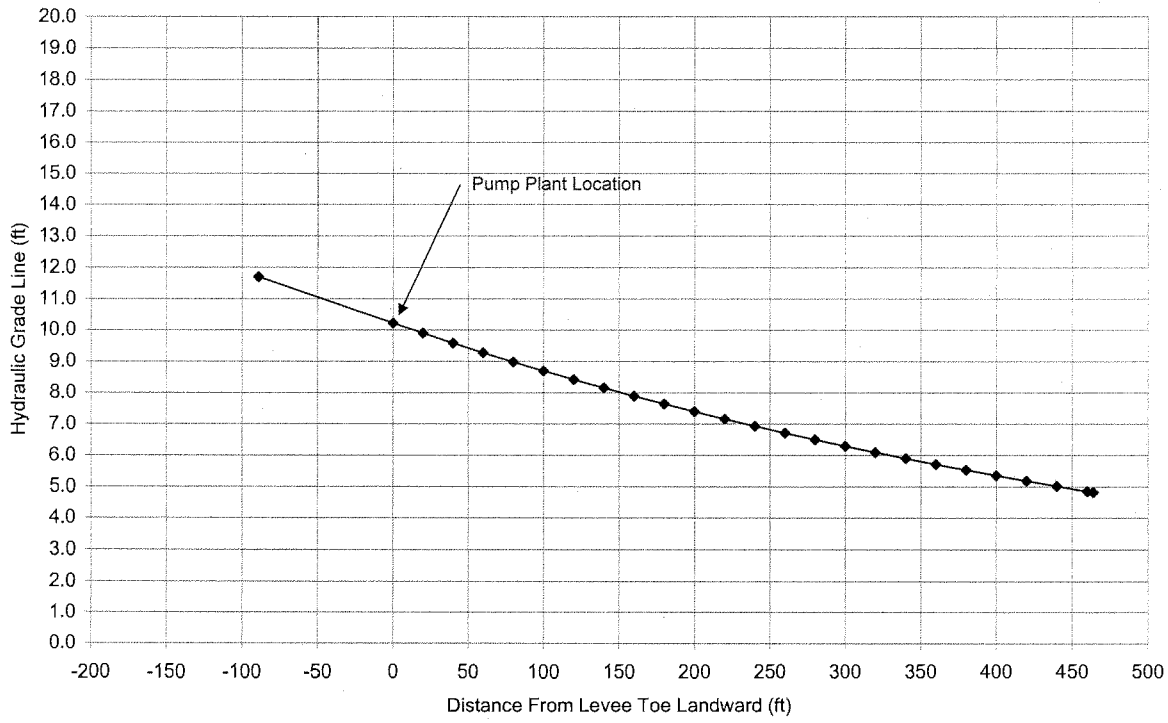
Station			Analysis										
			C_R Feet	C_L Feet	L_1 Feet	L_2 Feet	L_e Feet	L_t Feet	h_o Feet	i_o Ft/Ft	i_f Ft/Ft	FS in/in	
299+20	Existing Conditions		334	618	69	20	618	707	10.23	0.43	0.84	1.97	
299+20	n500 + 0		334	472	40	80	472	592	14.43	1.03	0.84	0.81	
299+20	n500 +3		334	472	40	80	472	592	16.83	1.20	0.84	0.70	
299+20	n500 + 5		334	472	40	80	472	592	18.42	1.32	0.84	0.64	


CENTRAL AVENUE
KANSAS
STREET
BRIDGE
NATIONAL BEEF PACKING COMPANY
℄ HEADER-WELL SYSTEM III
WELL NO. 1
WELL NO. 2
WELL NO. 3
WELL NO. 4
WELL NO. 5
WELL NO. 6
WELL NO. 7A
Power line to be removed by others
Near tracks to be temporarily removed by others to allow open cut excavation for header system
Near track to be temporarily removed by others to allow open cut excavation for header system
Power lines to be removed by others
6" Bedding rock
6" Bedding rock and 18" Type A riprap
FLOODWALL RAISE
(For details, see Dwg. No. D34)
RAMP OVER FLOODWALL RAISE
P.I. NO. 60
STA. 295+39.33
P.I. NO. 61
STA. 295+86.97
P.I. NO. 62
STA. 301+86.91
P.I. NO. 63
P.C. STA. 302+57.65
END FLOODWALL RAISE
P.I. NO. 59
STA. 294+92.49
Sta. 295+79
12" galv. steel pipe
Gatewell 295+00
Existing National Beef Pumping Plant
Sta. 295+80 (10" C.I.P.)
Sta. 295+52.35
For details, see Dwg. No. D37
24" C.I.P.
CENTRAL AVE. PUMPING PLANT AND OUTLET
(For details, see Dwg. No. D42)
CENTRAL (DOUBLE DECK)
467,200E 1,069,000 N
1,069,200 N
467,000E 1,069,400 N
1,069,600 N
Sta. 298+50
El. 760.65
EL. 761.70
EL. 761.65
S=0.0000
℄ Central Ave. Bridge
10" C.I.P.
2" galv. steel pipe
24" C.I.P.
TOP OF FLOODWALL RAISE
TOP OF RAMP
EL. 761.8
TOP OF EXISTING FLOODWALL
Ramp
STA. 302+57.65
EL. 761.65
END FLOODWALL RAISE
El. 760.65
Top of existing riprap riverside next to wall
Bottom of existing concrete sheet piling


KANSAS
CENTRAL AVENUE
BOTTOM OF BLANKET
Average ground surface
Levee begins
Floodwall ends
Ground surface at landside of floodwall
TOP OF FLOODWALL RAISE
TOP OF EXISTING FLOODWALL
Top of existing riprap riverside next to wall El. 750.15
Bottom of existing concrete sheet piling
Original
M.L.W.
10" C.I.P.
2" galv steel pipe
24" C.I.P.
Sta. 298+50
El. 760.65
EL. 761.65
Fill materials conc., bricks and ruble
LEAN CLAY-so, mst, gry
EL. 700
BEROCK EL 675± 13-187
678 STA 297
670 STA 302+50
UNDERGROUND EX

Hydraulic Grade Line for the Central Avenue Pump Plant Existing Conditions - Assuming No Wells			
Elevation Top of levee (MSL)	761.65	Existing Conditions	
Elevation at Landside Levee Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	11.7		
L1 (ft)	69		
L2 (ft)	20		
Le (ft)	618		
ho	10.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-89	11.70	761.70
Pump Plant Location	0	10.23	760.23
	20	9.90	759.90
	40	9.59	759.59
	60	9.28	759.28
	80	8.98	758.98
	100	8.70	758.70
	120	8.42	758.42
	140	8.15	758.15
	160	7.89	757.89
	180	7.64	757.64
	200	7.40	757.40
	220	7.16	757.16
	240	6.93	756.93
	260	6.71	756.71
	280	6.50	756.50
	300	6.29	756.29
	320	6.09	756.09
	340	5.90	755.90
	360	5.71	755.71
	380	5.53	755.53
	400	5.35	755.35
	420	5.18	755.18
	440	5.02	755.02
	460	4.86	754.86
	464	4.83	754.83

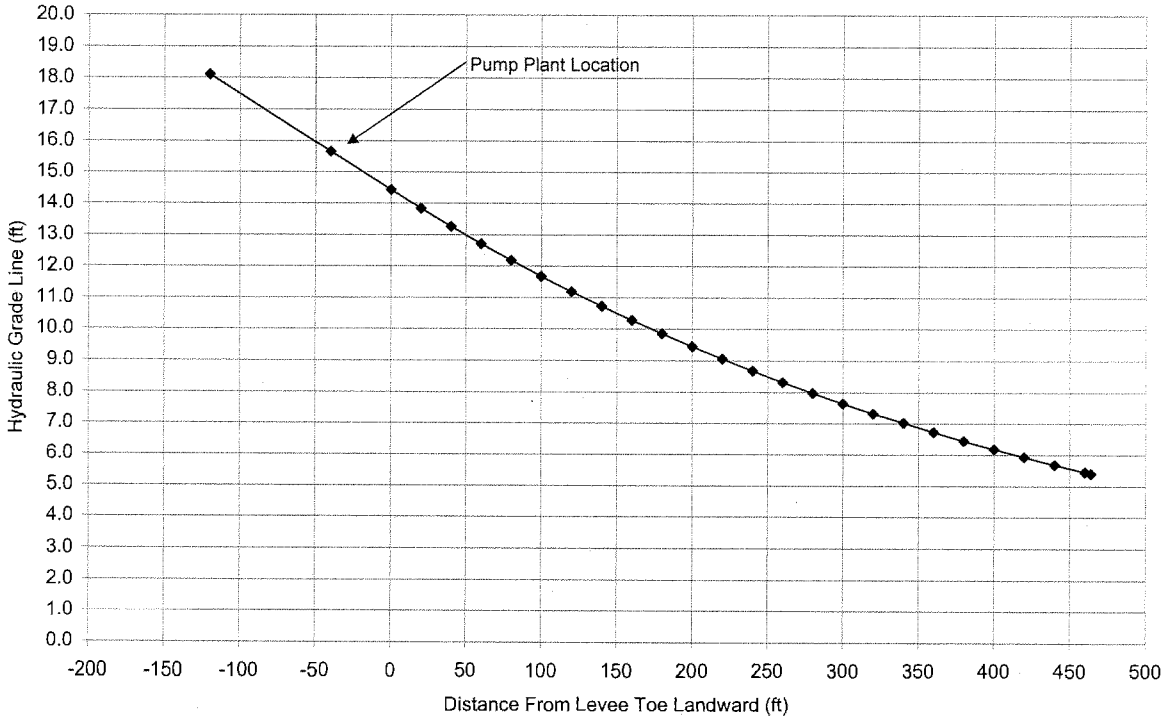
Existing Conditions Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



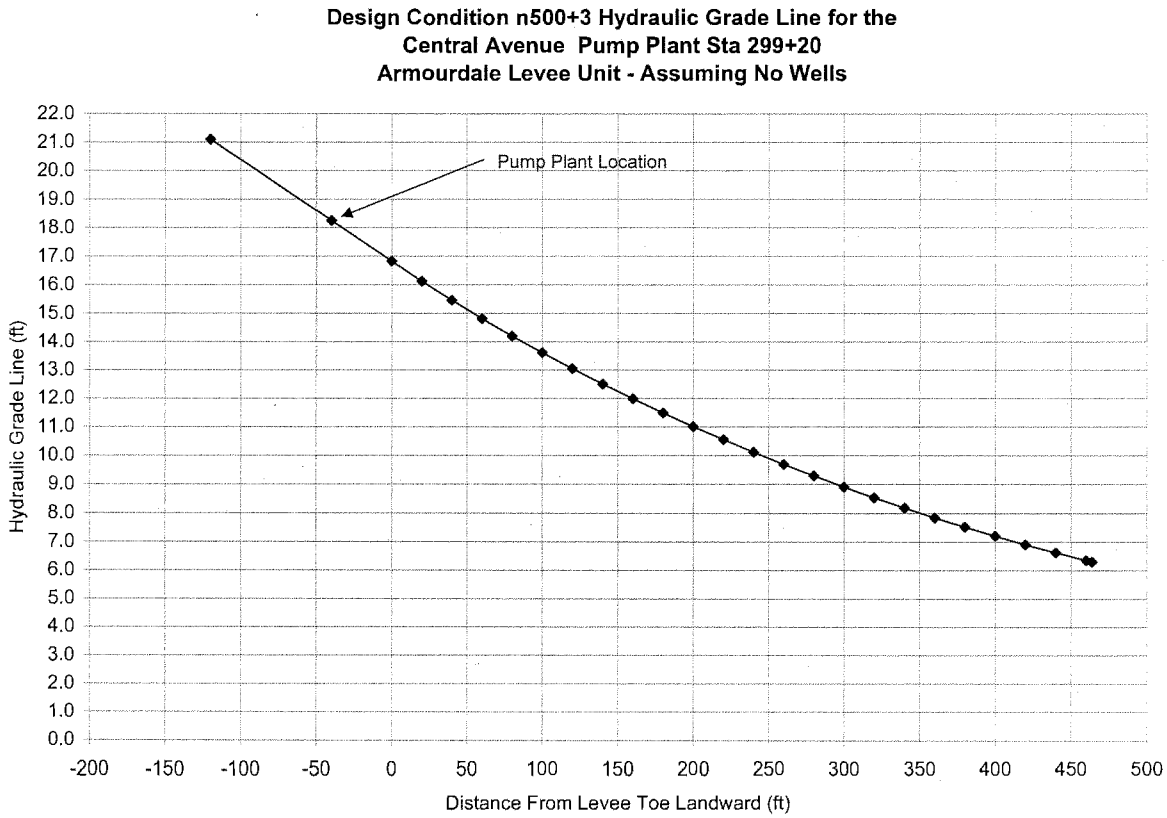
Hydraulic Grade Line for the Central Avenue Pump Plant Design Conditions n500+0 - Assuming No Wells

Elevation Top of levee (MSL)	760.1	n500+0	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	18.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	14.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	18.10	760.10
Pump Plant Location	-40	15.66	757.66
	0	14.43	756.43
	20	13.84	755.84
	40	13.26	755.26
	60	12.71	754.71
	80	12.18	754.18
	100	11.68	753.68
	120	11.19	753.19
	140	10.73	752.73
	160	10.28	752.28
	180	9.86	751.86
	200	9.45	751.45
	220	9.06	751.06
	240	8.68	750.68
	260	8.32	750.32
	280	7.97	749.97
	300	7.64	749.64
	320	7.33	749.33
	340	7.02	749.02
	360	6.73	748.73
	380	6.45	748.45
	400	6.18	748.18
	420	5.93	747.93
	440	5.68	747.68
	460	5.44	747.44
	464	5.40	747.40

Design Condition n500+0 Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells

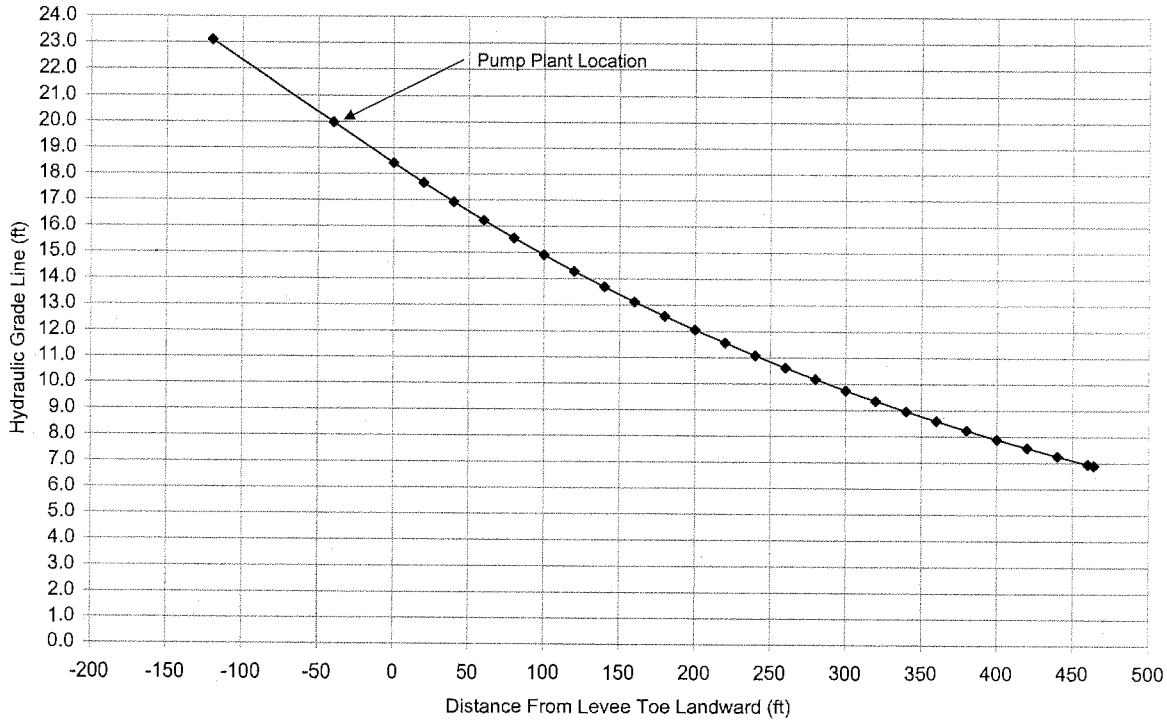


Hydraulic Grade Line for the Central Avenue Pump Plant Design Conditions n500+3 - Assuming No Wells			
Elevation New Top of levee (MSL)	763.1	n500 +3	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	21.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	16.83		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
	-120	21.10	763.10
Pump Plant Location	-40	18.25	760.25
	0	16.83	758.83
	20	16.13	758.13
	40	15.46	757.46
	60	14.82	756.82
	80	14.20	756.20
	100	13.61	755.61
	120	13.05	755.05
	140	12.51	754.51
	160	11.99	753.99
	180	11.49	753.49
	200	11.01	753.01
	220	10.56	752.56
	240	10.12	752.12
	260	9.70	751.70
	280	9.30	751.30
	300	8.91	750.91
	320	8.54	750.54
	340	8.19	750.19
	360	7.85	749.85
	380	7.52	749.52
	400	7.21	749.21
	420	6.91	748.91
	440	6.62	748.62
	460	6.35	748.35
	464	6.29	748.29


Design Condition n500+3 Hydraulic Grade Line for the
Central Avenue Pump Plant Sta 299+20
Armourdale Levee Unit - Assuming No Wells
Pump Plant Location
Hydraulic Grade Line (ft)
22.0
21.0
20.0
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500
Distance From Levee Toe Landward (ft)

Hydraulic Grade Line for the Central Avenue Pump Plant Design Conditions n500+5 - Assuming No Wells			
Elevation New Top of levee (MSL)	765.1	n500 +5	
Elevation at Landside Levee Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	728		
Driving Head (ft)	23.1		
L1 (ft)	40		
L2 (ft)	80		
Le (ft)	472		
ho	18.42		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance Point	-120	23.10	765.10
Pump Plant Location	-40	19.98	761.98
	0	18.42	760.42
	20	17.66	759.66
	40	16.92	758.92
	60	16.22	758.22
	80	15.55	757.55
	100	14.90	756.90
	120	14.28	756.28
	140	13.69	755.69
	160	13.12	755.12
	180	12.58	754.58
	200	12.06	754.06
	220	11.56	753.56
	240	11.08	753.08
	260	10.62	752.62
	280	10.18	752.18
	300	9.75	751.75
	320	9.35	751.35
	340	8.96	750.96
	360	8.59	750.59
	380	8.23	750.23
	400	7.89	749.89
	420	7.56	749.56
	440	7.25	749.25
	460	6.95	748.95
	464	6.89	748.89

Design Condition n500+5 Hydraulic Grade Line for the Central Avenue Pump Plant Sta 299+20 Armourdale Levee Unit - Assuming No Wells



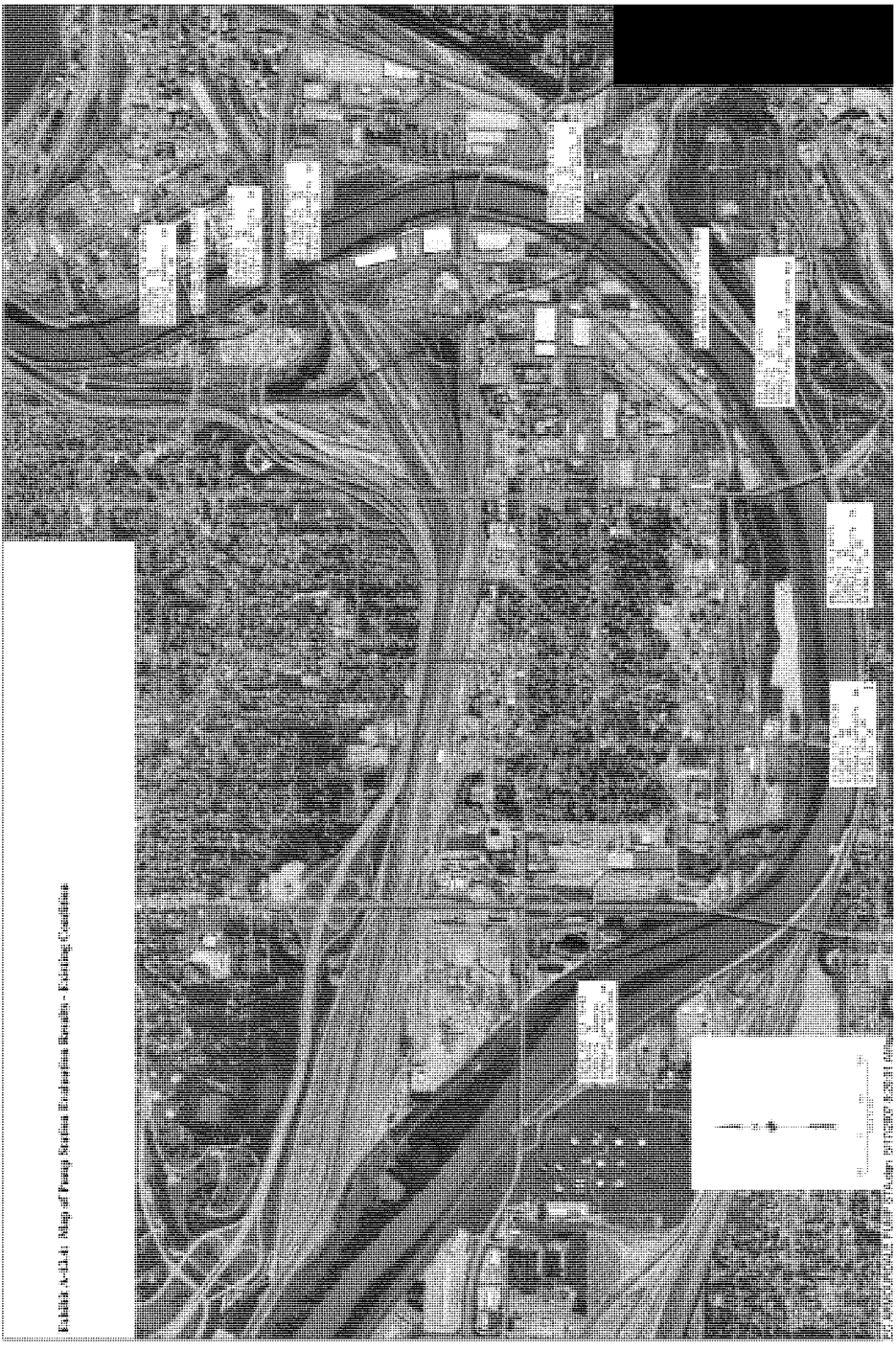
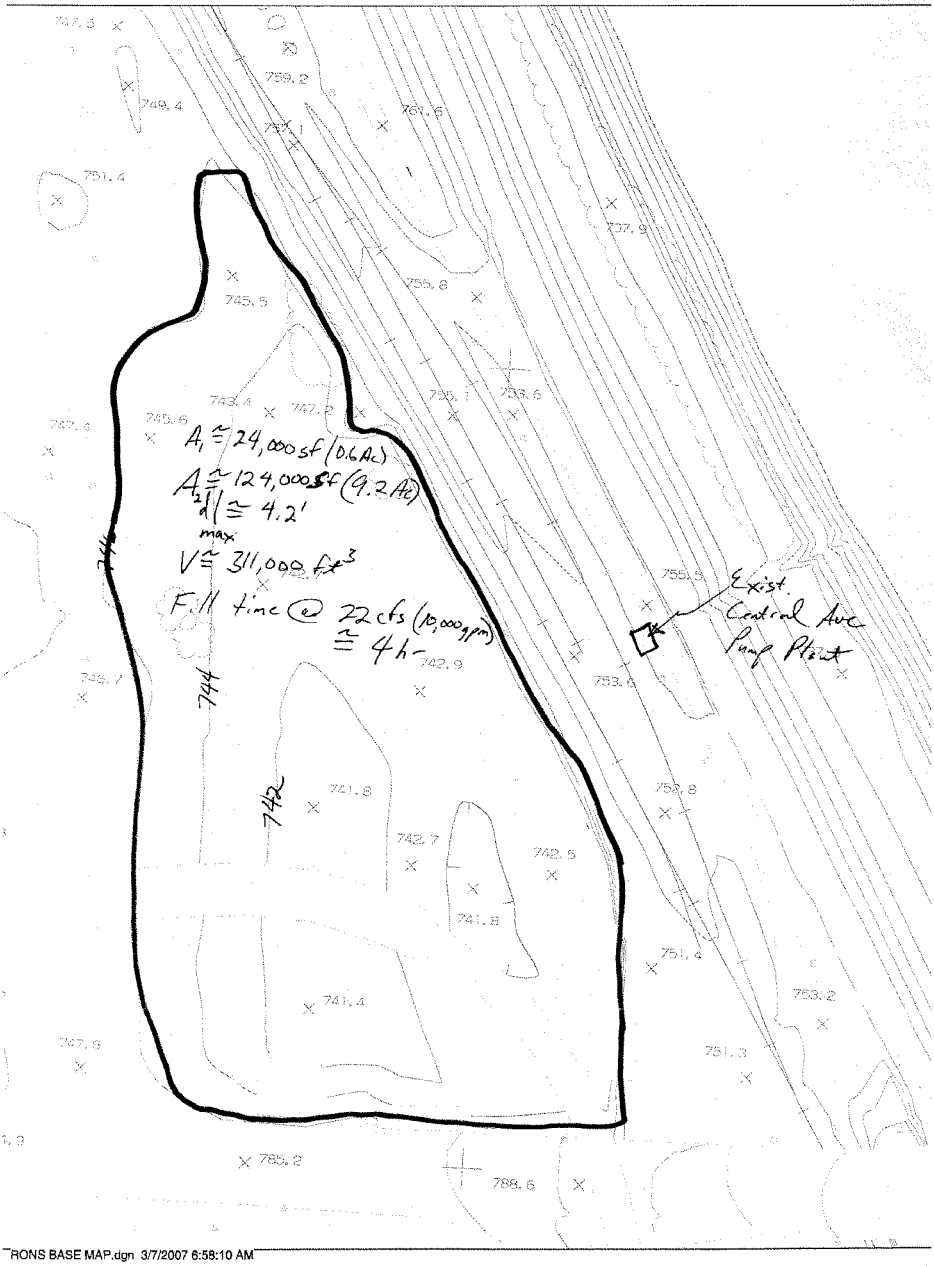


Exhibit A-1.1: Map of Pump Station Evaluation Results - Existing Condition

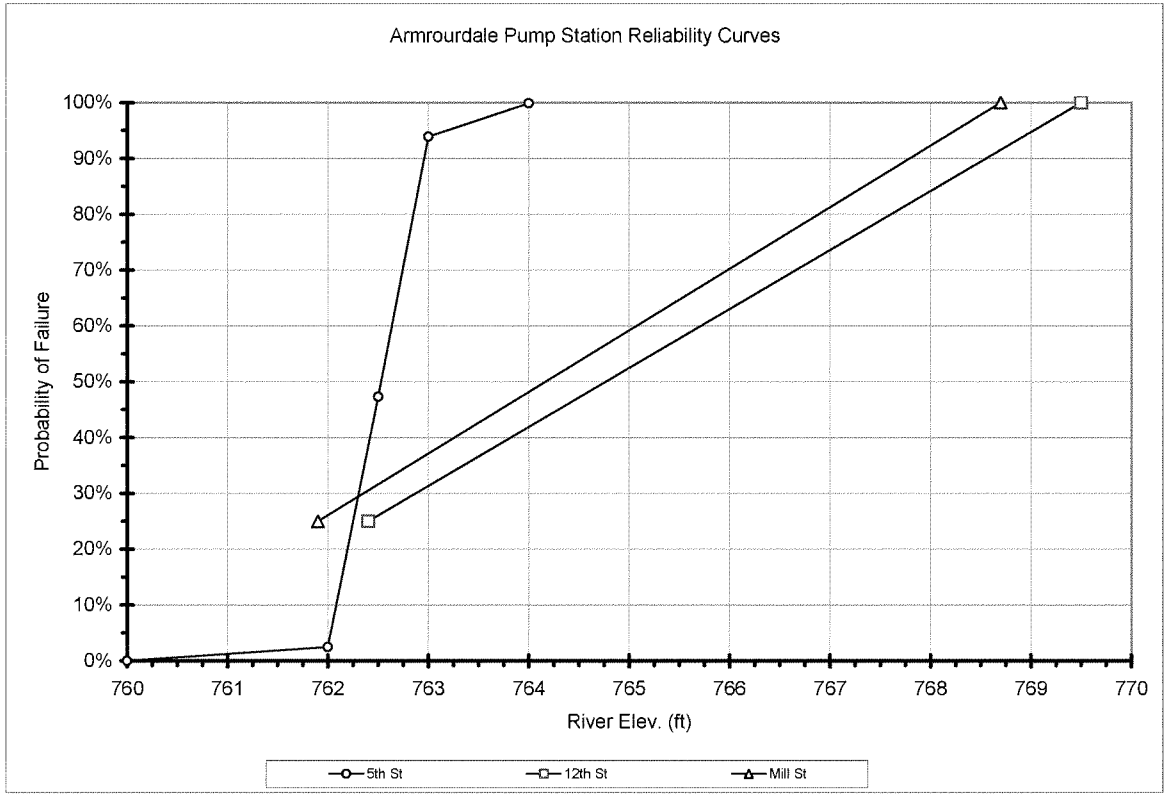
Exhibit A-15.5: Map of Pump Station Evaluation Results - Solids-3 Design Conditions

Exhibit A-13.6
Map of Ponding Area



RONS BASE MAP.dgn 3/7/2007 6:58:10 AM

Exhibit A-13.7



Chapter A-14

PUMP STATION ANALYSIS
CID

CHAPTER A-14
PUMP STATION ANALYSIS - CID

A-14.1 SOURCES OF INFORMATION

Corps staff visually inspected every pump station studied, including outlet works when accessible. Printed information such as pump curves or as-built drawings were unavailable or limited for several of the facilities. Ownership and maintenance of the pump stations is by the City of Kansas City, Missouri (KCMO), Unified Government of Wyandotte County / Kansas City, Kansas (UG/KCK), the Kaw Valley Drainage District (KVDD), or private, as indicated in Table A-14.1 below.

Table A-14.1
Summary of Pump Station Available Information

Name	Ownership	Station	As-Builts?	Pump Curves?
CENTRAL INDUSTRIAL DISTRICT (CID) Unit Pump Stations:				
Broadway	KCMO	24+77	Partial	Yes
Santa Fe	KCMO	52+85	Yes	Yes
Ohio	UG/KCK	83+52	Yes	Yes
Mistletoe	UG/KCK	37+06.5	Yes	Yes
New Central	UG/KCK	58+12	Yes	No
Stockyards #3	KVDD	74+21	Yes	Yes
Gateway 2000	Private	80+90	Yes	Yes
Stockyards #1 / Field Pump House	KVDD	98+05	No	No
Kemper Arena	KCMO	106+49	Yes	Yes

A-14.2 SUPPORTING INFORMATION AND CALCULATIONS

Supporting information for the pump station analysis is presented in the following exhibits included at this end of this chapter:

Exhibit A-14.1 – Pump Station Information and Photographs
Exhibit A-14.2 – Geotechnical Data (Hydraulic Grade Line Calculations)

Structural analysis calculations are available upon request from the Corps of Engineers.

A-14.3 METHODS OF ANALYSIS

Pump stations are generally evaluated both for structural reliability and, in cases of relief wells servicing actively pumped relief well systems, hydraulic capacity. No CID pump stations handle seepage flow from relief wells discharging below grade, therefore hydraulic capacity was not analyzed unless the local Sponsor indicated problems. Detailed analyses were conducted for existing conditions in an effort to define the risk of failure under current project conditions; and for the N500+3 design loading to determine the required modifications to accommodate the higher loading associated with a

prospective levee or floodwall raise. Two other design conditions, N500+0 and N500+5 were also considered, but primarily by engineering judgment rather than detailed analysis. The purposes of the two additional design points is to better define where the various design conditions fall on the net benefits curve, relative to the NED plan.

Structural analysis considered both strength and uplift (also called "stability" or "flotation" throughout the report) with water to the top of the existing line of protection. These analyses were applied to the pump station structures as well as outlet works (pipes, RCBs and gatewells). In some cases, as-built drawings and specifications were available, few assumptions had to be made, and the analysis was rather clear. In the case of Stockyards #1/ Field Pump House, details of the original construction were not available. Normally, with a lack of design documentation, assumptions would be made based on standard practice at the time of construction or previous findings on similar structures. In this particular case, since the pump station was deemed hydraulically unnecessary, it was not analyzed. Initial strength and uplift analyses were based upon geotechnical parameters applied across the levee unit and hydraulic grade lines (HGLs) with water to the top of the existing line of protection. Initial calculations neglected the affects of skin friction, water that would be present in the pump station under normal operating conditions, and the affect of existing relief wells (where present) to lower HGLs. When such analysis showed "marginal" results, the analysis was refined, considering all of the available forces (i.e. skin friction, wells, water weight) to resist failure. In those cases where pump stations have separate gatewells within the line of protection, the gatewell analysis is presented in the general structural portion of the feasibility report, along with drainage structures not associated with pump plants.

Strength analysis entailed examination of walls and slabs for shear and bending moment capacity. If the resulting FS for flexure and shear was 1.5 or greater, the member was considered to perform to 99.8% reliability when exposed to the existing condition event loading. If the resulting FS was less than 1.5, a reliability analysis was conducted.

Uplift analysis entailed examination of structures for flotation with water to the top of the existing or proposed line of protection. Several stations had subcomponents such as gatewells and discharge chambers that were constructed at different times. In those cases, the uplift analysis considered the subcomponents separately, and the recommended modification considered the ability of the subcomponents to be tied together to transfer forces. If the resulting FS was 1.1 or greater, the structure was considered to perform reliably.

Screening level hydrologic analyses were conducted at the beginning of the feasibility phase for interior drainage. Records and drainage studies by others were reviewed, and local Sponsors consulted. Since the focus of this study is an existing system and there were no interior drainage problems identified or reported by Sponsors, more detailed interior flood hydrology studies were not undertaken. It is also worth noting that there are several interior storm drainage systems that handle upland and lowland flow, ultimately draining to pump stations. In some cases, the total flow generated by the storm drainage system exceeds the capacity of the lowland system, including the pump

station. In other words, if the drainage systems were improved such that the flow could travel unimpeded, the pump stations would be overloaded. Research and discussion with Sponsors indicates that this situation has existed for many years, and no improvements are planned. Therefore, no pump station improvements were considered based solely upon interior flood hydrology.

Alternatives analysis was tied closely to geotechnical requirements for underseepage control (i.e. berms, cutoff trench, or relief wells). Care was taken to ensure that alternatives calling for additional relief wells considered all resultant costs, including pumping costs.

A-14.4 FINDINGS AND RECOMMENDATIONS

The analysis of the pump stations along the CID line of protection resulted in the findings and recommendations below. Existing and future condition findings and recommendations are also presented in Tables A-14.2 and A-14.3, respectively, located toward the end of the chapter.

A-14.4.1 Broadway Pump Station

The Broadway Pump Station is owned and operated by the City of Kansas City, MO and is located on the CID MO unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The floodwall section in this area is not expected to change, i.e. there is no raise proposed.

The structural evaluation indicates that the station is marginally acceptable for strength, and acceptable for flotation, with water to the top of the existing line of protection.

The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the floodwall and discharging to the Missouri River. The existing gravity discharge is a 54" reinforced concrete pipe. Corps staff was unable to locate drawings so as to determine the strength of the 54" line.

The Broadway Pump Station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings for the structure were available and used for the analysis, additional testing during the next project phase is not required. The discharge line, however, should be inspected and / or tested to determine its strength.

N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already roughly above this level. Only inspection of the discharge pipe would be required.

N500+3 and N500+5 design conditions would involve raising the line of protection by several feet. Since strength is marginal under existing conditions, improvements such as bracing or addition of a web wall would likely be required. Stability would likely be acceptable under this condition.

A-14.4.2 Santa Fe Pump Station

The Santa Fe Pump Station is owned and operated by the City of Kansas City, MO and is located on the CID MO unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The floodwall section in this area is not expected to change, i.e. there is no raise proposed.

The structural evaluation indicates that the station is acceptable for strength, and marginally acceptable for flotation, with water to the top of the existing line of protection.

The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the floodwall and discharging to the Missouri River. The existing gravity discharge is a 10' reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under existing conditions.

The Santa Fe Pump Station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already roughly above this level. No modifications would be required.

N500+5 design conditions would involve raising the line of protection by several feet. Strength would likely be acceptable under this condition. While stability was considered marginal under existing conditions, the decrease in factor of safety for stability would likely be very small for this design condition. Therefore, stability would also likely be acceptable, and only the discharge chamber and extension of the sluice gate stem would be required.

A-14.4.3 Ohio Pump Station (KCK FPS #1)

The Ohio Pump Station is owned and operated by the Unified Government of Wyandotte County / City of Kansas City, KS. While it is physically located in Kansas, the discharge crosses the state line and CID MO unit, and flows to the Missouri River. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. The levee section in this area is not expected to change, i.e. there is no levee raise proposed.

Structural evaluations indicate that the pump station is acceptable for flotation with water to the top of the existing line of protection. Strength of the pump station is unacceptable for existing conditions. Structural modifications are required to meet strength criteria.

The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the levee and discharging to the Missouri River. The existing gravity discharge is a 42" reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under existing conditions.

The Ohio Pump Station was designed and constructed prior to Corps of Engineers involvement in the flood control project, and was later modified by the Corps. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would essentially be existing conditions, since the existing line of protection is already above this level. Since strength is unacceptable for existing conditions, similar structural modifications as discussed above would be required.

N500+5 design conditions would involve raising the line of protection by several feet. Since strength is unacceptable for existing conditions, similar structural modifications as discussed above would be required. Stability would likely be acceptable under this condition. Therefore, only raising of the discharge chamber and extension of the sluice gate stem would be required.

A-14.4.4 Mistletoe Pump Station

The Mistletoe Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. It handles interior drainage and uncontrolled seepage flow. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A floodwall raise of roughly 1.5-ft (N500+3 level of protection) is proposed in this area.

Structural evaluations indicate that the pump station is acceptable for strength and flotation with water to the top of the existing and proposed lines of protection.

The existing pump discharge configuration is a discharge through flap valves into a discharge chamber connected to the gravity discharge pipe passing under the levee and discharging to the Kansas River. The existing gravity discharge is an 18" cast iron pipe. Although detailed inspection of the discharge line was not possible, unprotected ferrous pipe may experience pitting corrosion. For this reason, replacement of the pipe or lining with a cured in placed pipe (CIPP) designed for the fully deteriorated condition is recommended. The Mistletoe station is integral with the floodwall. For any line of protection raise, the top of the discharge chamber will need to be raised to coincide with the line of protection. Raising of the discharge chamber will require extension of the sluice gate stem.

The Mistletoe Pump Station was designed and constructed prior to Corps of Engineers involvement in the flood control project, and was later modified by the Corps. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would essentially be existing conditions, since the existing floodwall is already roughly 1.5-ft above this level. Only replacement or rehabilitation of the discharge pipe would be required.

N500+5 design conditions would involve raising the line of protection by roughly 3.5-ft. Since both strength and stability have more than adequate factors of safety for the N500+3 condition, both would likely be acceptable for this condition. Therefore, only raising of the discharge chamber and extension of the sluice gate stem would be required.

A-14.4.5 New Central Ave Pump Station (KCK FPS #16)

The New Central Ave. Pump Station is owned and operated by the Unified Government of Wyandotte County / City of Kansas City, KS and is located on the CID KS unit. It handles interior drainage. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A levee raise of 2-ft (N500+3 level of protection) is proposed in this area.

Structural evaluations indicate that the pump station is acceptable for strength and flotation with water to the top of the existing and proposed lines of protection.

The existing pump discharge configuration is a gravity drain under the levee and twin ductile iron pump discharge lines "up and over" the levee, discharging into a gatewell on the riverside levee crest. The existing gravity discharge is an 84" reinforced concrete pipe. Strength of the discharge pipe was found to be acceptable under both existing and proposed conditions. Strength and stability of the gatewell is adequate. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Raising the gatewell will require replacement of the sluice gate stem, installation of a new gate hoist, and installation of a new stem. The twin "up and over" discharge lines will need to be relocated up and over the new, higher level of protection. Pumps will also require replacement with more powerful pumps because the discharge lines are at a higher elevation.

The New Central Ave. Pump Station was designed and constructed in 1987 by local interests after the Corps of Engineers involvement in the flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would essentially be existing conditions, since the existing floodwall is already roughly 1 foot above this level.

N500+5 design conditions would involve raising the line of protection by roughly 4-ft. Strength would be marginally acceptable because the factor of safety would likely dip slightly below 1.5. Stability has a more than adequate factor of safety for N500+3 and would likely be acceptable for this condition. Therefore, raising the discharge chamber, extension of the sluice gate stem, relocating the up and over pipes, and new pumps would be required.

A-14.4.6 Stockyards #3 Pump Station

The Stockyards Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. It handles interior drainage and seepage flow from relief wells between Station 65+00 and 82+00. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A levee raise of 2-ft (N500+3 level of protection) is proposed in this area.

Structural evaluations indicate that the pump station is unacceptable for flotation with water to the top of the existing line of protection. Addition of weight or control of subsurface hydrostatic pressure is required to alleviate flotation issues. Strength of the pump station is also unacceptable for existing conditions. Structural modifications are required to meet strength criteria. Abandon the station and the relief well system in place and replace the entire system with area fill and re-grade.

The existing pump discharge configuration is a gatewell at the pump station and a pressurized pipe under the levee. Strength of the discharge pipe was found to be acceptable under existing conditions. The existing gravity discharge is a 24" cast iron pipe. Stability and the strength of the gatewell is unacceptable, for existing conditions. The pump station will be abandoned in place. The 24" pipe will be grouted as will all incoming pipes that service the relief well system. All mechanical and electrical equipment in the pump station will be removed.

The Stockyards Pump Station was designed and constructed as part of the original Corps of Engineers flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would essentially be existing conditions, since the existing conditions are 1 foot above this level. If there would be no levee raise, this station would not be abandoned and its flotation and discharge pipe problems would be addressed.

N500+5 design conditions would involve raising the level of protection by 4-ft. Since the pump station is to be abandoned and filled in at N500+3, it would then also be abandoned and filled in for this design condition.

A-14.4.7 Gateway 2000 Pump Station

The Gateway 2000 Pump Station is owned and operated by private interests and is located on the CID KS unit. It was designed to handle interior drainage in the area surrounding Gateway 2000. Structural evaluations indicate that the pump station is acceptable for flotation with water to the top of the existing line of protection. Strength of the pump station is also acceptable for existing conditions. A levee raise of 3.6-ft (N500+3 level of protection) is proposed in this area.

The existing pump discharge configuration is a discharge through flap valves into a gatewell/discharge chamber connected to the gravity discharge pipe passing under the levee. Strength of the discharge pipe was found to be acceptable under existing

conditions. The existing gravity discharge is a 60" reinforced concrete pipe. Stability of the gatewell is acceptable and the strength of the gatewell is acceptable, for existing conditions. Strength and stability of the gatewell is adequate. For any line of protection raise, the top of the gatewell will need to be raised to coincide with the new line of protection. Strength of the base slab will need to be increased by modifying. Adding FRP to the base slab is a recommended method for modification. Raising the gatewell will also require replacement of the sluice gate stem, installation of a new gate hoist, and installation of a new stem.

The Gateway 2000 Pump Station was designed and constructed in 1994 by local interests, after the Corps of Engineers involvement in the flood control project. Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

N500+0 design conditions would involve raising the level of protection by 0.5-ft. Since the pump station is adequate at protection level N500+3 it would be adequate at this design condition.

N500+5 design condition would involve raising the level of protection by 5.5-ft. Since the pump station is more than adequate at the protection level N500+3 it will likely be adequate at this design condition. The gatewell will likely require additional strengthening as indicated for N500+3 conditions.

A-14.4.8 Stockyards #1/ Field Pump House Pump Station

The Stockyards #1 Pump Station is owned and operated by the Kaw Valley Drainage District and is located on the CID KS unit. The station was originally designed to handle a small amount of interior drainage. The Sponsor reported no problems with the hydraulic capacity of this pump plant, therefore hydraulic analyses were not conducted. A floodwall raise of 2-ft (N500+3 level of protection) is proposed in this area.

The existing pump discharges to a 24" cast iron pipe. Stability and strength are adequate under existing conditions. This pump station pumps are likely inadequate and have not needed to run for over 20 years. This pump station will be abandoned in place. The 24" cast iron pipe and valve manhole will be grouted. All mechanical equipment will be removed.

N500+0 design conditions would essentially be existing conditions, since the existing conditions are 1 foot above this level. The pump station would still be abandoned in place.

N500+5 design conditions would involve raising the levee protection by 4-ft. The pump station would still be abandoned in place.

A-14.4.9 Kemper Arena Pump Station

The Kemper Arena Pump Station is owned and operated by the City of Kansas City, Missouri, and is located in the CID KS unit. The Station handles interior drainage from Kemper Arena and the American Royal area.

Structural evaluations indicate that the pump station is marginally acceptable for flotation with water to the top of the existing line of protection. For any line of protection raise, addition of weight or control of subsurface hydrostatic pressure is required to alleviate flotation issues. Strength of the pump station is acceptable for existing conditions. Structural modifications are required to meet strength criteria.

The pump station discharges into a 6' x 6' reinforced concrete box the passes under the floodwall, with a landside gatewell, at Sta. 106+49. Strength of the discharge pipe was found to be acceptable under existing conditions. Stability of the gatewell is acceptable and the strength of the gatewell is marginally unacceptable, for existing conditions. Structural modification, such as adding FRP to strengthen the walls, is required to meet strength criteria.

Since record drawings were available and used extensively for the analysis, additional testing during the next project phase is not required.

A-14.5 RELIABILITIES

Probability of failure curves for Broadway Pump Station and Ohio Pump Station were used in the economic model.

Calculations for the Broadway Pump Station revealed a 0.8% probability of failure for wall strength with water at the top of the flood protection.

Calculations for the Ohio Pump Station revealed a 100% probability of failure with water at top of flood protection, driven primarily by inadequate strength of key structural members.

TABLE A-14.2
Existing Conditions CID Pump Station Analysis Results

Pump Station	Sta.	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments
Broadway (KCMO)	24+77	1.8	1.11 R=99.15%	2.46	2.0	1.22	54" RCP Requires Insp.	
Santa Fe (KCMO)*	52+85	2.64	1.53	3.98	2.2	1.28[A]	120" RCP	Structure is supported by piles. Piles were not used to benefit against uplift.
Ohio (KVDD)	83+52	1.92	0.44 Not Reliable	0.76 Reliability not performed	3.63	1.59	42"RCP	Deterioration of station interior components due to corrosive environment observed during 2007 inspection
Mistletoe (KVDD)	37+06.5	1.61	2.25	3.54	2.13	1.62	18"CIP Requires Insp.	
New Central (KVDD)	58+12	1.59	2.19	4.16	2.86	2.0	84" RCP	
Stockyards #3 (KVDD)	74+21	1.54	1.97	1.76	2.1	1.16[B]	24" CIP Requires Insp.	
Gateway (KVDD)	80+90	2.3	>1.5	2.53	1.87	1.61	60"RCP	
Kemper Arena (KCMO)	106+49	2.77	1.99	1.71	1.92	1.15	6' x 6' RCB Shear 2.45 Moment 2.91	Station stability of concern with HGL above EC elevation

Green - FS equal to or greater than 1.5 for strength and greater than or equal to 1.10 for stability

Yellow - FS between 1.3 and 1.5 for strength and/or between 0.96 and 1.09 for stability

Red - FS <1.3 and/or FS < 0.95 for stability

Strength: 1.5 FS required per established criteria

Stability: 1.1 FS required per established criteria

*Pump station is pile supported. The piles were not analyzed.

TABLE A-14.3
Summary of N500+3 Conditions Results and Recommendations

Pump Station	Sta.	Minimum Shear Wall F.S.(W1,W2)	Minimum Wall Moment F.S.(W1/W2)	Minimum Base Slab Shear F.S	Minimum Base Slab Moment F.S	Stability (Flotation)	Outlet Pipe Reliability	Comments	HGL_{FC} - HGL_{EC} (ft)	BOB_{FC} - BOB_{EC} (ft)
Broadway (KCMO)	24+77	NA - See comments	NA - See comments	NA - See comments	NA - See comments	NA - See comments	54" RCP Requires Insp	By inspection, because strength is marginal for EC, improvements such as bracing or addition of web wall would be required	0	
Santa Fe (KCMO)*	52+85	NA - See comments	NA - See comments	NA - See comments	NA - See comments	NA - See comments	120" RCP	By inspection, stability likely acceptable and only the discharge chamber and extension of sluice gate stem required.	0	
Ohio (KVDD)	83+52	1.92	FS=0.44 Braced Frame in Wetwell	FS=0.76 R/C Overlay/ Shorten pump intakes	3.63	1.59	42" RCP	Address deterioration. Check with Paul/Katrina about attached gatewell. Verify that no additional fill will be placed over pipe. Pipe is okay with no additional fill.	0	0
Mistletoe (KVDD)	37+06.5	1.61^	2.25^	3.54^	2.13^	1.62^	18"CIP Requires Insp.	Raise of discharge chamber and extension of sluice gate stem. Recommend CIPP for economic analysis due to pipe age and unknown condition.	1.13	-3
New Central (KVDD)	58+12	1.57^	2.16^	4.16^	2.85^	2.03^	84" RCP		1.31	-2
Stockyards #3 (KVDD)	74+21	NA - See comments	NA - See comments	NA - See comments	NA - See comments	NA - See comments	24" CIP Requires Insp.	Abandon in place	2.07	2
Gateway (KVDD)	80+90	2.13^	>1.5	2.47^	1.82^	1.43^	60"RCP		2.84	3
Kemper Arena (KCMO)	106+49	NA - See comments	NA - See comments	NA - See comments	NA - See comments	NA - See comments	6' x 6' RCB Shear 2.45 Moment 2.91	By inspection, structural modifications and addition of weight or control of subsurface hydrostatic pressure required for future conditions	2	

Rehabilitation Required

FS equal to or greater than 1.5, no rehabilitation required

Strength: 1.5 FS required per established criteria for shear and moment

Stability: 1.1 FS required per established criteria

* Pump station is pile supported. The piles were not analyzed.

^ Deterministic Value was used to obtain the values shown, per e-mail sent by M.Parks on 5/9/08.

A-14.6 ANALYSIS OF NEW RELIEF WELL FLOWS

The following is a conceptual analysis of the impacts that proposed relief wells will impose on existing infrastructure in two locations located along the Kansas River, in both Missouri and Kansas. Maps of each location are included as Exhibit A-14.3 in the Supplemental Exhibits section.

A-14.6.1 Kansas River Station 107+00 to Station 116+70

Eighteen (18) surface discharging relief wells are proposed along the Kansas River between Sta. 107+00 and Sta. 116+70, adjacent to the American Royal Building (south of Kemper Arena) and the American Crane and Tractor building, south of the intersection of American Royal Drive and Genessee Street. The proposed quantity of stormwater that will be discharged by the relief wells is 18 cfs (8,100 gpm).

The well discharge elevations will be approximately 753. The top elevation of the current floodwall within this area is approximately 765. Therefore, the Kansas River will still have 12-ft of available freeboard before cresting the current floodwall prior to surface discharging occurring at this location. Once the river elevation equals the elevation of the relief wells, the wells will begin discharging to the surface.

Once the wells begin to discharge, stormwater will begin to pond within the existing rail yard and green space north and east of the existing American Crane and Tractor building. Approximately 91,000 cubic feet of storage is available in this area. Per the current information available for this area, it does not appear that an existing storm sewer system is available to drain this low area. However, 18 cfs is a relatively small amount of stormwater and should be able to be handled by most storm sewer systems, with minimal ponding occurring if there is in fact an existing drainage system available that drains this existing low area. Assuming full flow and no existing storm sewer (worst case), this low area would take approximately 1.4 hours to fill up, prior to overflowing onto the curb and gutter system along American Royal Drive.

Upon overflowing onto American Royal Drive, the existing enclosed storm sewer system along American Royal Drive has the capacity to handle the discharge from the wells. However, if the stormwater does not get into the enclosed system due to additional rainfall falling within the drainage area or clogging of the existing inlets, the stormwater will overtop the curb and pond in the parking lot west of the American Royal Building. There is approximately 31,000 cubic feet of storage available in this area. This low area is currently drained by an existing enclosed storm sewer system. Assuming that this inlet is clogged, it would take approximately 30 minutes to fill this area and then begin to flood the existing adjacent buildings. Presumably the existing inlets will be free of clutter and allow proper drainage of the well discharge. There are numerous inlets along American Royal Drive and the parking lot west of the American Royal Building that has adequate capacity to handle the additional stormwater. The vertical information for the existing storm sewer was not available for this analysis. The assumption was that the pipes were laid to the absolute minimum slope approved by APWA, which is 0.08% for

the 36" RCP that drains this area. Assuming no additional surface runoff other than the wells, the 36" RCP at a 0.08% slope has adequate capacity to handle the additional flow.

Therefore, based on the information available, the existing enclosed storm sewer downstream of Sta. 107+00-Sta. 116+70 has sufficient capacity to handle the additional 18 cfs from the surface discharge from the proposed relief wells, assuming that no additional surface runoff was occurring with the drainage area.

During the instance where the relief wells are flowing and a small rainfall event occurs, the existing enclosed storm sewer system would be required to handle an additional 9 cfs of surface runoff for a total runoff amount of 27 cfs. Assuming that the existing storm sewer system has a slope greater than 0.20%, the existing system will have adequate capacity to handle the relief well discharge, in addition to the small rainfall event runoff.

A-14.6.2 Kansas River, Station 127+00 to Station 166+83

Eighty three (83) surface discharging relief wells are proposed along the Kansas River between Sta. 127+00 and Sta. 166+83, adjacent to existing railroad buildings and infrastructure. The specific uses and vacancies for these buildings are not known at this time. The proposed quantity of stormwater that will be discharged by the relief wells is 108 cfs (48,474 gpm). These wells release stormwater into two watersheds, split by the KC Terminal Railroad Bridge at approximately Sta. 132+00. For the purpose of this report, the total 108 cfs has been prorated between each drainage area by stationing. Therefore, approximately 94.4 cfs (42,369 gpm) will drain to the south and approximately 13.6 cfs (6,104 gpm) will flow to the north.

A-14.6.2.1 Station 127+00 to Station 132+00

The well discharge elevations for this area will be approximately 758-ft, which is 7-ft below the current top of floodwall elevation of 765-ft.

As the river begins to rise above the top elevations of the well, stormwater will begin to discharge and eventually start to pool along Genessee Street. Per the current available information, there is an existing storm drainage system, but the details of said system are not known. It appears that this enclosed storm sewer discharges directly into the Turkey Creek Outfall Sewer. This area of ponding has an estimated storage volume of 156,000 cubic feet and will be two feet deep at its deepest point, if the existing storm sewer system is non-existent or inundated. If the stormwater does begin to pond, it will take 3.2 hours for this area to fill up and close Genessee Street. If the existing storm sewer system happened to be clogged or inundated and does not allow the proper passage of stormwater, nearby buildings will begin to experience flooding. However, the drainage area for this low area is quite significant. If there are no current flooding issues at this location, the enclosed storm sewer system should be adequately sized to handle the 13.4 cfs from the relief wells.

Therefore, presuming the enclosed storm sewer exists and is sufficiently sized for its drainage area, the area between Sta. 127+00 and Sta. 166+83 should not experience significant flooding due to the additional discharge from the relief wells assuming that no

additional surface runoff was occurring with the drainage area. Unfortunately, there is insufficient utility and topographical information to determine if a small rainfall event over the drainage area while the relief wells are flowing will cause any adverse flooding.

A-14.6.2.2 Station 132+00 to Station 166+83

The well discharge elevations for this area will range between 753-ft and 756-ft, which is 9 to 13-ft below the current top of floodwall elevation of 765-ft for this stretch of the river.

Once stormwater begins to discharge from the relief wells, it will travel through the existing rail yard and eventually start to pond along the southeast side of the rail yard. Approximately 303,000 cubic feet of storage (at a depth of one foot) is available for the stormwater to collect. Assuming no direct outfall from this low area, it would take 0.9 hours for this area to fill up and then begin to adversely affect nearby buildings. However, per storm sewer atlas maps, numerous existing enclosed storm sewer systems are located in the drainage area of these relief wells. Furthermore, as built conditions for these sewers were not available to analyze their current capacities. Similar to the northern drainage area, the southern drainage area is quite large, roughly 80 acres. Assuming that the existing storm sewer systems have been sized to handle the normal rainfall events, they should have adequate capacity to handle the 94.4 cfs that is proposed to discharge from the relief wells. Based on a quick Rational Method calculation, 80 acres of undeveloped land and a five minute time of concentration (worst case scenario) would produce over 175 cfs of runoff during the typical 10-year storm event. Presuming the storm sewers are adequately sized to handle the 10-year storm event, the enclosed system would be adequately sized to handle the additional stormwater generated from the relief wells. If the existing storm sewers are slightly undersized, the broad, large storage area should provide enough ponding for the stormwater to safely discharge without adversely affecting any buildings. Therefore, presuming the enclosed storm sewer is adequately sized to handle the 10-year storm event for its drainage area, the area between Sta. 132+00 and Sta. 166+83 should not experience significant flooding or infrastructure damage due to the additional discharge from the relief wells assuming that no additional surface runoff was occurring with the drainage area. Unfortunately, there is insufficient utility and topographical information to determine if a small rainfall event over the drainage area while the relief wells are flowing will cause any adverse flooding.

A-14.6.2.3 Relief Well Flow Recommendation

Based on the discussion presented above, it is recommended to allow the new relief wells to discharge at ground surface. No work is proposed to provide a new collector system nor modify or construct pump stations specifically for these flows.

A-14.7 REFERENCES

1. US Army Corps of Engineers (Feb. 28, 1995), *EM 1110-2-3102 CECW-EE General Principles of Pumping Station Design and Layout*, Kansas City District.

2. US Army Corps of Engineers (June 30, 1989), *EM 1110-2-3104 CECW-ED Structural and Architectural Design of Pumping Stations*, Kansas City District.

3. US Army Corps of Engineers (Nov. 30, 1999), *EM 1110-2-3105 CECW-EE Mechanical and Electrical Design of Pumping Stations (Change 2)*, Kansas City District.

4. US Army Corps of Engineers (Apr. 30, 2000), *EM 1110-2-1913 CECW-EG Design and Construction of Levees*, Kansas City District.

5. US Army Corps of Engineers (Mar. 31, 1998), *EM 1110-2-2902 CECW-ED Conduits, Culverts, and Pipes*, Kansas City District.

A-14.8 SUPPLEMENTAL EXHIBITS

EXHIBIT A-14.1

Pump Station Information and Photographs

1. Broadway
    a. Year Constructed: 1949[11]
    b. Owner/Operator: KCMO
    c. Levee Station: 24+75 (MO)
    d. Distance From Flood Protection Centerline: 35-ft[7]
    e. Station Capacity: 3,450gpm[1]
    f. Pumps: --
    g. Contributing Flows:
        i. Seepage flow (from seepage system between Sta. 22+82 and Sta. 0+00)[1] NOTE: According to KCMO staff no seepage flow comes into pump station [11]
        ii. Storm runoff (flow is contributed by the railroad yards area, 28ac, in the northeastern part of the Central Industrial District)[1]
    h. River Discharge: 54" RCP[1]
    i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee[7]
    j. Drawing Availability:
        i. Feb 1981 – General Plan View[7]
        ii. Dec 1955 – Gatewell Plans[7]
    k. General
        i. There is a floodwall at the pump plant location. [3]
        ii. The floodplain contains one small depression available for storage of surface runoff and seepage. The area which is protected by the floodwall is occupied by the Missouri Pacific Railroad yards and the Missouri Produce Company's railroad yards. [3]
        iii. Seepage begins at 29.5 and is, therefore, negligible at design river stage. [3]
        iv. The storm runoff is greater in design flood stage because there would be no infiltration over approximately one-half of the normally pervious area. [3]
        v. The City of Kansas City, Missouri stated that the Broadway Pump Station drains the hillside area and the railroad tracks, so there is almost always flow going through the station. However, the only time that the pumps were needed was in 1993. The pumps are the original pumps. [3]
        vi. There were plans to rehabilitate the pump station a few years ago, but the work was cancelled. The City did note, though, that new flap gates were installed because they had already been ordered. The City does not know if the remainder of the plans will be carried out in the future. [3]
        vii. The conditions of the service area for this pump plant have not gone through any significant changes since the design of the plant. The estimated percent impervious is from visual inspection of 1996 aerial photographs. [3]

2. Santa Fe Pump Station
    a. Year Constructed: 1949[11]
    b. Owner/Operator: KCMO[3]
    c. Levee Station: 52+85 (MO)
    d. Distance From Flood Protection Centerline: 45-ft[7]
    e. Station Capacity: 42,500gpm[1]
    f. Pumps: --
    g. Contributing Flows:
        i. Seepage flow from the seepage system between Sta. 78+00 to Sta. 22+82. [1]
        ii. Storm sewer system flow contributed by a bottoms area, 546ac, and a high bluff area in the northern portion of the Central Industrial District. [1]
    h. River Discharge: 10' RCP[1]
    i. Discharge Arrangement: Gatewell & pressurized pipe under floodwall[7]
    j. Drawing Availability:
        i. Feb 1981 – General Plan View[7]
        ii. Dec 1955 – Gatewell Plans[7]
    k. General
        i. There is a floodwall at this pump plant location. [3]
        ii. The Armour Packing Plant area has a pump plant that is adequate, but the sewer surcharges in the area where the flow would head to the Santa Fe Pump Plant. Therefore, the Armour Packing Plant area is considered undrained. [3]
        iii. In 1962, modifications were done to divert the sanitary flow to the West Side Sewage Treatment Plant. In doing so, the capacity of the stormwater side of the pump plant was decreased. Two of the pumps which were used previously for stormwater pumping became dedicated to sanitary flow pumping. [3]
        iv. HNTB performed a study on the Santa Fe Pump Station in 1994, which was followed by some renovation of the plant in about 1995. Pumps were rehabilitated, but capacities were not increased. [3]
        v. The drainage area determined for the 1994 HNTB report is slightly larger than that originally estimated by the COE. It was assumed in the report that the lesser area may have deducted non-contributing depressed areas that were present at the time. [3]
        vi. A small amount of additional development has taken place in the Santa Fe Pump Station service area which is reflected in the percent impervious. The estimated percent impervious was determined by visual inspection of 1996 aerial photographs. As the capacities of the pump station were not increased in 1994, the deficiencies still exist. [3]

vii. The pumping capacity may or may not be adequate based on the impacts of the increased percent impervious. [3]

3. Ohio (KCK FPS 1)
   a. Year Constructed: 1945 (Reconstructed 1950) [4]
   b. Owner/Operator: KCK
   c. Levee Station: 83+52 (MO) [1]
   d. Distance From Flood Protection Centerline: 170-ft[7]
   e. Station Capacity: 32,650gpm[4]
   f. Pumps:
      i. Worthington Impeller Centrifugal 16" design capacity 8,400gpm 870RPM, drive 100hp[4] Appears to have been removed.[9]
      ii. Fairbanks-Morse impeller axial flow 16" design cap. 6,800gpm 1,200RPM, drive 50hp[4]
      iii. Fairbanks-Morse impeller axial flow[4] 24" design cap. 17,4500gpm@17.3TDH 880RPM, drive 100hp[7]
   g. Contributing Flows:
      i. Seepage flow from the area between sta. 00+00 and approx. sta. 26+00, plus the seepage from approx. sta. 46+00 to approx. sta. 65+00. [1]
      ii. Storm sewer system flow contributed by the area, 110ac, just south and southwest of the junction of the Missouri and Kansas Rivers. [1]
   h. River Discharge: 42" RCP[1]
   i. Discharge Arrangement: Gatewell & pressurized pipe under levee[7]
   j. Drawing Availability:
      i. Oct 1950 – Plan & Profile[7]
      ii. Jul 1950 – Mechanical, Electrical & Structural Details[7]
      iii. Oct 1943 – Structural plan [5]
   k. General
      i. Approximately half of the topography is very flat, with the remainder having moderate slopes. There is one small depression located near the state line available for surface storage. [3]
      ii. The outlet is located at mile 378.43 on the Missouri River. Critical stage is 22.0-ft (where the river begins to back up into the pipe), but the pumps are not started until 28.0-ft. [3]
      iii. As noted in the 1948 "Supplement on Interior Drainage", one of the stormwater pumps cannot be depended on for any discharge in the Design Flood Condition as the total dynamic head is in excess of the maximum head for which it is rated. [3]
      iv. The City of Kansas City, Kansas owns and operates the pump plant. They call it Flood Pump Station #1. The equipment listed for the plant includes the original pumps, although

some rehabilitation work may have been done in recent years. HNTB is in the process of retrieving information pertaining to the possible rehabilitation. [3]

v. Sanitary flow now goes to the treatment plant and is, therefore, not included here. There is a minor reduction with respect to total flow contributing to the plant as a result. [3]

vi. Commercial development has contributed to a greater amount of impervious area within the service area. The estimated percent impervious is from visual inspection of 1996 aerial photography. The greater impervious area would result in an increase in runoff from the area. [3]

vii. The total flow reaching the plant remains the same in the Design River Condition because the sewer system capacity controls. [3]

4. Mistletoe Pump Station
   a. Year Constructed: ~1950
   b. Owner/Operator: KVDD
   c. Levee Station: 37+07 (KS) [1]
   d. Distance From Flood Protection Centerline: 0-ft[7]
   e. Station Capacity: 3150gpm[1]
   f. Pumps:
      i. 8" 2,660gpm@15-ft RDH 700RPM Centrifugal Sewage, dry pit[7] 15hp motor[10]
      ii. 8" 2,660gpm@15-ft RDH 700RPM Centrifugal Sewage, dry pit[7] 15hp motor[10]
      iii. Sump Pump 20gpm@20-ftTDH 116RPM[7]
   g. Contributing Flows:
      i. Seepage flow the area between approx. Sta 26+00 and approx. Sta. 46+00. [1] Floodwall Toe Drain[10]
      ii. Storm runoff flow contributed by the area, 7ac, immediately adjacent to the pump station. [1]
   h. River Discharge: 18" CIP[1]
   i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under floodwall[7]
   j. Drawing Availability:
      i. July 1981 – General Plan & Profile[7]
      ii. July 1950 – Mechanical, Electrical, Structural & Architectural Details[7]
   k. General
      i. The ponding that would occur in the plant at the design flood stage should not be damaging. It should be about 17 minutes in duration and cover about .003 acre-feet. [3]
      ii. The plant is owned and operated by KAW Valley Drainage District. Larry Brennan, manager of the district, stated that the only time the Mistletoe Plant ever ran was in 1993. [3]

iii. The only change with respect to plant design is that the sanitary flow has been diverted to the treatment plant. This will decrease the total flow draining to the pump plant. There have been no significant changes with regard to the stormwater contributing area of the pump plant. Estimated percent impervious was determined from visual inspection of 1996 aerial photography. [3]

5. New Central Avenue (KCK FPS 16)
   a. Year Constructed: 1987[3]
   b. Owner/Operator: KCK
   c. Levee Station: 58+12 (KS) [1]
   d. Distance From Flood Protection Centerline: 40-ft[5]
   e. Station Capacity: --
   f. Pumps:
      i. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5-ft head, drive 145hp[4]
      ii. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5-ft head, drive 145hp[4]
      iii. Cascade 1 stage 24MF 700RPM 19,000gpm @ 25.5-ft head, drive 145hp[4]
      iv. Sump Pump 3" discharge 200 gpm at 19-ft head[4]
   g. Contributing Flows:
      i. Storm sewer system flow [1]
   h. River Discharge: 84" RCP[1]
   i. Discharge Arrangement: Gravity drainage under levee pressurized pipe place up and over levee discharging into gatewell on riverside of the levee[9]
   j. Drawing Availability:
      i. Sep 1988 record drawings[5]
   k. General
      i. When the Kansas River stage is low enough to permit gravity flow, the Central Avenue Pump Plant is on standby. Water entering the stormwater collection system flows through the station inlet to the river by gravity. However, once the critical stage is reached, the gatewell sluice gate is closed and stormwater is pumped to the riverside of the gatewell sluice gate. This creates a high enough head in the wetwell to permit gravity flow to the river (going through the same outlet pipe as in the former gravity flow case). [3]
      ii. The City of Kansas City, Kansas calls this Flood Pump Station #16. [3]
      iii. The plant has a sump pump rated at 200 gpm for 19--ft of head. [3]

iv. The pump plant was designed for certain flows at total dynamic head conditions. It is thought that the drainage area conditions have not changed in the past 15 years. [3]

6. Stockyards #3
    a. Year Constructed: ~1950
    b. Owner/Operator: KVDD
    c. Levee Station: 74+21 (KS) [1]
    d. Distance From Flood Protection Centerline: 85-ft[7]
    e. Station Capacity: 10,300gpm[1]
    f. Pumps:
        i. 14" 5710 WX Fairbanks Morse pump, drive 50hp 860RPM[7]
        ii. 14" 5710 WX Fairbanks Morse pump, drive 50hp 860RPM[7]
        iii. 14" Submersible Fairbanks Morse Pump, 47.5hp 705RPM[10]
        iv. Sump Pump
    g. Contributing Flows:
        i. Seepage flow for the area between station 65+00 and station 82+00 coming through relief wells. [1]
        ii. Storm runoff drainage from the west side of the Central Industrial District (Kansas), less than 47ac. [1]
    h. River Discharge: 24" CIP[1]
    i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under levee[7]
    j. Drawing Availability:
        i. July 1980 – General Plan & Profile, Other Details (12 sheets) [7]
    k. General
        i. It was found by the COE that excessive damage occurs from the extended duration of ponding caused by high seepage rates over long periods and not by ponding of short duration caused by storm runoff. The designed pumping capacity was based upon this finding. [3]
        ii. KAW Valley calls this pump plant the Stockyards Pump Plant (used to be called Stockyards #3 Pump Plant). [3]
        iii. The Stockyards Pump Plant used to take relief well seepage flow, along with storm drainage, from the stockyards area. The construction of the Gateway 2000 facility cut off some of that storm drainage area, so the overall flow to the plant has decreased. In addition, the sanitary flow was diverted to the treatment plant. The estimated percent impervious is from visual inspection of 1996 aerial photography. [3]
        iv. The pump capacities are noted because they were previously inadequate. It is not currently known if the decreased drainage area and runoff will be enough to nullify that inadequacy. [3]
        v. New submersible pump installed in 1995.[10]

7. Gateway 2000
    a. Year Constructed: 1994
    b. Owner/Operator: Gateway 2000
    c. Levee Station: 80+90 (KS) [1]
    d. Distance From Flood Protection Centerline: 13-ft
    e. Station Capacity: 9,500gpm[1]
    f. Pumps:
        i. Flygt submersible L3531/705 14,100@8-ft of head and 9,500gpm@25-ft of head. [8]
        ii. Flygt submersible L3531/705 14,100@8-ft of head and 9,500gpm@25-ft of head.[8]
    g. Contributing Flows:
        i. Storm sewer system flow contributed by the Gateway 2000 property, approximately 80ac. [1]
    h. River Discharge: 60" RCP[1]
    i. Discharge Arrangement: Gatewell at pump station and pressurized pipe under the levee.
    j. Drawing Availability:
        i. Oct 1996 Plans – complete set[8]
    k. General
        i. The Gateway 2000 Pump Plant was constructed in 1994 to serve the area surrounding the Gateway 2000 facilities. [3]
        ii. The pump plant is located in Kansas and discharge through the CID Kansas Unit. However, a portion of the contributing service area is located in Missouri. [3]
        iii. The pump plant was designed to have certain pumping capacities at given river stages. Therefore, the exact service area information was not obtainable. It was estimated from drainage area mapping provided by the designer of the plant. The estimated percent impervious was determined by visual inspection of 1996 aerial photographs. However, the conditions of the approximate service area have not changed since the pump plant construction. [3]
        iv. Replaced Stockyards pump station #2 (pigeon roost)[10]

8. Stockyards #1
    a. Year Constructed: N/A
    b. Owner/Operator: KVDD
    c. Levee Station: 98+05 (KS) [1]
    d. Distance From Flood Protection Centerline: 75-ft[7]
    e. Station Capacity: N/A
    f. Pumps: N/A
    g. Contributing Flows:

i. Storm sewer system flow from the west side of the Central Industrial District (Kansas), less than 14ac. [1]

h. River Discharge: 24" CIP[1]

i. Discharge Arrangement:

j. Drawing Availability:

i. July 1980 – General Plan & Profile[7]

k. General

i. The capacity of the pump is actually 4.23 cfs, but the lack of power from the motor brings that down to the 1.47 cfs. [3]

ii. There is a total storage of 0.44 acre-feet required for ponding, with a maximum duration of 110 minutes. Part of this flow would drain to Valve No. 7. The ponding should not cause damage. [3]

iii. The outlet for the pump plant is at Kansas River Mile 1.92. [3]

iv. KAW Valley Drainage District calls this plant the Field Pump House (formerly called the Stockyards Pump House #1). Larry Brennan said that the pumps have not needed to run for over 20 years. [3]

v. Part of the drainage area contributing flow to the pump plant was cut off by Kemper Arena. The percent impervious estimated by visual inspection of 1996 aerial photographs shows no significant change, but the total flow from the smaller drainage area must be significantly less judging from the fact that it does not run very often. [3]

vi. The pump is likely still inadequate. [3]

vii. Pump pit, gatewell and valve box are all brick and motor construction. Pump pit is approximately 2' dia., both the gatewell and valve box are approximately 2'x2' in size.[10]

viii. Gate valve is located in discharge conduit at the levee. This valve is kept closed by KVDD. [10]

ix. Pump station has not been operational recently. During the 1993 flood crews tried, without success, to operate the pump. [10]

x. KVDD thought that most overland flow has been diverted to Kemper Pump Plant. [10]

9. Kemper (Area)

a. Year Constructed: 1975

b. Owner/Operator: KCMO

c. Levee Station: 106+49 (KS) [1]

d. Distance From Flood Protection Centerline: 180-ft[7]

e. Station Capacity: --

f. Pumps:

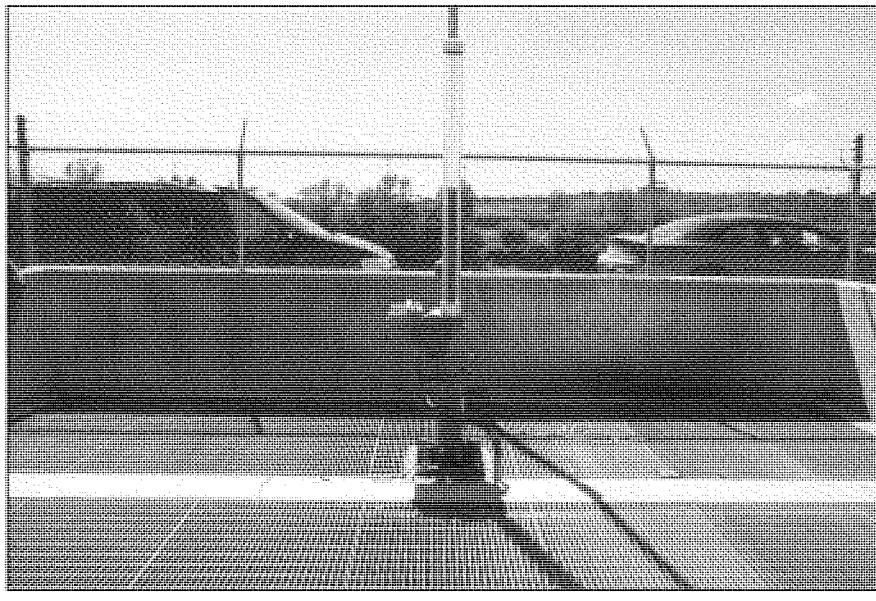
i. Four 36" column type, drive 350hp[12]

ii. Sump pump[12]

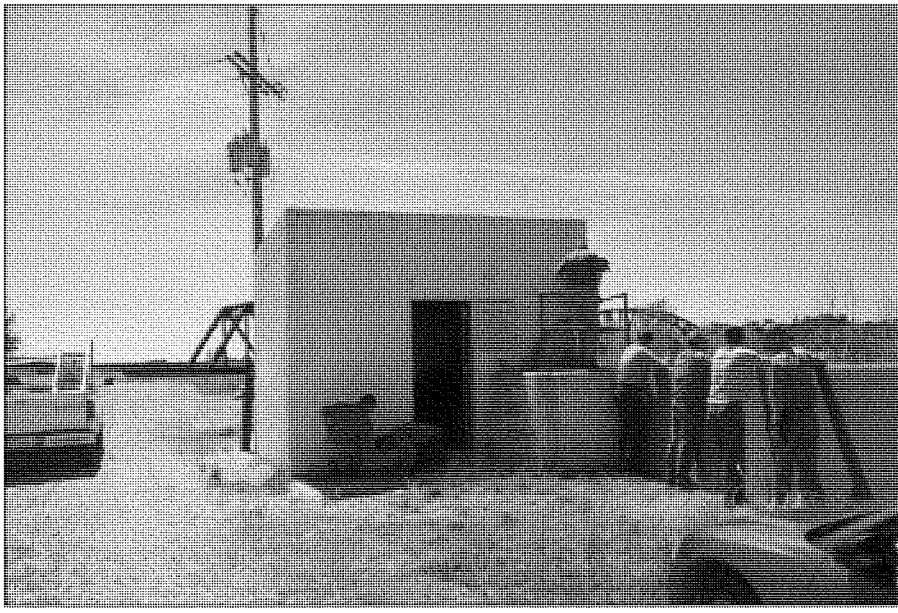
g. Contributing Flows:
    i. Storm runoff flow from the Kemper Arena and American Royal area, 51ac. [1]
h. River Discharge: 6'x6' RCB[1]
i. Discharge Arrangement: --
j. Drawing Availability:
    i. Jul 1980 – General Plan & Profile
k. General
    i. The pump plant is located in Missouri, with almost all of the drainage area in Missouri also. However, the discharge eventually goes through the Central Industrial District levee unit in Kansas. [3]
    ii. A representative of the City of Kansas City, Missouri noted that the pump plant was built in 1975 and has never had more than two pumps running. There are a total of four pumps at the plant. [3]
    iii. HNTB has been in contact with the designers in charge of runoff disposal from the new Butler Manufacturing building which will be constructed to the north of Kemper Arena. The runoff totals shown are preliminary estimates from the designers. Information is not readily available on the original design of the plant, but it is known that there are four vertical turbine pumps rated at 31,000 gpm each at 37-ft of head. There is a low river condition at which water is pumped to a point to be gravity discharged and a high river condition at the elevation noted. [3]
    iv. Current conditions are similar to design conditions for the Kemper Arena Pump Plant, with one exception. When the Butler Manufacturing building was constructed in the early 2000's, north of Kemper Arena, additional drainage was routed to the pump plant. A new 48" line from the site ties into the 6'x6' box between Kemper Arena and American Royal which feeds into the Kemper pump plant. With the exception of the Golden Ox parking lot, most of the area from Genessee Street and I-670 drains southward to the new pipe. An existing stormwater discharge pipe at the Livestock Exchange building was relocated around the new parking garage east of the Exchange building. The pipe was routed back to its original outlet location on the river. The project caused an approximate increase of flow to Kemper Pump Plant of 12.65 acres with a percent impervious of 87. The estimated percent impervious for present conditions was determined by visual inspection of 1996 aerial photographs. [3]

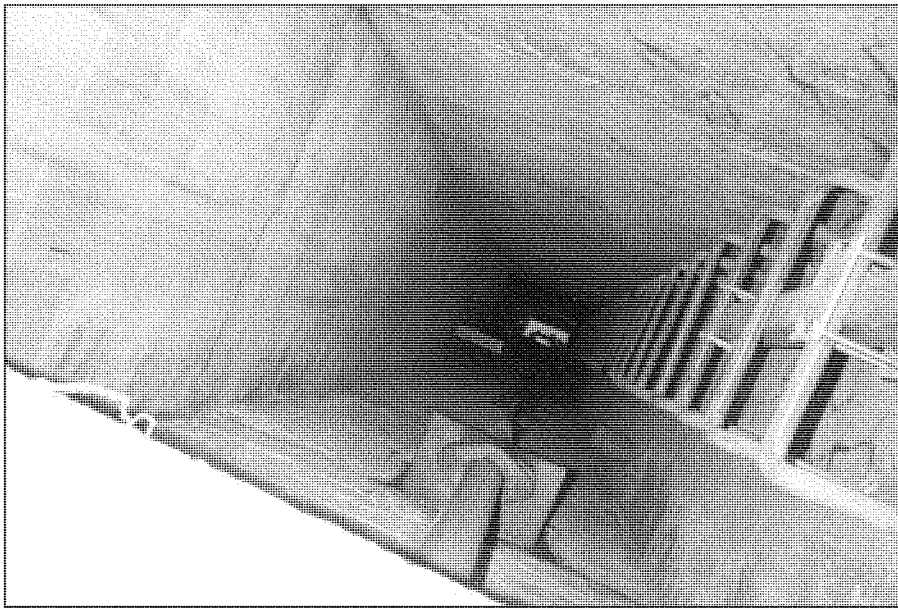
10. Pump Station Information References:
    a. HNTB Drainage Feature Inventory 2001 with updates
    b. HNTB Ownership Listing
    c. HNTB Latest
    d. KCK O&M Manuals from KCK Maintenance Library (Cary Houchins)
    e. KCK As-builts from KCK Sewer Department (Larry Henak)
    f. Verbal Conversations with Ernie Quinto
    g. O&M Drawings located at E:\sec\LocalProtectionProjects\KansasCity
    h. Gateway pump station As-builts supplied by Gateway (Chris Nijsse)
    i. Site visit with KCK 25-Jun-05
    j. Site visit with KVDD 04-Aug-05 (Ernie Quinto & Larry Brennen)
    k. Site visit with KCMO 15-Sep-05 (Mort Dasjordi & Steve Zulke)
    l. Periodic General Inspection Report No. 1, March 1981

Gateway 2000 Pump Station

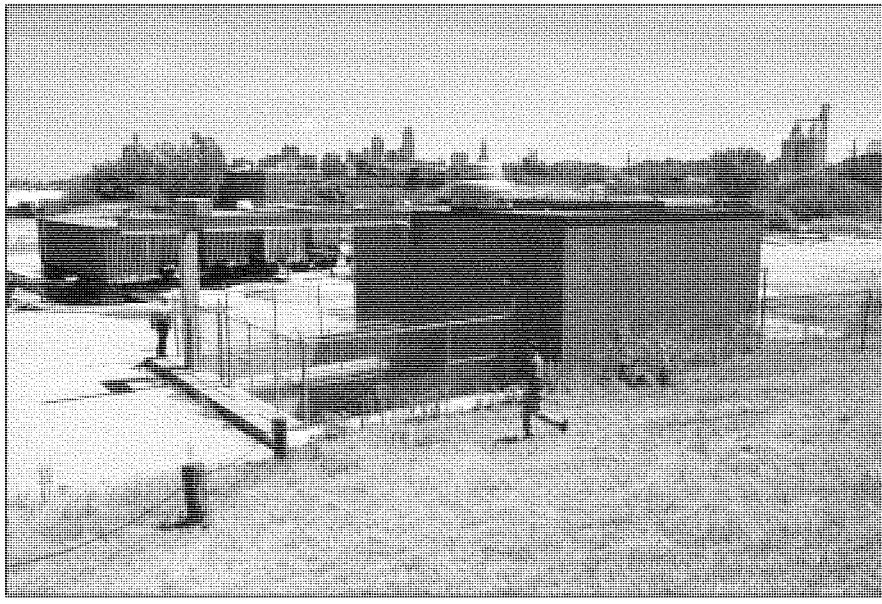


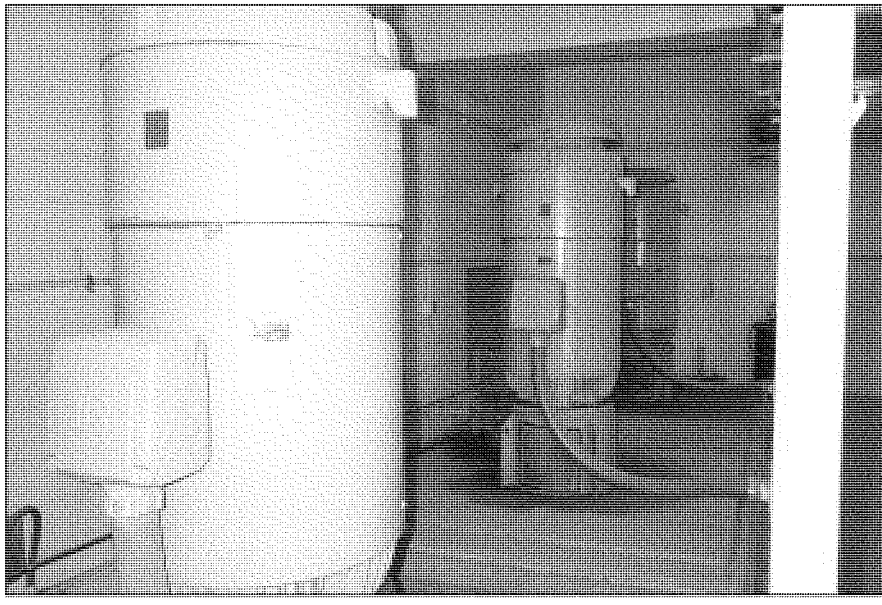
Mistletoe Pump Station



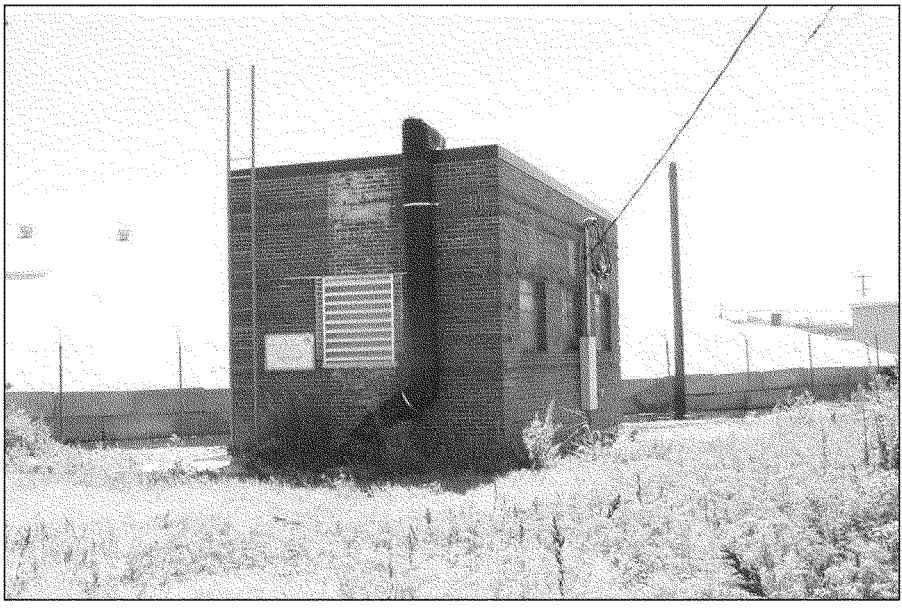


Central Avenue Pump Station

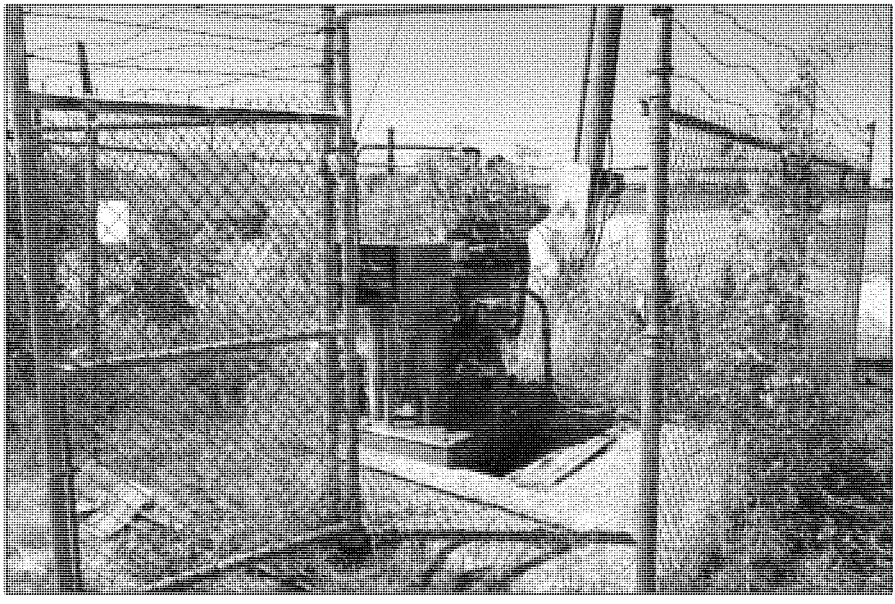


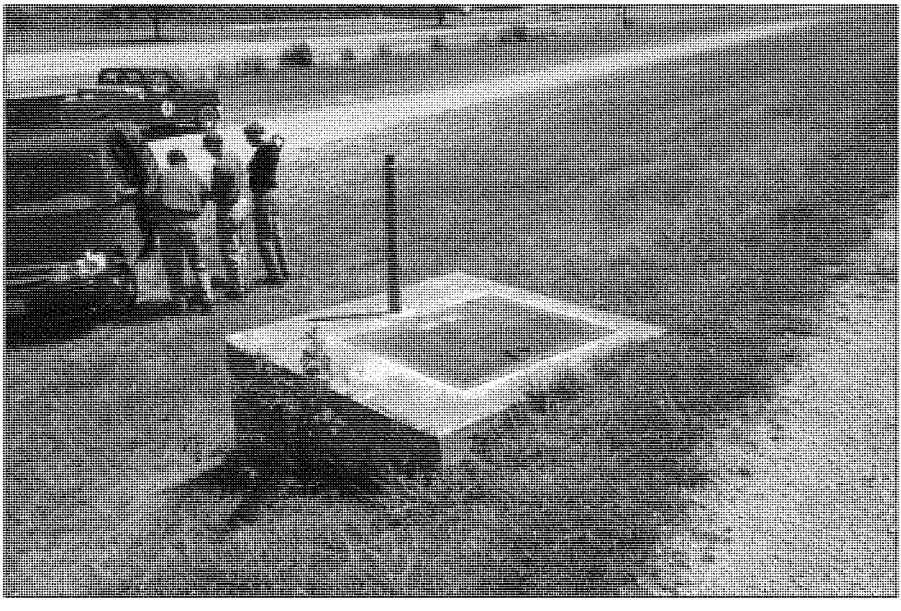


Ohio Avenue Pump Station

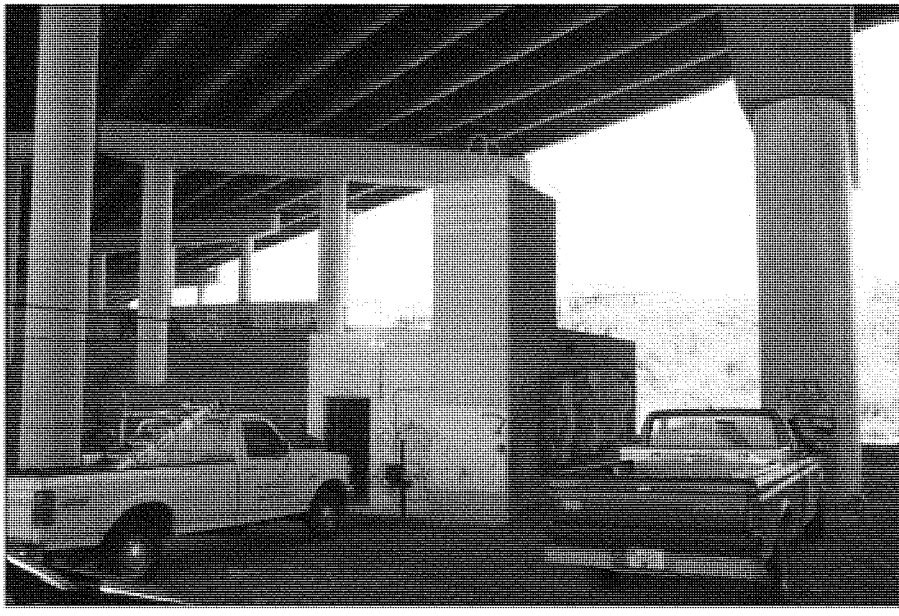


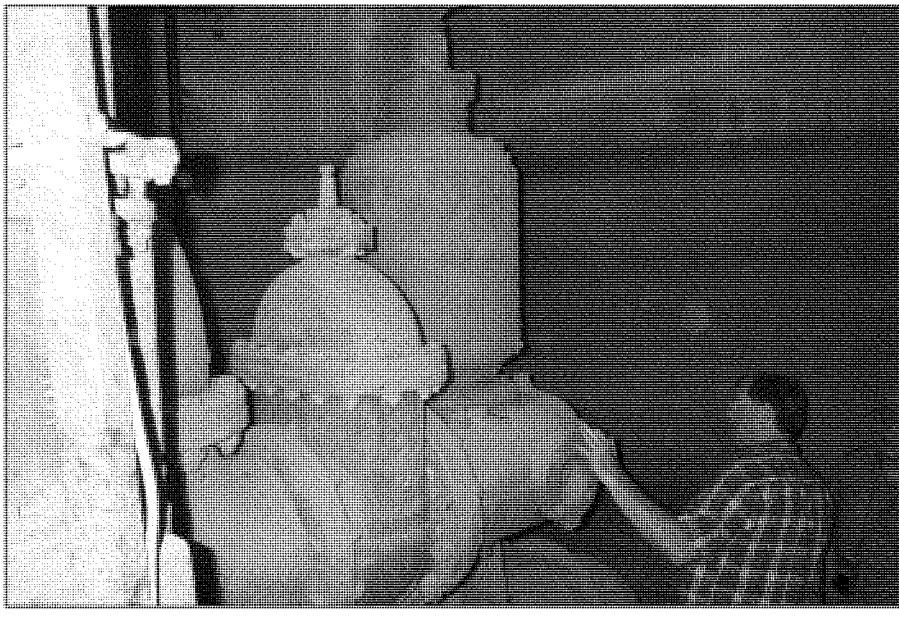
Stockyards #1/Field Pump Station



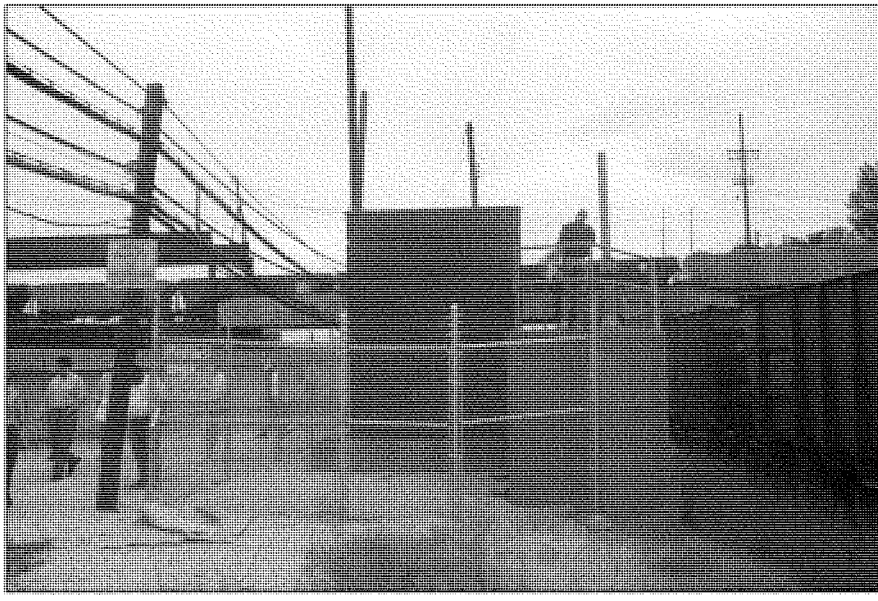


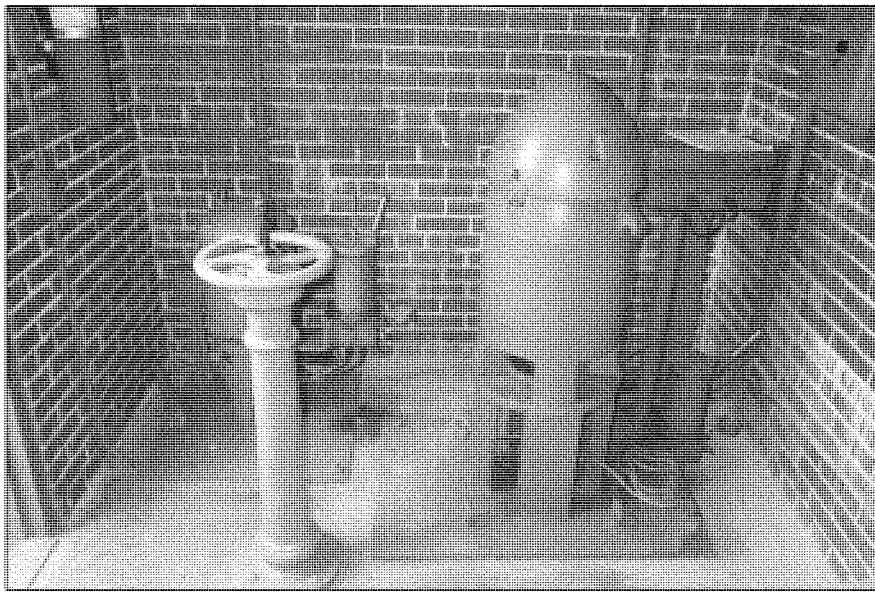
Stockyards #3 Pump Station



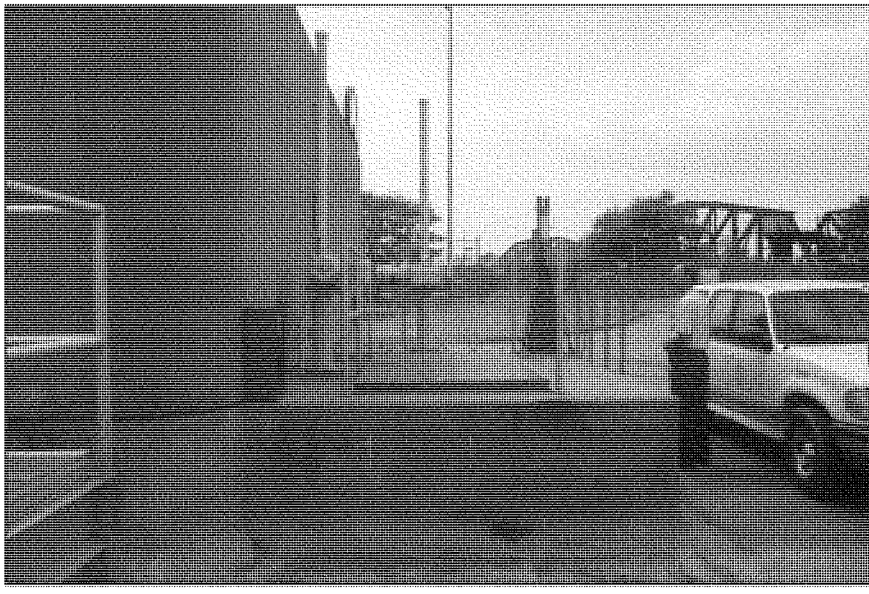


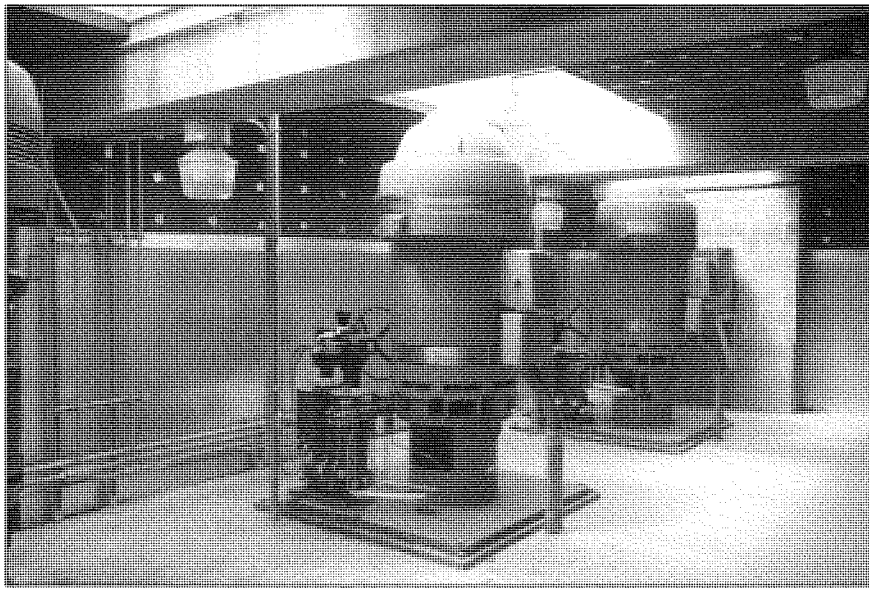
Broadway Pump Station





Kemper Pump Station





Santa Fe Pump Station

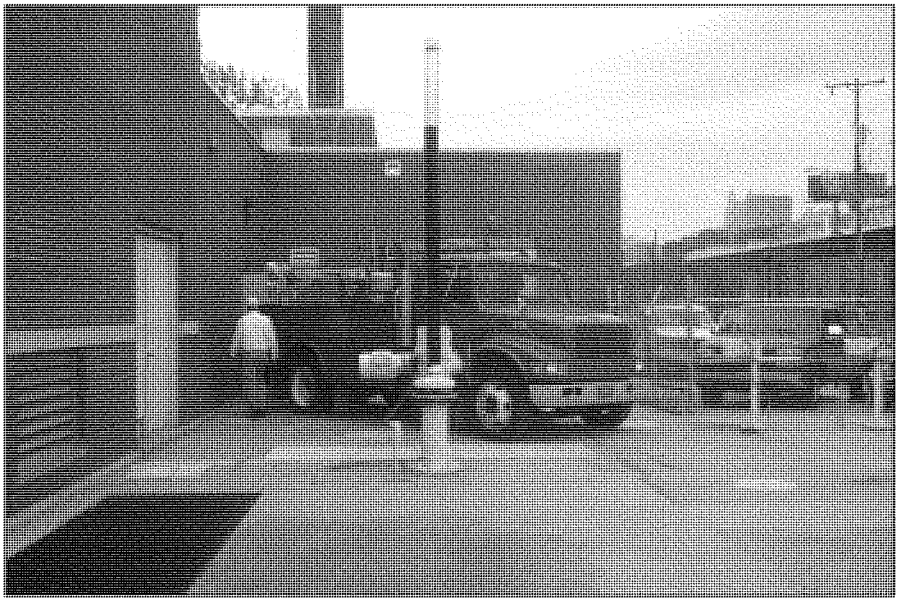


EXHIBIT A-14.2

Geotechnical Data (Hydraulic Gradeline Calculations)

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Broadway Pump Station Sta 24+76.9

Broadway Sta 24+76.9.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuczinskowski EC-3D
Date : 7/26/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 30 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Blanket details are not well known, very heterogeneous, mostly sand

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	759.00
500 year + 0':	
500 year + 3':	Note: Ignore all but the existing condition calculations
500 year + 5':	
Ground Surface Land Side:	752
Ground Surface River Side:	759
Bottom of Blanket:	745 (Assumed, based upon limited available information, blanket contains of mostly labels, no aquifer identified)
Top of Bedrock:	720 (Assumed, based upon limited available information)
Ground Surface at Pump Station:	759

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K1/K2)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

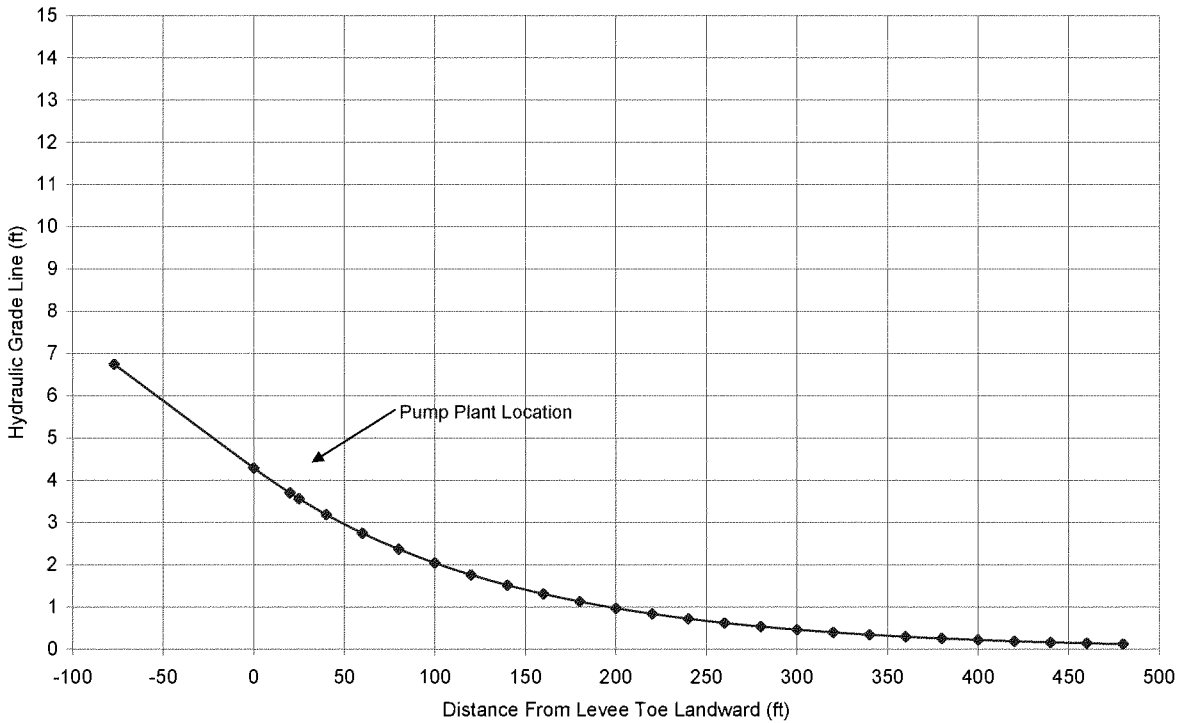
Saturated Unit Weight of Blanket: 115 pcf
Unit Weight of Water: 62.4 pcf

Station	Design Condition	Input Parameters: Permeability Ratio, River Side (K_1/K_2)	Permeability Ratio, Land Side (K_1/K_3)	Blanket Thickness (ft), River Side z_w	Blanket Thickness (ft), Protection z_{bo}	Blanket Thickness (ft), Land Side z_d	Aquifer Thickness (ft) d	Seepage Length (ft), River Side L_1	Seepage Length (ft), Protection L_2	Seepage Length (ft), Land Side L_3	Driving Head (ft) H
24+76.9	Existing Condition	100	100	5.0	6.1	7.3	25.0	70	15	1000	6.8
24+76.9	n500 + 0	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3
24+76.9	n500 +3	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3
24+76.9	n500 + 5	100	100	5.0	6.1	7.3	25.0	70	15	1000	-752.3

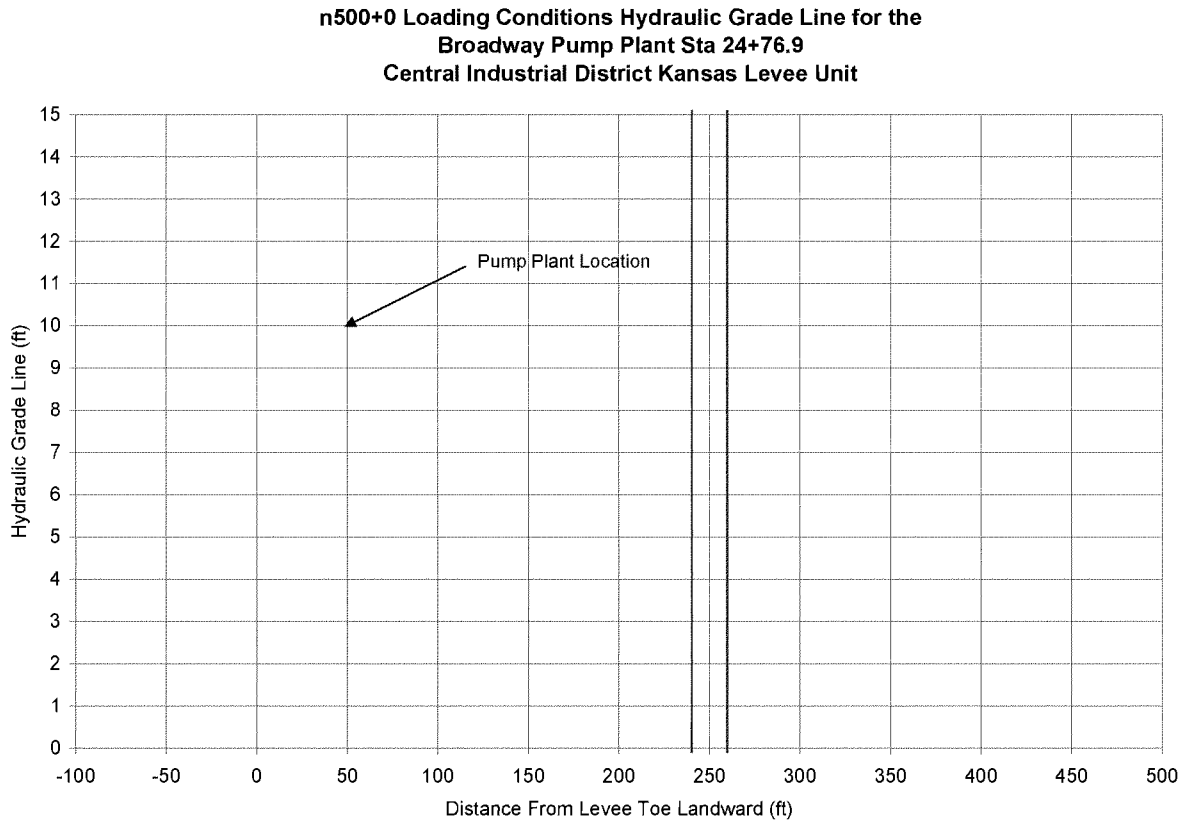
Station	Design Condition	Analysis: Factors e_x	Factors e_y	Effective Seepage Length (ft), River Side x_1	Effective Seepage Length (ft), Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_x	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ratio	Remarks
24+76.9	Existing Condition	0.008944	0.007428	62	135	4.3	0.59	0.84	1.42	
24+76.9	n500 + 0	0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	
24+76.9	n500 +3	0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	
24+76.9	n500 + 5	0.008944	0.007428	62	135	-478.3	-65.98	0.84	-0.01	

Hydraulic Grade Line for the Broadway Pump Plant Existing Conditions			
Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	6.75		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	4.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	6.75	759.00
	0	4.29	756.54
	20	3.70	755.95
Pump Plant Location	25	3.56	755.81
	40	3.19	755.44
	60	2.75	755.00
	80	2.37	754.62
	100	2.04	754.29
	120	1.76	754.01
	140	1.52	753.77
	160	1.31	753.56
	180	1.13	753.38
	200	0.97	753.22
	220	0.84	753.09
	240	0.72	752.97
	260	0.62	752.87
	280	0.54	752.79
	300	0.46	752.71
	320	0.40	752.65
	340	0.34	752.59
	360	0.30	752.55
	380	0.26	752.51
	400	0.22	752.47
	420	0.19	752.44
	440	0.16	752.41
	460	0.14	752.39
	480	0.12	752.37

Existing Conditions Hydraulic Grade Line for the Broadway Pump Plant Sta 24+76.9 Central Industrial District Kansas Levee Unit

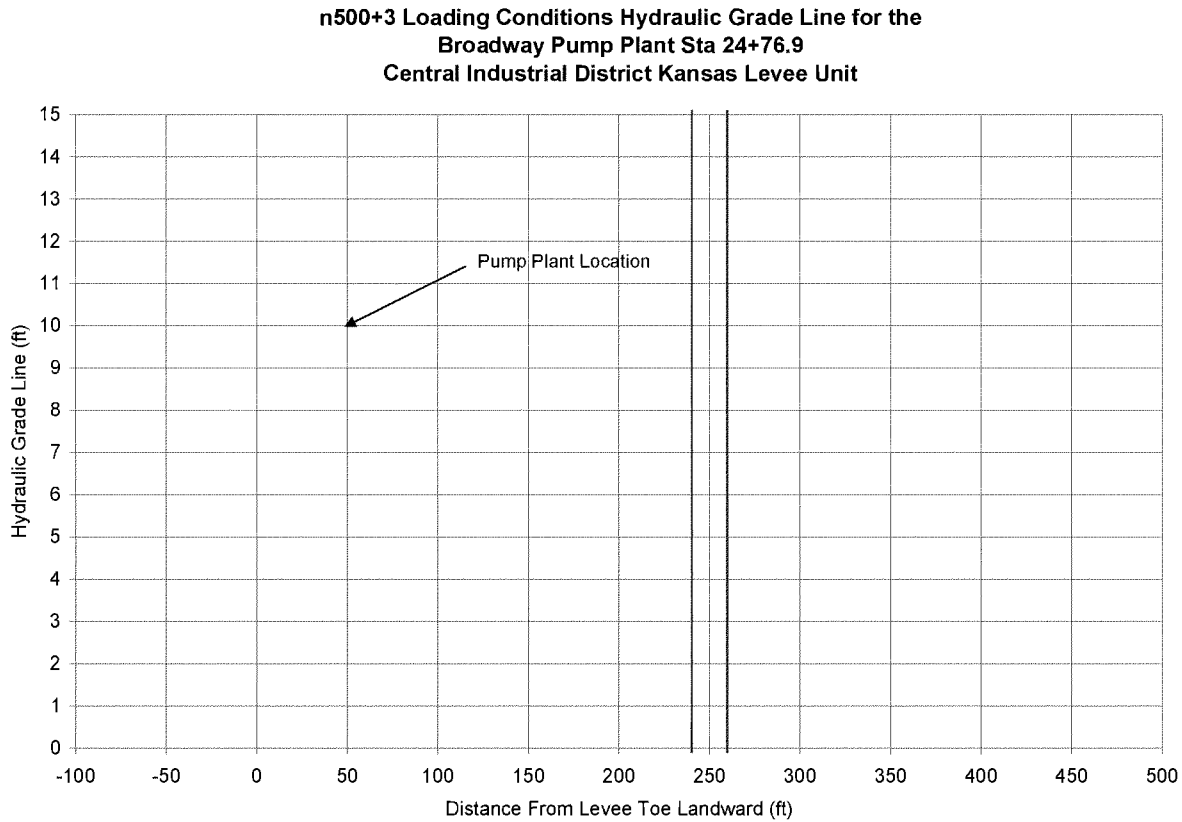


Hydraulic Grade Line for the Broadway Pump Plant n500+0 Loading Conditions			
Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_1	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72


n500+0 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

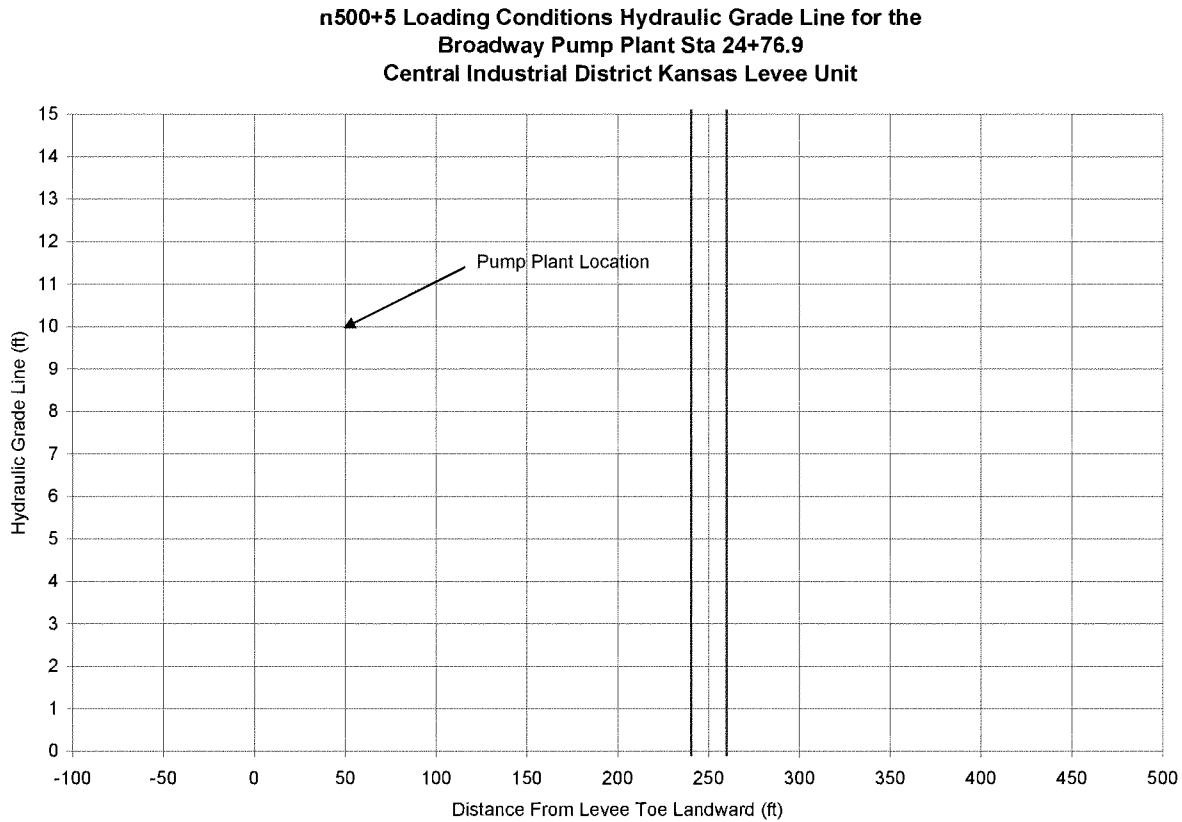
710

Hydraulic Grade Line for the Broadway Pump Plant n500+3 Loading Conditions			
Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_1	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72


n500+3 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

712

Hydraulic Grade Line for the Broadway Pump Plant n500+5 Loading Conditions			
Elevation Top of Protection (MSL)	759	Existing Conditions	
Elevation at Landside Toe (MSL)	752.25		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-752.25		
c_l	0.007428		
x_1 (ft)	62		
L_2 (ft)	15		
x_3 (ft)	135		
h_o	-478.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-77	-752.25	0.00
	-40	-620.46	131.79
	0	-478.34	273.91
	20	-412.31	339.94
Pump Plant Location	25	-397.28	354.97
	40	-355.39	396.86
	60	-306.33	445.92
	80	-264.04	488.21
	100	-227.59	524.66
	120	-196.17	556.08
	140	-169.09	583.16
	160	-145.75	606.50
	180	-125.63	626.62
	200	-108.28	643.97
	220	-93.34	658.91
	240	-80.45	671.80
	260	-69.35	682.90
	280	-59.77	692.48
	300	-51.52	700.73
	320	-44.41	707.84
	340	-38.28	713.97
	360	-32.99	719.26
	380	-28.44	723.81
	400	-24.51	727.74
	420	-21.13	731.12
	440	-18.21	734.04
	460	-15.70	736.55
	480	-13.53	738.72


n500+5 Loading Conditions Hydraulic Grade Line for the
Broadway Pump Plant Sta 24+76.9
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

714

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Mistletoe Pump Station Sta 37+07

Mistletoe Sta 37+07 rev 1.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station integral with floodwall
2. Floodwall raise proposed
3. L_2 not revised for minor change due to raise
4. Approx. 5 feet of trash fill exists landside, so reduce blanket thickness by 5 feet.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	760.65
500 year + 0':	759.00
500 year + 3':	762.00
500 year + 5':	764.00
Ground Surface Land Side:	745
Ground Surface River Side:	736
Bottom of Blanket:	722
Top of Bedrock:	670
Ground Surface at Pump Station:	755

Recommended Permeability Ratio
for Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Saturated Unit Weight of Blanket:	116 pcf
Unit Weight of Water:	62.4 pcf

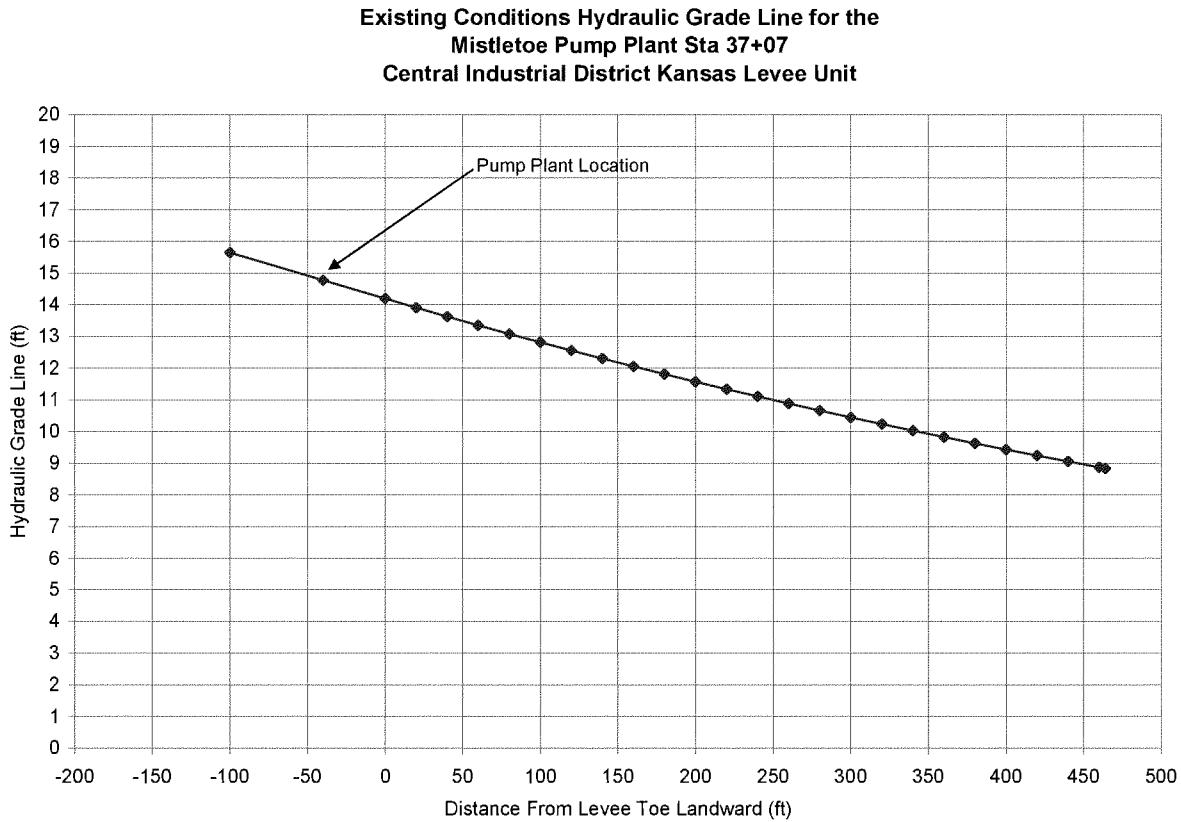
Station	Design Condition	Input Parameters: Permeability Ratio River Side (K_1/K_{bl})	Permeability Ratio Land Side (K_2/K_{bl})	Blanket Thickness (ft) River Side z_{bl}	Blanket Thickness (ft) Protection z_{bl}	Blanket Thickness (ft) Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft) River Side l_1	Seepage Length (ft) Protection l_2	Seepage Length (ft) Land Side l_3	Driving Head (ft) H
37+07	Existing Condition	800	800	14.0	18.5	23.0	52.0	0	100	1000	15.7
37+07	n500 + 0	800	800	14.0	18.5	23.0	52.0	0	100	1000	14.0
37+07	n500 +3	800	800	14.0	18.5	23.0	52.0	0	100	1000	17.0
37+07	n500 + 5	800	800	14.0	18.5	23.0	52.0	0	100	1000	19.0

Station	Design Condition	Analysis: Factors c_1	Factors c_2	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_2	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_0	Critical Hydraulic Gradient i_c	Gradient Factor of Safety i_c/i_0	Remarks
37+07	Existing Condition	0.001310	0.001022	0	978	14.2	0.62	0.84	1.37	
37+07	n500 + 0	0.001310	0.001022	0	978	12.7	0.55	0.84	1.53	
37+07	n500 +3	0.001310	0.001022	0	978	15.4	0.67	0.84	1.26	
37+07	n500 + 5	0.001310	0.001022	0	978	17.2	0.75	0.84	1.12	

716

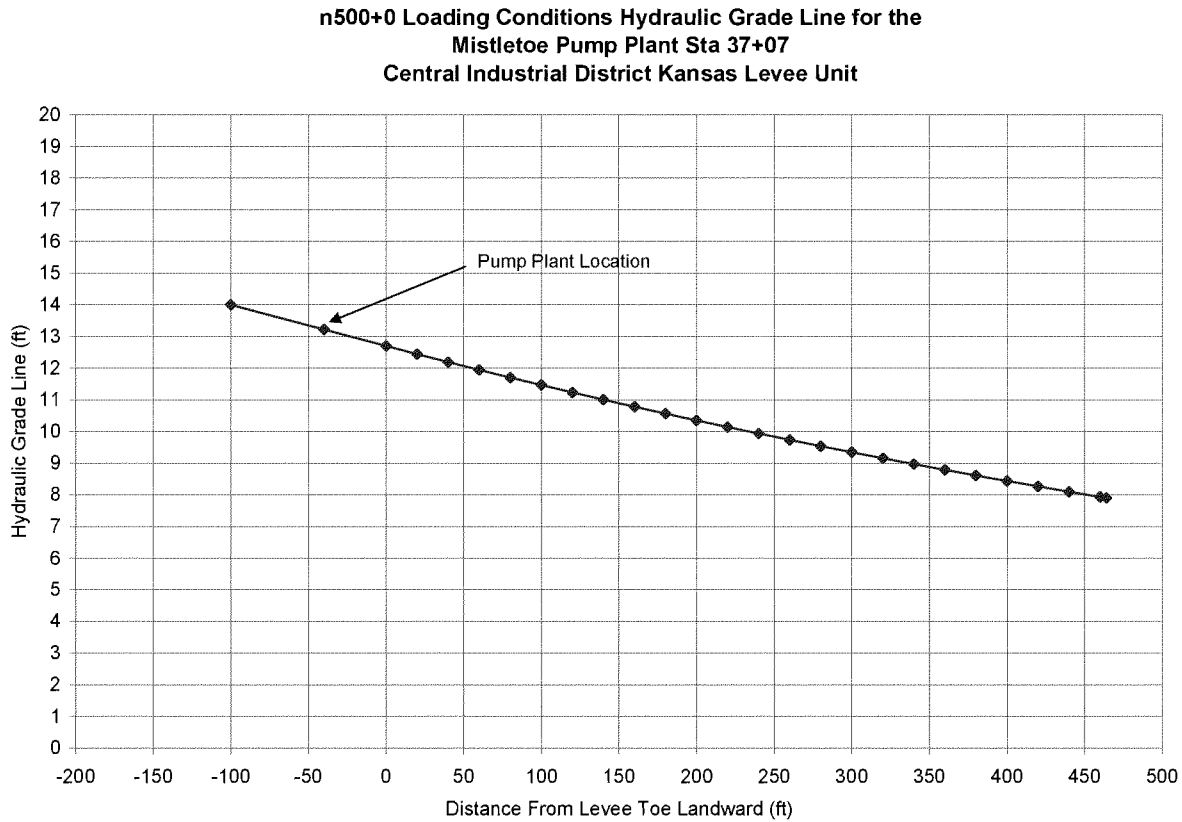
Hydraulic Grade Line for the Mistletoe Pump Plant
Existing Conditions

Elevation Top of Protection (MSL)	760.65	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.65		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	14.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	15.65	760.65
Pump Plant Location	-40	14.78	759.78
	0	14.20	759.20
	20	13.91	758.91
	40	13.63	758.63
	60	13.35	758.35
	80	13.08	758.08
	100	12.82	757.82
	120	12.56	757.56
	140	12.31	757.31
	160	12.06	757.06
	180	11.81	756.81
	200	11.57	756.57
	220	11.34	756.34
	240	11.11	756.11
	260	10.88	755.88
	280	10.66	755.66
	300	10.45	755.45
	320	10.24	755.24
	340	10.03	755.03
	360	9.83	754.83
	380	9.63	754.63
	400	9.43	754.43
	420	9.24	754.24
	440	9.05	754.05
	460	8.87	753.87
	464	8.84	753.84


Existing Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

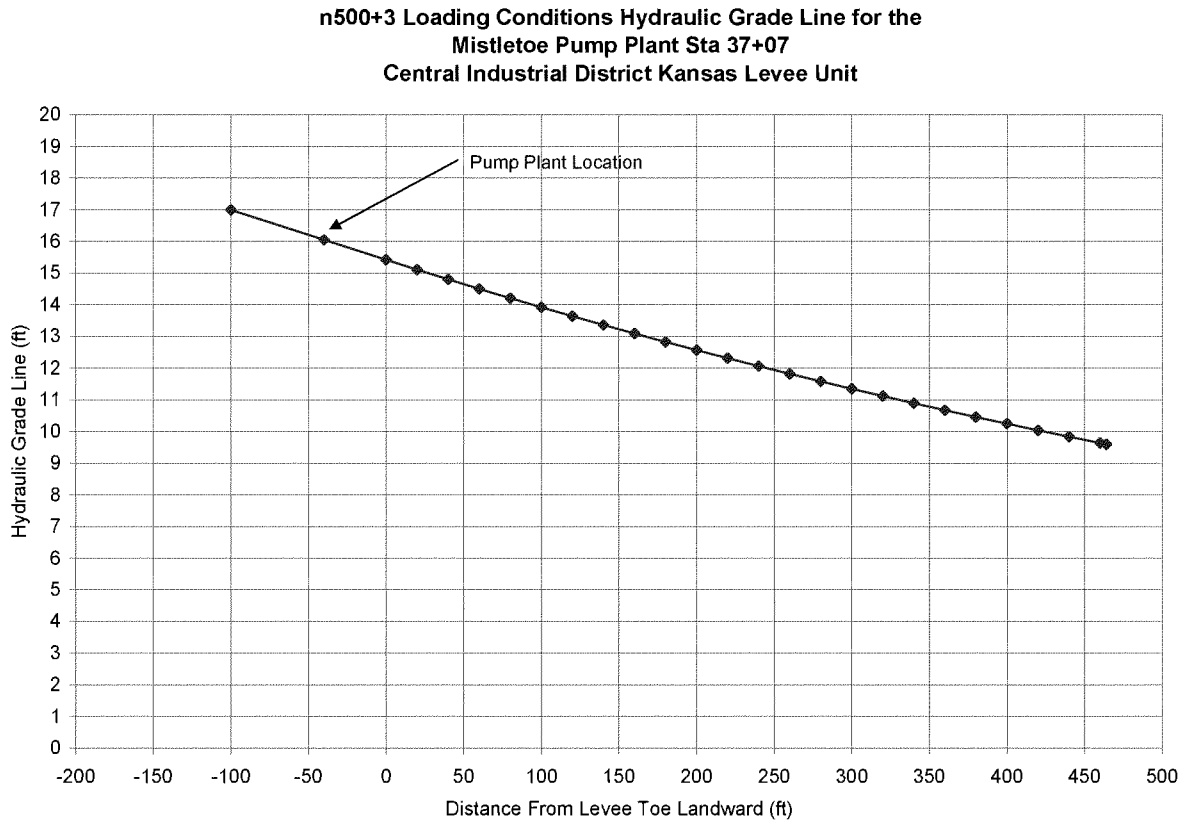
718

Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	759.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	14		
c_1	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	12.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	14.00	759.00
Pump Plant Location	-40	13.22	758.22
	0	12.70	757.70
	20	12.44	757.44
	40	12.19	757.19
	60	11.95	756.95
	80	11.70	756.70
	100	11.47	756.47
	120	11.24	756.24
	140	11.01	756.01
	160	10.78	755.78
	180	10.57	755.57
	200	10.35	755.35
	220	10.14	755.14
	240	9.94	754.94
	260	9.74	754.74
	280	9.54	754.54
	300	9.35	754.35
	320	9.16	754.16
	340	8.97	753.97
	360	8.79	753.79
	380	8.61	753.61
	400	8.44	753.44
	420	8.27	753.27
	440	8.10	753.10
	460	7.94	752.94
	464	7.90	752.90


n500+0 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

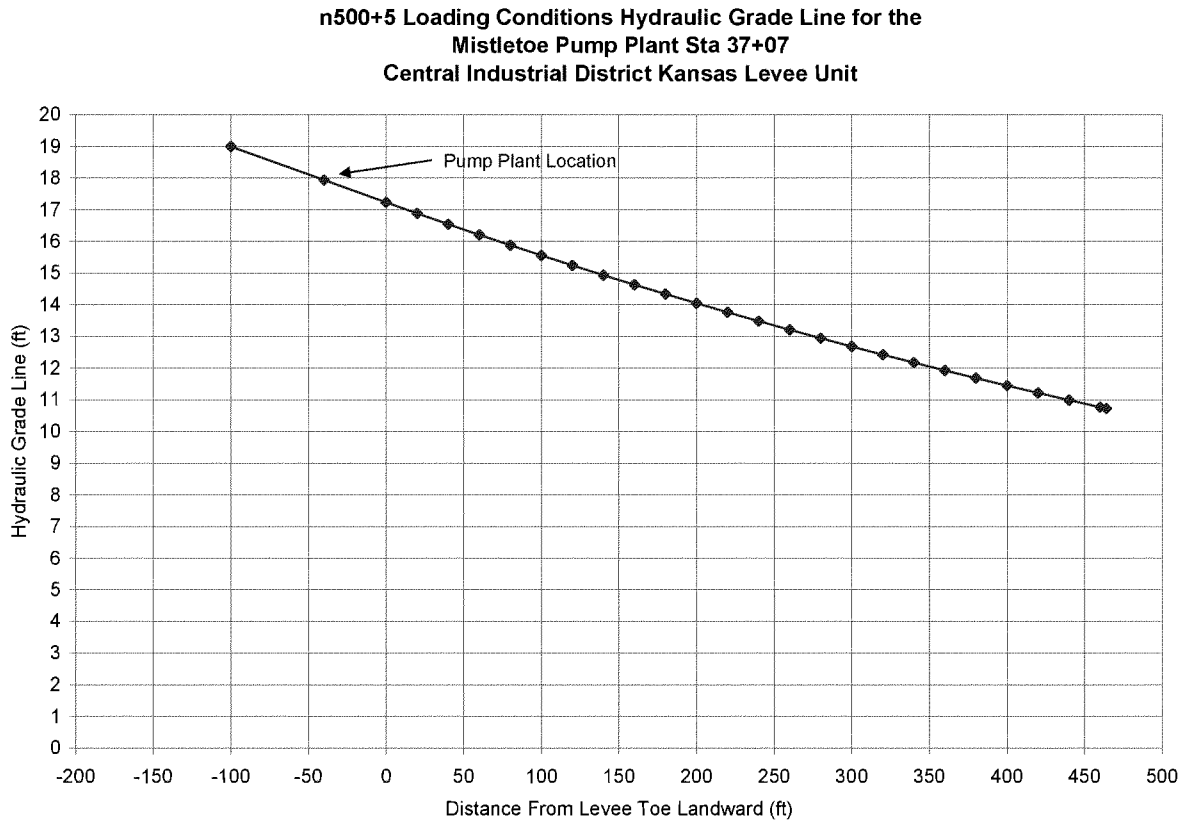
720

Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	762.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	17		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	15.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	17.00	762.00
Pump Plant Location	-40	16.05	761.05
	0	15.42	760.42
	20	15.11	760.11
	40	14.81	759.81
	60	14.51	759.51
	80	14.21	759.21
	100	13.92	758.92
	120	13.64	758.64
	140	13.37	758.37
	160	13.10	758.10
	180	12.83	757.83
	200	12.57	757.57
	220	12.32	757.32
	240	12.07	757.07
	260	11.82	756.82
	280	11.58	756.58
	300	11.35	756.35
	320	11.12	756.12
	340	10.89	755.89
	360	10.67	755.67
	380	10.46	755.46
	400	10.25	755.25
	420	10.04	755.04
	440	9.84	754.84
	460	9.64	754.64
	464	9.60	754.60


n500+3 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

722

Hydraulic Grade Line for the Mistletoe Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	764.00	Existing Conditions	
Elevation at Landside Toe (MSL)	745		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	19		
c_l	0.001022		
x_1 (ft)	0		
L_2 (ft)	100		
x_3 (ft)	978		
h_o	17.2		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-100	19.00	764.00
Pump Plant Location	-40	17.94	762.94
	0	17.24	762.24
	20	16.89	761.89
	40	16.55	761.55
	60	16.21	761.21
	80	15.88	760.88
	100	15.56	760.56
	120	15.25	760.25
	140	14.94	759.94
	160	14.64	759.64
	180	14.34	759.34
	200	14.05	759.05
	220	13.77	758.77
	240	13.49	758.49
	260	13.21	758.21
	280	12.95	757.95
	300	12.68	757.68
	320	12.43	757.43
	340	12.18	757.18
	360	11.93	756.93
	380	11.69	756.69
	400	11.45	756.45
	420	11.22	756.22
	440	10.99	755.99
	460	10.77	755.77
	464	10.73	755.73


n500+5 Loading Conditions Hydraulic Grade Line for the
Mistletoe Pump Plant Sta 37+07
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Santa Fe Pump Station Sta 52+86.7

Santa Fe Sta 52+86.7.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuzniakowski EC-4D
Date : 7/26/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 55 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_3 not revised for minor change due to raise
4. Blanket details are not well known, very heterogeneous, mostly sand

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	759.70
500 year + 0':	
500 year + 3':	Note: Ignore all but the existing condition calculations
500 year + 5':	
Ground Surface Land Side:	750
Ground Surface River Side:	748
Bottom of Blanket:	745 (Based upon limited available information, significant sand fill)
Top of Bedrock:	670 (assumed, based upon limited available information)
Ground Surface at Pump Station:	748

Recommended Permeability Ratio
For Foundation Blanket Materials
Chartie Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

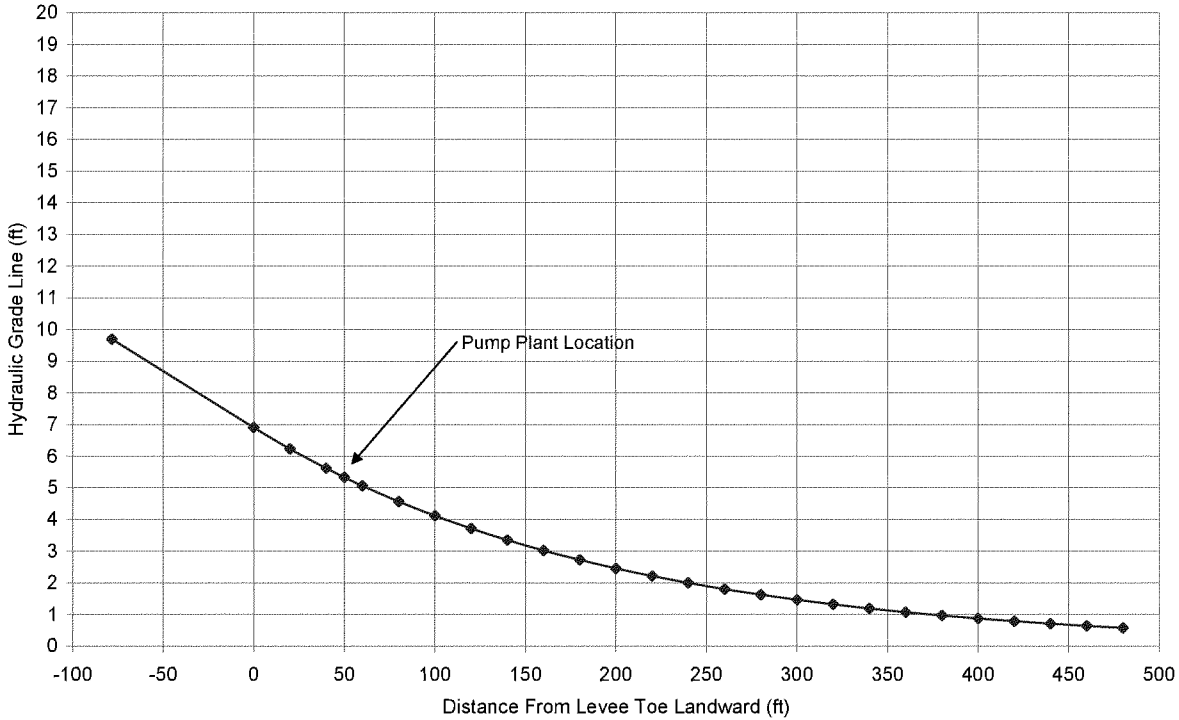
Station	Design Condition	Input Parameters: Permeability Ratio, River Side (K_f/K_{b1})	Permeability Ratio, Land Side (K_f/K_{b3})	Blanket Thickness (ft), River Side x_1	Blanket Thickness (ft), Protection x_m	Blanket Thickness (ft), Land Side x_3	Aquifer Thickness (ft) d	Seepage Length (ft), River Side L_1	Seepage Length (ft), Protection L_2	Seepage Length (ft), Land Side L_3	Driving Head (ft) H
52+86.7	Existing Condition	100	100	3.0	4.0	5.0	75.0	70	13	1000	9.7
52+86.7	n500 + 0	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0
52+86.7	n500 +3	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0
52+86.7	n500 + 5	100	100	3.0	4.0	5.0	75.0	70	13	1000	-750.0

Station	Design Condition	Analysis: Factors c_1	Factors c_3	Effective Seepage Length (ft), River Side x_1	Effective Seepage Length (ft), Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_0	Critical Hydraulic Gradient i_c	Gradient Factor of Safety i_c/i_0	Remarks
52+86.7	Existing Condition	0.006667	0.005164	65	194	6.9	1.38	0.84	0.61	
52+86.7	n500 + 0	0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	
52+86.7	n500 +3	0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	
52+86.7	n500 + 5	0.006667	0.005164	65	194	-534.0	-106.80	0.84	-0.01	

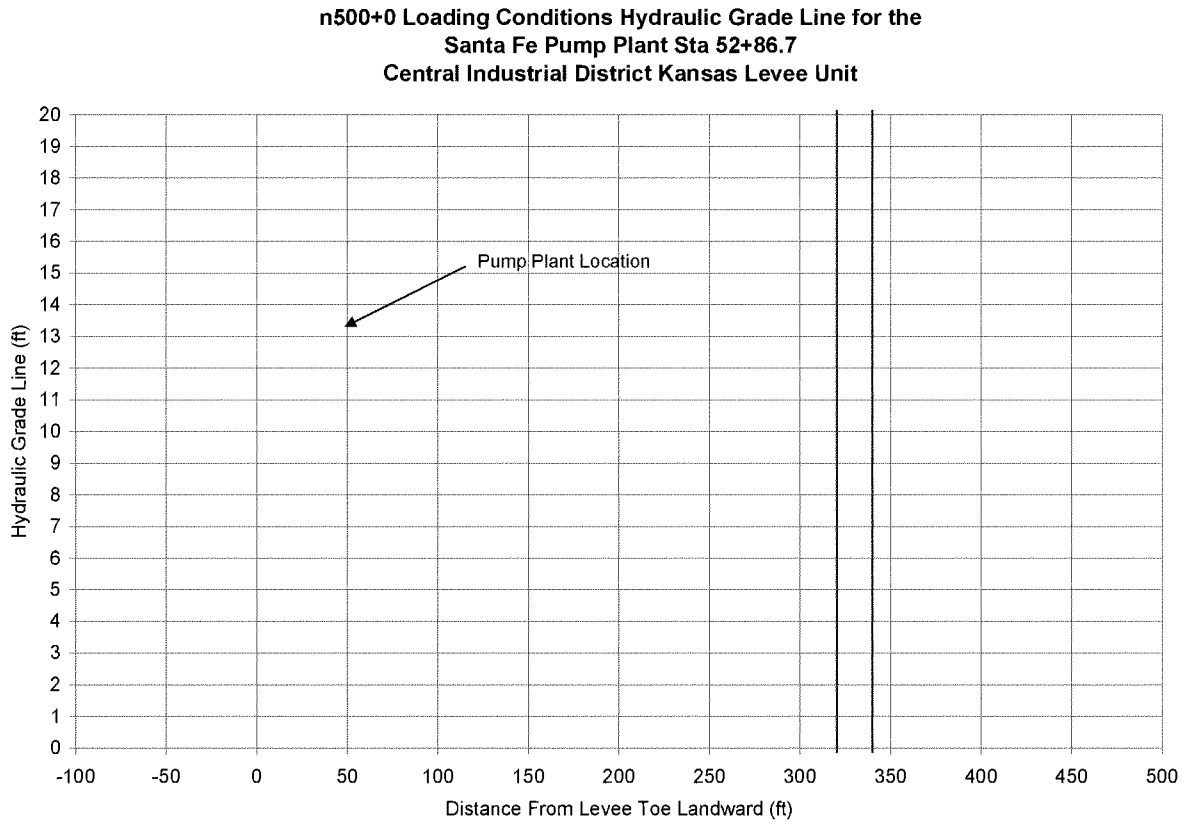
726

Hydraulic Grade Line for the Santa Fe Pump Plant Existing Conditions			
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	9.7		
c_1	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	6.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	9.70	759.70
	0	6.91	756.91
	20	6.23	756.23
	40	5.62	755.62
Pump Plant Location	50	5.33	755.33
	60	5.07	755.07
	80	4.57	754.57
	100	4.12	754.12
	120	3.72	753.72
	140	3.35	753.35
	160	3.02	753.02
	180	2.73	752.73
	200	2.46	752.46
	220	2.22	752.22
	240	2.00	752.00
	260	1.80	751.80
	280	1.63	751.63
	300	1.47	751.47
	320	1.32	751.32
	340	1.19	751.19
	360	1.08	751.08
	380	0.97	750.97
	400	0.88	750.88
	420	0.79	750.79
	440	0.71	750.71
	460	0.64	750.64
	480	0.58	750.58

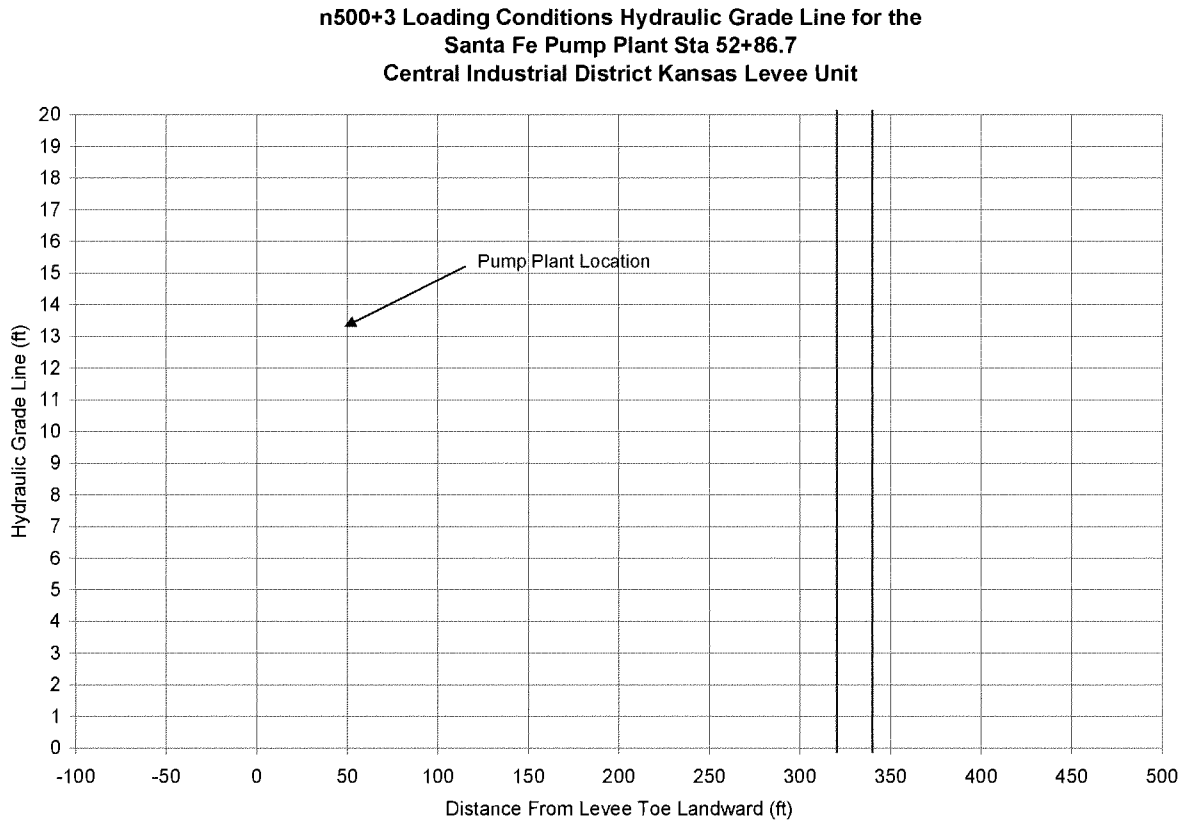
Existing Conditions Hydraulic Grade Line for the Santa Fe Pump Plant Sta 52+86.7 Central Industrial District Kansas Levee Unit



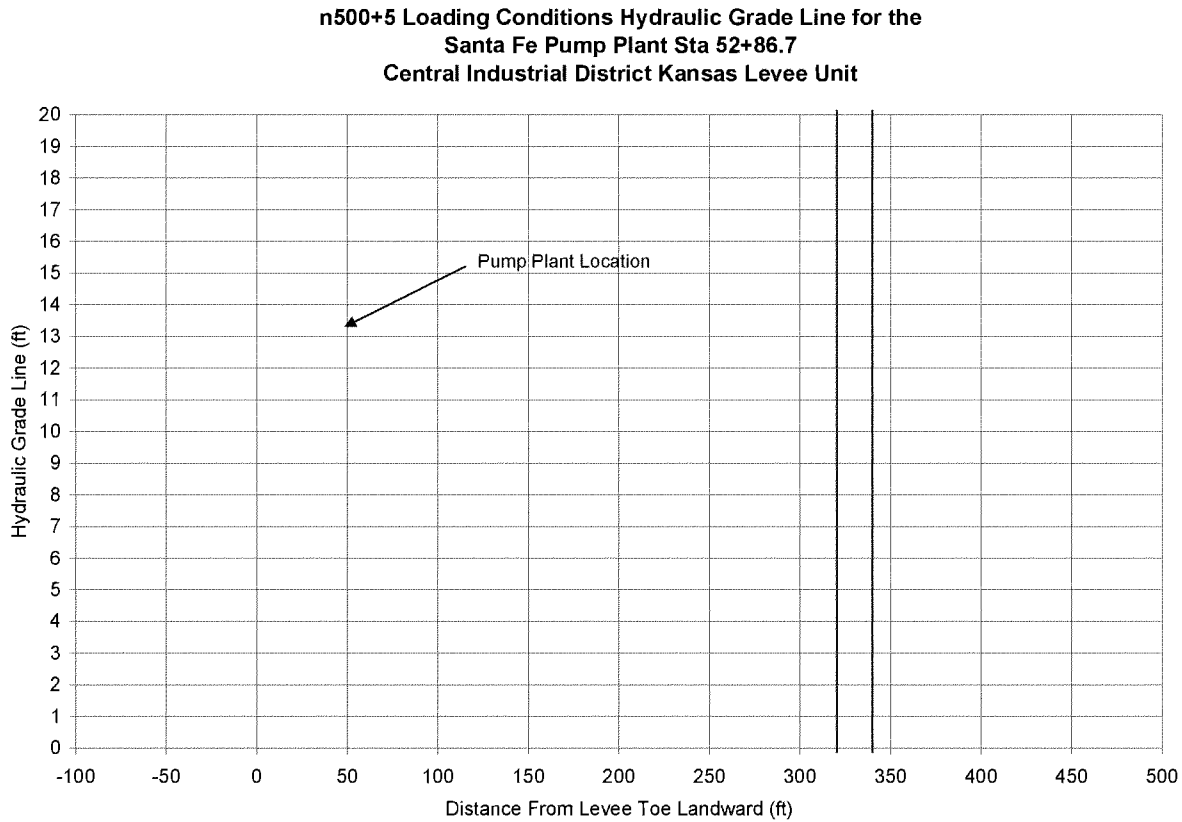
Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_1	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22


n500+0 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_i	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22


n500+3 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Hydraulic Grade Line for the Santa Fe Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	759.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	745		
Driving Head (ft)	-750		
c_1	0.005164		
x_1 (ft)	65		
L_2 (ft)	13		
x_3 (ft)	194		
h_o	-534.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-78	-750.00	0.00
	-40	-644.31	105.69
	0	-534.01	215.99
	20	-481.61	268.39
	40	-434.35	315.65
Pump Plant Location	50	-412.49	337.51
	60	-391.73	358.27
	80	-353.29	396.71
	100	-318.63	431.37
	120	-287.36	462.64
	140	-259.16	490.84
	160	-233.73	516.27
	180	-210.80	539.20
	200	-190.11	559.89
	220	-171.46	578.54
	240	-154.63	595.37
	260	-139.46	610.54
	280	-125.78	624.22
	300	-113.43	636.57
	320	-102.30	647.70
	340	-92.26	657.74
	360	-83.21	666.79
	380	-75.05	674.95
	400	-67.68	682.32
	420	-61.04	688.96
	440	-55.05	694.95
	460	-49.65	700.35
	480	-44.78	705.22


n500+5 Loading Conditions Hydraulic Grade Line for the
Santa Fe Pump Plant Sta 52+86.7
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the New Central Pump Station Sta 60+00

New Central Sta 60+00 rev 1.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station at Landward Toe of Levee
2. Raise proposal currently unknown
3. L_3 not revised for minor change due to raise
4. Approx. 10 feet of trash fill exists landside, so reduce blanket thickness by 5 feet.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	762.10
500 year + 0':	760.90
500 year + 3':	763.90
500 year + 5':	765.90
Ground Surface Land Side:	750 (Subtract 5' from ex. ds to deals tip. n.s. for H)
Ground Surface River Side:	738
Bottom of Blanket:	722
Top of Bedrock:	672
Ground Surface at Pump Station:	750

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Undersepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K_f/K_b)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

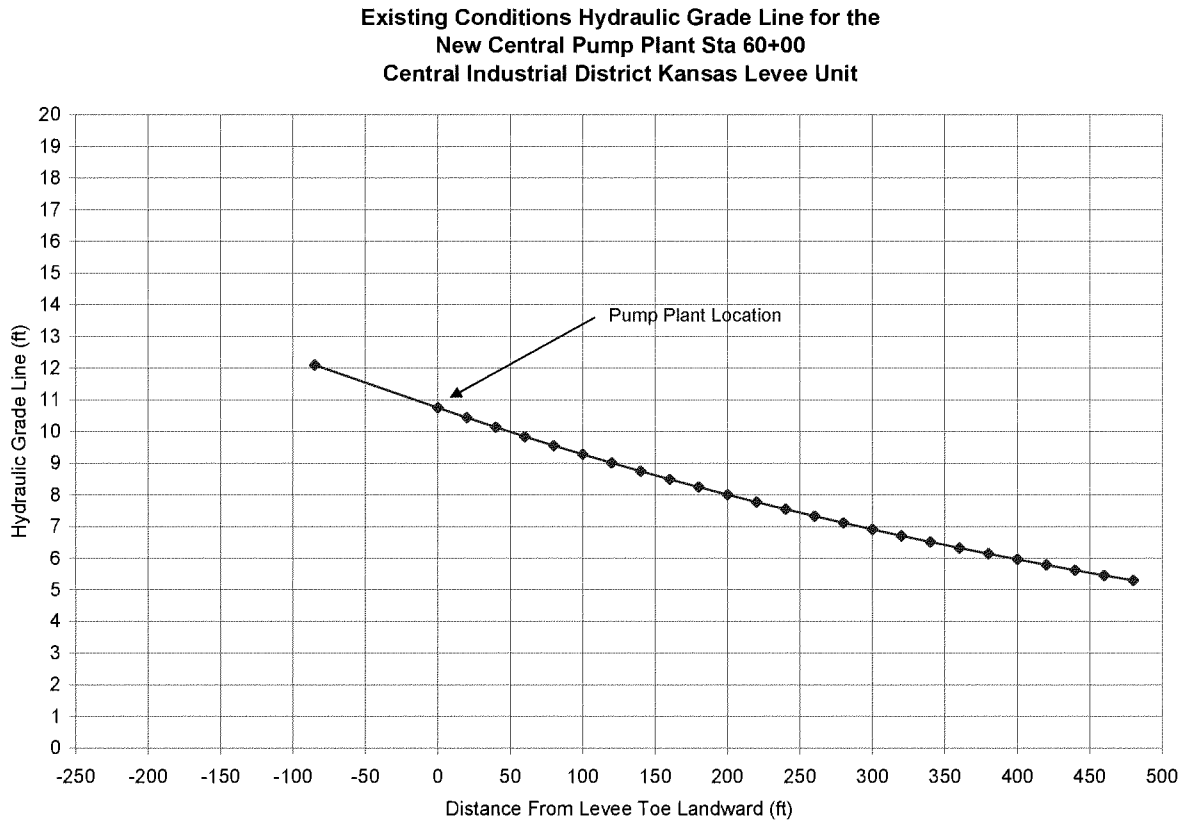
Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Station	Design Condition	Input Parameters: Permeability Ratio River Side (K_f/K_{bu})	Permeability Ratio Land Side (K_f/K_{bd})	Blanket Thickness (ft) River Side z_{bu}	Blanket Thickness (ft) Protection z_{bp}	Blanket Thickness (ft) Land Side z_{bd}	Aquifer Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Protection L_2	Seepage Length (ft) Land Side L_3	Driving Head (ft) H
58+12	Existing Condition	400	400	16.0	19.5	23.0	50.0	0	85	1000	12.1
58+12	n500 + 0	400	400	16.0	19.5	23.0	50.0	0	85	1000	10.9
58+12	n500 +3	400	400	16.0	19.5	23.0	50.0	0	85	1000	13.9
58+12	n500 + 5	400	400	16.0	19.5	23.0	50.0	0	85	1000	15.9

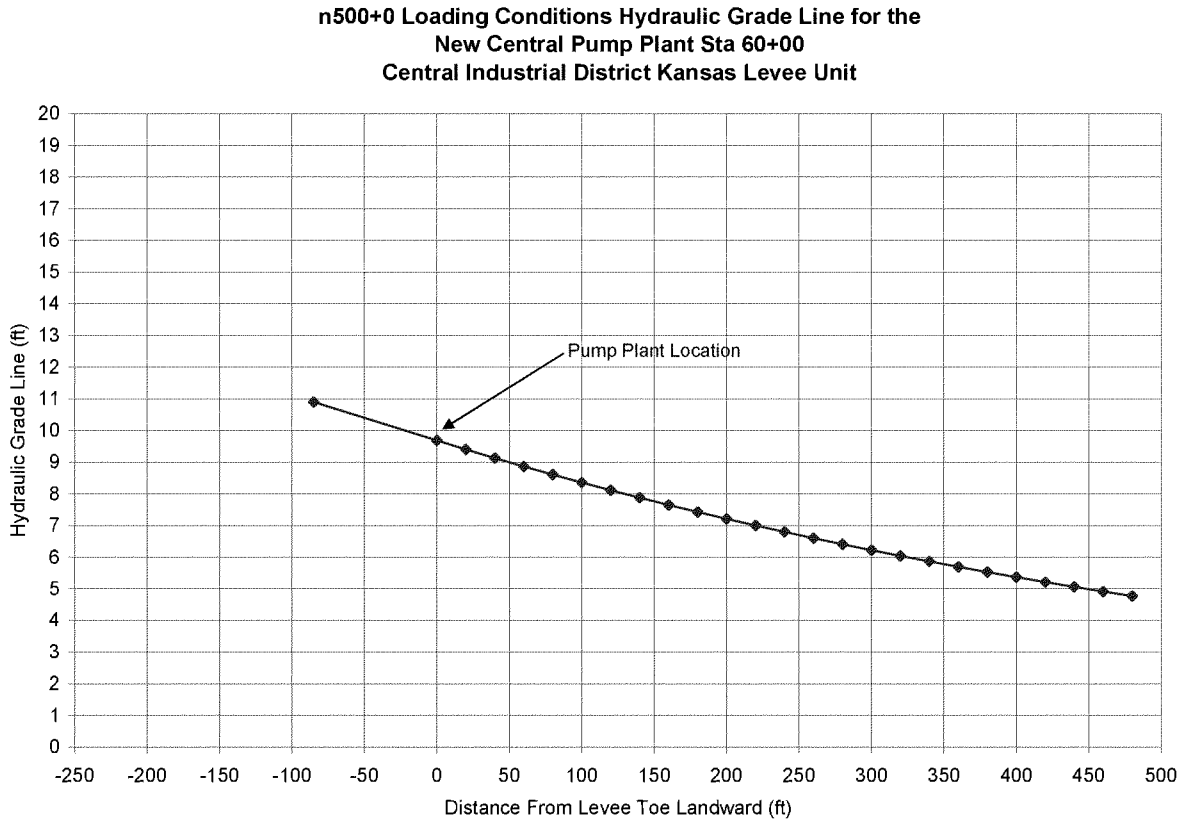
Analysis: Factors c_1	Factors c_2	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_2	Head at Toe (ft) h_x	Computed Hydraulic Gradient i_x	Critical Hydraulic Gradient i_c	Gradient Factor of Safety i_c/i_x	Remarks
0.001768	0.001474	0	678	10.8	0.47	0.84	1.80	
0.001768	0.001474	0	678	9.7	0.42	0.84	2.00	
0.001768	0.001474	0	678	12.4	0.54	0.84	1.57	
0.001768	0.001474	0	678	14.1	0.61	0.84	1.37	

736

Hydraulic Grade Line for the New Central Pump Plant Existing Conditions			
Elevation Top of Protection (MSL)	762.1	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	12.1		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	10.8		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	12.10	762.10
Pump Plant Location	0	10.75	760.75
	20	10.44	760.44
	40	10.14	760.14
	60	9.84	759.84
	80	9.56	759.56
	100	9.28	759.28
	120	9.01	759.01
	140	8.75	758.75
	160	8.49	758.49
	180	8.25	758.25
	200	8.01	758.01
	220	7.77	757.77
	240	7.55	757.55
	260	7.33	757.33
	280	7.12	757.12
	300	6.91	756.91
	320	6.71	756.71
	340	6.51	756.51
	360	6.32	756.32
	380	6.14	756.14
	400	5.96	755.96
	420	5.79	755.79
	440	5.62	755.62
	460	5.46	755.46
	480	5.30	755.30

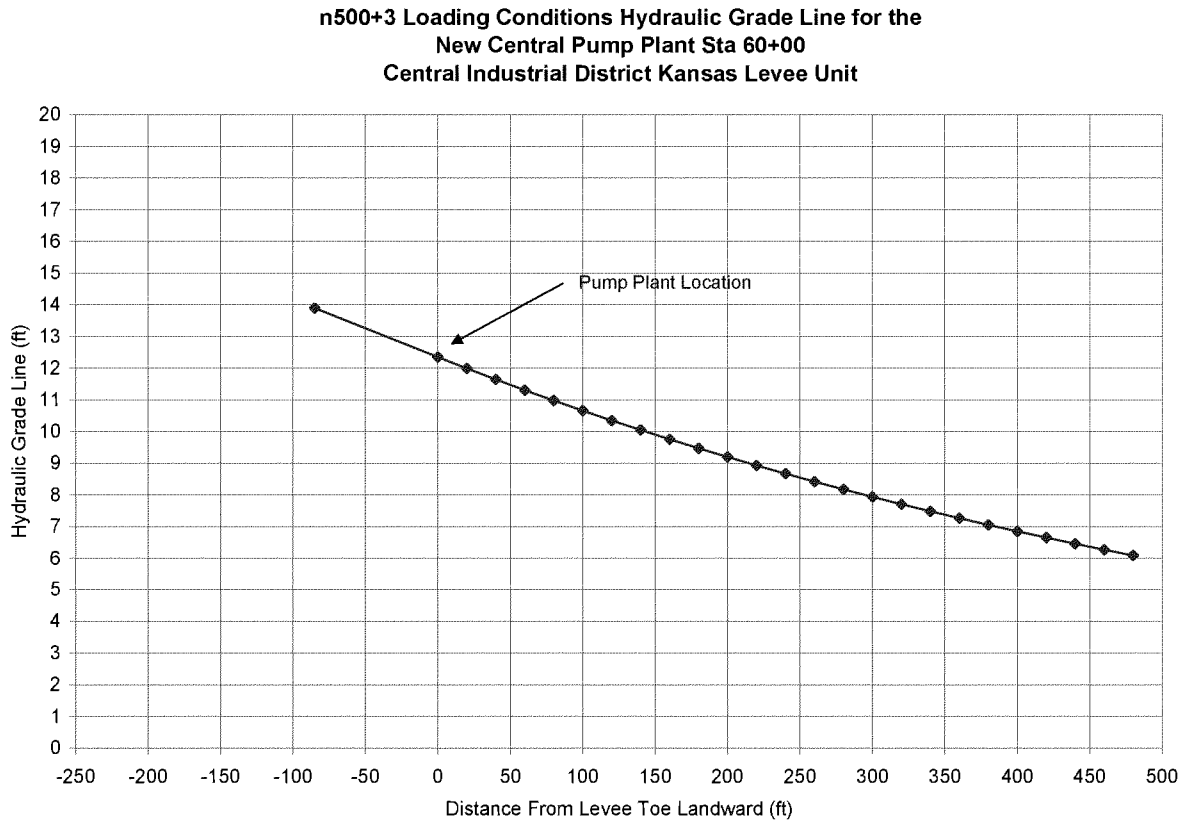

Existing Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Hydraulic Grade Line for the New Central Pump Plant n500+0 Loading Conditions			
Elevation Top of Protection (MSL)	760.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	10.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	9.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	10.90	760.90
Pump Plant Location	0	9.69	759.69
	20	9.40	759.40
	40	9.13	759.13
	60	8.87	758.87
	80	8.61	758.61
	100	8.36	758.36
	120	8.12	758.12
	140	7.88	757.88
	160	7.65	757.65
	180	7.43	757.43
	200	7.21	757.21
	220	7.00	757.00
	240	6.80	756.80
	260	6.60	756.60
	280	6.41	756.41
	300	6.22	756.22
	320	6.04	756.04
	340	5.87	755.87
	360	5.70	755.70
	380	5.53	755.53
	400	5.37	755.37
	420	5.21	755.21
	440	5.06	755.06
	460	4.92	754.92
	480	4.77	754.77


n500+0 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

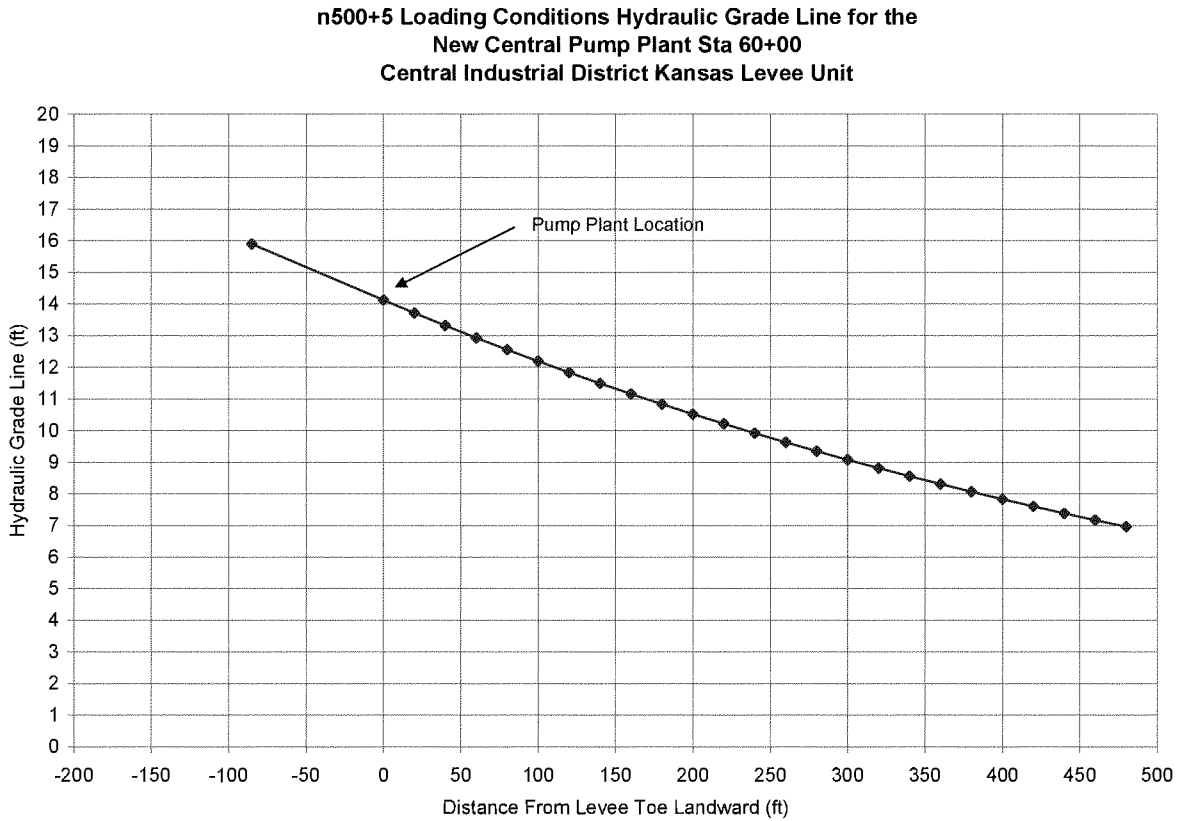
740

Hydraulic Grade Line for the New Central Pump Plant n500+3 Loading Conditions			
Elevation Top of Protection (MSL)	763.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	13.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	12.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	13.90	763.90
Pump Plant Location	0	12.35	762.35
	20	11.99	761.99
	40	11.64	761.64
	60	11.31	761.31
	80	10.98	760.98
	100	10.66	760.66
	120	10.35	760.35
	140	10.05	760.05
	160	9.76	759.76
	180	9.47	759.47
	200	9.20	759.20
	220	8.93	758.93
	240	8.67	758.67
	260	8.42	758.42
	280	8.17	758.17
	300	7.94	757.94
	320	7.71	757.71
	340	7.48	757.48
	360	7.26	757.26
	380	7.05	757.05
	400	6.85	756.85
	420	6.65	756.65
	440	6.46	756.46
	460	6.27	756.27
	480	6.09	756.09


n500+3 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

742

Hydraulic Grade Line for the New Central Pump Plant n500+5 Loading Conditions			
Elevation Top of Protection (MSL)	765.90	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.9		
c_l	0.001474		
x_1 (ft)	0		
L_2 (ft)	85		
x_3 (ft)	678		
h_o	14.1		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-85	15.90	765.90
Pump Plant Location	0	14.13	764.13
	20	13.72	763.72
	40	13.32	763.32
	60	12.93	762.93
	80	12.56	762.56
	100	12.19	762.19
	120	11.84	761.84
	140	11.49	761.49
	160	11.16	761.16
	180	10.84	760.84
	200	10.52	760.52
	220	10.22	760.22
	240	9.92	759.92
	260	9.63	759.63
	280	9.35	759.35
	300	9.08	759.08
	320	8.81	758.81
	340	8.56	758.56
	360	8.31	758.31
	380	8.07	758.07
	400	7.83	757.83
	420	7.61	757.61
	440	7.39	757.39
	460	7.17	757.17
	480	6.96	756.96


n500+5 Loading Conditions Hydraulic Grade Line for the
New Central Pump Plant Sta 60+00
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

744

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Stockyards No. 3 Pump Station Sta 74+22

Stockyards No 3 Sta 74+22 rev 1.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station at Landside Toe of Levee
2. Raise proposal currently unknown
3. L_3 not revised for minor change due to raise

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1988

Blanket Material	Assumed Permeability Ratio, (Kf/Kb)
SM	100
ML	700 to 400
ML-CL	400
CL	400 to 600
CH	600 to 1000

Elevations Used (From O&M Manual, except for n500 values):

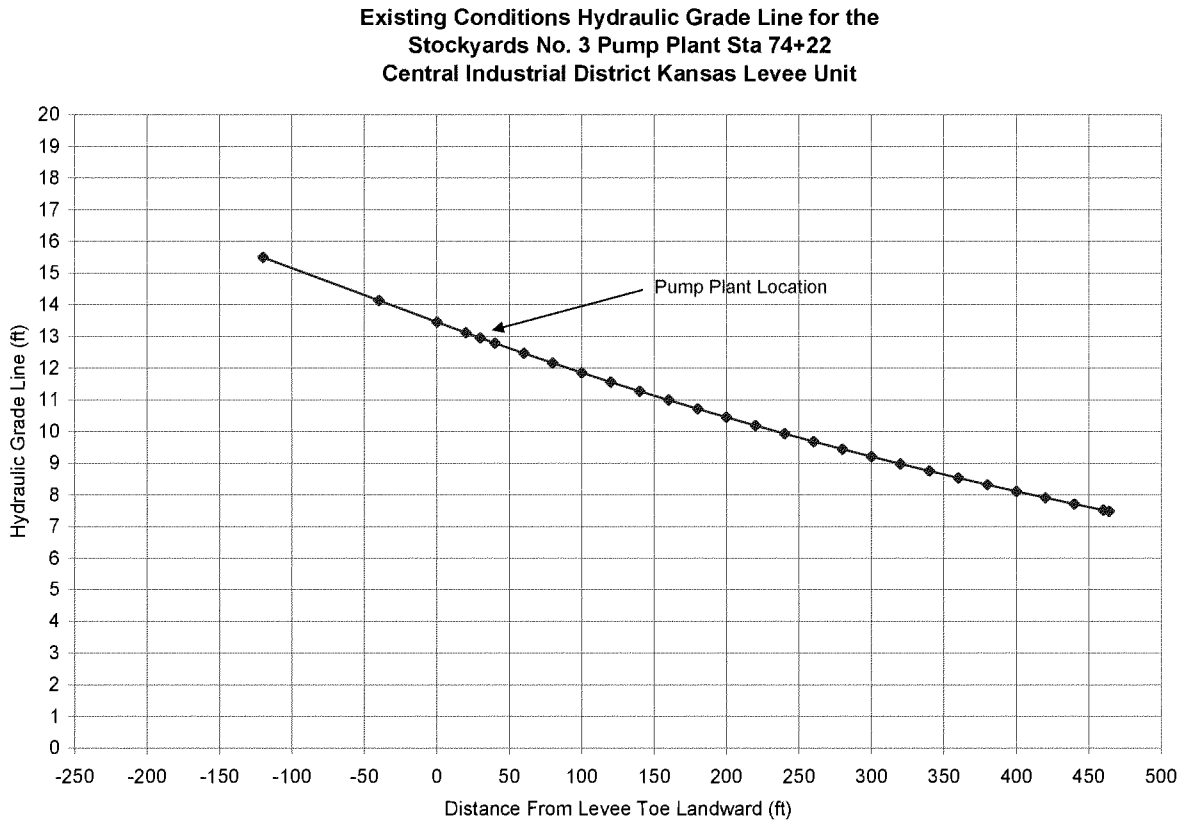
Top of Existing Protection:	762.50
500 year + 0':	762.50
500 year + 3':	765.50
500 year + 5':	767.50
Ground Surface Land Side:	747
Ground Surface River Side:	738
Bottom of Blanket:	722
Top of Bedrock:	672
Ground Surface at Pump Station:	742

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

Station	Design Condition	Input Parameters: Permeability Ratio, River Side (K_b/K_{bo})	Permeability Ratio, Land Side (K_b/K_{bd})	Blanket Thickness (ft), River Side x_{bc}	Blanket Thickness (ft), Protection z_{bo}	Blanket Thickness (ft), Land Side z_{bd}	Aquifer Thickness (ft) d	Seepage Length (ft), River Side L_1	Seepage Length (ft), Protection L_2	Seepage Length (ft), Land Side L_3	Driving Head (ft) H
74+22	Existing Condition	500	500	16.0	20.5	25.0	50.0	0	120	1000	15.5
74+22	n500 + 0	500	500	16.0	20.5	25.0	50.0	0	120	1000	15.3
74+22	n500 +3	500	500	16.0	20.5	25.0	50.0	0	120	1000	18.3
74+22	n500 + 5	500	500	16.0	20.5	25.0	50.0	0	120	1000	20.3

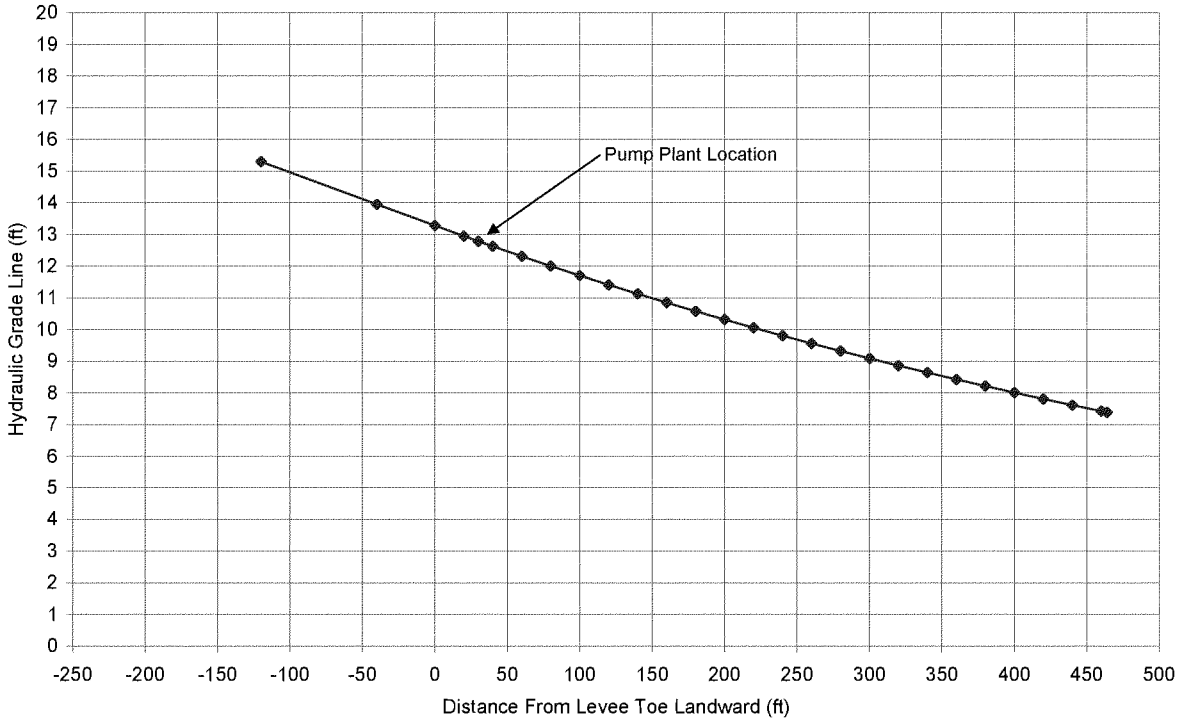
Station	Design Condition	Analysis: Factors c_1	Factors c_2	Effective Seepage Length (ft), River Side x_1	Effective Seepage Length (ft), Land Side x_2	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Gradient Factor of Safety i_c/i_o	Remarks
74+22	Existing Condition	0.001581	0.001265	0	791	13.5	0.54	0.84	1.57	
74+22	n500 + 0	0.001581	0.001265	0	791	13.3	0.53	0.84	1.59	
74+22	n500 +3	0.001581	0.001265	0	791	15.9	0.64	0.84	1.33	
74+22	n500 + 5	0.001581	0.001265	0	791	17.6	0.70	0.84	1.20	

Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Existing Conditions			
Elevation Top of Protection (MSL)	762.5	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.5		
c_1	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	13.5		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	15.50	762.50
	-40	14.14	761.14
	0	13.46	760.46
	20	13.12	760.12
Pump Plant Location	30	12.96	759.96
	40	12.79	759.79
	60	12.47	759.47
	80	12.16	759.16
	100	11.86	758.86
	120	11.56	758.56
	140	11.27	758.27
	160	10.99	757.99
	180	10.72	757.72
	200	10.45	757.45
	220	10.19	757.19
	240	9.93	756.93
	260	9.69	756.69
	280	9.44	756.44
	300	9.21	756.21
	320	8.98	755.98
	340	8.75	755.75
	360	8.53	755.53
	380	8.32	755.32
	400	8.11	755.11
	420	7.91	754.91
	440	7.71	754.71
	460	7.52	754.52
	464	7.48	754.48


Existing Conditions Hydraulic Grade Line for the
Stockyards No. 3 Pump Plant Sta 74+22
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

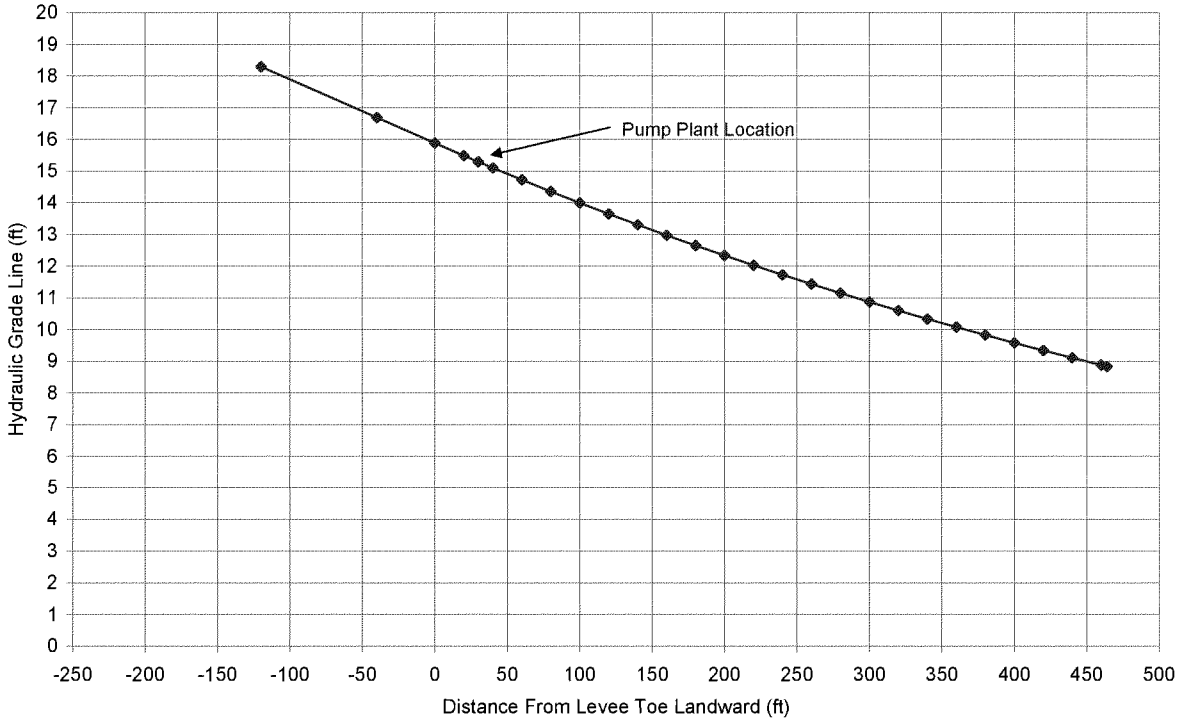
Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+0 Loading Conditions			
Elevation Top of Protection (MSL)	762.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	15.3		
c_i	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	13.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	15.30	762.30
	-40	13.96	760.96
	0	13.28	760.28
	20	12.95	759.95
Pump Plant Location	30	12.79	759.79
	40	12.63	759.63
	60	12.31	759.31
	80	12.01	759.01
	100	11.71	758.71
	120	11.41	758.41
	140	11.13	758.13
	160	10.85	757.85
	180	10.58	757.58
	200	10.31	757.31
	220	10.06	757.06
	240	9.81	756.81
	260	9.56	756.56
	280	9.32	756.32
	300	9.09	756.09
	320	8.86	755.86
	340	8.64	755.64
	360	8.42	755.42
	380	8.21	755.21
	400	8.01	755.01
	420	7.81	754.81
	440	7.61	754.61
	460	7.42	754.42
	464	7.39	754.39

n500+0 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



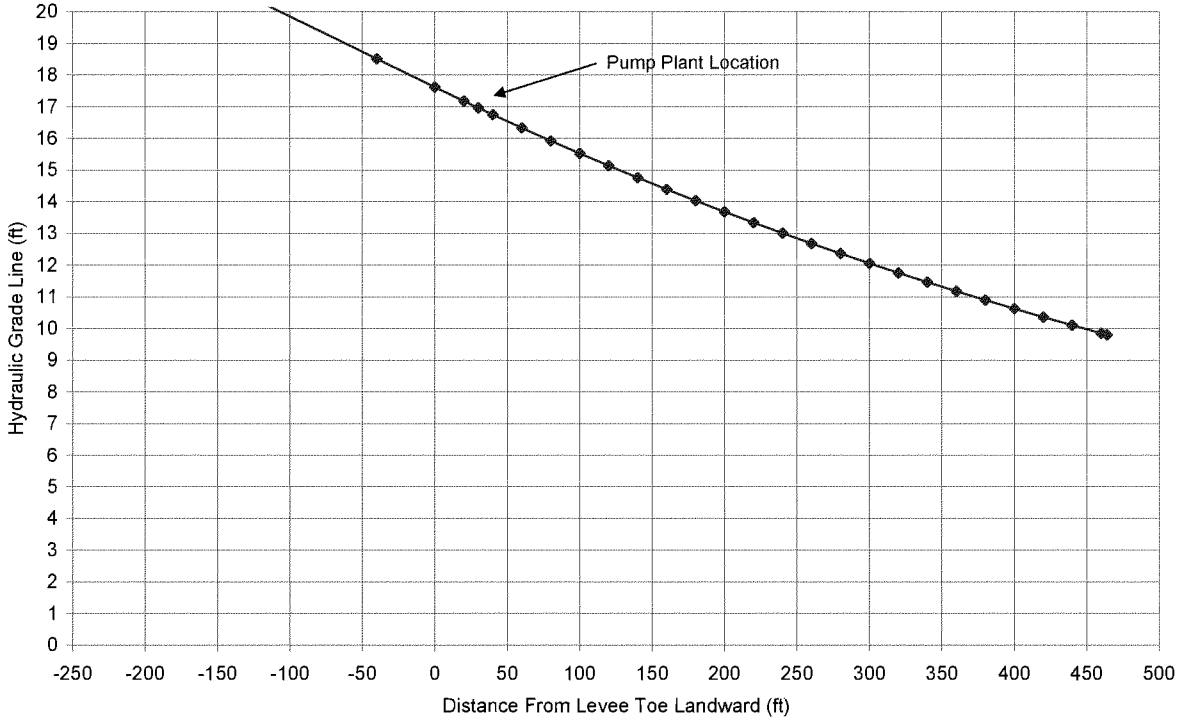
Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+3 Loading Conditions			
Elevation Top of Protection (MSL)	765.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	18.3		
c_i	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	15.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	18.30	765.30
	-40	16.69	763.69
	0	15.89	762.89
	20	15.49	762.49
Pump Plant Location	30	15.30	762.30
	40	15.10	762.10
	60	14.73	761.73
	80	14.36	761.36
	100	14.00	761.00
	120	13.65	760.65
	140	13.31	760.31
	160	12.98	759.98
	180	12.65	759.65
	200	12.34	759.34
	220	12.03	759.03
	240	11.73	758.73
	260	11.44	758.44
	280	11.15	758.15
	300	10.87	757.87
	320	10.60	757.60
	340	10.33	757.33
	360	10.08	757.08
	380	9.82	756.82
	400	9.58	756.58
	420	9.34	756.34
	440	9.11	756.11
	460	8.88	755.88
	464	8.83	755.83

n500+3 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Stockyards No. 3 Pump Plant n500+5 Loading Conditions			
Elevation Top of Protection (MSL)	767.30	Existing Conditions	
Elevation at Landside Toe (MSL)	747		
Elevation Bottom of Blanket (MSL)	722		
Driving Head (ft)	20.3		
c_l	0.001265		
x_1 (ft)	0		
L_2 (ft)	120		
x_3 (ft)	791		
h_o	17.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-120	20.30	767.30
	-40	18.52	765.52
	0	17.62	764.62
	20	17.18	764.18
Pump Plant Location	30	16.97	763.97
	40	16.76	763.76
	60	16.34	763.34
	80	15.93	762.93
	100	15.53	762.53
	120	15.14	762.14
	140	14.76	761.76
	160	14.40	761.40
	180	14.04	761.04
	200	13.69	760.69
	220	13.34	760.34
	240	13.01	760.01
	260	12.69	759.69
	280	12.37	759.37
	300	12.06	759.06
	320	11.76	758.76
	340	11.46	758.46
	360	11.18	758.18
	380	10.90	757.90
	400	10.63	757.63
	420	10.36	757.36
	440	10.10	757.10
	460	9.85	756.85
	464	9.80	756.80

n500+5 Loading Conditions Hydraulic Grade Line for the Stockyards No. 3 Pump Plant Sta 74+22 Central Industrial District Kansas Levee Unit



754

Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Gateway Station Sta 80+90

Gateway Sta 80+90 rev 1.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

NOTES:
1. Existing condition: Pump Station on Landside Crest of Levee Section
2. Raise proposed currently unknown
3. L_2 not revised for minor change due to raise
4. Floodwalls Exist along this reach. This analysis does NOT include drawdown effects of wells.

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	762.50
500 year + 0':	762.90
500 year + 3':	765.90
500 year + 5':	767.90
Ground Surface Land Side:	742
Ground Surface River Side:	738
Bottom of Blanket:	716
Top of Bedrock:	656
Ground Surface at Pump Station:	760

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

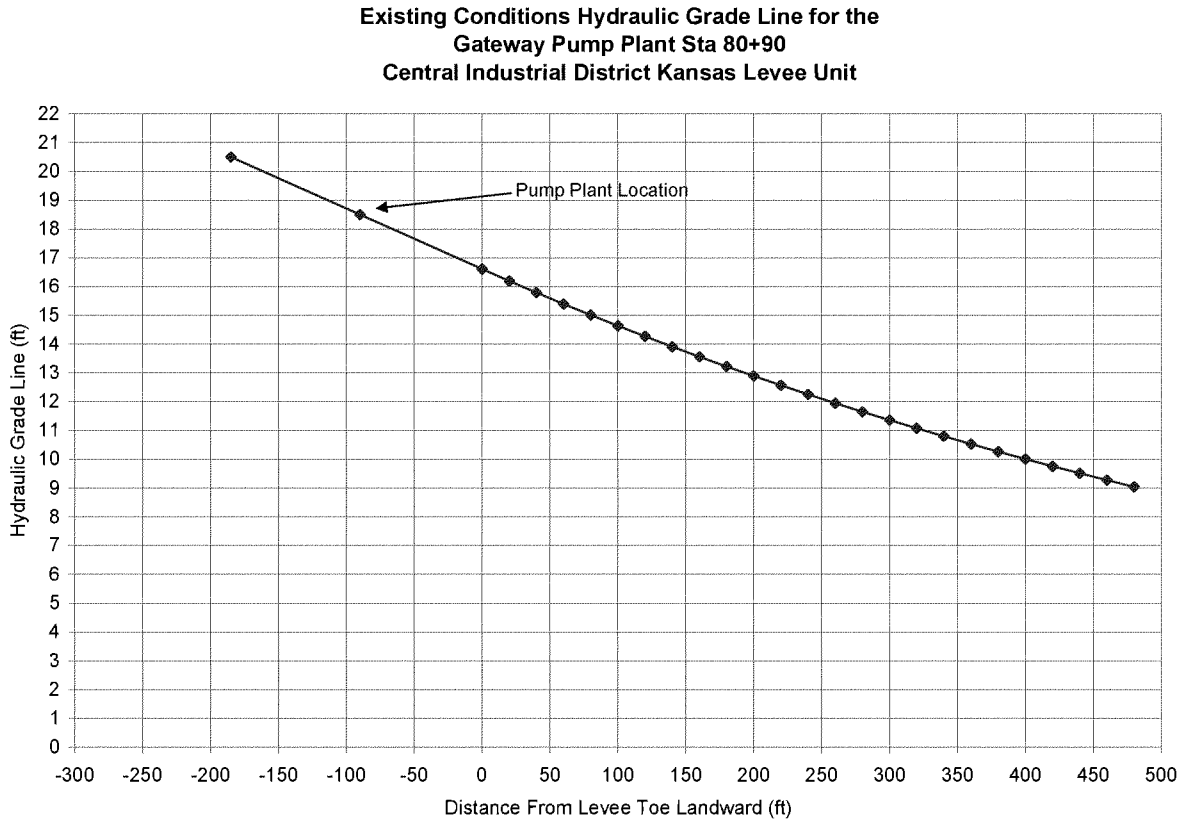
Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Saturated Unit Weight of Blanket: 115 pcf
Unit Weight of Water: 62.4 pcf

Station	Design Condition	Input Parameters: Permeability Ratio River Side (K_b/K_{bo})	Permeability Ratio Land Side (K_b/K_{bl})	Blanket Thickness (ft) River Side z_{bo}	Blanket Thickness (ft) Protection z_{bo}	Blanket Thickness (ft) Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Protection L_2	Seepage Length (ft) Land Side L_3	Driving Head (ft) H
80+90	Existing Condition	400	400	22.0	24.0	26.0	60.0	0	185	1000	20.5
80+90	n500 + 0	400	400	22.0	24.0	26.0	60.0	0	185	1000	20.9
80+90	n500 +3	400	400	22.0	24.0	26.0	60.0	0	185	1000	23.9
80+90	n500 + 5	400	400	22.0	24.0	26.0	60.0	0	185	1000	25.9

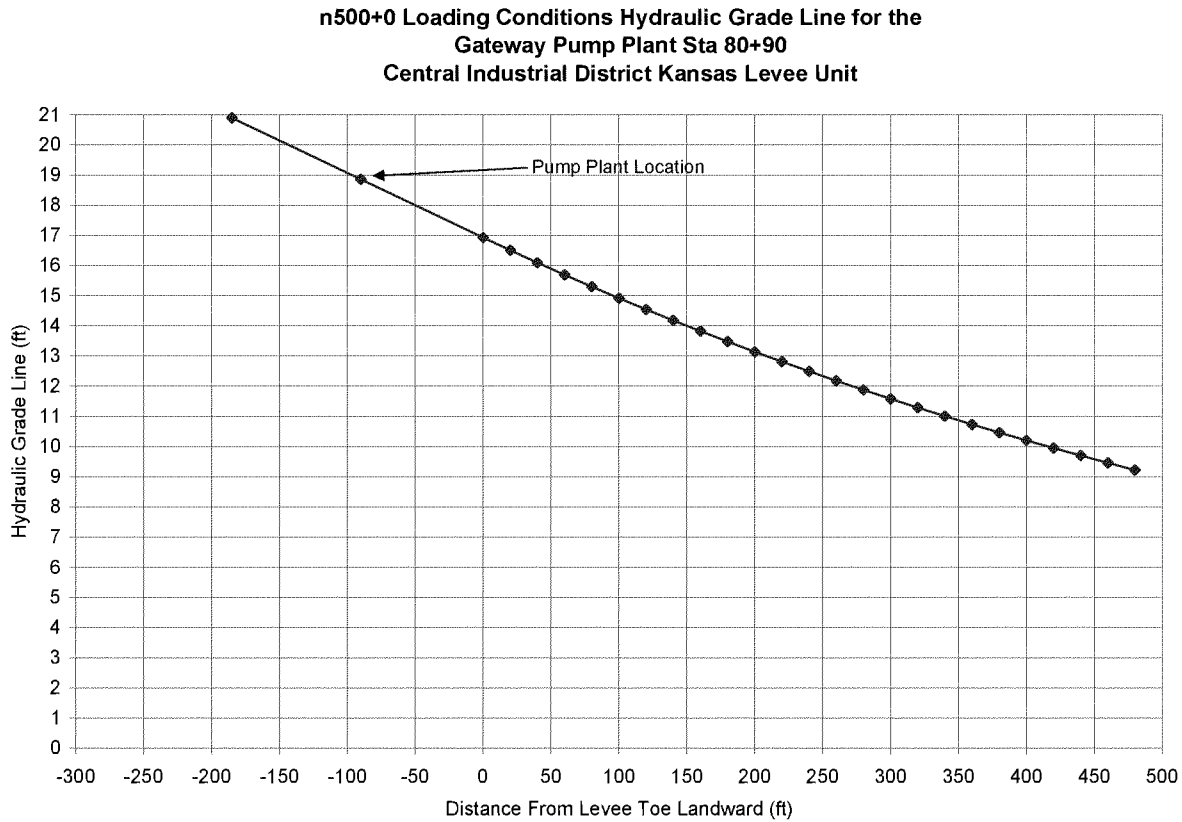
Station	Design Condition	Analysis: Factors c_f	Factors c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_g	Critical Hydraulic Gradient i_c	Gradient Factor of Safety tc/to	Remarks
80+90	Existing Condition	0.001376	0.001266	0	790	16.6	0.64	0.84	1.32	
80+90	n500 + 0	0.001376	0.001266	0	790	16.9	0.65	0.84	1.29	
80+90	n500 +3	0.001376	0.001266	0	790	19.4	0.74	0.84	1.13	
80+90	n500 + 5	0.001376	0.001266	0	790	21.0	0.81	0.84	1.04	

Hydraulic Grade Line for the Gateway Pump Plant			Existing Conditions
Elevation Top of Protection (MSL)	762.5	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	20.5		
c_1	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	16.6		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	20.50	762.50
Pump Plant Location	-90	18.50	760.50
	0	16.61	758.61
	20	16.19	758.19
	40	15.79	757.79
	60	15.40	757.40
	80	15.01	757.01
	100	14.63	756.63
	120	14.27	756.27
	140	13.91	755.91
	160	13.56	755.56
	180	13.23	755.23
	200	12.89	754.89
	220	12.57	754.57
	240	12.26	754.26
	260	11.95	753.95
	280	11.65	753.65
	300	11.36	753.36
	320	11.08	753.08
	340	10.80	752.80
	360	10.53	752.53
	380	10.27	752.27
	400	10.01	752.01
	420	9.76	751.76
	440	9.52	751.52
	460	9.28	751.28
	480	9.05	751.05


Existing Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

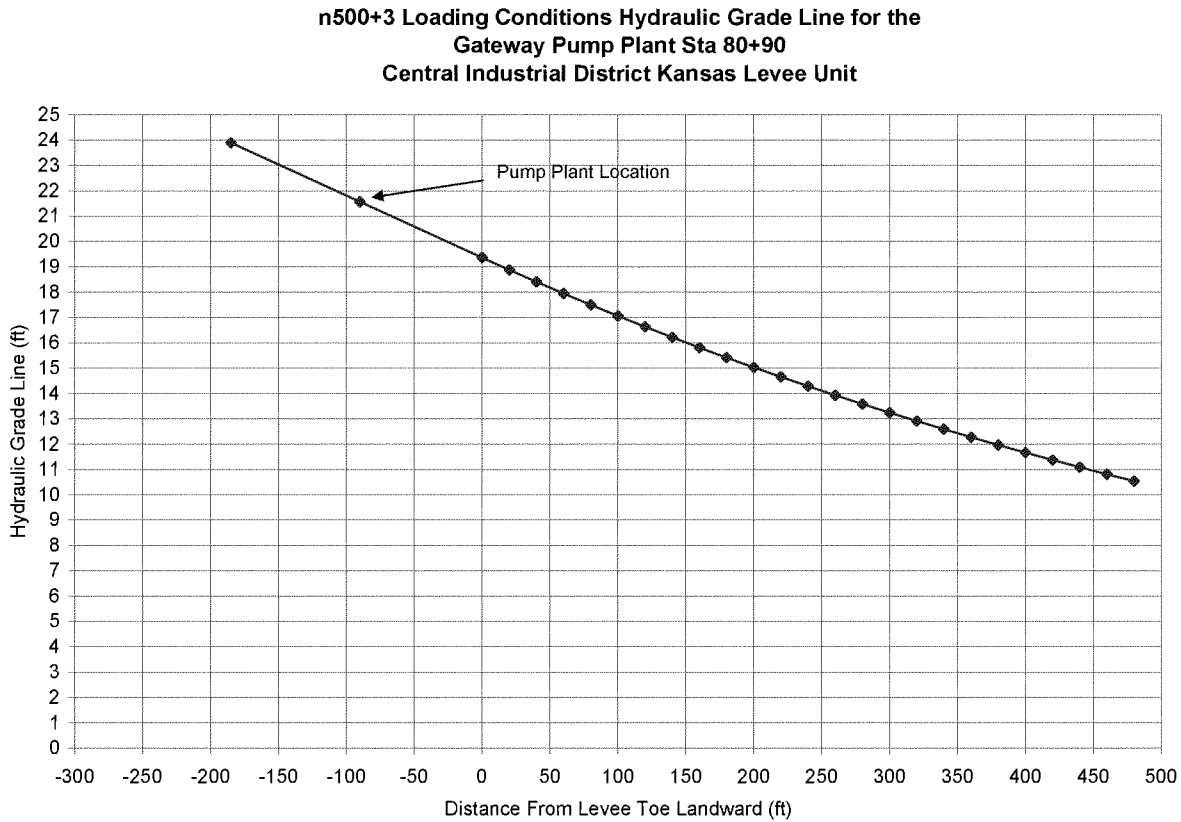
758

Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+0
Elevation Top of Protection (MSL)	762.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	20.9		
c_t	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	16.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	20.90	762.90
Pump Plant Location	-90	18.86	760.86
	0	16.93	758.93
	20	16.51	758.51
	40	16.10	758.10
	60	15.70	757.70
	80	15.30	757.30
	100	14.92	756.92
	120	14.55	756.55
	140	14.18	756.18
	160	13.83	755.83
	180	13.48	755.48
	200	13.15	755.15
	220	12.82	754.82
	240	12.50	754.50
	260	12.18	754.18
	280	11.88	753.88
	300	11.58	753.58
	320	11.29	753.29
	340	11.01	753.01
	360	10.74	752.74
	380	10.47	752.47
	400	10.21	752.21
	420	9.95	751.95
	440	9.70	751.70
	460	9.46	751.46
	480	9.22	751.22


n500+0 Loading Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

760

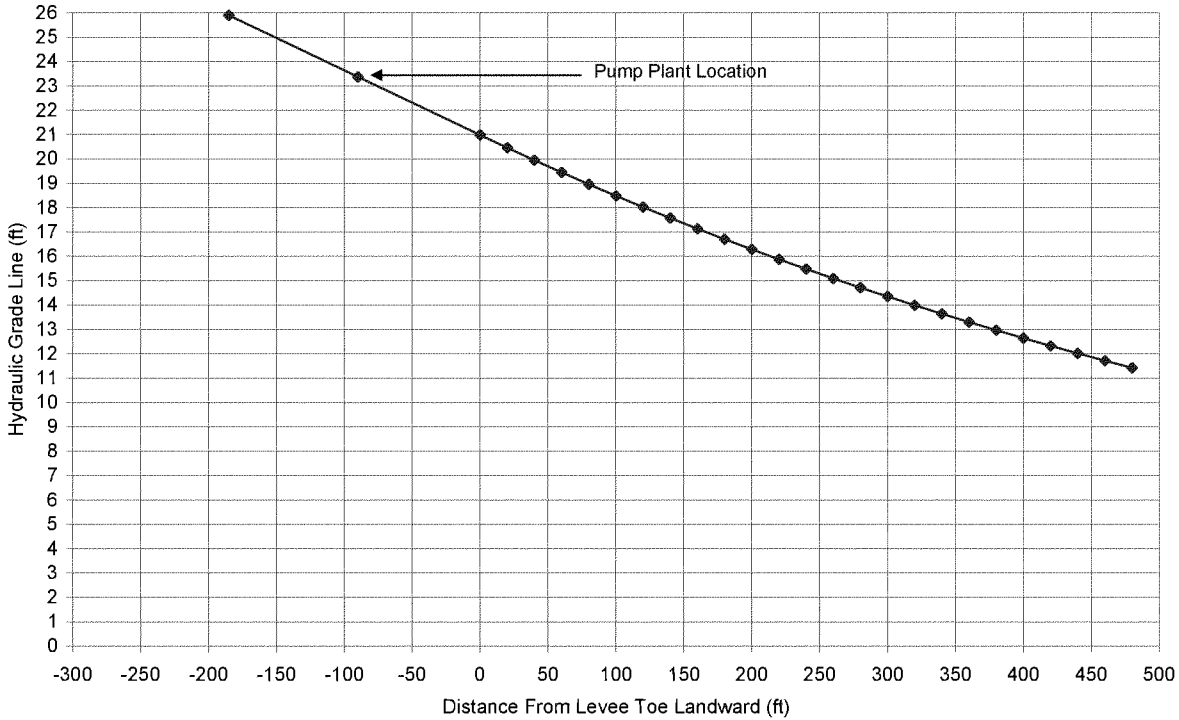
Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+3
Elevation Top of Protection (MSL)	765.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	23.9		
c_1	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	19.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	23.90	765.90
Pump Plant Location	-90	21.57	763.57
	0	19.36	761.36
	20	18.88	760.88
	40	18.41	760.41
	60	17.95	759.95
	80	17.50	759.50
	100	17.06	759.06
	120	16.64	758.64
	140	16.22	758.22
	160	15.81	757.81
	180	15.42	757.42
	200	15.03	757.03
	220	14.66	756.66
	240	14.29	756.29
	260	13.93	755.93
	280	13.59	755.59
	300	13.25	755.25
	320	12.91	754.91
	340	12.59	754.59
	360	12.28	754.28
	380	11.97	753.97
	400	11.67	753.67
	420	11.38	753.38
	440	11.09	753.09
	460	10.82	752.82
	480	10.55	752.55


n500+3 Loading Conditions Hydraulic Grade Line for the
Gateway Pump Plant Sta 80+90
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
350
400
450
500

762

Hydraulic Grade Line for the Gateway Pump Plant Loading Conditions			n500+5
Elevation Top of Protection (MSL)	767.90	Existing Conditions	
Elevation at Landside Toe (MSL)	742		
Elevation Bottom of Blanket (MSL)	716		
Driving Head (ft)	25.9		
c_l	0.001266		
x_1 (ft)	0		
L_2 (ft)	185		
x_3 (ft)	790		
h_o	21.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-185	25.90	767.90
Pump Plant Location	-90	23.38	765.38
	0	20.99	762.99
	20	20.46	762.46
	40	19.95	761.95
	60	19.45	761.45
	80	18.96	760.96
	100	18.49	760.49
	120	18.03	760.03
	140	17.58	759.58
	160	17.14	759.14
	180	16.71	758.71
	200	16.29	758.29
	220	15.88	757.88
	240	15.49	757.49
	260	15.10	757.10
	280	14.72	756.72
	300	14.35	756.35
	320	14.00	756.00
	340	13.65	755.65
	360	13.30	755.30
	380	12.97	754.97
	400	12.65	754.65
	420	12.33	754.33
	440	12.02	754.02
	460	11.72	753.72
	480	11.43	753.43

n500+5 Loading Conditions Hydraulic Grade Line for the Gateway Pump Plant Sta 80+90 Central Industrial District Kansas Levee Unit



Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Kansas - Hydraulic Grade Line for the Ohio Avenue Pump Station Sta 83+52 (CID-MO Stationing)

Ohio Sta 83+52.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

PDT
Name : Kuzniakowski EC-GD
Date : 6/19/2007

Peer Review
Checked by : EC-GD
Date :

NOTES:
1. Existing condition: Pump Station on Landside of Levee
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise
4. Shallow buried collector along landside toe of levee. Not expected to have much effect on hgl

Elevations Used(From O&M Manual, except for n500 values):

Top of Existing Protection:	760.55
500 year + 0':	
500 year + 3':	
500 year + 5':	
Ground Surface Land Side:	754
Ground Surface River Side:	752
Bottom of Blanket:	725
Top of Bedrock:	670
Ground Surface at Pump Station:	754

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	800 to 1000

Saturated Unit Weight of Blanket:	116 pcf
Unit Weight of Water:	62.4 pcf

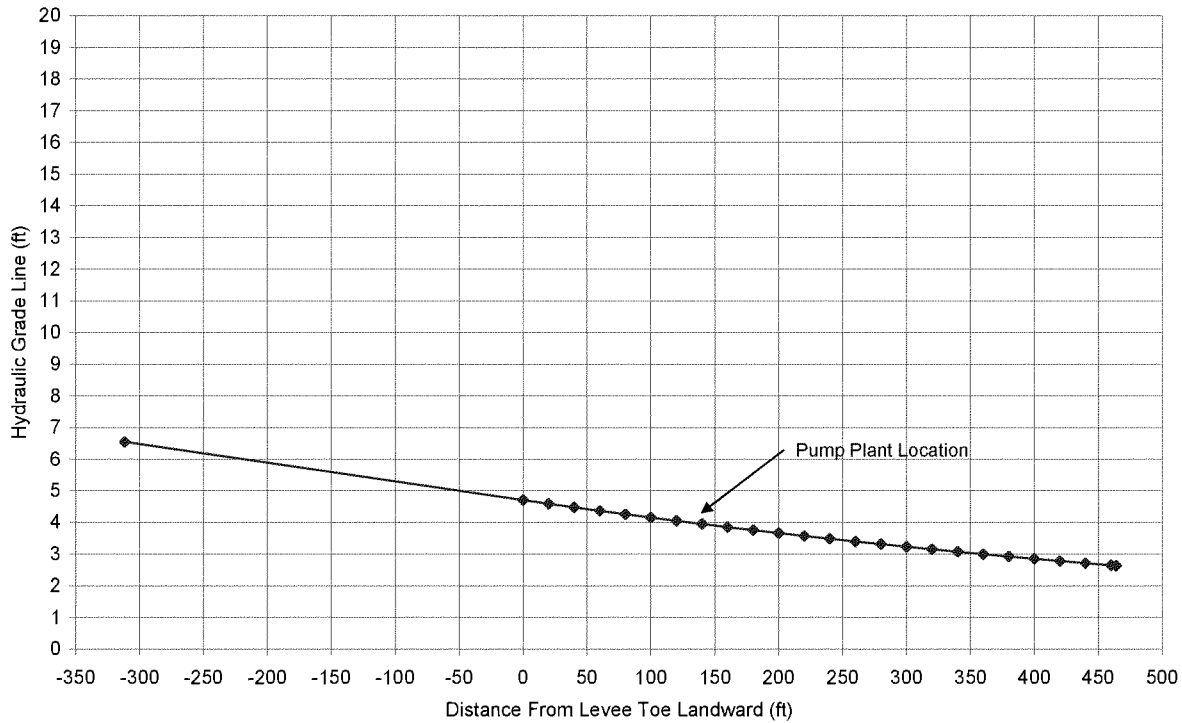
Station	Design Condition	Input Parameters: Permeability Ratio River Side (K_x/K_{bx})	Permeability Ratio Land Side (K_x/K_{bx})	Blanket Thickness (ft) River Side z_{bx}	Blanket Thickness (ft) Protection z_{bp}	Blanket Thickness (ft) Land Side z_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Protection L_2	Seepage Length (ft) Land Side L_3	Driving Head (ft) H
83+52	Existing Condition	400	400	27.0	28.0	29.0	55.0	250	70	1000	6.5
83+52	n500 + 0	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0
83+52	n500 +3	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0
83+52	n500 + 5	400	400	27.0	28.0	29.0	55.0	250	70	1000	-754.0

Analysis: Factors ε_1	Factors ε_2	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_x	Computed Hydraulic Gradient i_x	Critical Hydraulic Gradient i_c	Gradient Factor of Safety i_c/i_x	Remarks
0.001297	0.001252	242	799	4.7	0.16	0.84	5.19	
0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	
0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	
0.001297	0.001252	242	799	-542.4	-18.70	0.84	-0.05	

Hydraulic Grade Line for the Ohio Avenue Pump Plant
Existing Conditions

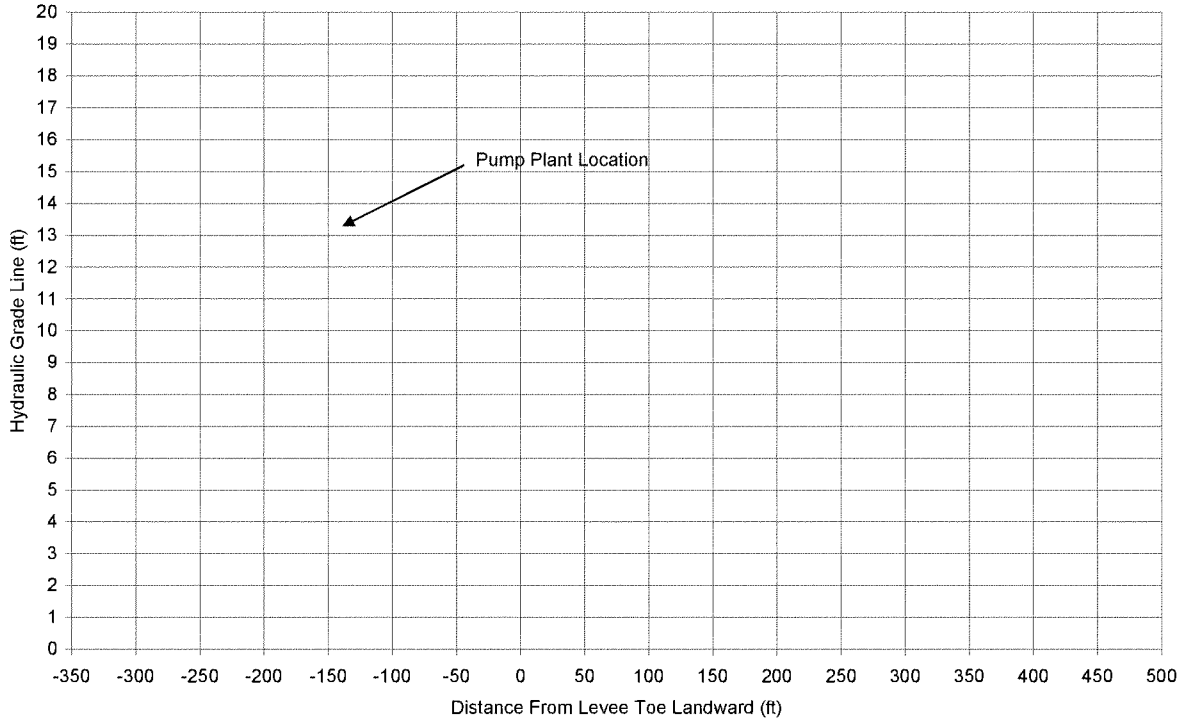
Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	6.55		
c_i	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	4.7		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	6.55	760.55
	0	4.71	758.71
	20	4.60	758.60
	40	4.48	758.48
	60	4.37	758.37
	80	4.26	758.26
	100	4.16	758.16
	120	4.05	758.05
Pump Plant Location	140	3.95	757.95
	160	3.86	757.86
	180	3.76	757.76
	200	3.67	757.67
	220	3.58	757.58
	240	3.49	757.49
	260	3.40	757.40
	280	3.32	757.32
	300	3.24	757.24
	320	3.16	757.16
	340	3.08	757.08
	360	3.00	757.00
	380	2.93	756.93
	400	2.86	756.86
	420	2.79	756.79
	440	2.72	756.72
	460	2.65	756.65
	464	2.64	756.64

Existing Conditions Hydraulic Grade Line for the
Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing)
Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Ohio Avenue Pump Plant n500+0 Loading Conditions			
Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_1	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	-40	-569.57	184.43
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

n500+0 Loading Conditions Hydraulic Grade Line for the Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing) Central Industrial District Kansas Levee Unit



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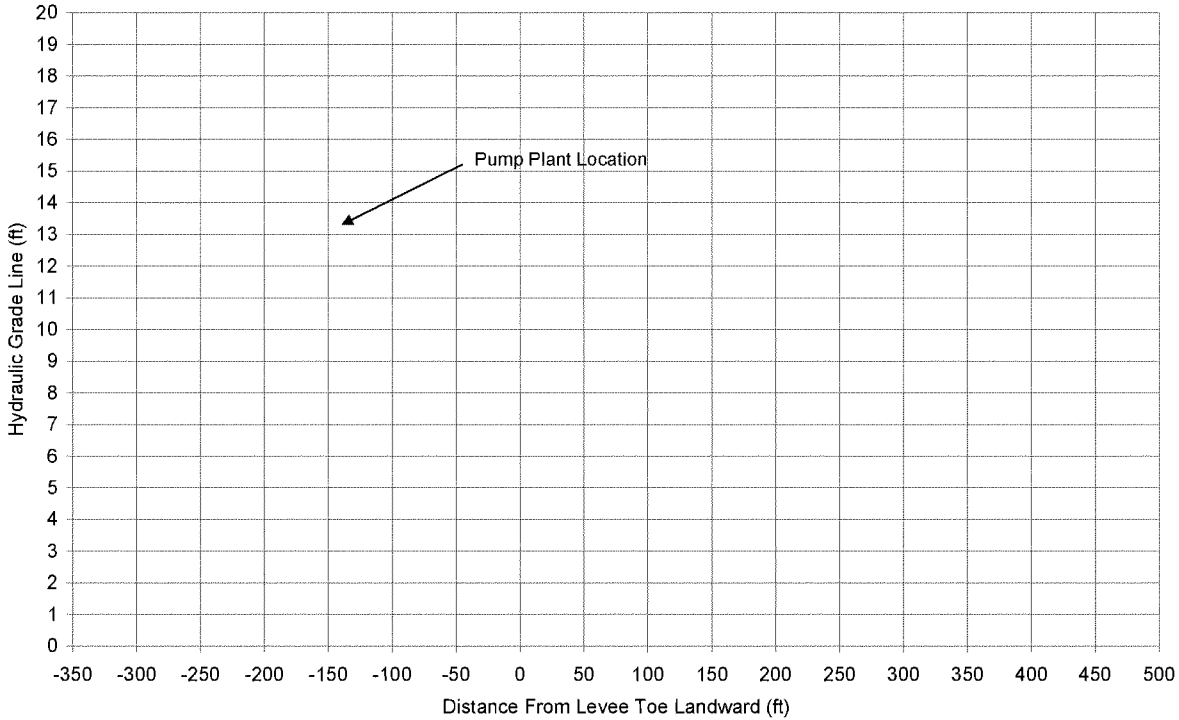
Hydraulic Grade Line for the Ohio Avenue Pump Plant n500+3 Loading Conditions

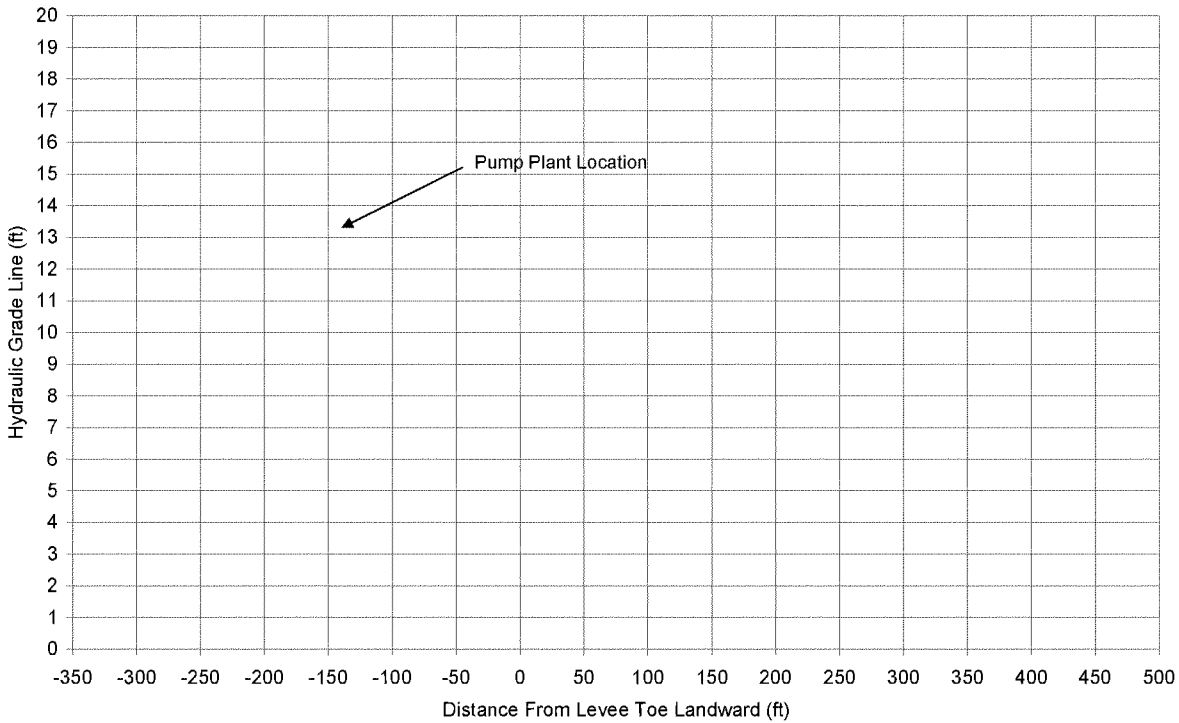
Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_1	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	-40	-569.57	184.43
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

n500+3 Loading Conditions Hydraulic Grade Line for the Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing) Central Industrial District Kansas Levee Unit



Hydraulic Grade Line for the Ohio Avenue Pump Plant n500+5 Loading Conditions			
Elevation Top of Protection (MSL)	760.55	Existing Conditions	
Elevation at Landside Toe (MSL)	754		
Elevation Bottom of Blanket (MSL)	725		
Driving Head (ft)	-754		
c_i	0.001252		
x_1 (ft)	242		
L_2 (ft)	70		
x_3 (ft)	799		
h_o	-542.4		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-312	-754.00	0.00
	0	-542.41	211.59
	20	-529.00	225.00
	40	-515.92	238.08
	60	-503.16	250.84
	80	-490.72	263.28
	100	-478.58	275.42
	120	-466.75	287.25
Pump Plant Location	140	-455.21	298.79
	160	-443.95	310.05
	180	-432.97	321.03
	200	-422.26	331.74
	220	-411.82	342.18
	240	-401.64	352.36
	260	-391.71	362.29
	280	-382.02	371.98
	300	-372.57	381.43
	320	-363.36	390.64
	340	-354.38	399.62
	360	-345.61	408.39
	380	-337.07	416.93
	400	-328.73	425.27
	420	-320.60	433.40
	440	-312.67	441.33
	460	-304.94	449.06
	464	-303.42	450.58

n500+5 Loading Conditions Hydraulic Grade Line for the Ohio Avenue Pump Plant Sta 83+52 (CID-MO Stationing) Central Industrial District Kansas Levee Unit



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Assumptions used for underseepage calculations:

1. Semi-pervious blanket both on the river side and landward side
2. Landward side blanket of infinite extent
3. If an underseepage blanket exists, 1/2 of the width is included in the levee

Factors used in underseepage calculations:

k_f = horizontal permeability of the pervious foundation
k_{br} = vertical permeability of the blanket, river side
k_{bl} = vertical permeability of the blanket, landward side
z_{br} = thickness of the blanket, river side
z_{bo} = thickness of the blanket under the levee
z_{bl} = thickness of the blanket, landward side
d = thickness of pervious foundation
H = Net head on levee
L_1 = distance to river from riverside levee toe
L_2 = base width of levee and berms
L_3 = length of blanket beyond landside levee toe
c_r = factor used in calculations for river side
c_l = factor used in calculations for landward side
x_1 = distance from effective seepage entry to riverside levee toe
x_3 = distance from landside levee toe to effective seepage exit
h_o = head at base of blanket, landward levee toe, measured above the ground
i_o = computed hydraulic gradient at landside levee toe
i_c = critical hydraulic gradient
γ_b = bouyant unit weight of blanket soils
γ_w = unit weight of water
h_x = pressure head at base of blanket measured above the ground surface
x = distance from levee toe, positive indicates landward

Equations:

$c_r = (k_{br}/k_f z_{br} d)^{1/2}$

$c_l = (k_{bl}/k_f z_{bl} d)^{1/2}$

$x_1 = \tanh(c_r L_1)/c_r$

$i_o = h_o/z_{bl}$

$i_c = \gamma_b/\gamma_w$

$h_o = H((x_3/(x_1+$

CID Missouri - Hydraulic Grade Line for the Kemper Arena Pump Station Sta 106+49 (KS Sta)

Kemper Sta 106+49 (KS Sta) rev 1.xls

UNDERSEEPAGE ANALYSIS

Note : This spreadsheet analysis should be used only if the blanket thickness is at least 1/4 of the height of the levee.

BDT
Name : Kuzniakowski EC-GD
Date : 7/27/2007

Peer Review
Checked by :
Date :

NOTES:
1. Existing condition: Pump Station 180 ft from Wall Centerline
2. Raise proposal currently unknown
3. L_2 not revised for minor change due to raise

Elevations Used (From O&M Manual, except for n500 values):

Top of Existing Protection:	763.00
500 year + 0':	764.70
500 year + 3':	767.70
500 year + 5':	769.70
Ground Surface Land Side:	750 (Subtract 5' for fill in blanket thickness, but not for H)
Ground Surface River Side:	740
Bottom of Blanket:	720
Top of Bedrock:	670
Ground Surface at Pump Station:	750

Recommended Permeability Ratio
For Foundation Blanket Materials
Chapter Underseepage Revision dated Oct 1998

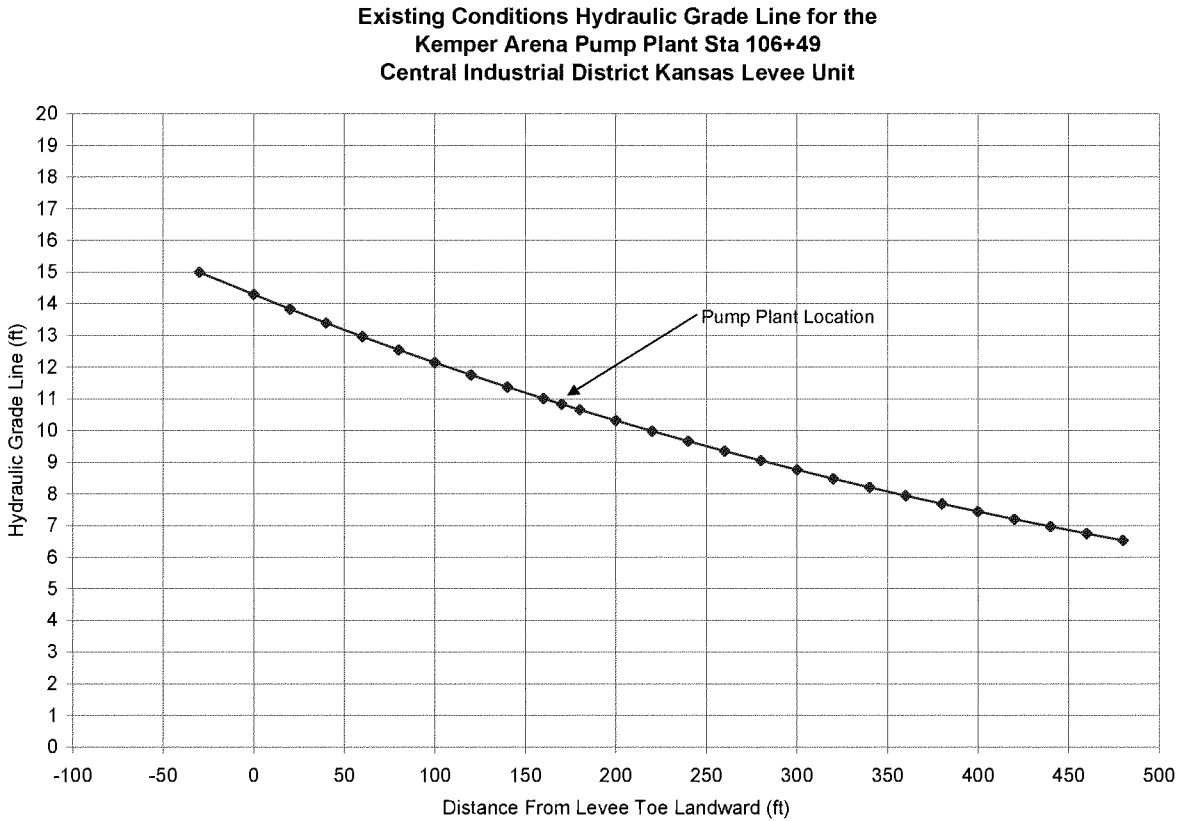
Blanket Material	Assumed Permeability Ratio, (K/Kb)
SM	100
ML	200 to 400
ML-CL	400
CL	400 to 600
CH	600 to 1000

Saturated Unit Weight of Blanket:	115 pcf
Unit Weight of Water:	62.4 pcf

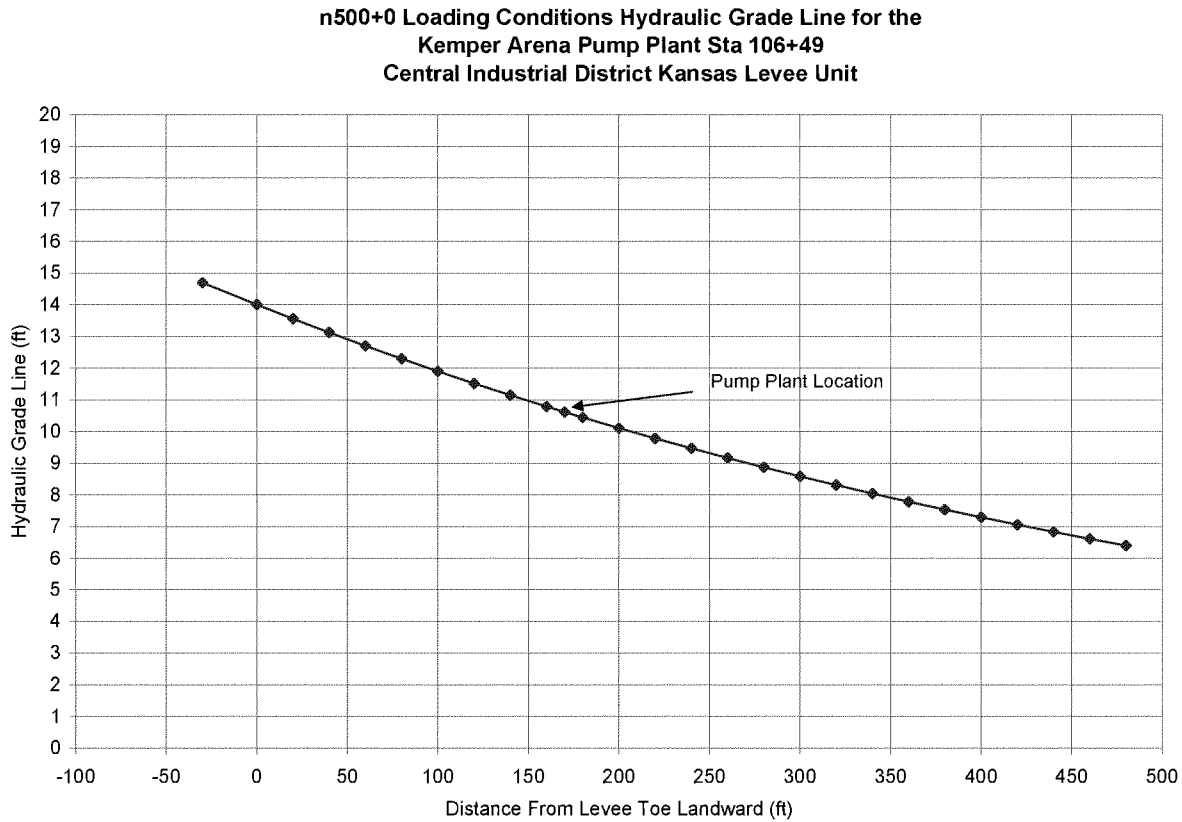
Station	Design Condition	Input Parameters: Permeability Ratio, River Side (k_w/k_{bw})	Permeability Ratio, Land Side (k_l/k_{bl})	Blanket Thickness (ft), River Side x_w	Blanket Thickness (ft), Protection x_{bw}	Blanket Thickness (ft), Land Side x_{bl}	Aquifer Thickness (ft) d	Seepage Length (ft), River Side L_1	Seepage Length (ft), Protection L_2	Seepage Length (ft), Land Side L_3	Driving Head (ft) H
106+49	Existing Condition	300	300	20.0	22.5	25.0	50.0	0	30	1000	15.0
106+49	n500 + 0	300	300	20.0	22.5	25.0	50.0	0	30	1000	14.7
106+49	n500 +3	300	300	20.0	22.5	25.0	50.0	0	30	1000	17.7
106+49	n500 + 5	300	300	20.0	22.5	25.0	50.0	0	30	1000	19.7

Station	Design Condition	Analysis: Factors c_r	Factors c_l	Effective Seepage Length (ft), River Side x_1	Effective Seepage Length (ft), Land Side x_3	Head at Toe (ft) h_0	Computed Hydraulic Gradient i_0	Critical Hydraulic Gradient i_c	Gradient Factor of Safety ic/io	Remarks
106+49	Existing Condition	0.001826	0.001633	0	612	14.3	0.57	0.84	1.47	
106+49	n500 + 0	0.001826	0.001633	0	612	14.0	0.56	0.84	1.50	
106+49	n500 +3	0.001826	0.001633	0	612	16.9	0.67	0.84	1.25	
106+49	n500 + 5	0.001826	0.001633	0	612	18.8	0.75	0.84	1.12	

Hydraulic Grade Line for the Kemper Arena Pump Plant Existing Conditions			
Elevation Top of Protection (MSL)	765.0	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	15		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	14.3		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	15.00	765.00
	0	14.30	764.30
	20	13.84	763.84
	40	13.40	763.40
	60	12.96	762.96
	80	12.55	762.55
	100	12.15	762.15
	120	11.75	761.75
	140	11.38	761.38
	160	11.01	761.01
Pump Plant Location	170	10.83	760.83
	180	10.66	760.66
	200	10.32	760.32
	220	9.98	759.98
	240	9.66	759.66
	260	9.35	759.35
	280	9.05	759.05
	300	8.76	758.76
	320	8.48	758.48
	340	8.21	758.21
	360	7.94	757.94
	380	7.69	757.69
	400	7.44	757.44
	420	7.20	757.20
	440	6.97	756.97
	460	6.75	756.75
	480	6.53	756.53

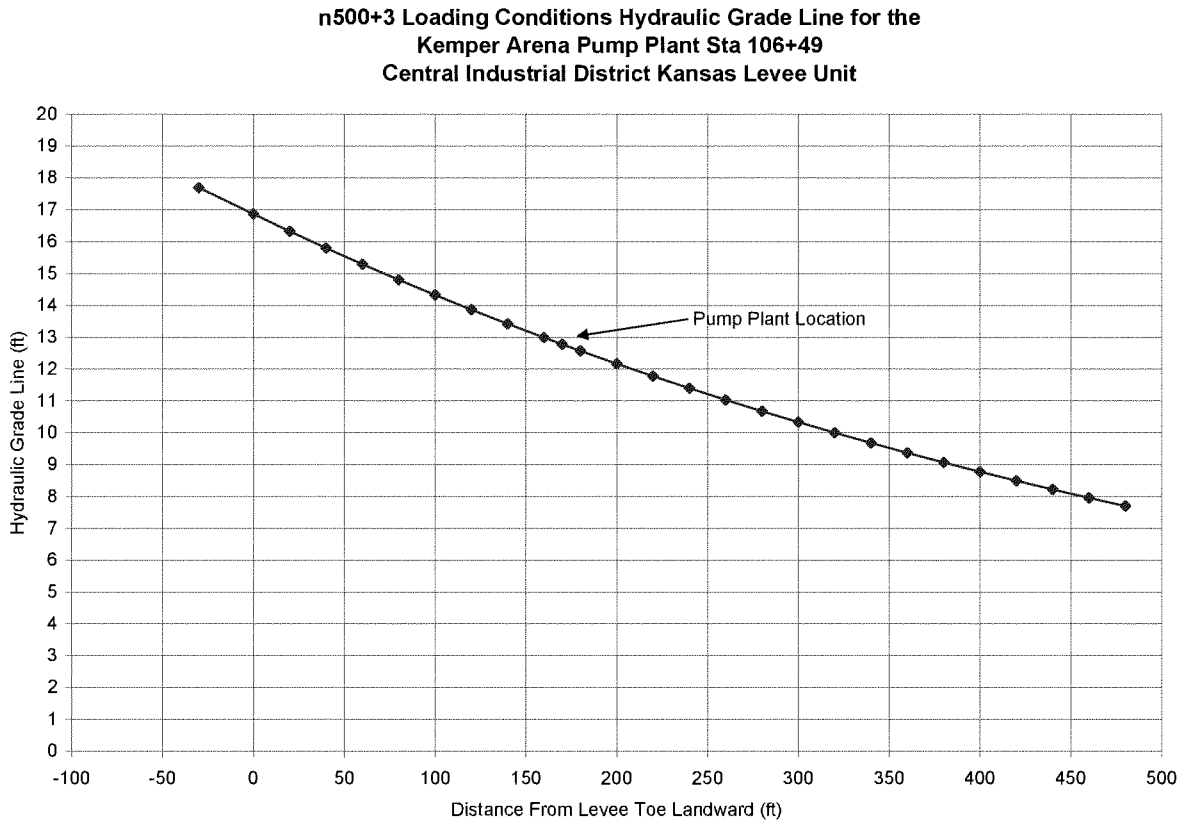

Existing Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
Pump Plant Location
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

Hydraulic Grade Line for the Kemper Arena Pump Plant n500+0 Loading Conditions			
Elevation Top of Protection (MSL)	764.70	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	14.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	14.0		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	14.70	764.70
	0	14.01	764.01
	20	13.56	763.56
	40	13.13	763.13
	60	12.71	762.71
	80	12.30	762.30
	100	11.90	761.90
	120	11.52	761.52
	140	11.15	761.15
	160	10.79	760.79
Pump Plant Location	170	10.62	760.62
	180	10.44	760.44
	200	10.11	760.11
	220	9.78	759.78
	240	9.47	759.47
	260	9.17	759.17
	280	8.87	758.87
	300	8.59	758.59
	320	8.31	758.31
	340	8.04	758.04
	360	7.78	757.78
	380	7.53	757.53
	400	7.29	757.29
	420	7.06	757.06
	440	6.83	756.83
	460	6.61	756.61
	480	6.40	756.40


n500+0 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
Pump Plant Location
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

780

Hydraulic Grade Line for the Kemper Arena Pump Plant n500+3 Loading Conditions			
Elevation Top of Protection (MSL)	767.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	17.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	16.9		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	17.70	767.70
	0	16.87	766.87
	20	16.33	766.33
	40	15.81	765.81
	60	15.30	765.30
	80	14.81	764.81
	100	14.33	764.33
	120	13.87	763.87
	140	13.42	763.42
	160	12.99	762.99
Pump Plant Location	170	12.78	762.78
	180	12.58	762.58
	200	12.17	762.17
	220	11.78	761.78
	240	11.40	761.40
	260	11.04	761.04
	280	10.68	760.68
	300	10.34	760.34
	320	10.01	760.01
	340	9.68	759.68
	360	9.37	759.37
	380	9.07	759.07
	400	8.78	758.78
	420	8.50	758.50
	440	8.23	758.23
	460	7.96	757.96
	480	7.71	757.71


n500+3 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
Pump Plant Location
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

782

Hydraulic Grade Line for the Kemper Arena Pump Plant n500+5 Loading Conditions			
Elevation Top of Protection (MSL)	769.7	Existing Conditions	
Elevation at Landside Toe (MSL)	750		
Elevation Bottom of Blanket (MSL)	720		
Driving Head (ft)	19.7		
c_l	0.001633		
x_1 (ft)	0		
L_2 (ft)	30		
x_3 (ft)	612		
h_o	18.8		
	Distance From Landward Levee Toe (ft)	Hydralic Gradient (ft)	MSL (ft)
Seepage Entrance	-30	19.70	769.70
	0	18.78	768.78
	20	18.18	768.18
	40	17.59	767.59
	50	17.31	767.31
	60	17.03	767.03
	80	16.48	766.48
	100	15.95	765.95
	120	15.44	765.44
	140	14.94	764.94
	160	14.46	764.46
Pump Plant Location	170	14.23	764.23
	180	14.00	764.00
	200	13.55	763.55
	220	13.11	763.11
	240	12.69	762.69
	260	12.28	762.28
	280	11.89	761.89
	300	11.51	761.51
	320	11.14	761.14
	340	10.78	760.78
	360	10.43	760.43
	380	10.10	760.10
	400	9.77	759.77
	420	9.46	759.46
	440	9.15	759.15
	460	8.86	758.86
	480	8.58	758.58

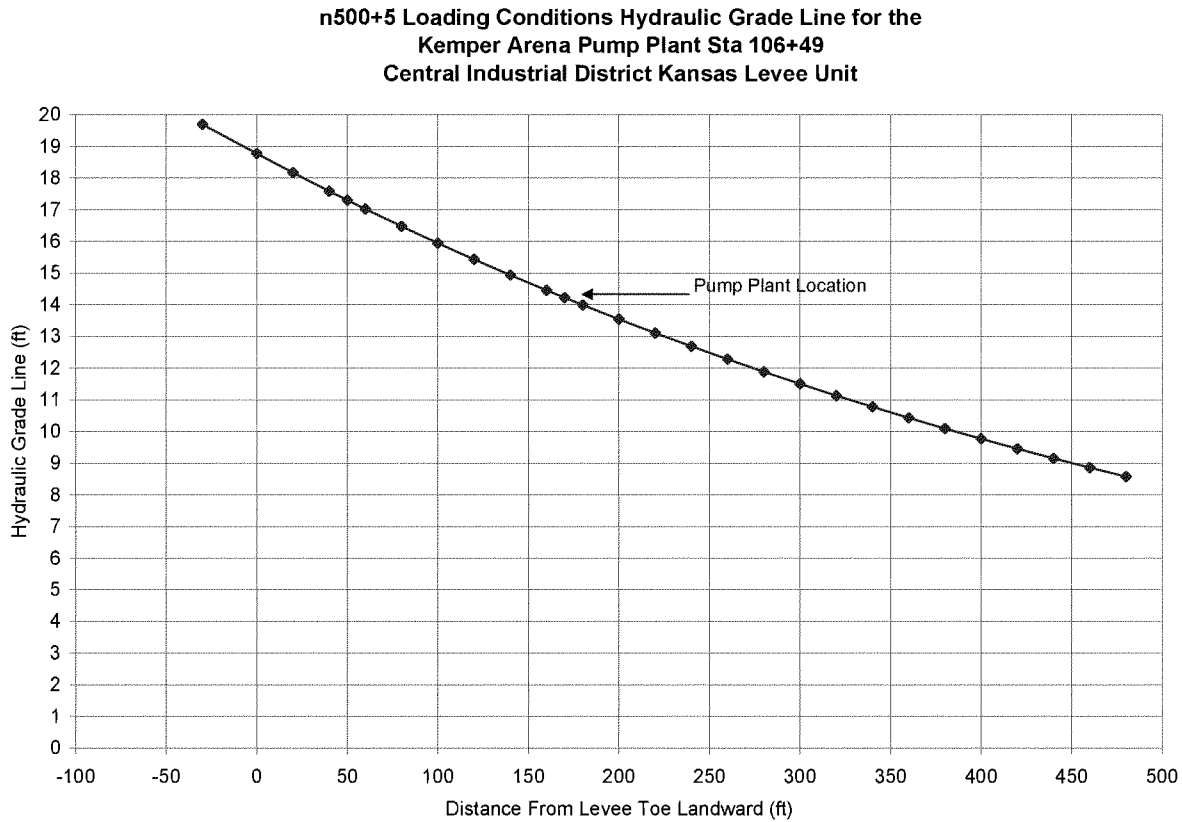
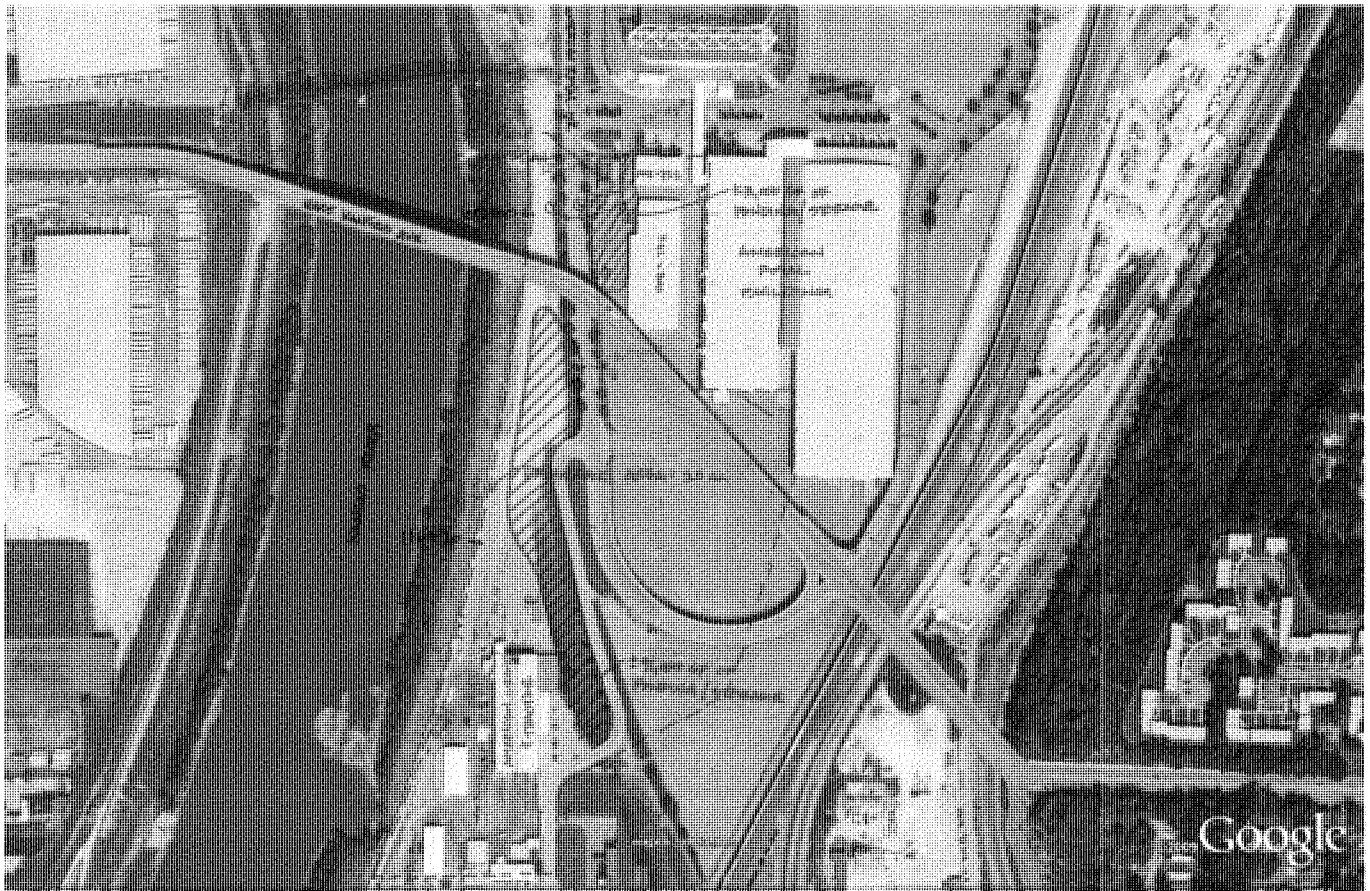

n500+5 Loading Conditions Hydraulic Grade Line for the
Kemper Arena Pump Plant Sta 106+49
Central Industrial District Kansas Levee Unit
Pump Plant Location
Hydraulic Grade Line (ft)
Distance From Levee Toe Landward (ft)
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-100
-50
0
50
100
150
200
250
300
350
400
450
500

EXHIBIT A-14.3

Central Industrial District, KS Relief Wells Maps

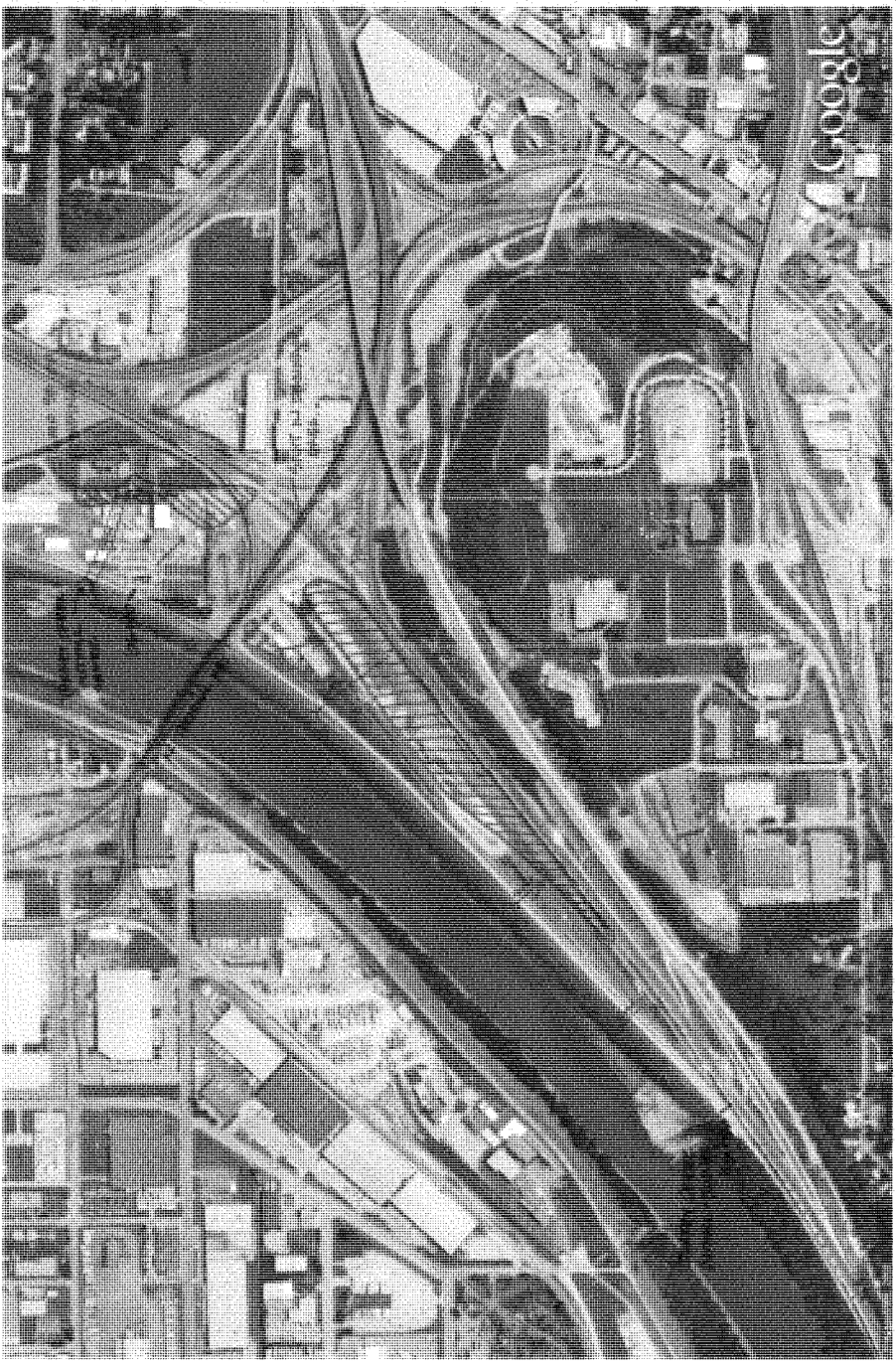


14-119

07/14/09 SPM

STA. 107+00 – 116+70

STA. 127+00 - 166+83



14-120

07/10/09 SPM

Chapter A-15

ACCESS ROADS ARMOURDALE

CHAPTER A-15
ACCESS ROADS - ARMOURDALE

A-15.1 Access Associated With Existing Levee

The Armourdale Unit is a highly developed manufacturing area with a significant road network. Contractor access into the area is via the West Kansas Avenue Bridge from the west, I-70 and US Highway 69 from the north, the East Kansas Avenue Bridge from the east, and US Highway 69 from the south. The bridges can be accessed from I-70, which runs east-west on the north side of the Armourdale area. Bridge load limits and clearances will be included during design of plans and specifications.

A-15.1.1 Construction Access

Direct access to the site will be via the existing road network shown by Exhibit A-15.1 in the Supplemental Exhibits Section at the end of this chapter. The main artery running east-west is Kansas Avenue. Osage Avenue is a smaller road but runs east-west as well. There are various north-south streets connecting the levee. Kaw Valley Drainage District (KVDD) will provide access, during and after construction, to the levee at the landward ramps along the levee unit listed in Table A-15.1.

TABLE A-15.1
Construction Access Points

Access Point	Station	Roads	Private/Public	Direction of Travel Once Access to Levee is Gained
A	32+10	Railroad Dirt Roads	Private	Upstream/Downstream
		26th Street	Public	Upstream/Downstream
		Bayard Street	Public	Upstream/Downstream
B	60+56	Kansas Ave	Public	Upstream
B	63+04	Kansas Ave	Public	Downstream
C	42+50	Procter & Gamble	Private	Upstream/Downstream
D	128+25	12th Street	Public	Upstream
D	131+10	12th Street	Public	Downstream
E	Left Blank Intentionally			
F	184+70	5th Street	Public	Upstream/Downstream
G	220+25	Sambol Meat Packing Parking Lot	Private	Downstream
		Osage Ave	Public	Downstream
H	252+35	Pitney Bowles Parking Lot	Private	Downstream
		via Kansas Ave	Public	Downstream
I	275+00	Railroad Dirt Roads	Private	Downstream
		via Kansas Ave	Public	Downstream
J	292+00	Central Ave	Public	Upstream/Downstream

Public access to these ramps is available over city streets and roadways except where noted. Coordination with the KVDD has shown that each access is maintained and usable for the project. The borrow area is located approximately 8 miles to the west of the site and is accessible via the public road system. In addition, multiple rights-of-entry through privately owned property will be required along the levee for direct access during construction. Temporary work area easements will be needed for location of construction trailers, contractor storage areas, and may also be needed for temporary construction roads.

Temporary aggregate construction roads will be constructed along the levee. These roads have little value for permanent use after construction is completed. They will be removed and replaced by permanent parking or seeded for turf establishment. Use of construction roads as permanent access to the levee is not viable since the current access is via existing ramps, which will be modified to accommodate any raise.

It is assumed that the city will require a road repair bond from the contractor in the amount of $500,000. The bond will be held by the city to use for any damages caused by the contractor's construction equipment. It is also assumed a full-time street sweeper will be required during the earth hauling operations. Inspections of the contractor's access route will be conducted prior to the start of construction and periodically during construction to determine the extent of any damage caused by the contractor. Costs are included in the estimate (see Cost Engineering Appendix of the Final Feasibility Report) for cleaning and repair of existing roads.

A-15.1.2 Ramps and Turnouts

Table A-15.2 contains a list of all ramps and access points and Table A-15.3 lists turnouts currently located along the Armourdale Unit. These tables provide the location of the feature according to station, what recommended action is proposed, and miscellaneous notes pertaining to the feature. According to EM 1110-2-1913, turnouts should be provided on one-lane access roads on levees at intervals of approximately 2,500 feet, provided there are no ramps within the reach. Considering where all the landward and riverward access ramps and turnouts are located, the current ramps and turnouts meet this criteria.

TABLE A-15.2
Ramps and Access Points Recommended Improvements

Access Point	Station	Landward or Riverward Access	Action	Notes
Ramp	11+90UE	Riverward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
Ramp	13+50UE	Riverward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
Ramp	16+55UE	Riverward	Abandon	
Ramp	15+20	Riverward	Reestablish for N500+3 raise	
A.P. A	32+10	Landward	Reestablish for N500+3 raise	
A.P. A	32+10	Riverward	Reestablish for N500+3 raise	
A.P. B	60+56	Landward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
A.P. B	63+04	Landward	Reestablish for N500+3 raise	
Ramp	75+53	Landward	Reestablish for N500+3 raise	
Ramp	77+05	Landward	Reestablish for N500+3 raise	
Ramp	113+00	Landward	Abandon	Ramp currently dead ends at fence around salvage yard
Ramp	121+70	Riverward	Reestablish for N500+3 raise	
A.P. D	128+25	Landward	Reestablish for N500+3 raise	
A.P. D	131+10	Landward	Reestablish for N500+3 raise	
Ramp	131+30	Landward	Abandon	Access is to private parking lot with no means of locking off access
Ramp	168+35	Riverward	Reestablish for N500+3 raise	
A.P. F	184+70	Landward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
Ramp	222+50	Riverward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
A.P. G	230+25	Landward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
Ramp	231+00	Riverward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
A.P. H	252+35	Landward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
Ramp	253+50	Riverward	Reestablish for N500+3 raise	Access is to new road at base of new floodwall
A.P. I	275+00	Landward	No action	Not a ramp, just an access point to slot area prior to new floodwall construction
A.P. J	292+00	Landward		Not a ramp, just an access point to the landside of the existing floodwall
Ramp	302+58	Landward	Reestablish for N500+3 raise	
Ramp	313+00	Landward	Reestablish for N500+3 raise	

TABLE A-15.3
Turnouts

Station	Landward or Riverward Access	Action
4+00UE	Landward	Reestablish for N500+3 raise
16+05	Landward	Reestablish for N500+3 raise
40+00	Landward	Abandon
110+00	Landward	Reestablish for N500+3 raise
157+70	Landward	Reestablish for N500+3 raise
168+35	Landward	Reestablish for N500+3 raise
247+15	Landward	Reestablish for N500+3 raise; new turnout will be riverside as new floodwall is going in here with a riverside access road
40+00LE	Landward	Reestablish for N500+3 raise

A-15.1.3 Private Roads and Access

When the levee is raised landward of the existing centerline, some private roads and access will be impacted. These roads and access will need to be reestablished if real estate interests are present. Table A-15.3 lists the known impacts and proposed remedies.

TABLE A-15.4
Access Impacts and Action

Station	Issue	Proposed Remedy
100+00	KC Hardwoods – Gravel road around "silo" is currently on levee slope	Reestablish access road on raised levee slope for N500+3 raise
131+00	Pallet Processing – Industry stores some pallets and accesses rear of their building using toe of existing levee	Real estate interest appears to not exist; therefore, access will not be reestablished
175+00 to 185+00	Schock Co. – Access Road from 5th Street P.S. to Schock loading docks will be impacted with levee raise	Reroute access road north of existing road and then back to toe of levee near corner of building
197+00 to 257+56	Levee and T-Wall raise will extend onto existing parking lots and distribution warehouse truck lots	Refer to Real Estate Appendix of Feasibility Report
275+00	Slot access under MO Pacific Bridge and Union Pacific Bridge will be eliminated with new floodwall	Coordinate with railroad and PBI Gordon regarding at-grade crossings of tracks onto PBI Gordon property

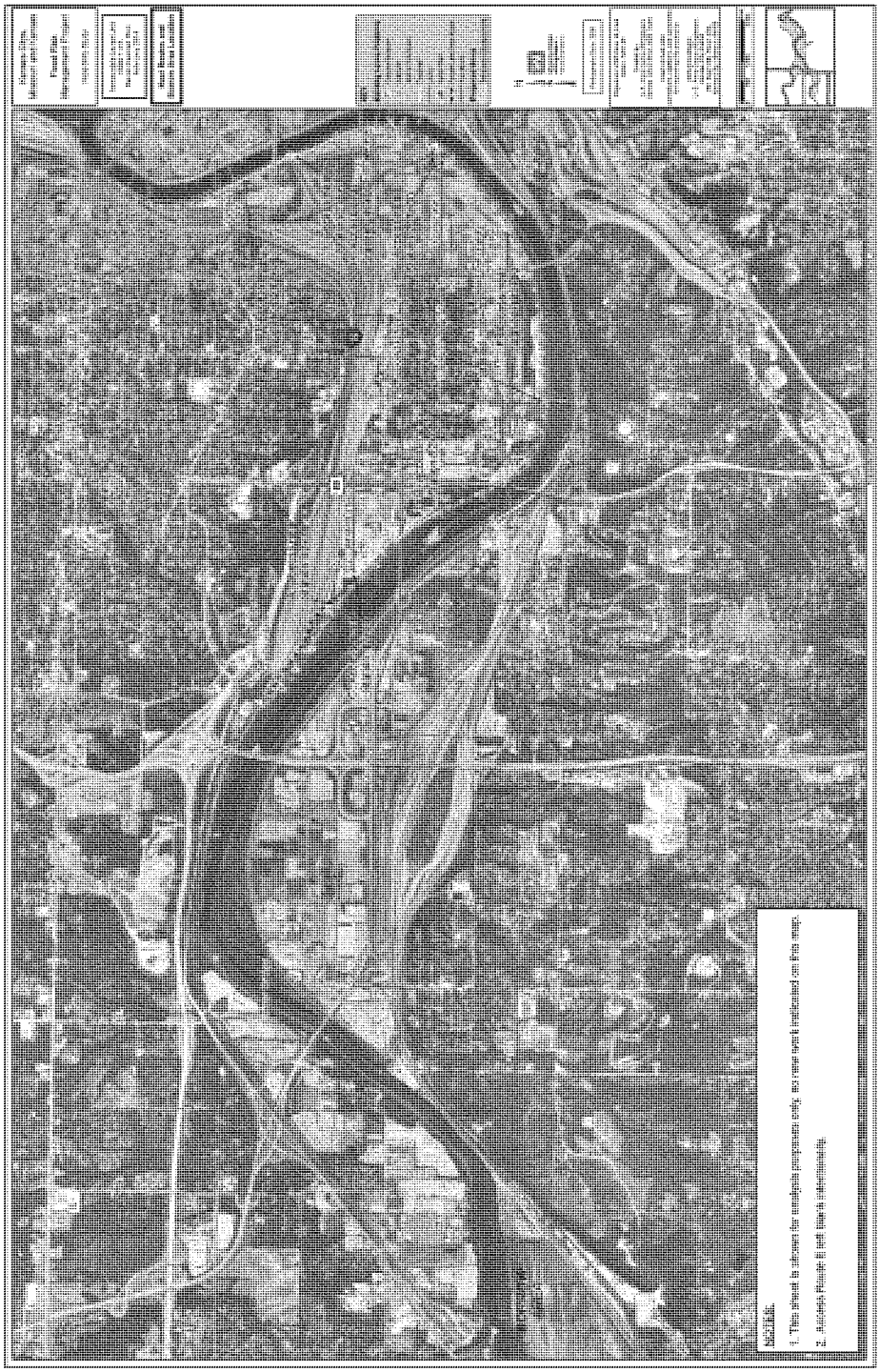
At Station 275+00, there is a route under the Union Pacific Bridge and MO Pacific Bridge through the "slot" area. The route will no longer be available with the construction of a new floodwall and stoplog closure structure. The railroad has an undeveloped crossing under the Union Pacific track (southern track) that gives them access to their railyard and maintenance buildings. This access will be left in place. During flood fights, the slot area has not been available due to flooding within the slot. Therefore, access around the slot with no additional access improvements, would be no different than current means. However, after the N500+3 recommendations are implemented, access around the slot area could be provided with at-grade crossings of the two railroad tracks passing over the tracks onto PBI Gordon access roads. No coordination with the railroad or with PBI Gordon was made during the feasibility study.

Additional information regarding impacts to parking lots and private real estate is included in the Real Estate Appendix of the Final Feasibility Report.

A-15.2 SUPPLEMENTAL EXHIBITS

EXHIBIT A-15.1

Armourdale Haul Routes


NOTES:
1. This sheet is shown for analysis purposes only, not one such indicated on this map.
2. Accesses Plan to Sheet Block is determined.

15-8

Chapter A-16

ACCESS ROADS
CID-KS

CHAPTER 16
ACCESS ROADS – CID KS

A-16.1 ACCESS ASSOCIATED WITH EXISTING LEVEE

The Central Industrial District Unit is a highly developed commercial area with a significant road network. Contractor access into the area from the east or west is primarily via I-670, I-70, Central Avenue, or Kansas Avenue. Access from the north and south primarily depends on city street access. Five bridges may be used to cross the Kansas River to get to the CID-KS levee unit. These include I-635, Highway 169, I-670, Central Avenue Bridge, and the James Street Bridge. Bridge load limits and clearances will be included during design of plans and specifications.

A-16.1.1 Construction Access

Direct access to the site will be via the existing city street network. The main artery running east-west is I-670 via exit 1B. Kaw Valley Drainage District (KVDD) will provide access, during and after construction, to the levee at the landward ramps listed in Table A-16.1.

TABLE A-16.1
Construction Access Points

Access Point	Station	Roads	Private/Public	Direction of Travel Once Access to Levee is Gained
A	17+00	Via Market Street	Public	Downstream/Upstream
B	27+00	Via N James Street	Public	Upstream
C	57+00	Via Central Ave	Public	Downstream/Upstream
D	78+00	Via I-670 Right of Way/Union Pacific Railroad Right of Way	Private	Upstream
E	133+00	Via W 25th Street	Public/Private	Upstream

Public access to these ramps is available over city streets and roadways except where noted. Coordination with the KVDD has shown that each access is maintained and usable for the project. The borrow area is located approximately 12 miles to the west of the site and is accessible via the public road system. Temporary work area easements will be needed for location of construction trailers, contractor staging areas, and may also be needed for temporary construction roads.

Temporary aggregate construction roads will be constructed along the levee. These roads have little value for permanent use after construction is completed. They will be

removed and replaced by permanent parking or seeded for turf establishment. Use of construction roads as permanent access to the levee is not viable since the current access is via existing ramps, which will be modified to accommodate any raise.

It is assumed that the city will require a road repair bond from the contractor in the amount of $500,000. The bond will be held by the city to use for any damages caused by the contractor's construction equipment. It is also assumed a full-time street sweeper will be required during the earth hauling operations. Inspections of the contractor's access route will be conducted prior to the start of construction and periodically during construction to determine the extent of any damage caused by the contractor. Costs are included in the estimate (see Cost Engineering Appendix of the Final Feasibility Report) for cleaning and repair of existing roads.

A-16.1.2 Ramps and Turnouts

According to EM 1110-2-1913, turnouts should be provided on one-lane access roads on levees at intervals of approximately 2,500 feet, provided there are no ramps within the reach. All reaches have an existing ramp at less than or approximately 2,500 feet except between approximate Stations 27+00 to 57+00. However, a staging area will be constructed at Station 40+00, which could double as a turnout. Considering where all the access ramps are located, the requirement for turnouts has been met.

A-16.1.3 Private Roads and Access

When the levee is raised landward of the existing centerline, some private roads and access will be impacted. These roads and access will need to be reestablished if real estate interests are present. Table A-16.2 lists the known impacts and proposed remedies.

TABLE A-16.2
Access Impacts and Action

Access Point	Station	Roads	Impact	Action
A	17+00	Via Market Street	None	None
B	27+00	Via N James Street	None	None
C	57+00	Via Central Ave	Removed during construction	Reestablish access to public street
D	78+00	Via I-670 Right of Way/Union Pacific Railroad Right of Way	Removed during construction	Reestablish access
E	133+00	Via W 25th Street	None	None

Additional information regarding impacts to parking lots and private real estate is included in the Real Estate Appendix of the Feasibility Report.

A-16.1.4 Haul Routes

Haul Routes most likely taken by the contractor are listed in Table A-16.3. Routes coincide with access points listed in Table A-16.1. All haul routes are via local, state, or federal highway systems. There are no hospitals, schools, or places of worship immediately along these projected haul routes. There is one place of worship and two schools that are located approximately 1, 2, and 3 blocks, respectively, located off of the projected haul route.

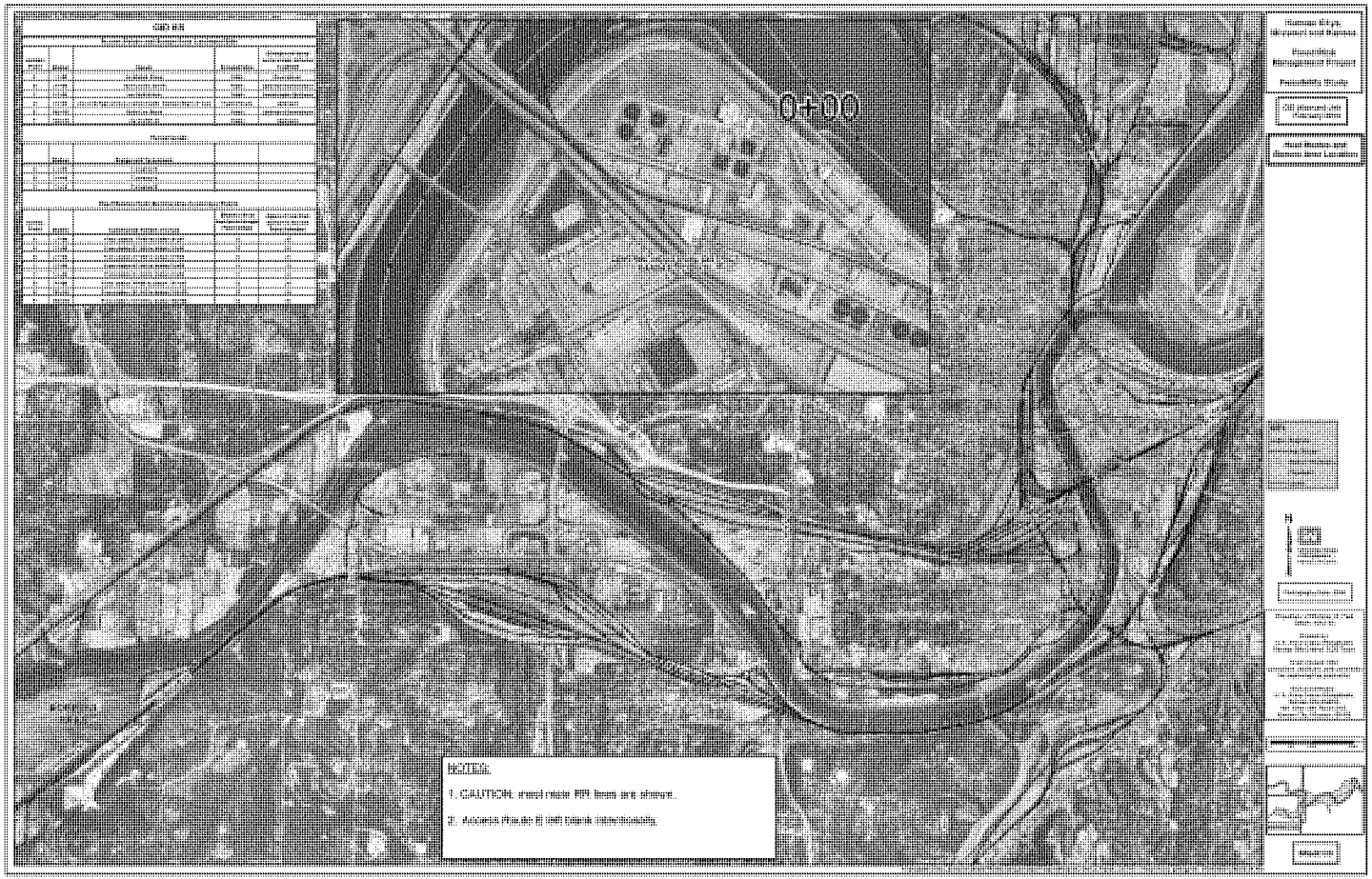
TABLE A-16.3
Haul Routes

Access Point	Station	Constructed Portion of Levee	Distance from Borrow to Access Point (miles)	Approximate Time from Borrow to Access Point (minutes)
A	17+00	From Station 17+00 to Station 0+00	12.4	25
		From Station 17+00 to Station 27+00	12.4	25
B	27+00	From Station 27+00 to Station 57+00	11.7	23
C	57+00	From Station 57+00 to Station 78+00	11.2	23
		From Station 57+00 to Station 27+00	11.2	23
D	78+00	From Station 78+00 to Station 101+50	11.7	24

A-16.2 SUPPLEMENTAL EXHIBITS

EXHIBIT A-16.1

CID-KS Haul Routes


0+00
NOTES:
1. CAUTION: most main RR lines are shown.
2. Access Route E left bank intermittently.

16-6

APPENDIX B
Socioeconomic Analysis

Kansas Citys, Missouri and Kansas
Flood Risk Management

Final Feasibility Report

April 2014

US Army Corps
of Engineers®

Prepared by:
U.S. Army Corps of Engineers
Kansas City District
600 Federal Building
601 E 12th Street
Kansas City, MO 64106-2896

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List of Acronyms and Abbreviations

AAD	Average Annual Damages
ACS	American Community Survey
AEP	Annual Exceedance Probability
BNSF	Burlington Northern & Santa Fe Railway Company
cfs	Cubic feet per second
CID	Central Industrial District
CSVR	Contents-to-structure value ratio
CWPM	Civil Works Policy Memorandum
DDF	Depth-Damage Function
EAB	Expected Annual Benefit
EAD	Expected Annual Damage
ER	Engineering Regulation
FEMA	Federal Emergency Management Agency
IWR	Institute for Water Resources
MSA	Metropolitan Statistical Area
msl	Mean sea level
NED	National Economic Development
OMRR&R	Operation, Maintenance, Repair, Rehabilitation, and Replacement
OSE	Other Social Effects
P&G	Principles and Guidelines
RED	Regional Economic Development
RM	River mile
UPRR	Union Pacific Railroad
USACE	United States Army Corps of Engineers

1.0 Introduction

1.1 Federal Project Overview & Study Purpose

The Kansas Citys Flood Control Project was originally authorized by Section 5 of the Flood Control Act approved 22 June 1936, while subsequent modifications were authorized under Section 216 of the 1970 Flood Control Act. The system consists of seven separable units protecting much of the central industrial district of the Kansas City area. Protection of both public safety and property requires that the seven units function as an integrated and unified system during flood events.

The entire seven-levee system withstood the Missouri River Flood of 1993, but the general performance of the system was severely tested during the event. Not only were the stages extreme, but also durations were lengthy. Concerns arose about the level of protection against overtopping and seepage. Further, there was a concern that the levees may provide less than the authorized level of protection. In response to these concerns, a reconnaissance study was undertaken and completed in 2000, which produced recommendations supportive of further feasibility analysis.

The initial investigation of all seven units resulted in the identification of specific remedies and improvements that could be most readily analyzed and evaluated. In order to enable the feasibility study of the overall system to progress in an efficient manner, the study was separated into Phase 1 and Phase 2 efforts.

The Phase 1 study effort resulted in the Interim Feasibility Report (Interim Report), published in September 2006, which addressed four of the seven levee units: Argentine, North Kansas City, East Bottoms, and Fairfax-Jersey Creek. A fifth levee unit, the Birmingham unit, was determined to meet the authorized level of protection assuming continued adequate operations and maintenance efforts. Study of the two remaining units – CID and Armourdale – was deferred to a second phase. The Interim Report summarizing the Phase 1 analysis presented a series of recommendations that were subsequently authorized by the Water Resources Development Act of 2007.

The final Feasibility Report is the culmination of Phase 2 of the ongoing feasibility study. The study area of Phase 2 includes the Armourdale unit on the Kansas River at Kansas City, Kansas, and the Central Industrial District (CID) unit on the Kansas and Missouri Rivers at Kansas City, Kansas, and Kansas City, Missouri, along with all of their appurtenant features, including levees, floodwalls, pump stations, relief wells, closure structures, and berms. The Armourdale and CID units are two of three units comprising an integrated, interdependent flood risk reduction system on the Kansas River. The third unit in the Kansas River system is the Argentine unit, which was among the units evaluated in the Phase 1 interim report. In order to discuss the Kansas River

units as a system, the present report will display certain data for the Argentine unit that was covered in greater detail in the Phase 1 interim report.

1.2 Study Guidance

Pertinent guidance governing economic analysis procedures includes:

- "Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies" (P&G), dated March 1983;
- Engineering Regulation (ER) 1105-2-100, "Planning Guidance Notebook," dated 22 April 2000 (partially updated subsequently);
- Engineering Regulation (ER) 1105-2-101, "Risk-Based Analysis for Flood Damage Reduction Studies," dated 3 January 2006
- Engineer Manual (EM) 1110-2-1619, "Risk-Based Analysis for Flood Damage Reduction Studies," dated 1 August 1996.

1.3 Study Area Location

The Kansas Citys, Missouri and Kansas, Local Flood Protection Project consists of seven separate levee/floodwall units located along both banks of the Missouri and Kansas Rivers. The Phase 1 report evaluated the Argentine, Fairfax-Jersey Creek, North Kansas City, East Bottoms, and Birmingham units whereas the Phase 2 report will focus on the Armourdale and Central Industrial District units. The CID unit, located near the confluence of the Missouri and Kansas Rivers, can be impacted by both Missouri River flooding and Kansas River flooding. For purposes of this appendix, the total project study area will refer to all seven units while the Phase 2 study area will only refer to the Armourdale and CID reaches. Table 1 below lists the study reaches as well as their beginning and ending river mile.

Table 1: **Study Reaches**

Reach Name	Beginning Station	Ending Station	Bank
Phase 2 Units			
Kansas River:			
Armourdale	0.6	7.7	Left
CID-Kansas	0.0	3.0	Right
Missouri River:			
CID-Missouri	365.8	367.4	Right
Phase 1 Units			
Kansas River:			
Argentine	4.6	10.0	Right
Missouri River:			
Fairfax-Jersey Creek	367.5	373.9	Right
North Kansas City	362.6	370.7	Left
East Bottoms	356.6	366.0	Right
Birmingham	353.2	360.4	Left

1.4 Phase 2 Project Description

1.4.1 Armourdale Project Description

The Armourdale unit is located in Wyandotte County, Kansas, along the left bank of the Kansas River from river mile 7.0 to river mile 0.3, near the confluence of the Kansas and Missouri Rivers. The original levees and floodwalls were constructed under the jurisdiction of the Kaw Valley Drainage District (Kaw is a colloquial, regional name referring to the Kansas River) and then modified and expanded in the initial and follow-on Federal projects. The primary components of the unit consist of earthen levee, floodwalls, riprap and toe protection on riverward slope of levees, toe drains along the concrete floodwalls, sandbag gaps, stop log gaps, drainage structures, relief wells and pumping plants. Construction of the Federal project began in 1949 and was completed in 1951. More recent improvements, separately authorized under the 1962 modification, were completed in 1976. The levees and floodwalls of the Armourdale unit are currently authorized to pass a maximum Kansas River flow of 390,000 cfs coincident with a Missouri River flow of up to 220,000 cfs.

1.4.2 Central Industrial District (CID) Project Description

Although the CID unit is one levee unit, it is operated and managed as two separate and distinct sections: the CID-Kansas section, and the CID-Missouri section. The CID-Kansas section (CID-KS), is located in Wyandotte County, Kansas, and extends along the right bank of the Kansas River from mile 3.4 to the mouth, then downstream along the right bank of the Missouri River to the Missouri and Kansas State Line. The unit consists of levee sections and floodwalls, riprap and levee toe protection and a surfaced levee crown and ramps, a stop log gap, a sandbag gap, pumping plants, drainage structures, and relief wells. This section was originally developed by the Kaw Valley Drainage District, and initial Federal improvements entered construction in 1948. Most of the Federal improvements including repairs to levee unit damage from the 1951 flood were completed by 1955. The most recent improvements authorized under the 1962 modification were completed in 1979. The CID-KS section is authorized to pass a Kansas River discharge of 390,000 cfs coincident with a Missouri River flow of 220,000 cfs.

The CID-Missouri section (CID-MO) is located in Kansas City, Jackson County, Missouri. This section extends along the right bank of the Missouri River (river mile 365.7) to the Kansas-Missouri state line (river mile 367.2). The CID-MO section consists of levee, floodwalls, a levee drainage system and pumping plants, sandbag and stop log gaps, toe and bank protection, and slope protection on the riverward slope. The initial construction began in 1946. Significant improvements and repair of 1951 flood damage followed the initial construction and were completed in 1955. The CID-MO section is designed to pass a Missouri River flow of 540,000 cfs.

2.0 Socioeconomic Description

Census 2010 data and American Community Survey (ACS) 2007-2011 5-year estimates for 11 census tracts were compiled to describe the socioeconomic characteristics for both Phase 1 and Phase 2 levee units. Census and ACS data were also compiled for counties in the study area and for the Kansas City MO-KS Metropolitan Statistical Area (MSA). Although census tracts cover areas that may typically be somewhat larger than the area protected by a levee unit, the census tracts surveyed for the Kansas Citys units have similar alignments. Table 2 lists the census tracts that define the study area.

Population and housing units from the 2010 Census are available for census tracts and allow a fairly accurate accounting of study area population, as summarized in Table 3. By this reckoning, the 2010 population of the Phase 2 study area was 4,654 and there were 1,887 households. The Phase 2 study reach with the larger population is Armourdale; however the Armourdale unit has suffered moderate decline over the last decade while the CID unit has experienced significant growth. Table 4 and Table 5 summarize a range of population characteristics from the 2010 Census for the Census tracts comprising the entire project area, whereas Table 6 describes the housing characteristics. If 2010 Census data were not available, American Community Survey 5-year estimates were used.

Table 2: **Census Areas Included In Study Area**

Phase 2 Units			
State	**County**	**Census Tract**	**Portions of Study Area Included**
Kansas	Wyandotte	425.01	Armourdale
Kansas	Wyandotte	425.02	Armourdale
Kansas	Wyandotte	426	Armourdale
Kansas	Wyandotte	400.02	CID
Missouri	Jackson	152	CID
Kansas	Wyandotte	430	CID (Turkey Creek)
Kansas	Wyandotte	452	CID (Turkey Creek)
Missouri	Jackson	46	CID (Turkey Creek)
Missouri	Jackson	153	CID (Turkey Creek)
Phase 1 Units			
State	**County**	**Census Tract**	**Portions of Study Area Included**
Kansas	Wyandotte	428	Argentine
Kansas	Wyandotte	438.04	Argentine
Kansas	Wyandotte	400.01	Fairfax-Jersey Creek
Missouri	Clay	221	North Kansas City
Missouri	Jackson	3	East Bottoms
Missouri	Jackson	155	East Bottoms

Table 3: **Study Area Population Trends**

Phase 2 Units	**Armourdale**	**Central Industrial District**	**CID - Turkey Creek**
2010 Population	2,924	1,730	8,626
2000 Population	3,212	936	10,002
% Change 2000-2010	-8.97%	84.83%	-13.76%
2010 Households	872	1,015	4,454
2000 Households	986	483	4,706
% Change 2000-2010	-11.56%	110.14%	-5.35%
Phase 1 Units	**Argentine**	**North Kansas City**	**East Bottoms**
2010 Population	3,345	4,283	2,808
2000 Population	3,480	4,883	3,276
% Change 2000-2010	-3.88%	-12.29%	-14.29%
2010 Households	1,258	2,416	1,178
2000 Households	1,282	2,669	1,282
% Change 2000-2010	-1.87%	-9.48%	-8.11%

Table 4: **Selected Population Characteristics**

Phase 2 Units	**Armourdale**	**Central Industrial District**	**CID - Turkey Creek**
Population	2,924	1,730	9,712
Median Age	13.5 to 51.5	30.5 to 51.5	30.4 to 43.0
% 18 and under	33.1%	4.8%	47.4%
% 65 and above	7.8%	1.0%	8.5%
Racial Distribution			
% White	58.3%	77.2%	62.8%
% Black or African American	2.4%	15.4%	14.2%
% American Indian and Alaska Natvie	2.1%	0.5%	0.9%
% Asian	1.4%	1.9%	2.6%
% Native Hawaiian and Other Pacific Islander	0.0%	0.1%	0.1%
% Some Other Race	32.4%	2.0%	15.3%
% Multi Race	3.4%	2.9%	4.1%
% Hispanic	64.7%	4.9%	36.1%
Education Levels			
% High School Grads (age 25+)	57.9%	90.8%	80.2%
% College Grads (age 25+)	4.9%	50.1%	30.4%
Phase 1 Units	**Argentine**	**North Kansas City**	**East Bottoms**
Population	3,345	4,283	2,808
Median Age	29.6 to 30.5	40.3	31.2 to 36.8
% 18 and under	33.7%	15.5%	28.1%
% 65 and above	13.4%	16.5%	11.4%
Racial Distribution			
% White	55.4%	76.4%	58.7%
% Black or African American	17.8%	11.1%	17.8%
% American Indian and Alaska Natvie	1.1%	0.8%	0.8%
% Asian	1.4%	3.3%	8.0%
% Native Hawaiian and Other Pacific Islander	0.0%	0.3%	0.3%
% Some Other Race	18.1%	4.6%	10.6%
% Multi Race	6.1%	3.4%	3.8%
% Hispanic	45.6%	11.4%	20.8%
Education Levels			
% High School Grads (age 25+)	68.9%	87.6%	71.9%
% College Grads (age 25+)	10.6%	23.0%	14.3%

Table 5: **Study Area Unemployment, Poverty, and Personal Income**

Phase 2 Units	**Armourdale**	**Central Industrial District**	**CID - Turkey Creek**
Median Household Income	$32,559	$41,118	$32,974 to $50,121
Unemployment Rate	12.4%	7.1%	10.1%
Per Capita Income	$12,614	$37,689	$26,048
% of Individuals Living Below Poverty Level	29.9%	17.4%	25.2%
Phase 1 Units	**Argentine**	**North Kansas City**	**East Bottoms**
Median Household Income	$31,742	$42,886	$25,389 to $29,718
Unemployment Rate	19.8%	8.2%	13.3%
Per Capita Income	$15,472	$29,193	$15,310
% of Individuals Living Below Poverty Level	32.5%	6.6%	46.5%

Table 6: **Housing Characteristics for Study Area**

Phase 2 Units	**Armourdale**	**Central Industrial District**	**CID - Turkey Creek**
Total Housing Units	1,025	1,110	5,091
% Occupied	85.1%	91.4%	87.5%
% Vacant	14.9%	8.6%	12.5%
% Owner Occupied	57.0%	18.3%	43.3%
% Renter Occupied	43.0%	81.7%	56.7%
% Built 1939 or earlier	29.6%	58.8%	49.7%
% Moved in 2004 or earlier	42.8%	89.0%	52.6%
Average Household Size	3.28	1.41	2.11
Median Value of Owner-Occupied Housing	$47,600	$233,200	$81,000 to $200,500
Phase 1 Units	**Argentine**	**North Kansas City**	**East Bottoms**
Total Housing Units	1,373	2,744	1,467
% Occupied	91.6%	88.0%	85.8%
% Vacant	8.4%	12.0%	21.0%
% Owner Occupied	37.0%	26.7%	38.5%
% Renter Occupied	13.9%	73.3%	61.5%
% Built 1939 or earlier	14.7%	18.3%	57.0%
% Moved in 2004 or earlier	35.2%	66.7%	46.9%
Average Household Size	2.81	1.86	2.27
Median Value of Owner-Occupied Housing	$85,600 to $137,500	$119,000	$41,300 to $156,900

2.1 Armourdale Unit

2.1.1 Armourdale Land Use and Location

The Armourdale unit is located on the left bank of the Kansas River in Wyandotte County, Kansas and is encompassed by census tracts 425.01, 425.02, and 426, with a land area of 3.8 square miles. This unit protects the Armourdale area of Kansas City, Kansas, an area of mixed residential, commercial, industrial and public development There are several very large, complex manufacturing and commercial facilities (e.g., Proctor and Gamble, Colgate Palmolive), a power plant facility (Kansas City, Kansas Board of Public Utilities Kaw Power Station/Municipal Plant) and the Kansas City Southern and Union Pacific rail yards and main line tracks located in the study area. There are many small retail and commercial businesses typically found in and around residential neighborhoods, and more than 900 residential units in the area.

2.1.2 Armourdale Population, Income and Employment Characteristics

Population in the Armourdale unit decreased from 3,213 in 2000 to 2,924 in 2010 (a 9.9 percent decrease). Compared to Wyandotte County and Kansas City metropolitan residents in general, Armourdale residents on average are slightly younger and less educated. The median age for residents in the census tracts is 28.7. Only 52.1 percent of residents above the age of 25 are high school graduates. High school graduation rates for Wyandotte County and the Kansas City MSA are 78.6 percent and 90.0 percent respectively. Almost 65 percent of Armourdale residents classify themselves as Hispanic or Latino, which is significantly higher than the city (8.2 percent) and county (26.4 percent) percentages.

The median household income for Armourdale households in 2010 was $32,813, much lower than Wyandotte County ($38,503) and the Kansas City MSA ($55,749) households. Moreover, 34 percent of residents find themselves living below the poverty level with a per capita income of $11,261.

2.1.3 Armourdale Housing Characteristics

The 1,025 housing units in the Armourdale unit had a vacancy rate of 15 percent, higher than the 12.5 percent vacancy rate for Wyandotte County and the 9.5 percent rate for the Kansas City MSA. The median value of owner-occupied housing units in 2010 was $40,700 compared with a median value of $97,600 for Wyandotte County and $158,000 for the Kansas City MSA. More than 33 percent of the housing units in the Armourdale Levee unit were built before 1940 versus 20.8 percent for Wyandotte County and 13.1 percent for the Kansas City MSA.

2.2 Central Industrial District Unit

2.2.1 CID Land Use and Location

The Central Industrial District unit is located on the right banks of the Missouri and Kansas Rivers near their confluence. The protected area lies on both sides of the state line between Missouri and Kansas, and includes most of the central industrial districts of both the City of Kansas City, Missouri (Jackson County portion) and the City of Kansas City, Kansas in

Wyandotte County. The area encompasses census tracts 152 (Missouri) and 400.02 (Kansas), with a total land area of 1.8 square miles.

The CID contains commercial, industrial, and public type development. This is an older, historical area, that was devastated during the 1951 flood, but that has been experiencing recent development, revitalization, and renovation of existing commercial and industrial properties. There has also been high interest in recent years in developing and using some of the large older commercial/industrial buildings as residential loft space. The higher value investment is in two convention centers/entertainment venues (American Royal Building and Kemper Arena), some large warehouse facilities, several industrial sites, a few large commercial businesses, and public works facilities. The area is also the home to the world headquarters of Butler Manufacturing.

2.2.2 CID Population, Income, and Employment Characteristics

In 2010, the Central Industrial District had a population of 1,730 representing a noteworthy increase (84.8 percent) over the 2000 population of 936. CID residents appear to be slightly younger and more educated than Kansas City MSA, Wyandotte County, and Jackson County residents with a median age of 28.7 and high school graduation rate of 93.5 percent. The predominately white racial distribution of the population (77.2 percent) is similar to that of the Kansas City MSA (78.4 percent) but higher than percentages for Wyandotte County (54.6 percent) and Jackson County (66.9 percent).

The unemployment rate (6.3 percent) for CID residents is slightly lower than comparable county and city rates, while the percentage of individuals living below poverty level (11.8 percent) is analogous to the Jackson County (11.9 percent) and Kansas City MSA (11.1 percent) rates. However, the per capita income of $36,707 for area residents compares favorably to both county and city levels.

2.2.3 CID Housing Characteristics

The average owner-occupied home value of $227,400 is considerably higher than the average values for Wyandotte County, Jackson County, and the Kansas City MSA and the percentage of renter-occupied units in the area (81.7 percent) is also significantly higher than all comparable levels. These numbers are primarily due to the recent influx of new apartment and loft development in the CID area, which coincides with the appreciably lower percentages of residents living in the same unit as five years prior and smaller average household size.

2.3 Study Area Economy and Access

The Kansas City metropolitan area has a diverse and varied economic base. As a centrally located market, it is a major warehouse and distribution center and a leading agribusiness center. It ranks first in the nation as a farm distribution center and as a market for hard wheat. In addition to its agribusiness activities, the metropolitan area has major industrial activities such as auto and truck assembly, steel and metal fabrication, envelope and greeting card production, and

food processing. The metropolitan area also fosters a growing non-manufacturing sector. Wholesale and retail industries and service organizations are now chief employers in the area.

The metropolitan area has a major network of interstates and major highways that provides excellent access to each of the levee units. The CID unit is accessed by means of Interstate 70 on the north, by Interstate 35 on the West, and by Interstate 670, which crosses the center portion of the protected area. U.S. Highway 69 and Interstate 35 provide access to the Argentine unit, and U.S. 69, U.S. 169, and Interstate 70 serve the Armourdale unit. Interstate 70 and the Fairfax Bridge/U.S. 69 provide major highway access to the Fairfax-Jersey Creek unit. Missouri Highway 210, Burlington Avenue, the Bond and Heart of America Bridges, and Interstates 35 and 435 provide access to the North Kansas City unit. The East Bottoms unit is served by Interstates 29, 35, and 435, and the Birmingham unit has ready access by means of Missouri Highway 210 and Interstates 29, 35 and 435. Kansas City International Airport, less than 20 miles north of the study area, is easily accessible via the interstate system. Major rail service is available to each of the units, and the Charles B. Wheeler (Downtown) Airport is located in the North Kansas City unit. The Greater Kansas City Area is generally considered to be the nation's second largest rail center, second only to Chicago. The trunk lines serving Kansas City have main line tracks in the areas protected by the Kansas City Levees. Greater Kansas City is also among the top five trucking centers in the nation.

Table 7 summarizes the study area's industrial structure according to the percentage employed in each industry. The figures below are based on 2007-2011 American Community Survey 5-Year Estimates.

Table 7: **Study Area Employment by Industry**

Workers By Industry	Armourdale	Central Industrial District	CID - Turkey Creek	Argentine	North Kansas City	East Bottoms	% of Study Area
Civilian Employed Population (Age 16+)	1,090	1,086	5,480	1,167	2,463	1,221	12,507
Agriculture, forestry, fishing and hunting, and mining	16	0	9	19	14	0	0.5%
Construction	194	19	503	44	92	78	7.4%
Manufacturing	198	168	558	201	264	198	12.7%
Wholesale trade	19	25	95	70	214	75	4.0%
Retail trade	82	107	553	99	188	93	9.0%
Transportation and warehousing, and utilities	69	60	194	58	69	54	4.0%
Information	0	113	216	0	62	18	3.3%
Finance and insurance, and real estate and rental and leasing	17	106	421	50	202	42	6.7%
Professional, scientific, management, and administrative	136	182	812	113	273	182	13.6%
Educational services, and health care, and social assistance	129	183	953	308	536	182	18.3%
Arts, entertainment, and recreation, and accommodation, and food services	69	86	745	81	410	181	12.6%
Other services, except public administration	147	29	286	32	81	118	5.5%
Public administration	14	8	135	92	58	0	2.5%

2.4 Study Area Investment

Table 8 below shows a summary of study area investment as reported in the Phase 1 Interim Feasibility Report. The price level for the report was October 2004. Table 9 provides the study area investment totals in current dollars based on the most recent survey data for the Phase 2 units and updating the Phase 1 units to the October 2012 price level.

As indicated above in relation to Table 3, after the Phase 1 report was completed, it was determined that the area in the Turkey Creek floodplain along Southwest Boulevard needed to be added to the CID unit to capture the potential effects of river flows escaping the CID unit. The totals for this area that was not considered during Phase 1 are broken out from the remainder of the CID in Table 10.

In accordance with Section 308 of the Water Resources Development Act of 1990, the structure inventory reflected in Tables 8 through 10 includes no structures located within the 1 percent flood plain, with first-floor elevations below the 1 percent flood event that have been built new or have been substantially improved since 1999. Such structures also are not included in the benefits prepared for this economic analysis.

Table 8: **Interim Feasibility Report Investment**

Levee Unit	Number of Structures	Structure/ Infrastructure Investment	Cotents/Other Investment	Levee Unit Totals
Argentine	723	$ 588.0	$ 1,898.0	$ 2,486.0
Armourdale	**1,349**	**$ 628.0**	**$ 1,555.0**	**$ 2,182.0**
CID	**287**	**$ 386.0**	**$ 377.0**	**$ 763.0**
Fairfax-Jersey Creek	348	$ 656.0	$ 2,303.0	$ 2,960.0
North Kansas City	1,658	$ 1,438.0	$ 1,519.0	$ 2,957.0
East Bottoms	751	$ 1,438.0	$ 2,981.0	$ 4,561.0
Birmingham	209	$ 1,580.0	$ 126.0	$ 386.0
Study Area Totals	**5,325**	**$ 6,714.0**	**$ 10,759.0**	**$ 16,295.0**

October 2004 prices ($ Million)

Table 9: **2012 Updated Investment**

Levee Unit	Number of Structures	Structure/ Infrastructure Investment	Cotents/Other Investment	Levee Unit Totals
Argentine	723	$ 775.3	$ 2,277.8	$ 3,053.1
Armourdale	**1,468**	**$ 1,241.4**	**$ 1,320.5**	**$ 2,561.9**
CID	**526**	**$ 1,067.7**	**$ 1,747.8**	**$ 2,815.5**
Fairfax-Jersey Creek	348	$ 864.9	$ 2,763.8	$ 3,628.8
North Kansas City	1,658	$ 1,896.0	$ 1,823.0	$ 3,719.0
East Bottoms	751	$ 2,083.2	$ 3,577.5	$ 5,660.7
Birmingham	209	$ 342.8	$ 151.2	$ 494.0
Study Area Totals	**5,683**	**$ 8,271.3**	**$ 13,661.6**	**$ 21,933.0**

October 2012 prices ($ Million)

The Phase 2 study area collectively protects property with an estimated value of $5.38 billion (October 2012 prices), as summarized in Table 10. This total includes 951 residential structures and 1,043 businesses and public facilities as well as 279 miles of streets and railroads that would be subject to flood damage.

Table 10: **Phase 2 Study Area Investment**

	Armourdale	Central Industrial District	CID - Turkey Creek	Phase 2 Total
Non-Residential (businesses and public facilities)				
Quantity	538	341	164	**1,043**
Structures	$ 388,508.4	$ 547,169.7	$ 202,020.1	$ 1,137,698.2
Contents	$ 1,260,810.4	$ 1,431,042.6	$ 307,821.9	$ 2,999,674.9
Other	$ 22,700.5	$ 3,304.9	$ 0.7	$ 26,006.1
Total Value	**$ 1,672,019.3**	**$ 1,981,517.2**	**$ 509,842.7**	**$ 4,163,379.2**
Residential				
Quantity	930	0	21	**951**
Structures	$ 52,882.1	$ -	$ 8,824.7	$ 61,706.8
Contents	$ 26,396.9	$ -	$ 3,886.1	$ 30,283.0
Other	$ 10,576.4	$ -	$ 1,764.9	$ 12,341.4
Total Value	**$ 89,855.4**	**$ -**	**$ 14,475.7**	**$ 104,331.1**
Roads & Streets (railroads, highways, city streets)				
Miles	167.0	73.4	39.0	**279.4**
Total Value	$ 799,978.9	$ 243,507.6	$ 66,147.1	$ 1,109,633.6
Total Value	**$ 2,561,853.6**	**$ 2,225,024.8**	**$ 590,465.5**	**$ 5,377,343.9**

October 2012 prices ($1,000s)

3.0 Phase 1 Economic Justification Update

3.1 Purpose and Scope of Update

Although the Phase 1 units are not the subject of the present report, we are presenting updated investment figures for these units so that they will be on the same basis as the Phase 2 units discussed in this report.

The last approved economic justification data of record for the Kansas Citys Levees project is the FY12 NWK Economic Update covering the Phase 1 units. This update was approved by CENWD in July 2012. The economic update was a Level 1 update under the current methodology for updating benefit-to-cost ratios for budget development guidance. A Level 1 update is a reaffirmation that the last approved set of published benefits, in this case the 2006 Chief's Report for the Phase 1 units, remains valid based on primarily qualitative analysis. There were no significant changes in Phase 1 assumptions concerning economic development, hydrologic/hydraulic, structural and geotechnical engineering parameters, NEPA requirements, or plan formulation. An updated cost estimate was prepared for the update and used in the benefit-cost analysis.

For the update, a field survey was completed by NWK staff in July-August 2011 noting changes in the property base throughout the study area since the feasibility study. Specifically, we noted (1) changes in occupancy for existing structures; (2) obvious changes in activity level at existing businesses; (3) changes in the type of land use; and (4) new construction. The windshield survey was supplemented by additional research on the internet concerning occupancy. In general, relatively few changes in the economic property base had occurred in the Kansas Citys Levees area since publication of the Chief's Report. None of the developments that had occurred would materially alter the assumptions that framed and supported the 2006 economic analysis. Thus, there was no compelling reason to change the estimate of annual benefits.

3.2 Phase 1 Project Benefits and Costs

3.2.1 Phase 1 Project Benefits as of 2012

Since there was no compelling reason to change the estimate of annual benefits, the previously reported benefits total of $41,336,400, reflecting a 7 percent interest rate, was maintained. At the current Federal water resources interest rate of 3.75 percent, the benefits total $41,444,700 using the same data as shown in Table 11. These totals reflect the October 2005 price level used in the feasibility report.

3.2.2 Phase 1 Project Costs as of 2012

The current estimate of total Phase 1 project costs, in October 2012 prices, is $96,697,900. This total represents an increase of 21.7 percent over the total of $79,431,000 in the approved Chief's Report. Using CWCCIS (31 March 2012 version) index numbers from the composite account for October 2005 and October 2012, the price level increase is 23.8 percent. Thus, the Phase 1 project cost changes are generally nominal rather than real cost adjustments. Note that the first costs have been deflated to October 2005 prices so that the annual costs will be on an equivalent basis to the annual benefits.

Downward adjustments in costs to North Kansas City were the result of the need for fewer relief wells than originally proposed. An existing pump station, eliminating the need to construct a new pump station, can handle the resulting lower well flows resulting in significant cost savings. In all, the total first costs have dropped from $8.2 million to $5.5 million.

First costs for the Argentine unit have not increased more than those for the overall project, but assumptions for interest during construction and annual cost savings have changed. It is now recognized that work on the Argentine unit cannot begin until after completion of Phase II work on the CID and Armourdale units. Project completion for the Argentine unit has therefore been pushed out to 2026, with a significant increase in interest during construction. This has increased Argentine's total first costs from $52.9 million to $63.3 million.

For the Fairfax-Jersey Creek unit, the estimate for addressing the structural/geotechnical issues identified has increased. It should be noted, however, that this portion of the current cost estimate is in progress and is at a more preliminary level than the rest of the total project

estimate. As it currently stands, the total first costs have risen from $16.7 million to $25.2 million.

3.2.3 Phase 1 Benefit-Cost Ratio

At the current FY 2013 Federal interest rate of 3.75 percent, the benefit-cost ratio for the Phase 1 project is 9.5, as shown in Table 11, with net annual benefits of $37,076,400. In the approved Chief's Report, the ratio for the then-current interest rate of 5.125 percent was 8.0. The drop in interest rates since then has resulted in a higher benefit-cost ratio at the current rate.

At 7 percent, the benefit-cost ratio in the Chief's Report was 6.0, while the 7 percent benefit cost-ratio is 5.4 in this update. Since benefits are unchanged, the growth in annual costs has accounted for the drop in the benefit-cost ratio, which nevertheless remains strong. Net benefits total $33,724,000.

It should be emphasized, once more, that the discussion provided here in section 3 is primarily in reference to the economic survey and analysis supporting the previously completed study and authorized project for Phase 1, as well as the economic update of the Phase 1 project approved in 2012, and the newly-added inventory for the Turkey Creek portion of the CID unit. The primary relevance of this discussion to the Phase 2 units is to portray the surveyed investment for all portions of the Kansas Citys project (both Phase 1 and 2) on an equivalent price level basis.

Table 11: **Phase 1 Economic Justification**

October 2005 prices (First Costs October 2012); $1,000's				
	Interest Rate = 3.75%		**Interest Rate = 7%**	
	Total	Remaining	Total	Remaining
Argentine				
First Costs	$63,293.7	$63,293.7	$63,293.7	$63,293.7
Annual Benefits	$18,177.6	$18,177.6	$18,148.8	$18,148.8
Annual Costs	$ 3,066.2	$ 3,066.2	$ 5,483.1	$ 5,483.1
Benefit-Cost Ratio	5.9	5.6	3.3	3.3
Net Benefits	$15,111.4	$14,957.3	$12,665.7	$12,665.7
Fairfax-Jersey Creek				
First Costs	$25,228.8	$18,761.6	$25,228.8	$18,761.6
Annual Benefits	$12,023.6	$12,023.6	$11,991.0	$11,991.0
Annual Costs	$ 960.0	$ 725.7	$ 1,616.7	$ 1,235.8
Benefit-Cost Ratio	12.5	16.6	7.4	9.7
Net Benefits	$11,063.6	$11,297.9	$10,374.3	$10,755.2
East Bottoms				
First Costs	$ 2,001.1	$ 1,786.6	$ 2,001.1	$ 1,786.6
Annual Benefits	$ 4,363.2	$ 4,363.2	$ 4,344.2	$ 4,344.2
Annual Costs	$ 104.0	$ 96.3	$ 153.8	$ 141.2
Benefit-Cost Ratio	41.9	45.3	28.2	30.8
Net Benefits	$ 4,259.2	$ 4,266.9	$ 4,190.4	$ 4,203.0
North Kansas City				
First Costs	$ 5,544.2	$ 2,793.5	$ 5,544.2	$ 2,793.5
Annual Benefits	$ 6,880.3	$ 6,880.3	$ 6,852.4	$ 6,852.4
Annual Costs	$ 238.1	$ 138.4	$ 358.6	$ 196.6
Benefit-Cost Ratio	28.9	49.7	19.1	34.9
Net Benefits	$ 6,642.2	$ 6,741.9	$ 6,493.8	$ 6,655.8
Phase 1 Project Total				
First Costs	$96,697.9	$87,265.4	$96,697.9	$87,265.4
Annual Benefits	$41,444.7	$41,444.7	$41,336.4	$41,336.4
Annual Costs	$ 4,368.3	$ 4,026.6	$ 7,612.4	$ 7,056.8
Benefit-Cost Ratio	9.5	10.3	5.4	5.9
Net Benefits	$37,076.4	$37,418.1	$33,724.0	$34,279.6

4.0 Previous Flood Events

4.1 Pre-1929 Flooding

Floods in the Missouri and Kansas River Basin carry great quantities of silt and debris, and are of comparatively low velocity and of several days duration. Flow data at the USGS gauge on the Hannibal Bridge in Kansas City is available for the period 1929 to present. Before 1929 the

major flood events in the Kansas Citys area occurred in 1844 (believed to have been approximately 17.0 feet above flood stage), 1881 (6.8 feet above), 1903 (14.0 feet above), and 1908 (9.3 feet above). Although the 1844 event is considered the greatest known event in the lower Missouri Basin, there was little development in the area. However, the wharves at nearby Independence, Missouri were destroyed, and Westport Landing (early downtown Kansas City area) thus gained most of the Santa Fe Trail trade. In the 1903 flood, 19 lives were lost in the Kansas Citys area, and an estimated $23,000,000 in property damages (1903 prices) was sustained. The flood of 1903 had an estimated discharge of 543,000 cubic feet per second (cfs).

4.2 The 1951 Flood

The 1951 flood, with a Missouri River discharge of 573,000 cfs, and 469,000 cfs on the Kansas River, exceeded the other previous events except for the flood of 1844. A two-month period of above-normal precipitation followed by unprecedented intense rains over a 72-hour period in early July caused the flooding. In the early morning hours of Friday, July 13, 1951, the Kansas River poured over the dikes in the Argentine District and about 2,000 residents fled to nearby bluffs. Early that morning, too, after more than 9,100 people were evacuated from the Armourdale district, water began to overtop a 4-mile stretch of the levee and inundated the Armourdale area with depths of 15 to 30 feet. On Kansas Avenue, the floodwater was reported to be "waist-high on top of a two story building." About 400-800 people who had decided to stay had to be rescued by boats, out of trees, and from ledges and rooftops. Intense sandbagging efforts to save the West Bottoms failed and later that morning, the Central Industrial District was flooded. In the East and West Bottoms areas, manufacturing and wholesale districts, railroad yards and the Kansas City stockyards were devastated. Packing plants were flooded, and the floodwaters swept away thousands of hogs and cattle. Railroad transportation was halted due to the flooding with severe damage to tracks, rail cars, and rail yards. The American Royal building was inundated by 15 feet of water. Only two highway bridges remained in operation in the area, and runaway barges were a threat to these remaining bridges.

The flood threat moved on to the Municipal Airport (now the Charles B. Wheeler (Downtown) Airport), the Fairfax District and North Kansas City by Friday night. Planes were evacuated, and about 4,000 North Kansas City residents were ordered to evacuate. Although work to support the dike using bulldozers and trucks continued through the night, the Jersey Creek dike collapsed early on Saturday July 15, and water poured into the Fairfax District. In an effort to protect the downtown airport and Municipal Air terminal, junked cars were dumped onto levees. Of the five industrial districts, only North Kansas City was completely saved (the Municipal Air Terminal escaped the worst of the damage). Emergency operations also prevented flooding of the Northeast (East Bottoms) and Birmingham Industrial Districts. Water stood for several days in the flooded units and the Kansas River stretched from the Armourdale bluff to the Argentine bluff, with very little to be seen above the floodwater. About 11 square miles were flooded in the Kansas Citys area. Although at least 5 persons died in the Kansas Citys area, about 15,000 people were evacuated. Many of these residents were left homeless and were relocated to trailers

and other temporary housing, some for nearly two years. According to the Kansas City District's post-flood report for the Kansas Citys 7 levees area, the flood caused an estimated $461 million in damage (roughly $8 billion in 2013 prices). In addition to at least 5 deaths and 17,500 evacuations, more than 5,700 homes and nearly 1,500 businesses were flooded in the 7 levees area alone. Consequently, July 13, 1951 became known as "Black Friday."

4.3 The 1993 Flood

The 1993 flood event crested at 48.9 feet on July 27, 1993, with a Missouri River discharge of 543,000 cfs. Despite the discharge being less than for the 1951 flood, the 1993 crest of 48.9 feet exceeded the 1951 crest stage of 46.2 feet. All the levees in the Kansas Citys project held, although water levels on several units were encroaching in established freeboard. Every one of the levees sustained some damage. An estimated $4.57 billion in damages were prevented by the Kansas Citys Federal levee system (The Great Flood of 1993, Post-Flood Report, U.S. Army Corps of Engineers, Sept. 1994). Main stem reservoirs on the Missouri above Kansas City prevented an estimated $3.8 billion in damages, much of that in the Kansas Citys area. Just outside the study area in Kansas City, Kansas, several low-lying trailer courts and other homes near Kansas River mile 10 were damaged or destroyed. An estimated 600 mobile homes and 200 other homes were affected. Damages to Kansas City, Kansas utilities reached several million dollars. Kansas City, Missouri reported more than $15 million in damage to public infrastructure. Kemper Arena and the American Royal Building suffered about $2.5 million in water damage to flooring and electrical circuits. The downtown airport sustained damages of nearly $3 million, and pollution control and public works facilities sustained an estimated $8 million in damage. Since the levees in the Kansas Citys project did not overtop or breach, these reported damages sustained were due to underseepage, interior drainage, and possibly Turkey Creek overbank flows.

5.0 Damage Analysis Database Preparation

5.1 Data Collection Methodology

The data collection effort for Phase 1 carried out primarily in 2002-2004, included data for all seven of the Kansas Citys project leveed areas, including the Phase 2 units. The Phase 2 data collection involved three steps: (1) evaluation of the data obtained during Phase 1 of the feasibility study; (2) obtaining structure characteristics from relevant county and state tax records, and GIS data and available mapping from the city and/or county; and (3) design and execution of a structure-by-structure field survey.

It is important to note that the data collection methodology for Phase 2 represented as few changes as possible over the Phase 1 methodology. The Phase 1 project was approved by ASA (CW) and was subsequently authorized; various portions of the project are now in design or construction. It would be inappropriate to produce a Phase 2 economic analysis that is methodologically inconsistent with the Phase 1 analysis. Apart from correcting a handful of

minor errors found in the Phase 1 analysis and using the vehicles depth-damage function released in EGM 09-04 in 2009, the data collection and development methodology for Phase 2 are identical to Phase 1.

5.1.1 Phase 1 Survey

Due to the massive extent of the investment in the study area, intense efforts were required to prepare for, closely manage, and coordinate, conduct, and complete the economic field survey for the Kansas Citys feasibility study to determine study area investment and its damage potential. For Phase I of the survey, the data collection efforts for the commercial, industrial, and public facilities were accomplished by architectural-engineering (AE) contract. Corps in-house economics staff members completed the data collection for residential investment, public investment in streets and highways, and commercial investment in railroad tracks, with contract assistance for research and data input. These surveys were detailed in the socioeconomics appendix to the 2006 feasibility report, but the information is repeated here for reference.

5.1.1.1 Phase 1 Commercial, Industrial, and Public Facility Economic Data Collection

Economic data collection efforts for the commercial, industrial, and public facilities were based on a mix of direct interviews of large, high value businesses, direct interviews of a representative sample of other typical businesses in the study area, visual field observation, estimates based on similar investment and damages for comparable types of businesses, and visual observation and estimates using Marshall and Swift commercial valuation software. Business specific data obtained during the reconnaissance phase were also evaluated for use in the feasibility study.

For Phase 1 of the survey, the survey team leader (an experienced former Corps economist) conducted an initial windshield survey of all development in each levee unit, with extensive identification of individual major businesses. Based on several factors, including his visual observation, the lists of major businesses identified by the study sponsor representatives, available reconnaissance phase data, and the color-coded parcel valuation maps prepared by the GIS staff, the survey team leader identified the largest and/or highest value businesses in each levee unit and a mix of other businesses that would comprise a representative sample of typical businesses in the study area. These businesses were compiled as a "master list" of commercial, industrial and public properties that would be given priority for data acquisition. From the master list, the survey team leader determined an initial subset of these master list businesses that either would specifically need to be interviewed due to their size and complex nature, or would be included in order to develop and interview a representative sample of the typical business types found in the study area. Survey team members were then sent out with survey forms for face-to-face interviews with the specifically identified master list businesses.

Survey forms completed and returned provided detailed information about property values, location of damageable investment, and damageability of the investment at various depths of flooding in relation to the first floor. Data included the type of business, depreciated structure investment value, investment values by physical location (basement, first floor, second floor) for

inventory, office equipment, production equipment, and other contents. Survey data also included estimates of potential damage to structure, inventory, equipment and other contents with various potential depths of flooding in relation to first floors. Information on historical flood events and historical damages were obtained in some cases.

The AE contractor developed descriptive and location data for each master list business by visual observation during a windshield survey, review of the aerial survey maps, and available Phase 1 field notes. The data items developed included the following: levee unit location and river mile location of structure, structure number, name and address of business occupant as available, number of buildings, ground elevation and first floor above ground height, type of construction material, estimated effective age and condition of the building. Ground elevation for each structure was determined based on the aerial maps with either 2-foot contours and spot elevations (Kansas) or 4-foot contours and spot elevations (Missouri).

The investment values for structure and contents for the master list businesses were completed using the methodologies proposed and described above. For other businesses, the estimated valuations were based either on a unit cost per square foot from similar business types that had returned survey forms or by using the Marshall and Swift Commercial Estimator 7 computer program. The required data input included zip code, stories in building, total building area, occupancy group, occupancy type, occupancy code number, occupancy percentage, story height, construction class, and quality.

Descriptive and location data for the remaining businesses and public entities (not on the master list) in the rest of the study area were obtained by windshield survey and from available mapping. Each structure or group of structures was assigned a structure number corresponding to the aerial map structure number for identification purposes. Square footage estimates were calculated by scaling the structure footprint outline shown on the map, combined with descriptive data from the visual survey. Quality control review of such calculations was conducted by comparing square footages for a sample of the structures to available GIS data on square footage. First floor heights above ground, structure ground elevations, and low entry elevations were identified using the aerial maps and visual inspections. Valuation estimates were developed either by using Marshall and Swift estimation software or by estimating based on locally-obtained data for similar structures/investment types.

5.1.1.2 Phase 1 Residential Data Collection

Corps of Engineers Economics staff conducted a field survey of residential structures in the study area. Local realtors in both Kansas and Missouri who sell homes in the levee unit areas were contacted to obtain the typical sales prices for residences by type of residential structure (one-story with basement, two-story without basement, etc.). Realtors also provided typical market values for residential lots in the individual market areas. For comparison purposes, Multiple Listing Service data was also obtained from a real estate appraiser about recent comparable sales for residential units in the levee units. To verify that the realtor provided

market values were reasonable and representative of depreciated replacement values, Marshall & Swift depreciated replacement values were developed for a random sample of residential structures. The realtor-provided market values were determined to reasonably reflect depreciated replacement value. Based on the values provided by the local realtors, typical residential structure market values (not including the typical market land value) were developed by structure type and by levee unit area.

Elevations of the lowest openings and first floors relative to the ground were noted by visual observation of structures during the field survey. Ground elevations for residential structures were determined from the available mapping.

5.1.2 Tax Data

Property parcels within the 0.2 percent-chance floodplain were identified and looked up in their respective county appraiser's property database. The Wyandotte County Appraiser's property database provided assessment values along the necessary structure details to calculate replacement costs. This data included structure characteristics such as occupancy, age, construction quality/class, area (square feet), number of stories, exterior wall type, basement type, garage type, and condition. The Jackson County, Missouri Appraiser's property database also provided structure assessment values but no supporting data. Values from the tax data were updated as the study progressed.

5.1.3 Phase 2 Field Survey

Kansas City District economics staff carried out a structure-by-structure field survey of all buildings in the seven levee areas (both Phase 1 and Phase 2 units) over several days in July-August 2011. The purpose of the survey was to build on the data from the Phase 1 survey and county tax records and obtain a generalized reality check on the usefulness of the appraised values by assessing whether obvious mismatches between data and reality occurred. Specifically we looked for any obvious discrepancies in existing data, changes in structure occupancies, or changes in activity level at existing businesses. Finally, the nature of the activity was not always obvious from the tax data or business name, so properties were inspected for additional clues, as well as for the presence of significant outside inventory or equipment. The windshield survey was supplemented by additional research on the Internet concerning occupancy. Notes from the completed field survey were subsequently integrated with the tax data to form an adjusted initial structure inventory for the study area.

5.2 Data Development – Elevations

In the second phase of the database preparation for the economic analysis, the raw data obtained from the county tax records, GIS maps, field surveys and business surveys were further developed, refined, and organized to produce the three key variables for each property to be used in the damage analysis: beginning damage elevations, property values, and depth-damage relationships. The risk analysis program used for the damage analysis also requires specification of uncertainty factors for each of these variables.

Each property in a flood risk management analysis is assigned a mean sea level (msl) ground elevation. This includes streets and railroads as well as buildings. Buildings additionally are assigned a first-floor elevation expressed as a foundation height above the ground elevation. Damage computations take into account ground elevation, first-floor elevation, and lowest-opening elevation if it is different from the other two elevations. Property elevations help determine depths of flooding for each flood event evaluated.

Each structure in the study area was assigned a ground elevation using the contour maps. All structures were also assigned a station or river mile for the purpose of allowing the stage-damage relationship for the structure to be transferred to the index point of the reach in the damage analysis.

In addition to the ground elevations and stations, each structure was also assigned a foundation height relative to the ground elevation. The foundation heights were estimated in half-foot intervals by visual observation during the field survey. The first-floor elevation (which is usually the beginning damage elevation) in the economic analysis model was determined by adding the foundation height to the ground elevation.

Using a flooded area map based on a 0.2 percent chance event, all city streets in the floodplain were evaluated on a block-by-block basis, assigning an average elevation for each block. Highways and railroad track were similarly divided into short segments and assigned an average elevation for each section.

The first-floor elevation for each type of structure is characterized by an uncertainty factor, usually expressed as a standard deviation around a normally distributed variable. Ground elevations were assigned in accordance with EM 1110-2-1619 dated 1 August 1996. Based on contour interval mapping for each levee unit, the recommended standard deviations and normal distributions were assigned (0.3 feet for Kansas units and 0.6 feet for Missouri units).

5.3 Data Development – Valuation

Guidance for Corps of Engineers economic analyses defines asset value of depreciated replacement value, which is defined as the cost of replacing an item today with an item of identical effective age (i.e., not a brand new item, unless the item being replaced is brand new). As the term implies, the concept is to identify the replacement cost of the item and then depreciate this value according to the item's condition and age. This concept of value is applied to values for all structures, whether residential or non-residential, as well as major production or office equipment and vehicles. Inventories of businesses, including raw materials, work in progress, and furnished goods, are valued in terms of replacement value.

The economic expression of values for each property category also must include uncertainty factors to be used in the risk analysis. Most economic variables in flood risk management studies are believed to be distributed normally, so the uncertainty around a median value is expressed as a standard deviation. In cases where the samples available for estimating variables

are very small and the distribution of the variable is unknown, uncertainty may instead be expressed as a triangular distribution with most likely, maximum, and minimum value estimates.

5.3.1 Residential Structures Valuation

For this analysis, a detailed RS Means analysis of depreciated replacement value was performed on all homes in the Armourdale and CID floodplain. The detailed data obtained from the county tax database on each of the homes included square footage, construction type, number of stories, wall type, basements, garages, porches, heating and air conditioning, floor coverings and interior walls, and lump sum adjustments such as number of bathrooms and fireplaces. The RS Means database provides values associated with each of these characteristics and allows computation of detailed replacement values for each home. Additional data from the tax database on age and condition were then used in conjunction with the RS Means material to estimate depreciation and calculate a depreciated replacement value.

Structure value uncertainty in this analysis generally is related to uncertainty in either the assessment of residential construction quality or depreciation estimates. Uncertainties were determined from the range of values provided by local realtors contacted for the different structure types and levee unit areas.

5.3.2 Residential Contents Valuation

Residential content values are normally expressed in terms of a contents-to-structure value ratio (CSVR). For example, if a home appraised at $100,000 has a CSVR of 0.5, the home is assumed to have contents valued at $50,000. The CSVR is a standard technique used in the insurance industry for estimating contents values in the absence of detailed data. Due to the nature of the residential depth-damage relationships (developed by the Institute for Water Resources described further below) used in this analysis, a nominal residential CSVR of 1.0 is used in the risk analysis, following the guidance accompanying the IWR functions. The IWR functions are formulated so that no CSVR is actually used to compute content values in the analysis (the function of the nominal CSVR of 1.0 is only to ensure that the depth-damage functions result in correct calculations in the risk analysis). For purely informational purposes of estimating investment values in the study area, residential contents value is assumed to equal 50 percent of structure value. The IWR functions are used for all 1, 1 ½, and 2-story homes, with or without basement.

Uncertainties in residential contents valuation are not specified in this analysis for those homes affected by the IWR functions, following the guidance for the functions warning against the use of any uncertainty factors because of how the functions are constructed and used in the risk analysis. That approach has been followed in this analysis.

For the residential other category in HEC-FDA, each residential unit was assumed to have a vehicle of typical average value and typical landscape investment subject to damage. Most families today own more than one vehicle, and with imminent threat of flooding, it is likely that

a family would load belongings into one of the vehicles and evacuate the area. Thus, for purposes of the study, vehicles subject to flood damage were limited to one per residential structure. Most homes in the protected areas have typical shrub plantings, lawns, and gardens that would also be damaged by flooding. The vehicle and typical landscaping investment value in the Residential Other category was assumed to be about 20 percent of structure value, with 5 percent as one standard deviation of error for uncertainty. In support of this ratio, we determined that the average used vehicle sale value in 2012 was approximately $9,300 (Automotive Dealer Exchange Services of America, September 2012). The $9,300 would be slightly more than 18 percent of the average home value in the Phase 2 study area of $51,100, and since the $9,300 does not account for those who have new cars and does not include landscaping, 20 percent of structure value appears to be a reasonable assumption for this category of damages.

5.3.3 Commercial Structures Valuation

The values of all commercial and public structures in Wyandotte County, Kansas inventoried for this analysis are estimated using information from the county tax database in conjunction with RSMeans commercial valuation reference products. Characteristics for each building taken from the tax database included occupancy type (e.g., warehouse, church, retail store, office building, etc.), construction quality, exterior wall, and square footage. These characteristics were the basis for calculation of replacement values for each structure using the RS Means reference data. The next step involved obtaining data on age, typical building life, and condition for each structure from the tax database and using these characteristics to develop effective age and corresponding depreciation factor. Application of the depreciation factor to the replacement value resulted in a depreciated replacement value for each building.

The values obtained from the Phase I survey for Jackson County, Missouri commercial and public structures were updated using CCI update factors and were then compared against the appraised values, field survey notes, and other sources such as internet, newspaper articles, and personal interviews with adjustments being made where necessary.

Structure value uncertainties were determined in Phase 1 by completing a set of Marshall & Swift valuations for a representative sample of master list businesses/properties that had returned completed survey forms. The Marshall & Swift valuations were compared to the survey values and a standard deviation was computed. The 30 businesses sampled for the uncertainty determinations were selected to be representative of the various types of structures located within the study area, and the process considered such factors as construction material, size, location, and building effective age. Based on statistical analysis of the sampled data set, a normal distribution was selected with a standard deviation of 41.3 percent. This standard deviation was used as the depreciated structure value uncertainty for all commercial structures within the study area that did not provide survey data.

5.3.4 Commercial Contents Valuation

Commercial and public contents include assets such as office equipment, major production equipment, and rolling stock, as well as inventories items including raw materials, work in progress, and finished goods. All properties in this analysis were assigned content values in terms of contents-to-structure-value ratio (CSVR). For firms and facilities that provided more detailed data to us via interviews and survey forms, this ratio was developed indirectly from data on asset and inventory values obtained from the companies. It should be stressed that in these cases, the structure and content values were developed first, and then the CSVRs were derived from those values. Although computation of CSVRs was an additional step not required for valuation of these businesses and facilities, the CSVRs were developed in order to treat all data in the database consistently (the majority of businesses derive content values from CSVRs) and facilitate simpler data handling for the risk analysis.

Since obtaining first-hand data from all companies in the study area would not be realistic, content values for the majority of businesses and facilities must be derived from something other than company-specific data. For contents valuation of these firms, this analysis primarily utilizes CSVRs developed by the New Orleans District Corps of Engineers, which has accomplished a great deal of analysis over several major studies concerning typical content values and depth-damage functions for both structures and contents in a broad range of industries. The data used in this analysis were published in the report "Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios in Support of the Lower Atchafalaya Reevaluation and Morganza to the Gulf, Louisiana, Feasibility Studies," dated May 1997. During the Phase I Interim Feasibility Report, this source was recommended by Corps subject experts from HQUSACE and other districts for use. The New Orleans report, which includes two sets of CSVRs for various business types, is probably the most methodical available attempt to develop CSVRs. One set of CSVRs was based on estimates by expert panels, while the other set was based on interviews with business owners or operators in the Baton Rouge area. The informative expert panel data from the report is limited somewhat by its use of only one prototypical business as a basis for estimating CSVRs in each broad industry category. We instead chose to use CSVRs from the New Orleans owner/operator data. These data were based on post-flood surveys conducted in the aftermath of an urban, freshwater, main stem (long duration) flood event in Louisiana. These flooding characteristics transfer well to the Kansas City context of flooding. The owner/operators interviewed represented many of the same types of businesses and facilities as are found in the Kansas City structure inventory. Seven broad business categories are included: restaurants, grocers, retail and services, professional offices, repairs and home use businesses, warehouses and contractors, and public facilities. Development of the owner/operator data for each of these categories included interviews with 10 businesses, usually representing several specific types of businesses within each broad category. Table 12 summarizes these ratios.

In contrast to residential valuation, values were not added to commercial and public contents as an "other" category to account for vehicles and landscaping. Each home is assumed to have vehicles, and many residents of the study area also work there. Therefore, the addition of vehicles at places of business would entail considerable double counting. Landscaping is not included since no generalized data are available relating to typical ratios of landscaping costs to business structure values.

Uncertainty in contents valuation for firms not contacted is assumed to be subject to a normal distribution and is characterized by standard deviations accompanying the CSVRs in the New Orleans data, as seen in Table 12.

Table 12: **Commercial Content-to-Structure Value Ratios**

Category	CSVR	Standard Deviation
Eating and Recreation	3.06	1.62
Groceries and Gas Stations	1.28	0.76
Professional Businesses	0.78	0.58
Public and Semi-Public	0.82	1.39
Repairs and Home Use	2.51	0.86
Retail and Personal Services	1.48	1.23
Warehouse and Contractor Services	3.72	1.45
Source: "Depth-Damage Relationships for Structures, Contents, and Vehicles and Content-to-Structure Value Ratios in Support of the Lower Atchafalaya Reevaluation and Morganza to the Gulf, Louisiana, Feasibility Studies," dated May 1997.		

5.3.5 Streets and Railroads Valuation

Roads are perhaps the most commonly damaged infrastructure facility in a flood event. Damage to roads and other paved surfaces may be caused by floodwaters overtopping, eroding and scouring road surfaces, shoulders, and embankment slopes. In addition to obvious washout areas, as the ground begins to dry out after flooding, pavement buckling and other problem areas can become apparent. Curbs, gutters and sidewalks along the streets and roads can be damaged by uprooted trees and by heavy equipment during cleanup. Also associated with road damage would be damage to traffic signs and stoplights. For purposes of this study, damages to roads and paved parking lots were estimated for the Armourdale and CID units. Damages to traffic signs and traffic signals were not included. Miles of roads by type and elevation for each levee unit were determined during the reconnaissance phase of the study. Estimates of the average investment per mile for the various types of roads were developed from road construction cost estimates obtained from the Missouri Department of Transportation and from representatives of Kansas City highway engineering firms, and were applied to the estimated miles of roadways by type in each levee unit area subject to flooding. Uncertainties in investment value were determined based on the ranges of values provided for the different types of roads. The analysis

uses a 20 percent maximum or minimum variation for interstates and heavy-duty concrete roads, and 35 percent variation for arterial and local/collector streets.

Data for railroad tracks were developed in a similar manner. Miles of track were determined by elevation and by levee unit area during the reconnaissance phase of the study. Estimates of investment value per mile for different types of track were obtained during interviews with study area railroad representatives. Investment per mile was estimated to range from $1 million to more than $2.5 million, depending on the number of electric time locking switches in the track. Since the study area encompasses heavily urbanized areas and major rail yards, it is reasonable to assume that track with electronic switching would be more prevalent than track without electronic switching. Based on values per mile and the miles of track with the different levels of electronic switching provided by railroad representatives interviewed, an average value per mile of $1.75 million (October 2004 price level) was developed and applied to the total miles of railroad tracks in the levee unit areas. This value has been updated to $2.275 million based on CWCCIS index numbers for Roads, Railroads, and Bridge. On average, approximately 42 percent of this value represents the value of main line tracks, and about 57 percent represents the value of electronic switching and other equipment. Uncertainties were based on the range of values obtained from the railroad representatives interviewed.

The Kansas Citys study area also contains significant rail yards and main line track. For Kansas River flooding, there is not much advance warning. Therefore, if a call went out to evacuate the study area, railroad representatives from different rail companies stated independently that it would be nearly impossible to move the boxcars out of the study area rail yards because there is not enough locomotive power available to move them in a short timeframe. The majority of cars and their commodities would be left on the track and thus subject to damage. For purposes of this analysis we estimated that approximately 25 percent of the boxcars would be moved out of harm's way, and that all locomotives would be moved out, except for locomotives in the repair facilities. Railroad car damage would mainly be damage to wheel assemblies once they get wet, along with some other minor damage to the cars themselves. The railroad representatives interviewed estimated the cost for replacement of flooded wheel assemblies at $80,000 ($97,900 in October 2013 prices) per car. Estimated numbers of rail cars in each yard were determined from examination of aerial photos and comparison with the information obtained from the Argentine rail yard. Boxcar commodities would also be damaged in a major flood event. To estimate boxcar commodities damage, the top inbound and outbound commodities by rail carload were researched for the Kansas City area, and a weighted value per rail car of $21,700 ($26,500 in October 2013 prices) was developed. The top commodities included motor vehicles and motor vehicle parts, grain and grain products, bituminous coal, miscellaneous coal and petroleum, and mineral products. The Kansas City rail yards are also major containerized shipping centers. Containers waiting processing were estimated by field observation, examining aerial photos, and counting stacks of containers. It was also assumed that approximately 25 percent of these containers would be moved in a major flood event. Container unit damage in a

major flood event was estimated at about 5 percent of total investment in containers for purposes of the analysis. Container commodity damage was assumed to be similar to boxcar damage; container content values and damages are for finished goods and would likely be much higher for raw materials.

5.3.6 Emergency and Disaster Relief Costs Valuation

These other costs of flooding are much more difficult to determine and estimate than physical flood damages. In the Kansas Citys study area, actual study area historical data about these types of costs are neither readily available nor easily estimated because the last damaging flood event in any of the study area units was in 1951. However, we estimated emergency costs for the study area units based on an evaluation of actual data collected about the 1993 flood along the Missouri and Mississippi Rivers. Several Corps published reports about the 1993 flood were consulted to obtain estimates of typical emergency costs in Missouri. These reports included the *1993 Interagency Floodplain Management Review Committee Report* (Galloway Report); *Impacts of the Great Flood of 1993*, U.S. Army Corps of Engineers Lower Mississippi Valley Division, May 1996; and the *Flood Plain Management Assessment of the Upper Mississippi River and Lower Missouri Rivers and Tributaries*, U. S. Army Corps of Engineers, June 1995. We specifically compared 1993 flood damages with 1993 agency emergency costs as reported in these documents. The 1993 emergency cost category data included the following: Federal Emergency Management Agency disaster administrative costs (costs of temporary disaster field offices and temporary hires, but not including costs for permanent administrative staff or permanent office and equipment costs), Department of Health and Human Services 1993 flood disaster costs, Corps flood emergency and emergency operations costs, and Environmental Protection Agency 1993 flood costs relative to underground storage tanks, oil spill response, and Abatement, Control, Compliance program operations. Based on the data provided in the reports, emergency costs, as a percent of total physical flood damages, ranged from a low of 12.4 percent to a high of 15 percent, with an average of 13.4 percent for all states impacted by the 1993 flood. We assumed that the 1993 flood data were typical for a flood event of that magnitude (approximately a 0.2 percent probability event) and that the data provided an historical basis for estimating these types of costs that could be incurred in the Kansas Citys highly developed urban study area for a flood event of similar magnitude. For the Kansas Citys feasibility study, emergency costs were estimated at 13 percent of primary damages for a 0.2 percent event. This percentage is similar to the average percentage described above for all states impacted in the 1993 flood and is also similar to the percentage used to estimate these costs in the Pearl River Study (USACE Mobile District). We did not obtain or include data about emergency costs for local police and emergency services. Estimated emergency costs for a 0.2 percent event were entered into the HEC-FDA study file for each levee unit area as the maximum emergency costs that could be incurred, and a depth percent damage relationship was applied in the HEC-FDA model to estimate emergency costs for other probability events. The equivalent annual emergency costs incurred over the period of analysis were computed in HEC-FDA.

5.3.7 Cleanup Costs

Based on data obtained from studies and approved reports by other Corps Districts, cleanup costs, with levee overtopping and approximately a 0.2 percent event in each of the Kansas Citys levee units, were estimated at 2 percent of total investment in structures and contents. Estimated cleanup costs for each levee unit were entered into the HEC-FDA study file, along with an appropriate depth-damage relationship, for integration with the hydrologic data and to determine annual cleanup costs incurred in each unit over the period of analysis.

5.3.8 Floodplain Relocation/Reoccupation Costs

Based on our research, the 1993 emergency costs described above included hazardous and toxic waste type cleanup costs, but in our opinion, did not include normal business and residential cleanup costs or relocation and reoccupation costs for floodplain residents. Relocation and reoccupation costs (and cleanup costs to a certain extent) were included in a different category identified as "disaster relief" in the 1993 flood data published in the three reports cited above. This disaster relief category reported all government human resource disaster relief payments. However, in our evaluation of the 1993 data, we determined that the 1993 disaster relief category data overlapped somewhat with actual physical flood damages and use of that data would result in the potential for double counting. Thus, estimates of costs for relocation and reoccupation of floodplains were instead obtained from extensive interviews with Federal Emergency Management Agency (FEMA) officials and published FEMA data for recent Missouri declared disasters, including the 1993 flood in Missouri. FEMA estimates were based on monies provided for disaster housing assistance and individual and family grant assistance. Estimated FEMA assistance per disaster per housing unit ranged from a low of $5,500 to a high of nearly $16,000, and an estimated average of $7,500 per housing unit. The average cost per housing unit was applied to the number of housing units in each of the Kansas Citys levee unit areas for use in this study as an estimate of relocation and reoccupation of floodplain costs that occur with floods of the magnitude of the 1993 event. This average value per housing unit was also comparable to a value computed per vulnerable residence in the floodplain based on information and data for Missouri counties provided in the *1993 Interagency Floodplain Management Review Committee Report* noted above. Relocation and reoccupation costs for non-residential occupants (commercial, industrial, public) were not estimated and were not included in the analysis.

5.3.9 Traffic Disruption Costs

Flooding or even the threat of flooding and public safety concerns may cause road closures and detouring of traffic. Traffic detours can last for the duration of actual flooding plus the time required for road cleanup and road repairs. As described in *National Economic Development Procedures Manual-Urban Flood Damage*, IWR Report 88-R-2, March 1988, the costs of traffic disruption are based on the vehicle operating costs for the additional miles traveled because of the detour, and on traffic delay costs per passenger. Lowest point elevations for major routes in each unit were compared with levee unit overtopping elevations and flood event interior stages with overtopping to determine which roads would be closed and by which flood events. Kansas

City District Hydrology and Hydraulics Section staff made estimates of flood durations for various events, and further durations of potential road closures were estimated for cleanup and repair activities. Daily traffic counts for major roads and highways that would likely be closed in the event of levee failure/overtopping were obtained from Missouri and Kansas state transportation departments, together with estimates of the number of trucks versus cars in the count. The average number of passengers per vehicle was determined based on an urban weighted average (1.42 persons) provided in a Kansas Department of Transportation study. Detour routes were measured and compared with mileage for the non-disrupted route to determine the additional miles that would be traveled with the detour. Additional operating costs per mile were estimated at $0.375 for cars and $0.625 for trucks. Additional time to travel the detour was computed based on an estimated detour route speed limit and distance compared with the non-disrupted route speed limit and distance. The published average local wage rate for the metropolitan area of $16.56 was used for truck drivers and one-third of the average local wage rate was used for adult car passengers. Estimated traffic disruption costs for each unit were entered into the HEC-FDA study file, and integrated with the hydrologic data, to determine estimated annual traffic disruption costs during the period of analysis.

5.4 Data Development – Depth-Damage Functions

The goal of this portion of the analysis is the production of depth-damage relationships or functions for each type of item susceptible to inundation. An item that has experienced prolonged submersion might be a total loss, or badly damaged but salvageable, or even relatively unaffected in some cases. Depth-damage functions give estimated percentages of value affected by each foot of flooding; e.g., 2 feet of inundation might be associated with damage amounting to 20 percent of total property value. The relationships are developed for each type of occupancy within each economic category and are usually broken down by structure and contents. Uncertainty in the depth-damage percentages must also be specified in terms of either a standard deviation or minimum and maximum values for each foot of flooding.

5.4.1 Residential Depth-Damages

Residential damages for most homes in this analysis are based on depth-damage percentages released in Economic Guidance Memorandum 04-01, "Generic Depth-Damage Relationships for Residential Structures with Basements," dated 10 October 2003. This EGM summarized data developed by the Institute for Water Resources (IWR) using post-flood residential damage claim records provided by the Federal Emergency Management Agency (FEMA). The functions account for both structural and content damage to homes. Based as they are on post-flood damage claims data, the functions should also account for any emergency flood avoidance actions taken by residents such as evacuation or flood proofing. The IWR functions pertain to all six residential occupancy types selected for this analysis: 1-story with and without basement, 1 1/2-stories with and without basement, and 2 stories with and without basement.

For the residential other category, depth-damage percentages are based on a weighted average from the damage functions produced for the Economic Guidance Memorandum 09-04, "Generic

Depth-Damage Relationships for Vehicles," dated 22 June 2009. The EGM summarized data provided by the Flood Damage Data Collection Program from residential post-flood damage surveys that included data collected for vehicles kept at residences in ten communities that experienced major flooding.

5.4.2 Commercial Depth-Damages

A customized individual occupancy type for use in the risk analysis was developed for companies and facilities that provided specific information on values, elevations and damage potential in our discussions with them. Each major asset or inventory item was valued and assigned a depth-damage function with uncertainty (usually expressed as a triangular distribution with minimum and maximum values) indexed to a given elevation within the structure. However, most businesses and facilities in a large urban floodplain inventory cannot be characterized by company-specific data, and the treatment of depth-damage relationships for these businesses is similar to the contents valuation process for the same businesses described above in section 5.3.4. The New Orleans District report discussed there is also the source for many of the depth-damage functions used in this analysis and is considered relevant to the study area for the same reasons. The functions are based on a wide range of expertise, including panels made up of experienced subject experts on construction and post-flood cleanup, owner/operators of businesses, and FEMA post-flood depth-damage functions for the same region. It was determined that the New Orleans owner/operator curves were appropriate because flooding characteristics (urban, freshwater, and long duration) were similar. The owner/operators interviewed also represented many of the same types of businesses and facilities as those included in the Kansas Citys structure inventory. Depth-damage functions are included for each of the three types of non-residential structure (masonry, steel, and wood) and seven types of non-residential contents (restaurants, grocers, retail and services, professional offices, warehouses and contractors, repair and home use establishments, and public facilities). The New Orleans functions include median, maximum, and minimum values that serve as the basis for triangular damage uncertainty distributions in the risk analysis. To account for the high depths of flooding in the Kansas Citys study area, depth-damage relationships were extended by means of extrapolation and professional judgment.

It should be noted that some of the functions assume that damage occurs at an elevation of zero. One reason for this is that surface flows do, in fact, damage some items. Examples include finished goods inventories stored on the floor (particularly items such as food or drugs), inventories that are very sensitive to humidity even if not directly touching the water, or equipment with electrical wiring on the floor. Another reason is that the depth-damage functions typically are structured in depth increments of a half-foot, if not a foot. If damage occurs with depths of only two or three inches (as it usually would), these depths would more readily round to zero than to one foot or one half foot. Damage percentages paired with an elevation of zero, therefore, might in actuality be accounting for very shallow flows of greater than zero depth.

5.4.3 Streets and Railroads Depth-Damages

Depth-percent damage relationships for roads in the study area were based on previously developed Kansas City District curves used in other approved studies. Omaha District curves were obtained for comparison and to develop estimated uncertainties in the depth-damage relationship. Large paved parking lots associated with commercial and industrial structures in the study area were measured from aerial maps to determine square footage, and a value per square foot representing the cost to resurface the lot (obtained from Kansas City District Cost Engineering staff) was applied to the square footage to determine investment value. Road depth percent damage relationships were used for the large paved parking lots identified. Damages to roads and parking lots in each levee unit area are included in the public category of damage.

For railroads, previously developed Kansas City District depth-percent damage curves were used, and compared with Omaha District curves to determine estimated uncertainties. Separate depth damage relationships for railroad tracks and electronic equipment were used. Estimated damages to railroad tracks are also included in the public category of damage in each levee unit area.

5.4.4 Emergency and Disaster Relief Depth-Damages

The depth percent damage relationship for emergency and disaster relief was developed as follows. First, study area primary damages resulting from various probability events were computed as a percentage of the estimated 0.2 percent probability event primary damages in the study area. Each resulting percentage was then paired with the average flood depth in the study area flooded units for each probability event in order to develop a depth-percent damage relationship. Thus, emergency costs estimated to be incurred for any exceedance probability event would be approximately 13 percent of the primary physical damages for that specific exceedance probability event.

5.5 Costs of Flooding Not Included in the Analysis

Although the accounting of flood losses for this analysis is quite comprehensive, certain costs of flooding are not included in this economic analysis. Usually this is because of one or more of the following reasons: (a) difficulty of monetizing the damages; (b) difficulty of estimating the scale of damage and relating it to specific flood events for use in a function; or (c) uncertainty that improvements in the Federal project would significantly affect the costs involved. Costs not included in the analysis include:

- Damages to some utilities – Damages to sewer systems and underground utility lines would occur in each reach, with damages likely in the millions of dollars with additional cleanup costs. Much of this damage would likely occur in a large flood event anyhow, even with a stronger or higher levee.
- Damage to levee units – The levee units that are the focus of this study would themselves be subject to damages in large flood events, and damage could easily reach the tens of millions of dollars. But again, much of this damage might still occur even with augmented levees.

6.0 Model Configuration

6.1 Study Configuration

6.1.1 Analysis Years and Period of Analysis

In addition to the existing conditions of 2012, we also analyze a base condition and a future condition. The base year for the economic analysis – i.e., the year when the project would be completed and operational – is 2030. The future condition year (selected to be 2049 in this study) is normally used to project changes in hydrologic/hydraulic conditions and economic development for a specific future year, usually about 20 to 30 years out from the base year.

In this analysis, the economic database for the existing condition is also used to characterize the base and future conditions. These conditions initially were defined separately in order to allow the addition of planned development late in the study completion period based on the most current information about future development. Since economic development plans potentially affecting the future without-project condition tend to be fluid and speculative, we establish our assumptions in this area as late in the study as possible. Ultimately, however, while there were many possible projects on the horizon as we completed this study, none met our criteria for inclusion: (a) high likelihood of implementation, (b) firm identification of a location, and (c) availability of information on industrial classification and estimated investment. Therefore, the economic database used in the existing conditions analysis is carried through to the base and future conditions without change. However, in the future condition there are changes in the engineering data used in the risk-based analysis for structural and geotechnical conditions caused by the continuing neglect of the deficiencies in the CID levee reach. Moreover, there is also a minor change in the hydraulic and hydrologic conditions due to the increase in uncertainties regarding river stages.

6.1.2 Interest Rate and Price Level

Annualized estimates of damages, benefits and costs in this analysis assume the FY 2014 Federal interest rate of 3.5 percent and a period of analysis of 50 years. The selection of a 50-year period of analysis is based on the period over which hydrologic/hydraulic conditions can be projected as well as on official guidance for evaluation of Federal levees, which specifies a maximum period of 50 years. All estimates are expressed in October 2013 prices unless otherwise noted.

6.1.3 Study Streams and Reaches

Study reaches serve the basic purpose of allowing the aggregation of stage-damage data for all properties located in a particular portion of the stream's floodplain. Each reach is assigned an index point, and all property evaluations in that reach are adjusted to the elevations at the index point. These adjustments in elevation compensate for variations in the lay of the land along the stream and particularly the gradual drop in ground elevations typically encountered when going downstream.

The Phase 2 study damage streams and their reaches used in this study are summarized in Table 13, which indicates the beginning and ending river mile or station for each reach as well as the index point.

Table 13: **Study Damage Reaches**

Damage Reach Name	Beginning Station	Ending Station	Bank	Index Location
Kansas River:				
Armourdale	0.6	7.7	Left	5.2
Missouri River:				
Central Industrial District	365.8	370.7	Both	368.8

6.1.4 Economic Categories

The economic structure inventory in this study is categorized in terms of three basic land uses: residential, commercial (including businesses, non-profit institutions such as churches and schools, public facilities and utilities), and public (notably, streets and railroads). Four categories of non-physical costs of flooding, cleanup, emergency, floodplain evacuation and reoccupation, and traffic disruption, are also included in the analysis.

6.2 Risk Analysis Preparation

The comprehensive structure inventory for the study area – including elevations, values, and depth-damage functions for each property – was entered into the HEC-FDA risk analysis program for damage computations. HEC-FDA refers to the Flood Damage Analysis software developed by the Hydrologic Engineering Center for use by the Corps of Engineers. The basic assumption underlying use of a risk analysis program is that the field data in flood risk studies are based on imperfect knowledge and that key variables for which median or most likely values are specified could, in reality, take on a range of values above and below the specified values. The economic structure inventory is loaded into HEC-FDA and integrated with hydraulic and hydrologic data characterizing flood potential as well as geotechnical and structural data characterizing the levee units. All engineering and economic data are entered into the program in terms of median or most likely values and accompanied by appropriate uncertainty parameters specifying the assumed range of theoretically possible values for each variable. The subsequent risk analysis simulates tens of thousands of theoretical flood events, synthetically extending the period of record to thousands of years and thereby producing results that embody uncertainties in assumptions and the dynamic interaction of variables over time. For each event, the program samples the range of possible values for each variable and determines (a) whether the flood event results in damages, and (b) how much damage occurs.

Damages are initially expressed as a stage-damage relationship; i.e., each foot of potential flooding at an index point is associated with an estimated amount of "primary damage." But the ultimate goal is expression of damages in an annualized equivalent form. The calculation of

average annual damages conceptually involves a weighted average in which the primary damages for each event are multiplied by the incremental probability of that event and the product is summed and averaged over 100 years. This total represents an estimate of the average damages that could be expected in any given year over the long term. The average annual damage total can then be compared on an equal basis to an annualized cost for the planned project to obtain a benefit-cost ratio.

An additional result of the risk analysis is a set of statistics characterizing project performance in terms of reliability or non-exceedance probability. The program estimates the probability that a levee unit will successfully contain certain specified flood events of interest such as the 1 percent chance event (i.e., the event magnitude with a 1 percent chance of occurring in any year).

6.2.1 Hydrologic and Hydraulic Data

Water surface profiles relating Kansas River stages to frequencies or probabilities of occurrence throughout the study area were provided for each of eight events, including the 0.10, 0.01, 0.005, 0.002, 0.0013, 0.001, 0.0008 and 0.0007 probability events. The profiles are referenced to 2008 conditions, although it should be noted that no increases in these stages are forecasted through the period of analysis and the same profiles are used for existing, base year, and future conditions.

The exceedance-probability relationship for the Kansas River was evaluated using the graphical method, which involves specifying a discharge-probability relationship (including a discharge for the 0.999 probability event) for each index point along with the equivalent record length for the stream.

The risk-based economic analysis is based on each levee's lowest point. The low point for each unit is identified by developing a water surface profile that corresponds to the overtopping discharge and then comparing the water surface profile to the top of levee elevation profile to find the location at which the top of levee falls below the water surface profile. The initial overtopping elevation in each area is adjusted to the economic index point for that reach. The resulting adjusted initial overtopping elevation at the index point essentially serves as the top of levee elevation for that reach, although it will not be necessarily be the same as the actual top of levee elevation at the index point or other sections of the levee.

6.2.2 Geotechnical and Structural Data

In addition to the top of levee elevation, geotechnical and structural probabilities of failure below the top of levee/floodwall elevations must also be considered. Existing older levees and floodwalls may have deteriorated and can no longer be assumed to hold water to the stage initially designed for. Geotechnical and structural engineers determined the most likely expected modes and sites of failure prior to overtopping in each unit. A full range of conditional probabilities of failure versus river stage elevation were determined by geotechnical and structural engineer PDT members for each site/mode of failure in each unit, in accordance with

existing guidance. The probability of failure versus exterior stage relationships developed for major features and/or sites that were considered to have high probability of failure were then translated to the index point of each reach, and each individual potential failure site/mode was determined to be independent. The probabilities of failure for each site/mode were then combined using a formula contained in ETL-1110-2-556, Risk-Based Analyses for Geotechnical Engineering for Support of Planning Studies, to derive a single combined probability of failure versus river stage curve that accounted for all sites or modes of potential failure (Formula: Pr(f)=1-(1-p1)(1-p2) . . .(1-pn)). The resulting combined probability of failure versus river stage curve was entered into the HEC-FDA study file in the "Levee Features" section.

Table 14: **Armourdale Probability of Failure Functions – Existing Conditions**

WSE	p(F) site 1	p(F) site 2	p(F) site 3	p(F) site 4	p(F) site 5	p(f) combined
@ index point (RM 5.2)	Undersee page Station 276+00	Stability Station 222+00	Mill St Pump Station 156+75	12th St Pump Station 129+20	5th Ave Pump Station 185+70	
748.06	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
764.37	0.0000	0.0000	0.2408	0.2486	0.0000	0.4300
766.37	0.0027	0.0000	0.4022	0.4518	0.0250	0.6800
766.87	0.0038	0.0000	0.4574	0.5046	0.4732	0.9800
767.37	0.0050	0.0005	0.5125	0.5574	0.9390	0.9900
768.37	0.0075	0.0025	0.6228	0.6630	0.9990	1.0000
770.56	0.0110	0.0515	0.8643	0.8944	0.9990	1.0000
771.70	0.0310	0.2500	0.9901	1.0000	0.9990	1.0000

Table 15: **CID Probability of Failure Functions – Existing Conditions**

@ index point (RM 368.8)	Structural Gatewell 67+65	Structural Stoplog Gap 104+51.5	Structural Stoplog Gap 132+20	Structural Stoplog Gap 166+31	
757.30	0.0000	0.0000	0.0000	0.0000	0.0000
757.42	0.0000	0.0000	0.0000	0.0200	0.0200
758.54	0.0540	0.0000	0.0000	0.0506	0.1018
759.09	0.1090	0.0000	0.0000	0.0841	0.1839
759.30	0.1300	0.0000	0.1692	0.0989	0.3486
759.43	0.1430	0.0000	0.2739	0.1080	0.4449
759.52	0.1520	0.0000	0.3464	0.1144	0.5091
759.67	0.1670	0.0000	0.4672	0.1278	0.6129
759.81	0.1810	0.0000	0.5800	0.1433	0.7053
760.00	0.2000	0.0000	0.6897	0.1644	0.7926
760.16	0.2533	0.0050	0.7821	0.1822	0.8676
760.30	0.3000	0.0074	0.8630	0.1978	0.9236

7.0 Damage Analysis Results

It should be emphasized that the damages summarized in this section are risk-based, and the results obtained in the risk analysis can appear at odds with nominal data that do not reflect the uncertainties involved. An additional factor distinguishing damage potential in the risk context from data based on nominal top of levee and flood event elevations is that the risk model assumes that a flood can occur from geotechnical or structural failure as well as by overtopping. Geotechnical and structural deficiencies are important existing issues in both the Armourdale and CID units.

7.1 Existing Condition Results

As computed in the HEC-FDA risk analysis model, equivalent annual damages (EAD) total $64,259,940 for the Phase 2 study area. The distribution of EAD among the damage categories for both Phase 2 units is summarized in Table 16. Table 17 summarizes the engineering performance statistics emerging from the risk analysis. Both aspects of the results are discussed below for each study reach.

In general, the analysis produces two conclusions regarding the engineering performance of the Armourdale and CID units:

- Hydraulically, both units are high enough to offer moderate protection against all but the most extreme events.
- However, significant geotechnical and structural concerns are compromising the performance of the units.

Table 16: **Equivalent Annual Damages – Existing Conditions**

Damage Category	Armourdale	Central Industrial District	Phase 2 Study Area
Commercial	$38,979.51	$ 7,421.69	$46,401.20
Residential	$ 1,948.50	$ 0.87	$ 1,949.37
Public	$ 6,289.82	$ 539.72	$ 6,829.54
Emergency	$ 6,492.34	$ 734.91	$ 7,227.25
Cleanup	$ 1,412.41	$ 167.82	$ 1,580.23
Evacuation	$ 180.70	$ 2.38	$ 183.08
Traffic	$ 88.77	$ 0.51	$ 89.28
Total	**$55,392.04**	**$ 8,867.90**	**$64,259.94**
October 2013 prices, 3.5% interest rate; $1000s			

Table 17: **Engineering Performance – Existing Conditions**

	Armourdale	Central Industrial District	Argentine
Annual Exceedance Probability* (median)	0.0350	0.0033	0.0110
Return interval (years)	29	303	91
Long Term Risk (chance of exceedance during indicated period)			
over 10 years	0.3138	0.0461	0.1250
over 30 years	0.6768	0.1321	0.2838
over 50 years	0.8478	0.2103	0.4871
Conditional Exceedance Probability - Overtopping or Breach**			
10.0% event	0.1641	0.0000	0.0000
4.0% event	0.2220	0.0004	0.0320
2.0% event	0.3165	0.0140	0.2140
1.0% event	0.5453	0.1134	0.5121
0.4% event	0.8139	0.4236	0.8057
0.2% event	0.9194	0.6657	0.9132
Conditional Exceedance Probability - Overtopping Only**			
10.0% event	0.0000	0.0003	
4.0% event	0.0000	0.0003	
2.0% event	0.0062	0.0054	
1.0% event	0.0794	0.0663	
0.4% event	0.3670	0.3346	
0.2% event	0.6141	0.5875	

*Annual exceedance probability is the chance of experiencing any flood event - of whatever magnitude - within any year

**Conditional exceedance probability is the probability that specified flood event would overtop or breach the levee

7.1.1 Armourdale Economic Performance Without-Project

Equivalent annual damages total $55,392,040 for Armourdale. Over 70 percent of this total is accounted for by commercial structures. Public structures (including streets and railroads) make up nearly 12 percent of damages, while homes only account for 3.5 percent.

7.1.2 Armourdale Engineering Performance Without-Project

The median annual exceedance probability for Armourdale is currently 0.035. In other words, there is a 3.5 percent chance of a damaging flood in any year given the levee's existing state. In the 1 percent-chance flood event, the probability of only overtopping is 7.9 percent. However, the levee has five critical sections with deficiencies. Given these deficiencies, the probably of overtopping or breaching the levee for that event rises to 54.53 percent. The long-term risk of a damaging flood over 30-year period currently exceeds 1 in 2.

7.1.3 Central Industrial District Economic Performance Without-Project

The total equivalent annual damages for the Central Industrial District reach are $8,867,900. Again commercial structures make up the vast majority of the total with 84 percent of damages. Emergency costs and public structures account for another 8 percent and 6 percent, respectively. All other damage categories combine to total less than 2 percent.

7.1.4 Central Industrial District Engineering Performance Without-Project

The median annual exceedance probability for CID is currently 0.0033. In the 1 percent-chance flood event, the probability of only overtopping is 6.6 percent. However, there are four critical sections with structural deficiencies. Given these deficiencies, the probably of overtopping or breaching the levee for that event rises to 11.34 percent. The long-term risk of a damaging flood over 50-year period is currently over 1 in 5.

7.2 Future Without-Project Condition

Future condition profiles were determined to be the same as existing condition profiles, with the exception of any impacts from the recently completed Missouri River Levee System L-385 Federal Levee. The L-385 project, on the opposite bank of the Missouri River and just upstream of the Fairfax-Jersey Creek unit, was determined to have minimal effect on future condition profiles in the Kansas Citys study area. Some very slight variations in profiles occurred in the far upstream portion of the Kansas Citys study area, and these were accounted for in the future condition profiles. It should be noted that uncertainties about river stage, however, were increased from 1.5 feet in the existing condition to 1.8 feet in the future condition. This was based on calculation procedures outlined in EM 1110-2-1619 and further described in the Hydrology and Hydraulics Engineering Appendix.

Additionally, we are assuming no changes in the geotechnical/structural conditions from the existing condition to the future without project condition and no significant increase in economic development is projected for the 50-year period of analysis as much of the protected area is

essentially built out. These assumptions provide the framework of the future without-project scenario in which the analysis of future flooding impacts is conducted.

8.0 Alternatives Screening

8.1 Overview of Evaluation Procedures

Economic costs and benefits resulting from a project are evaluated in terms of their impacts on national wealth, without regard to where in the United States the impacts may occur. National Economic Development (NED) benefits must result directly from a project and must represent net increases in the economic value of goods and services to the national economy, not simply to a locality. For example, if a flood interrupts auto production at a plant in one community, that community suffers a loss. But if the affected company replaces the interrupted production at another plant in another city, the community's loss does not represent a net loss to the national economy, and the prevention of such a loss cannot be claimed as a NED benefit.

NED costs represent the costs of diverting resources from other uses in implementing or operating and maintaining the project, as well as the costs of uncompensated economic losses resulting from detrimental effects of the project. NED benefits, the benefit-cost ratio, and the net NED benefits are calculated during the evaluation process. Net benefits represent the amount by which the NED benefits exceed NED costs, thereby defining the plan's contribution to the nation's economic output. The plan with the highest net benefits is considered the recommended plan, assuming technical feasibility, environmental soundness, and public acceptability. Note that the plan with highest net benefits is not necessarily the plan with the highest benefit-cost ratio. The benefit-cost ratio helps identify which plans have likely economic feasibility and can be carried forward for further analysis, but is not decisive in identifying the NED plan from among those plans that are economically feasible.

8.2 Alternatives Formulation Process – Economic Background

8.2.1 Initial Economic Analysis

Preliminary economic analyses were prepared in 2006 to assist in the screening of the initial array of alternatives. The selection of management measures and development of alternatives was limited by the planning constraint of achieving and maintaining a uniform level of flood risk management for the Kansas Citys system. The 500+3 plan recommended in the Interim Feasibility Report was applied as the maximum desired plan. Costs and benefit estimates were prepared for different scales of levee raise and associated modifications leading up to this maximum. Although these lower raise alternative plans do not meet all of the study objectives they were necessary for comparison to ensure the identification of the plan, or plans, that meet economic criteria within each levee unit and the overall system. Cost estimates for the Argentine Unit were included to allow for comparison of the Kansas River three-unit system total. The results are shown in Table 18.

Table 18: **Initial Screening Analysis of Alternative Raise Profiles**

500+1 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$33,042,548	$2,093,795	$16,322,473	7.8	$14,228,678
Armourdale	$51,723,299	$3,371,286	$5,234,014	1.6	$1,862,728
CID-KS	$39,959,191	$2,563,797	$3,266,651	1.3	$702,854
TOTAL	$124,725,038	$8,028,878	$24,823,138	3.1	$16,794,260
500+2 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$33,945,404	$2,150,335	$16,560,871	7.7	$14,410,536
Armourdale	$61,233,118	$3,984,373	$5,553,332	1.4	$1,568,959
CID-KS	$40,482,623	$2,597,032	$3,454,202	1.3	$857,170
TOTAL	$135,661,145	$8,731,740	$25,568,405	2.9	$16,836,665
500+3 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$35,313,745	$2,278,318	$17,081,997	7.5	$14,803,679
Armourdale	$63,411,583	$4,138,267	$5,744,664	1.4	$1,606,397
CID-KS	$41,759,697	$2,686,581	$3,608,586	1.3	$922,005
TOTAL	$140,485,025	$9,103,166	$26,435,247	2.9	$17,332,081

October 2006 Prices; 5.125% Interest Rate

8.2.2 Verification of the Systems Analysis

During the Phase 2 economic analysis, it was recognized that the current annual costs and benefits are significantly higher than in the 2006 screening, especially in the Armourdale Unit, even after adjustment for inflation. The earlier calculation of economic benefits was derived from overtopping failure impacts only. Potential geotechnical and structural failure modes identified and evaluated since that time can lead to flooding risks and impacts at lower elevations than overtopping, thus increasing the benefits. Similarly, the relative project costs are greater due to the measures required to address these additional project concerns. The addition of new potential failure modes, and the plan formulation to address them, represents a change in the Future With Project condition upon which the initial economic analysis was based. A new comparison of the costs and benefits of the different levee height alternatives was required to verify the optimization of net benefits.

Updated economic benefits were determined for the 500+1 and 500+2 profile raises. A review of the Recommended Plan cost estimate was conducted to determine costs for the lower raise alternatives. As the different alternatives are in the same locations, requiring essentially the same easements, equipment, contracting, design effort, etc., there is only a relatively small cost savings of building a levee one or two feet lower. The primary cost differences are related to material quantities of earth and concrete for the levee and floodwall raises and underseepage berms, and the number of required relief wells. An update to screening table is presented below.

As shown in the Table 19, each individual unit, and the three-unit Kansas River system collectively, continue to show rising net benefits at the 500+3 profile raise alternative.

Table 19: **Updated Screening Analysis of Alternative Raise Profiles**

500+1 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$59,813	$3,279	$17,367	5.3	$14,088
Armourdale	$219,948	$12,429	$48,466	3.8	$35,257
CID-KS	$74,135	$4,190	$5,430	1.3	$1,240
Total	$353,896	$19,898	$70,483	3.5	$50,585
500+2 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$61,447	$3,369	$17,621	5.2	$14,252
Armourdale	$223,814	$12,640	$48,466	3.8	$35,826
CID-KS	$81,157	$4,574	$6,532	1.4	$1,959
Total	$366,418	$20,582	$72,619	3.5	$52,037
500+3 Profile Raise					
Unit	**First Cost**	**Annual Costs**	**Annual Benefits**	**B/C**	**Net Benefits**
Argentine	$63,924	$3,504	$18,175	5.2	$14,671
Armourdale	$232,984	$13,141	$50,007	3.8	$36,866
CID-KS	$83,682	$4,711	$7,389	1.6	$2,678
Total	$380,590	$21,356	$75,571	3.5	$54,215

October 2012 Prices; 3.75% Interest Rate

8.3 General Description of Alternatives

8.3.1 Armourdale

In most of the levee sections of the unit initially evaluated, only one alternative plan was identified as technically feasible and effective to perform the raise and address the respective impacts to appurtenant structural and geotechnical features. These individual section measures are thus common to all final alternative plans for the overall unit. Similarly, structural and hydraulic pump station modifications are necessary based on the new unit height and are common to the final array of measures.

The final evaluation of measures focused only on those unit sections where more than one set of measures was identified and carried forward. In five separate sections of the unit, multiple raise alternatives were identified as feasible. These reaches and their alternatives are shown in the table below.

Table 20: **Armourdale Alternatives**

Start Station	End Station	Remaining Alternatives
10+00 UE	16+48 UE	1. Landside levee raise 2. Riverside levee raise **3. Replace levee with floodwall**
77+80	81+00	1. Landside levee raise 2. T-wall on levee **3. Replace levee with floodwall**
95+00	105+00	1. Landside levee raise **2. T-wall on levee**
120+00		1. Landside levee raise **2. Landside levee raise with retaining wall**
230+00		1. Landside levee raise **2. T-wall on levee**
240+00	257+66	1. Landside levee raise 2. T-wall on levee **3. Replace levee sections with floodwall**
42+50 LE	61+00 LE	1. Landside levee raise **2. New sandbag gap closure at Sta 42+50 LE**

Note: **Bold font** indicates selected alternative

In the majority of these remaining sections, the feasible alternatives create access limitations and real estate related conflicts that could involve potentially costly relocations. Experience on similar projects in the Kansas City area, and other locations, has shown that real estate access and relocations involving railroads are both very costly and time consuming. This is an important consideration in the final alternative evaluation and selection. Following is a brief discussion of the alternatives in the Armourdale unit.

Sta. 10+00UE to 16+48UE

A landside levee raise would require relocation of railroad tracks and a riverside levee raise would require modification of two large outfall structures. Replacement of the existing levee with a floodwall eliminates all real estate conflicts. Alternative 3 was carried forward.

Sta. 77+80 to 81+00

A landside levee raise would require relocation of railroad tracks. A T-wall on the levee limits top of levee road accessibility to this area of the unit. The access cannot be rerouted to the landside due to the railroad tracks. Replacement of the levee with a new floodwall eliminates the real estate conflicts and maintains access. Again, Alternative 3 was carried forward.

Sta. 95+00 to 105+00

A landside levee raise would encroach upon an area needed for access to an adjacent business, Kansas City Hardwoods. A T-Wall on top of the levee limits top of levee road access, but access could be rerouted on the landside in the same area as the business access. Thus, Alternative 2 was carried forward.

Sta. 120+00

A landside levee raise would encroach upon an adjacent business, KC Railcar. The use of a retaining wall at the landside toe would limit the increase in levee width and avoid this conflict. Alternative 2 is recommended.

Sta. 230+00

A landside levee raise would encroach upon an area needed for access to an adjacent business, Sambol Meat Packing. A T-Wall on top of the levee would eliminate the increase in levee width and avoid this conflict. Thus, Alternative 2 is recommended.

Sta. 240+00 to 257+66

This section contains two existing levee sections separated by an existing floodwall section. The floodwall has already been identified for replacement as it cannot be structurally modified for raise. A landside raise of the levee sections would encroach upon areas used by adjacent businesses for storage and access. A T-Wall on top of the levee would limit top of levee access. Landside access in this section is already difficult due to the operations of multiple adjacent businesses and the Kansas Avenue bridge approach. Replacement of the levee sections with new floodwall eliminates the real estate conflicts, creates additional area for landside access, and provides for a uniform raise measure for the entire section. Alternative 3 was therefore carried forward.

Sta. 42+50LE to 61+00LE

Even though a landside levee raise would be a very short increase in height, access and implementation of the project would conflict with the adjacent railroad track. A new sandbag gap closure at Sta. 42+50 eliminates this minor unit modification and potentially costly real estate conflict. Alternative 2 was carried forward.

8.3.2 Central Industrial District

Six alternative plans were analyzed in the final evaluation. Each plan includes the same raises of the earthen levee and floodwall sections, the same area fill locations, and the same pump station modification and abandonments. The differences among the plans are related to the new tieback measure; whether or not this measure is included, where the tieback connection is located along the existing alignment, the effect of the new tieback on the proposed relief well system, and what

alignment tieback is constructed on between the existing unit and the bluff. The six alternatives are described as follows:

1. Unit stops at Sta. 130+00 and turns to bluff (adds 4 stop log gaps and 15 new relief wells)
2. Unit continues to Sta. 166+80 (adds 83 relief wells/new pump plant/1 stop log gap)
3. Unit stops at Sta. 138+95 and turns to bluff (adds 2 stop log gaps and 30 new relief wells)
4. Unit stops at Sta. 130+00 and turns to bluff (adds 4 stop log gaps/smaller pump station)
5. Unit continues to Sta. 166+80 (adds 83 relief wells/new pump plant/1 stop log gap/header pipe)
6. Unit stops at Sta. 138+95 and turns to bluff (adds 2 stop log gaps and 30 new relief wells with a new pump plant)

The primary differences between the six plans in the final array is whether or not to modify and raise the existing floodwall upstream of station 130+00, or to essentially shorten the unit by constructing a new tieback to the bluff along the eastern edge of the study area. The existing floodwall in this section has already been modified and raised in the past. Although the foundation analysis determined that additional raise could be supported, the actual implementation would be technically very complex. The area inside the unit along this section contains multiple railroad tracks and one abandoned and dilapidated railroad storage warehouse which provide limited economic benefits. Following is a brief discussion of the alternatives in the CID unit.

No Tieback

Alternatives 2 and 5 assume that the existing wall is raised and no tieback is constructed. Each plan includes a new pump station to handle the flow from the additional 83 relief wells and a new stop log closure structure constructed upstream of the existing closure at the end of the unit. Alternative 5 has a different configuration of header piping to collect flows from the relief wells. Both alternatives have the same future with and without-project conditions. Implementation of either alternative will provide reliable flood risk management up to three feet above the 0.2 percent elevation along the full extent of the existing unit alignment. Without project implementation, the reliability of the unit does not meet current criteria and the entire CID study area is subject to inundation from flood events less than the system design event. These alternatives meet all project objectives and are within the project constraints.

Tieback at Sta. 130+00

Alternatives 1 and 4 assume that a tieback is constructed to the bluff starting at Sta. 130+00, immediately downstream of the Kansas City Terminal Bridge. The existing floodwall upstream of Sta. 130+00, including the existing stop log closure at the KC Terminal Bridge, would not be raised. The tieback would require 4 new stop log closure structures and 15 new relief wells. Alternative 4 assumes that a new small pump station would be needed to handle additional relief well flows. Alternative 1 does not include a pump station. Both alternatives have the same

future with and without-project conditions. Implementation of either alternative will provide reliable flood risk management up to three feet above the 0.2 percent flood elevation along the existing unit alignment downstream of Station 130+00. Upstream of this location, the existing floodwall would remain in place and continue to provide benefits up to its current elevation. If a flood exceeded this height, this section would overtop causing inundation of the railroad tracks. The new tieback would prevent these floodwaters from entering the rest of the study area up to three feet above the 0.2 percent elevation.

Tieback at Sta. 138+95

Alternatives 3 and 6 assume that a tieback is constructed to the bluff starting at Sta. 138+95. The existing floodwall upstream of this location would not be raised. Under both alternatives, the existing stop log closure at the Kansas City Terminal Bridge would be raised. The tieback itself would be shorter than in other alternatives, and require only two new stop log closure structures. However, an additional 30 new relief wells are needed. Alternative 6 assumes that a new pump station would be needed to handle additional relief well flows. Alternative 3 does not include a pump station. Both alternatives have the same future with and without-project conditions. Implementation of either alternative will provide reliable flood risk management up to three feet above the 0.2 percent flood elevation along the existing unit alignment downstream of Station 138+95. Upstream of this location, the existing floodwall would remain in place and continue to provide protection up to its current elevation. If a flood exceeded this height, this section would overtop causing inundation of the railroad tracks. The new tieback would prevent these floodwaters from entering the rest of the study area up to three feet above the 0.2 percent elevation. Without project implementation, the reliability of the unit does not meet current criteria and the entire CID study area is subject to inundation from flood events less than the system design event. These alternatives meet all project objectives and are within the project constraints.

Cost Screening Evaluation

In July 2008, screening level cost estimates were prepared for the six final alternatives. The results are presented in the following table (October 2008 price level).

Table 21: **CID Preliminary Cost Screening Evaluation**

Alternative	Preliminary Cost ($1000s)
1	$98,624
2	$130,026
3	$89,918
4	$102,580
5	$130,834
6	$96,136

New Pump Station Analysis

Following the initial plan evaluation and cost estimates, further analysis was conducted to determine the technical necessity of a new pump station to handle relief well flows. A review of the existing interior storm drainage system showed that if all proposed new relief wells were installed as surface discharging, there would be adequate capacity to carry the expected flows to existing sewer outlets and pumping facilities. Removing the new pump station from the proposed alternative plans eliminates Alternatives 4, 5, and 6 from further consideration as they are otherwise identical to Alternatives 1, 2, and 3, respectively. Furthermore, with no pump station the estimated cost of Alternative 2 is reduced by approximately $8.9 million, for a new estimate of $121.1 million.

8.4 Screening Benefits Comparison

8.4.1 Benefits Computation

To determine the economic justification of the alternatives, each alternative was entered into the HEC-FDA risk analysis model. The Monte Carlo analysis in HEC-FDA was then run to determine damage reduction and residual damages – i.e., damages that would continue to occur in the with-project condition even with implementation of that alternative. The residual damages that would continue to occur in the with-project condition were expressed as equivalent annual damages that account for both the base year condition and the discounted present-worth of the future year condition. The difference between the without-condition EAD and the residual EAD for each alternative represents the damages reduced or benefits of the alternative.

Screening benefits in this analysis were based on physical inundation reduction to homes, businesses, public facilities, railroads, and streets, as well as emergency costs and relocation/reoccupation costs.

8.4.2 Engineering Data Considerations

Like the economic data, top of levee elevations and hydraulic and hydrologic data also were unchanged from the without-project condition. Given the structural and geotechnical character of all identified deficiencies in the Armourdale and CID units of the Kansas Citys levee system, the most important variable in determining the performance of alternatives in this analysis was the probability of failure function. Each reach has a single probability of failure function accounting for the multiple locations of concern within the reach by combining them at the index point. For the alternatives in this analysis, the probability of failure function for the existing condition at each problem site is modified, consistent with Corps guidance, to reflect the repair by specifying a probability of failure of 0.002 at top of levee and a zero probability of failure at all points below three feet top of levee. The risk program interpolates probabilities between these two points. The modified individual functions then go into a revised combined probability of failure function at the index point, and the risk simulation is repeated to determine residual damages with the project in place and damages reduced (i.e., benefits).

8.5 Screening Cost Estimates

Screening-level costs are summarized in Table 22. Costs were prepared by cost engineering staff for each of the alternatives. All costs include interest during construction computations, which assume project completion in 2030. All screening costs reflect an October 2013 price level, and the annualized totals reflect the current Federal interest rate of 3.5 percent as well as a 50-year period of analysis.

The costs for operation, maintenance, repair, rehabilitation, and replacement (OMRR&R) were estimated for each alternative and are based on life cycle cost analysis. The analyses only include the net additional OMRR&R costs the sponsors would be expected to incur based on the new proposed unit modifications. The analyses considered and accounted for the new additional OMRR&R in each year of occurrence, and then computed a present worth value of the future OMRR&R costs. The present worth value was then annualized using the current federal interest rate of 3.5 percent and a 50 year period of analysis.

The new additional (net) OMRR&R costs that the sponsors would be responsible for are due to the new additional relief wells required for each alternative. Each new well is assumed to be maintained every 4 years at an estimated cost of $5,000 per well and replaced after 40 years at a cost equal to the current construction cost. Replacement costs include 10 percent E&D and 7 percent S&A. There are 59 additional new wells in the Armourdale alternative and 47 for CID. The sponsor would continue to incur costs for any existing relief wells but these costs are ongoing for the existing project and are not included in the analysis of the proposed project.

Table 22: **Screening Cost Estimates**

Oct 2013 prices; 3.5% interest rate; 50 year period of analysis; $1000s

	Armourdale	**Central Industrial District**	**Total Phase 2**
Construction	$ 151,809.0	$ 51,726.0	$ 203,535.0
Lands, Damages, & Relocations (LERRD)	$ 3,413.0	$ 2,053.0	$ 5,466.0
Planning, Engineering & Design (PED)	$ 11,934.0	$ 4,188.0	$ 16,122.0
Construction Management (S&A)	$ 10,724.0	$ 3,643.0	$ 14,367.0
Contingencies	$ 54,769.0	$ 19,142.0	$ 73,911.0
Total First Cost	**$ 232,650.0**	**$ 80,752.0**	**$ 313,402.0**
Interest During Construction (IDC)	$ 52,388.5	$ 18,488.5	$ 70,877.0
OMRR&R	$ 191.6	$ 144.9	$ 336.5
Total Annual Costs	**$ 12,343.8**	**$ 4,375.9**	**$ 16,719.7**

Total first costs - PED + LERRD + construction + S&A

Annual costs = ((Total first costs + IDC) x I&A factor of 0.004457) + OMRR&R

Annual OMRR&R costs include only additional costs over and above existing costs

8.6 Summary of Economic Screening of Alternatives

Table 23 displays a summary of total annual costs (including increases in OMRR&R costs), annual benefits, benefit-cost ratios, and net benefits for each of the Phase 2 alternatives evaluated. The Phase 1 Argentine unit benefits and costs are included in the table for the purpose of showing a Kansas River system benefit-cost ratio.

Table 23: **Economic Screening Summary**

Oct 2013 prices; 3.5% interest rate; 50 year period of analysis; $1000s

Levee Unit Alternative	Annual Costs	Annual Benefits	Benefit-Cost Ratio	Net Benefits
Armourdale				
Nominal 500+3 Raise	$12,343.8	$51,457.1	4.2	$39,113.2
Central Industrial District				
Nominal 500+3 Raise	$ 4,375.9	$ 5,229.6	1.2	$ 853.7
Total Phase 2 Study Area	**$16,719.7**	**$56,686.6**	**3.4**	**$39,966.9**
Argentine 500+3 Raise	$ 3,821.5	$18,180.0	4.8	$14,358.5
Kansas River System	**$20,541.2**	**$74,866.6**	**3.6**	**$54,325.4**

8.7 Central Industrial District Sensitivity Analysis

The Central Industrial District is located near the confluence of the Kansas and Missouri Rivers with the protected area lying on both sides of the Missouri and Kansas state line. While the Recommended Plan for the CID unit is economically justified, a further question may arise concerning separable justification for the Kansas and the Missouri sections of the unit. Given that both sections are mutually dependent in protecting the same area, the CID-Kansas and CID-Missouri sections cannot be cleanly separated in analysis of overall CID damages. Therefore, it was determined to treat the sections as one unit. However, because the sections have separate project sponsors, costs have been computed for the two sections individually. Therefore, a sensitivity analysis for the two sections has also been prepared to approximate total economic outputs for each unit. In order to do this, the structure inventory was divided into two reaches based on whether the structures would experience Kansas River or Missouri River flooding first and subsequently re-run in HEC-FDA. The sensitivity analysis demonstrates, as can been seen in Table 24, that even if the two units were separated both would still be economically justified.

Table 24: **CID Sensitivity Analysis**

Oct 2013 prices; 3.5% interest rate; 50 year period of analysis; $1000s

Levee Unit Alternative	Annual Costs	Annual Benefits	Benefit-Cost Ratio	Net Benefits
CID-Kansas				
Nominal 500+3 Raise	$4,345.9	$5,166.8	1.2	$ 820.9
CID-Missouri				
Nominal 500+3 Raise	$ 29.9	$ 62.6	2.1	$ 32.7

8.8 Economic Performance of Screening Alternatives Considered

The economic performance and effectiveness of the final array of alternatives in each unit are compared in Table 25 below. The table displays the probabilistic values of equivalent annual damage (EAD) and EAD reduced, thus showing the impact of uncertainty in evaluation of project benefits. The damages reduced represent the project benefits, and are shown in terms of annualized equivalent values as computed in the HEC-FDA program.

Table 25: **Economic Performance of Alternatives**

Oct 2013 prices; 3.5% interest rate; 50 year period of analysis; $1000s

	Equivalent Annual Damages			Probability EAD Reduced		
Plan	Without Plan	With Plan	Damage Reduced	0.75	0.50	0.25
Armourdale						
Nominal 500+3 Raise	$55,392.04	$3,935.00	$51,457.05	$36,287.34	$49,899.39	$63,998.28
Central Industrial District						
Nominal 500+3 Raise	$ 8,867.90	$3,638.32	$ 5,229.58	$ 1,583.39	$ 3,769.20	$ 7,442.57
Total	**$64,259.94**	**$7,573.32**	**$56,686.63**	**$37,870.73**	**$53,668.59**	**$71,440.85**

8.9 Engineering Performance of Screening Alternatives Considered

As shown in Table 26, implementation of the NED plan would improve the engineering performance for both units, with annual exceedance probability of 0.035 reduced to 0.0012 for Armourdale and a corresponding reduction from 0.0033 to 0.0012 for CID. Conditional exceedance probability in a 1 percent event would be reduced from 0.5453 and 0.1134 for Armourdale and CID in the future without-project condition to 0.0139 and 0.0073 respectively.

Table 26: **Engineering Performance of Alternatives**

	Armourdale	Central Industrial District	Argentine
Annual Exceedance Probability* (median)	0.0012	0.0012	0.0010
Return interval (years)	833	833	1000
Long Term Risk (chance of exceedance during indicated period)			
over 10 years	0.0139	0.0184	0.0194
over 30 years	0.0410	0.0543	0.0478
over 50 years	0.0674	0.0888	0.0933
Conditional Exceedance Probability - Overtopping or Breach**			
10.0% event	0.0000	0.0003	0.0000
4.0% event	0.0000	0.0003	0.0000
2.0% event	0.0004	0.0003	0.0005
1.0% event	0.0139	0.0073	0.0147
0.4% event	0.1451	0.1017	0.1517
0.2% event	0.3479	0.2888	0.3586

*Annual exceedance probability is the chance of experiencing any flood event - of whatever magnitude - within any year

**Conditional exceedance probability is the probability that specified flood event would overtop or breach the levee

8.10 Future With-Project Condition Summary

8.10.1 NED Effects of Selected Plan

The overall NED contribution to the national economy is about $40 million, which are the total net benefits of the Phase 2 project. The project would reduce the existing condition EAD of $64.3 million to $7.6 million in residual EAD.

8.10.2 RED Effects of Selected Plan

Construction of the alternatives considered would contribute to the long term stability of both the Armourdale and CID units. Plans considered do not require acquisition or relocation of residents or businesses and there would be no impacts to the local tax bases due to demolition or removal of structures. With increased levee unit reliability and performance, existing businesses would be expected to continue their existing occupancy in each unit and new businesses and investment would be more easily attracted to the units in the future, resulting in a stronger tax base. With continued industrial and commercial stability enhanced by the increased reliability against flooding, existing neighborhoods and populations would also be expected to remain relatively stable, barring impacts from other sources. Temporary increases in employment would be expected during construction and the temporary presence of construction workers for the project may bring a temporary increase in demand for some services in the local area, but also a

temporary increase in business volume, profits, and sales tax receipts at the local retail and service establishments.

8.11 Residual Risk

Although floodplain users and occupants may desire total protection from flooding, it cannot be overemphasized that this is an unachievable goal. No flood risk management project can guarantee total elimination of flooding. Therefore, it is important for floodplain users and occupants to be aware of the level of flood risk that remains even after implementation of a recommend project.

The selected plan has substantial economic benefits and reduces study area damages in the existing conditions by more than 89 percent. The probability and occurrence of flooding will be greatly diminished. But significant residual equivalent annual damages remain totaling $7.7 million. There would be a 1 in 11 chance of exceedance for CID and Argentine and 1 in 15 chance of exceedance for Armourdale over a 50-year period. For comparison purposes the future without-project condition indicates a 1 in 2 chance of exceedance for Armourdale and Argentine and 1 in 5 chance of exceedance for CID.

The median annual exceedance probability of 0.0012 for both Phase 2 units and .0010 for Argentine indicates that there is a 0.12 and 0.10 percent chance of a damaging flood event in any given year for those respective units.

If the capacity of the federal levee system is exceeded in a particular event, most of the areas inside the levees would be affected due to the flat floodplain topography in these areas. If the amount of water that gets through or over the levees is sufficient to produce severe flood depths, damages in the study area probably would reach $60 million or more. Large-scale evacuations of neighborhoods would be necessary, followed by relocation assistance. A number of important sections of streets and railroad would be closed and in some cases inundated. Inundation depths can range from in excess of 6 to 10 feet in CID and 10 to 15 feet in Armourdale.

Effective emergency planning in advance is the best way to protect communities and minimize the damage from these rare flood events.

9.0 Conclusion

It can be seen in Table 23 that in addition to the strong benefit-cost ratio for the Kansas River system-wide project, each unit is also individually justified. The combined Phase 2 portion of the total project has a benefit-cost ratio of 3.4, while Armourdale unit's benefit-cost ratio is 4.2 and the CID portion stands at 1.2. With Phase 2 net benefits of $40.0 million, the project represents a strong contribution to national economic outputs.

10.0 Economic Update Plan

ER 1105-2-100, paragraph D-4, requires a plan for conducting updates of the project economic justification. Economic updates, revisiting estimated damages, benefits, and costs, will be required every three years. Updates are not intended to involve major economic analyses or extensive reworking of the feasibility study analysis. They are intended to verify the continuing validity of important assumptions on which the economic justification is founded as well as to convert data to current price levels. It is currently expected that the first economic update would be required in FY 2017.

Project economic justification updates will include the following tasks:

1. Data gathering – Information supporting the floodplain inventory will be updated as follows:
   - Windshield survey of study area – A windshield survey including all major portions of the study area will be carried out to initially identify major changes in the scale or condition of residential and nonresidential properties and transportation networks.
   - Discussions with local leaders and research – City and/or Chamber of Commerce staff will be consulted to further help indentify major changes of the previous three years pertaining to the economic structure inventory and particularly to major nonresidential properties. Discussions will encompass verification of continuing operations at major properties, identification of significant changes in operational scale at major businesses and facilities, and identification of significant new development including major new businesses, public facilities, residential developments, and roads and streets. Business owner/operators may also be consulted briefly for general information on operational scale.
   - Additional research – Available information on the internet will be consulted, including totals for new construction permits.

2. Economic structure inventory revisions – Completion of the first task may lead to the conclusion that there have been no changes since the feasibility report that would affect the overall economic justification of the project, in which case a Level 1 update will be prepared that will reaffirm the feasibility study benefits, the economic database will be revised based on the first task, as follows:
   - Major existing businesses and public facilities – For the enterprises accounting for a large share of damages and project benefits, structure values will be updated using RSMeans square foot cost values and depreciation factors. Contents will be adjusted as appropriate based on information gathered from the first task.
   - Other existing nonresidential properties – Structure values of the remaining businesses and public facilities will be updated using CCI (Construction Cost Index, Engineering News Record) factors to bring replacement costs up to current price levels. An average depreciation factor for the three-year increment based on RSMeans data will then be applied to produce updated depreciated replacement structure values.

- Residential values – Residential updating will be based on the sample of 301 properties used to determine depreciated replacement structure values in the feasibility analysis. Depreciated replacement structure values of the sample properties will be updated individually using RSMeans cost per square foot values and appropriate individual depreciation rates. The average sample change in replacement value versus the feasibility or previous update will be applied to replacement values for the remainder of the residential database. An average depreciation factor will be applied to produce updated depreciated replacement values.
- Transportation network – Use of generalized indices such as CWCCIS will be used for specific infrastructure benefit categories such as roads and rail lines provided these benefit categories do not constitute a major portion of overall project benefits.
- New development – For significant new additions to or reductions in the property base, including large businesses and facilities, major new roads and streets, and significant new residential projects, appropriate adjustments will be made to the property inventory when properties could account for a disproportionately large share of benefits in view of their structure and content values as well as their damage susceptibility.
- Other categories of benefits – For disaster relief costs, the average percentage change in value will be computed for the residential category and applied to the relief costs. For production losses, the average percentage change in value for the nonresidential category will be computed and applied. Emergency costs will be updated using an average encompassing both the residential and nonresidential percentage changes.

3. Benefit-cost data computation
   - HEC-FDA analysis – The HEC-FDA program will be loaded with the updated property database and new damage and benefit estimates will be produced.
   - Costs and benefit-cost ratios – An updated cost estimate will be prepared by engineering staff and annualized (this will be true whether or not the benefits need to be revised rather than reaffirmed). Benefit-cost ratios and net benefits will be calculated based on the updated benefits and costs.

4. Population – Estimates of affected population and population at risk will be updated if significant new Census data are available.

U.S. Army Corps of Engineers, Kansas City District

Final Feasibility Report

APPENDIX C

REAL ESTATE PLAN

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

APPENDIX C
REAL ESTATE PLAN
KANSAS CITY MO & KS FLOOD RISK MANAGEMENT PROJECT
PHASE II
FINAL FEASIBILITY REPORT
ARMOURDALE UNIT, CENTRAL INDUSTRIAL DISTRICT
KANSAS CENTRAL INDUSTRIAL DISTRICT
MISSOURI CENTRAL INDUSTRIAL DISTRICT
KANSAS CITY, KANSAS AND KANSAS CITY, MISSOURI
MARCH 2014

1. Purpose:

The purpose of this appendix is to include information on any real estate requirements that may be involved for the identified project. Phase II of the Kansas Cities Flood Risk Management Project is located in Kansas City, Kansas and Kansas City, Missouri and consists of the study areas known as the Armourdale Levee Unit and Central Industrial District (Kansas Unit and Missouri Unit).

This Real Estate Plan is in support of the Kansas Cities' Levee Project Phase II Final Feasibility Report. The Kaw Valley Drainage District is the Non-federal Sponsor (NFS) for the Armourdale Levee Unit and the Central Industrial District – Kansas Unit (CID-KS). The City of Kansas City, Missouri is the NFS for the Central Industrial District – Missouri Unit (CID-MO). Phase I of this project consisted of five units known as the Argentine Levee Unit, East Bottoms Levee Unit, Fairfax Unit, Jersey Creek Unit and North Kansas City Levee Unit. The Armourdale Levee Unit and Central Industrial District – Kansas and Missouri (CID) Levee Unit and Floodwall will make up Phase II of this project.

The purpose of the project is to raise the level of protection to provide greater protection against rare flood events as identified in the Reconnaissance Study and in support of overtopping and underseepage concerns observed during the 1993 flood.

The Armourdale Unit:

The Armourdale Unit is located in Wyandotte County, Kansas on the left bank of the Kansas River between approximate Kansas River miles 7.0 and 0.25. Mile 0.0 would be at the confluence with the Missouri River. The levee project begins at Station 5+00 UE (Upper End) where it ties into high ground across the Union Pacific Railroad tracks just downstream of the I-635 bridge, and extends downstream to Station 45+00 LE (Lower End). The unit protects numerous commercial and light industrial properties as well as a significant residential population. The total length of the unit is approximately 6.6 miles.

Construction of the current levee system began May 1949 and was completed in February 1951. More recent improvements, separately authorized as the 1962 Modification, were completed April 1976.

Central Industrial District – Kansas & Missouri Unit (CID-KS/MO):

The Central Industrial District – Kansas (CID-KS) flood protection unit is located in Wyandotte County, Kansas, and extends from the Kansas/Missouri state line along the right bank of the Missouri River to the mouth of the Kansas River. It then continues upstream along the right bank of the Kansas River to mile 3.4. The Kaw Valley Drainage District is the local agency

responsible for operation and maintenance of the unit. The original unit was constructed by the Kaw Valley Drainage District prior to May, 1948. The bulk of the improvements were completed by November, 1955. The most recent improvements were completed in December, 1979.

The unit consists of a system of levees and floodwalls, underseepage control with relief wells, a stoplog gap, a sandbag gap, pump plants, and drainage structures. The levee is approximately 1.8 miles long and the floodwalls total about 7,900 feet.

The Central Industrial District – Missouri (CID-MO) flood protection unit is located in Kansas City, Missouri within Jackson County. The unit extends along the right bank of the Missouri River, upstream from the Grand Avenue Viaduct (river mile 365.7), to the Kansas/Missouri state line (river mile 367.2). The City Council passed four resolutions between 1941 and 1947 to provide the required assurances of local cooperation. The initial construction began in March, 1946 and was completed in September, 1947. Significant improvements and repair of 1951 flood damage followed the initial construction and were completed in November, 1955.

The unit consists of a system of levees, floodwalls, underseepage control, a sandbag gap and stoplog gaps, pump stations, and conduits. The levees total about 430 feet in length and the floodwalls are about 1.45 miles long.

The Real Estate Plan is tentative in nature; it is for planning purposes only and both the final real property acquisition lines and the real estate cost estimates provided are subject to change even after approval of the Feasibility Report.

2. Project Authorization:

This Real Estate information is provided as an appendix to the Kansas City, Kansas and Kansas City, Missouri, Flood Risk Management. The project is authorized by Section 216 of the 1970 Flood Control Act which provides authority to reexamine completed civil works projects and in accordance with ER405-1-12 paragraph 12-16, dated 1 May, 1998.

The Kansas Cities project is authorized as seven levees units. This project extends over the lowest 10 miles of the Kansas River (at the confluence with the Missouri River) and a 20 mile reach of the Missouri River flanking the mouth of the Kansas River.

3. Project Location and Description:

The Kansas City metropolitan area is a centrally located market; it is a major warehouse and distribution center and a leading agribusiness center. In addition to agribusiness activities, the metropolitan area has major industrial activities such as auto and truck assembly, steel and metal fabrication, and food processing. Wholesale and retail industries and service organizations are now chief employers in the area. The area has a network of interstates and major highways that provides excellent access to the levee units.

The Armourdale and CID units work in concert with the Argentine Unit to create a three-unit system along the lower 10 miles of the Kansas River. The units were designed and constructed in conjunction with each other, but are independently operated to some extent. The total protected area is characterized by dense industrial and commercial development. Some limited residential habitation is also present. Communities (or portions thereof) within the study area include Kansas City, Kansas, and Kansas City, Missouri.

4. Description of Lands, Easements, Rights-of-Way, Relocation and Disposal (LERRD):
Armourdale Unit:

The recommended alternative for this unit is the nominal 500-year+3 landside levee raise, which includes underseepage control improvements, to provide equal levels of protection among the Kansas River units. The existing flood damage reduction of the Armourdale unit is primarily earthen levee and floodwall. This raise would include increasing the height of the existing earthen levee and floodwall replacement. I-walls would be installed near the top of the levee in areas of proposed levee raises with insufficient real estate for landside expansion. In addition to levee and floodwall modifications, this raise could include pump plant modifications and/or replacement, and the installation of relief wells or a buried collector system to relieve underseepage pressures. I-walls would be constructed in the levee crest. Floodwall excavation would occur up to approximately 50 feet landward of the toe of the existing levee. The placement of borrow soil would occur up to about 75 feet landward of the toe of the existing levee for the earthen levee raise.

Project purposes would require the expansion of the current Kaw Valley Drainage District easements. Required estates will include permanent and temporary easements for flood protection, borrow and work area easements and utility relocation. There will be a requirement for a ponding area located at 297+00 to about 303+00. This area is owned by APAC and the recommended estate is a Flowage Easement (Occasional Flooding). A disposal site is not required as the project plan is for all aggregates to be used as fill, all trees and branches to be burned on site and all top soils to be used on the new surfaces. Recommended Estates to be acquired by the NFS(s) are explained at the end of this section. All recommended estates are Standard Estates.

Table 3.1 below outlines the additional rights-of-way needed for the Armourdale Unit.

TABLE 3.1 – Real Estate Required for Armourdale Unit

Type of Easement	Acres	Number of Parcels	Estimated Value
Temporary Construction Easement (3 years)	18.47	29	$108,615.00
Permanent Flood Protection Easements	14.32	26	$1,068,506.00
Total Estimate Real Estate Costs			**$1,177,121.00**

Access to into the project area will be via West Kansas Avenue Bridge from the west and I-70 and US Highway 69 from the south. The bridges can be accessed from I-70, which runs east-west on the north side of the Armourdale area. Direct access to the site will be via an existing road. Temporary access easements will need to be acquired for the private access roads. Kaw Valley Drainage District will provide access to the levee during and after construction of the project.

Temporary aggregate construction roads will be constructed along the levee. These roads have little value for permanent use after construction is completed. They will be removed and replaced by permanent parking or seeded for turf establishment. Use of construction roads as permanent access to the levee is not viable since the current access is via existing ramps, which will be modified to accommodate any raise.

CID-KS Unit:

The recommended alternative is the nominal 500-year+3 landside levee raise, which includes underseepage control improvements, to provide equal levels of protection among the Kansas River units. The existing flood damage reduction of the CID unit is primarily earthen levee and floodwall. This raise would include increasing the height of the existing earthen levee and floodwall replacement. No I-walls would be installed for the CID levee improvements. Pump plants, relief wells, and a buried collector would provide underseepage control. The placement of borrow soil would occur up to about 75 feet landward of the toe of the existing levee for the earthen levee raise.

Project purposes would require the expansion of the current Kaw Valley Drainage District CID-KS easements. Required estates will include permanent and temporary easements. It will also include relocation of utilities and facilities. No fee simple acquisition is required for levee right-of-way on this project. Plus, a disposal site is not required as the project plan is for all aggregates to be used as fill, all trees and branches to be burned on site and all top soils to be used on the new surfaces. All recommended estates are Standard Estates. Estates to be acquired by the NFS(s) are explained at the end of this section.

Access to the project area from the east or west is primarily via I-670, I-70, Central Avenue, or Kansas Avenue. Direct access to the site will be by the existing city street network. The main artery running east-west is I-670 via exit 1B. Kaw Valley Drainage District will provide access during and after construction to the levee at the landward ramps. Public access to these ramps is available over city streets and roadways. There will be some temporary access easements required. Some private roads and accesses will be impacted.

Prospective borrow areas were identified by the Sponsor and screened through joint Corps and Sponsor Efforts. The proposed levee raise accounts for about half of the fill required and stability or underseepage berms account for the other half. Subsurface investigation of the borrow area provided the required geotechnical information for the materials to be used in the levee. It was decided that the area owned by the Water District One of Johnson County (WaterOne) location was the best location for borrow material. The WaterOne site is located approximately 4 miles from the project site. This site will provide borrow for both phase 1 and phase II of the project. It is estimated that roughly 33 acres will be needed for Armourdale's levee raise and roughly 62 acres will be needed for CID's levee raise.

Table 3.2 below outlines the additional rights-of-way needed for the CID Unit.

TABLE 3.2 – Real Estate Required for CID Unit			
Type of Easement	**Acres**	**Number of Parcels**	**Estimated Value**
Temporary Construction Easement (3 years)	48.09	30	$282,798.00
Permanent Flood Protection Easement	14.17	12	$833,281.00
Total Estimate Real Estate Costs			**$1,116,079.00**

CID-MO Unit:

Project purposes would require the expansion of the current City of Kansas City, Missouri easements. The only required estates will be temporary easements.

In addition, multiple rights-of-entry through privately owned property will be required along the levee for direct access during construction. Temporary work area easements will be needed for location of construction trailers, contractor storage areas, and may also be needed for temporary construction roads.

Standard Estates to be acquired by the NFS for the project:

a. Flood Protection Levee Easement: A perpetual and assignable right and easement (the lands to be described) to construct, maintain, repair, operate, patrol and replace a flood protection levee, including all appurtenances thereto; reserving, however, to the owners, their heirs and assigns, all such rights and privileges in the land as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however to existing easements for public roads and highways, public utilities, railroads, and pipelines.

b. Borrow Easement: A perpetual and assignable right and easement to clear, borrow, excavate and remove soil, dirt, and other materials from (the land described in Schedule A) (Tracts Nos. _____, _____ and _____); subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges in said land as may be used without interfering with or abridging the rights and easement hereby acquired.

c. Temporary Work Area (Construction) Easement: A temporary easement and right-of-way in on, over and across (the land to be described) for a period not to exceed three (3) years, beginning with date possession of the land is granted to the United States, for use by the United States, its representatives, agents, and contractors as a borrow area construction staging area, storage area, and work area, including the right to move, store, and remove equipment and supplies, and erect and remove temporary structures on the land and to perform any other work necessary and incident to the construction of the Kansas City, Kansas and Kansas City, Missouri Flood Damage Reduction Project, together with the right to trim, cut, fell and remove therefrom all trees, underbrush, obstructions, and any other vegetation, structures, or obstacles within the limits of the right-of-way; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used without interfering with or abridging the rights and easement hereby acquired; subject, however, to existing easements for public roads and highways, public utilities, railroads and pipelines. (The width of the work area easements will vary but the approximate width is 15-30 feet.)

d. Flowage Easement (Occasional Flooding). The perpetual right, power, privilege and easement occasionally to overflow, flood and submerge (and to maintain mosquito control) in connection with the operation and maintenance of the project as authorized by the Act of Congress approved, together with all right, title and interest in and to the structure; and improvements now situated on the land, except fencing *(and also excepting; (here identify those structures not designed for human habitation which the District Engineer determines may remain on the land));* provided that no structures for human habitation shall be constructed or maintained on the land, that no other structures shall be constructed or maintained on the land except as may be approved in writing by the representative of the United States in charge of the project, and that no excavation shall be conducted and no landfill placed on the land without such approval as to the location and method of excavation and/or placement of landfill; the above estate is taken subject to existing easements for public roads and highways, public utilities, railroads and pipelines; reserving, however, to the landowners, their heirs and assigns, all such rights and privileges as may be used and enjoyed without interfering with the use of the project

for the purposes authorized by Congress or abridging the rights and easement hereby acquired; provided further that any use of the land shall be subject to Federal and State laws with respect to pollution.

5. Non-Federally-Owned Land:
Kaw Valley Drainage District owns fee over approximately 6 acres along the project area and holds permanent flood protection levee easement over the remainder of the existing project area. These lands are available for the project but not sufficient to cover all improvements, so additional easements will be required as stated in Section 4. As per ER 405-1-12, Chapter 12, paragraph 12-38 "The non-Federal sponsor shall not receive credit for the value of any LER, including incidental costs that have been provided previously as an item of cooperation for another Federal project." At this time all lands identified as belonging to the NFS have been provided previously. Credit will only be received for newly required LERRD.

6. Existing Federal Projects:
Armourdale Unit:
The Armourdale Unit is located along the left bank of the Kansas River from mile 6.7 (Matoon Creek) to mile 0.3, near the confluence of the Kansas and Missouri Rivers. The Kaw Valley Drainage District of Wyandotte County, Kansas, furnished the required assurances of local cooperation by resolution dated June 15, 1938. The original levees and floodwalls were constructed under the jurisdiction of the Kaw Valley Drainage District. The flood protection unit consists of a system of levees, flood walls, riprap and toe protection on riverward slopes of levees, toe drains along the concrete flood walls, two sandbag gaps and four stoplog gaps, drainage structures, 45 relief wells and eight pumping plants. The floodwalls range from 13 to 27 feet tall and are approximately 6,600 feet long. The levees in three sections total about 5.5 miles long with a sandbag gap between the two sections upstream of the floodwall. The uppermost levee section is a tieback from high ground on the left bank of Matoon Creek to the Union Pacific Railroad tracks. The second section extends from the railroad embankment near the mouth of Matoon Creek downstream along the left bank of the Kansas River to the floodwall. The floodwall begins north of the Chicago, Rock Island and Pacific Railroad Bridge and extends downstream to connect with the third levee section. Construction of the Federal project began in May 1949 and was completed in February 1951. More recent improvements, separately authorized as the 1962 Modification, were completed in April 1976. The levees and floodwalls of the Armourdale Unit are modified to safely pass a Kansas River flow of 390,000 cfs coincident with a Missouri River flow of 220,000 cfs.

The new project area of interest covers almost the entire length of the existing levee unit and is identified by levee stationing 0+05 to 322+85 UE and 39+71 to 42+50 LE. The project alternative involves differing levels of protection, to include: levee raise, T-walls on levee, modified floodwalls, stability berms, underseepage berms, collector drains, and relief wells for the various reaches of the levee. Many of these improvements will require additional right-of-way agreements with land owners.

CID-KS Unit:
The CID-KS Unit is located along the right bank of the Kansas River from mile 3.1 to mile 0.0, near the confluence of the Missouri River. The Kaw Valley Drainage District of Wyandotte County, Kansas, furnished the required assurances of local cooperation by resolution dated 15

May 1950. The original levees and floodwalls were constructed under the jurisdiction of the Kaw Valley Drainage District. The flood protection unit consists of a system of levees, flood walls, riprap and toe protection and a surface levee crown and ramps, toe drains along the concrete flood walls, two sandbag gaps and two stoplog gaps, twenty-two drainage structures, ten relief wells and ten pumping plants. The floodwalls range from 13 to 27 feet tall and are approximately 6,600 feet long. The levee is approximately 1.8 miles long and the three sections of floodwall totals about 7,900 feet. The Unit is currently designed to pass a Kansas River discharge of 390,000 cfs coincident with a Missouri River flow of 220,000 cfs. Construction of the Federal project began in May 1948 and was completed in May 1950. The majority of the improvements were completed November 1955 and the most recent improvements were completed in December 1979.

The new project area of interest covers the total length of the existing levee unit and is identified by levee stationing 0+00 to 165+00. The project alternative involves differing levels of protection, to include: levee raise, modified floodwalls, stability berms, underseepage berms, collector drains, area fills, and relief wells for the various reaches of the levee. Many of these improvements will require additional right-of-way agreements with land owners.

CID-MO Unit:

The CID-MO Unit is located along the right bank of the Missouri River upstream from the Grand Avenue Viaduct (river mile 365.7) to the Kansas-Missouri state line (river mile 367.2). The initial construction began March 1946 and was completed in September 1947. Significant improvements and repair of the 1951 flood damage followed the initial construction and were completed in November 1955. The levees and floodwalls of the CID-MO Unit are designed to pass a Missouri River flow of 540,000 cfs.

The project alternative involves differing levels of protection, to include: levee raise, T-walls on levee, modified floodwalls, stability berms, underseepage berms, collector drains, and relief wells for the various reaches of the levee. Many of these improvements will require additional right-of-way agreements with land owners.

LERRD credit has already been claimed for existing right-of-way by the sponsors and the sponsors have the sufficiency to acquire new right-of-way for the proposed project. The minimum estate will be the addition of permanent flood protection levee easement, flowage easement and temporary work area easements for access, construction and the borrow area. Acquisition will be the responsibility of the NFS. The need for a disposal area has not been identified.

Past and Present Sponsors for this area of interest is Kaw Valley Drainage District and Kansas City, Missouri. They hold the Flood Protection Levee Easement for this project in perpetuity. Both sponsors have agreements in place with the railroads to shut down operations during extreme flooding events. For the proposed alternative, some active railroad tracks will incur operational interruptions but no relocations. The plan is for the sponsors to work with the railroad and minimize down time by doing work in phases.

7. Federally Owned Land:

There are currently no federally owned lands within the project area.

8. Navigational Servitude:

Navigation Servitude is not applicable and will not be exercised for this project.

9. Real Estate Maps:
Project maps are included as Exhibit A and landowner list is included at Exhibit B. Real Estate mapping depicts the project area, tract boundaries required to support the project, significant utilities to be relocated and known or potential HTRW lands.

10. Flooding Induced by Project:
The feasibility study requires the analysis of any induced damages due to raises in the water surface profile caused by raises of the studied levee unit. A Physical Takings Analysis is required where significant induced flooding is anticipated. A Physical Takings Analysis has been performed for the Armourdale Unit and it has been determined that because the project requires a restriction be placed upon the APAC property, i.e., no landfill above certain elevations so that water may be retained in the designated "ponding areas," this requirement would rise to the level of a taking without the acquisition of a restrictive easement. It is recommended that a standard estate for a flowage easement (occasional flooding) and a temporary easement for construction is required should the project require excavation to lower the elevation at the proposed ponding area. No induced flooding is being caused by actions in the CID-KS/MO Unit Feasibility study; therefore, no induced damages are expected and no Physical Takings Analysis will be required.

11. Baseline Cost Estimate on Acquisition of LERRD:
For a summary of total real estate costs see table below. Temporary work area easements, permanent flood protection levee easements and a flowage easement will be required and are included in the cost estimate along with incidental costs for NFS and in-house Federal administrative costs. When there was an absence of specific information relating to suspect contaminated areas, for this cost estimate, it was prudent to value the land as clean. Known areas of contamination will be avoided to the fullest extent possible. The NFS will be responsible for all clean up of environmental contamination and are aware of this fact.

Below is a summary of the Real Estate Cost Estimate for LERRD, NFS incidental costs, In-House Labor costs and Contingencies. LERRD values are based on an estimate only. An update to the Gross Appraisal was conducted and approved by Kevin Keller, appraiser for the Kansas City District Corps of Engineers Office in March of 2014. The Utility and Facility Relocation costs were provide by cost estimators at the Kansas City District Corps of Engineers.

TABLE 11.1 - Baseline Cost Estimate

Type of Costs	Total Costs
LERRD Estimate for Armourdale Levee Unit	
Permanent Flood Protection Easement	$1,068,506.00
Temporary Construction/Access Easement (3 Years)	$108,615.00
Total Armourdale Land Value Estimate	*$1,177,121.00*
25% Contingency	$294,280.00
Total Land Value Estimate (Rounded)	**$1,472,000.00**
Utility/Facility Relocations Estimate	
Utility/Facility Relocation Estimate	$1,388,788.00
30.79% Contingency (Rounded)	$427,608.00
Total Utility/Facility Relocation Estimate (Rounded)	**$1,816,396.00**
LERRD Administrative Costs	
Non-Federal Sponsor Incidental Costs	$788,800.00
Federal Incidental Costs	$58,000.00
Total Incidental Costs	*$846,800.00*
27% Contingency	$228,636.00
Total Incidental Costs w/ Contingency (Rounded)	**$1,075,436.00**
Total Real Estate Acquisition Costs w/ Contingency (Armourdale)	$4,363,832.00
Type of Costs	Total Costs
LERRD Estimate for CID KS/MO Levee Unit	
Permanent Flood Protection Easement	$833,281.00
Temporary Construction/Access Easement (3 Years)	$282,798.00
CID Land Value Estimate (Rounded)	*$1,116,079.00*
25% Contingency	$279,020.00
Total Land Value Estimate (Rounded)	**$1,396,000.00**
Utility/Facility Relocations Estimate	
Utility/Facility Relocation Estimate	$246,410.00
31.07% Contingency	$76,560.00
Total Utility/Facility Relocation Estimate (Rounded)	**$322,970.00**
LERRD Administrative Costs	
Non-Federal Sponsor Incidental Costs	$572,000.00
Federal Incidental Costs	$42,000.00
Total Incidental Costs	*$614,000.00*
31% Contingency	$190,340.00
Total Incidental Costs w/ Contingency (Rounded)	**$804,000.00**
Total Real Estate Acquisition Costs w/ Contingency (CID)	$2,522,970.00

12. Mitigation:

There are anticipated mitigation activities for the proposed project.

13. Relocation Assistance (P.L. 91-646):
There are no P.L. 91-646 relocations anticipated for the proposed project.

14. Mineral Activity Impacted Present/Future:
There is no current or anticipated mineral activity or timber value impact in the vicinity of the proposed project.

15. Assessment of Non-Federal Sponsor Legal Capability:
The Corps has worked with both of the NFSs to complete the "Assessment of the Non-Federal Sponsor's Real Estate Acquisition Capabilities Checklist" (See Exhibit C). Both NFS have the legal authority to acquire and hold title to real property and condemnation authority and is fully capable of contracting for all real estate needs for the project. The sponsors do not anticipate requiring USACE assistance with acquiring real estate. Financial capability of both sponsors is addressed in the main report.

16. Zoning Ordinances Considered in Support of LERRD Requirements:
Predominate zoning within the boundaries of the project is commercial, light industrial, and business industrial. All land associated with the project are located within a flood zone. Zoning issues are not anticipated for this project.

17. Real Estate Acquisition Schedule:
The project sponsor is responsible for acquiring real estate interests required for the project. A time frame for land acquisition has been outlined for the area of interest ranging from 1 year to 1 ½ years and can begin when final plans and specs have been completed and the Project Partnership Agreement (PPA) has been executed. The Project Manager has coordinated with the Sponsors on the formulation of this schedule and has received their concurrence.

- April 2014 – Feasibility Report Approval by the Civil Works Review Board.
- July 2014 – Approval of the Report of the Chief of Engineers recommending the project to Congress for authorization.
- October 2015 – Execution of Project Design Agreement with Local Sponsor and initiation of Pre-Construction Engineering and Design Phase (pending availability of design phase funding).
- October 2018 – Execution of the PPA with the Local Sponsor (pending construction authorization and availability of construction funding); Initiation of Land and Easement Acquisition by the Local Sponsor.
- September 2019 – Certification of Real Estate.
- October 2019 – Initiation of Construction.
- October 2029 – Completion of Project Construction.

18. Facility/Utility Relocation:
In accordance with Policy Guidance Letter No. 31, a preliminary attorney's opinion of compensability has not been prepared because the relocations do not exceed 30% of the total

project cost. The estimated costs for utility relocations for Armourdale and CID, KS is 3% and 8% respectively. A real estate assessment has been prepared and is on file in the Real Estate Division.

"ANY CONCLUSION OR CATEGORIZATION CONTAINED IN THIS REAL ESTATE PLAN, OR ELSEWHERE IN THIS PROJECT REPORT, THAT AN ITEM IS A UTILITY OR FACILITY RELOCATION TO BE PERFORMED BY THE NON-FEDERAL SPONSOR AS PART OF ITS LERRD RESPONSIBILITIES IS PRELIMINARY ONLY. THE GOVERNMENT WILL MAKE A FINAL DETERMINATION OF THE RELOCATIONS NECESSARY FOR THE CONSTRUCTION, OPERATION, OR MAINTENANCE OF THE PROJECT AFTER FURTHER ANALYSIS AND COMPLETION AND APPROVAL OF FINAL ATTORNEY'S OPINIONS OF COMPENSABILITY FOR EACH OF THE IMPACTED UTILITIES AND FACILITIES."

The Armourdale portion of the project identifies three utility relocations eligible for compensation. They are as follows:

Table 1: Utilities that Require Relocation for Armourdale Unit

Station (ft.)	Type	Owner
-9+63 (0+75 UE)	Sewer Force Main	City of KC, KS
129+20	12th Street Pump Plant Outfall	City of KC, KS
156+75	Mill Street Plant Outfall	City of KC, KS

The CID-KS portion of the project identifies six utility relocations eligible for compensation. They are as follows:

Table 2: Utilities that Require Relocation for CID-KS Unit

Station (ft.)	Type	Owner
25+90	8" Gas Line	KGS
26+10	Gas Line	KGS
26+10	2-12" Water Lines	City of KC, KS
57+10	12" Water Line	City of KC, KS
57+10	Power Line	KCP&L
Approx 124+60	Power Line	KCP&L

There are no utility relocations anticipated for the CID-MO Unit of the project.

19. Impact of HTRW:

The National Environmental Policy Act (NEPA) of 1969 mandates that an Environmental Impact Statement (EIS) be prepared for federally funded projects that may be impacted. An EIS was completed in 2006. An HTRW assessment was conducted by the Kansas City District. The following information is a summary of HTRW information for the Armourdale and Central Industrial District Levee Units. Detailed HTRW information can be found in Appendix E of the Final Feasibility Report

Areas of contamination will be avoided to the fullest extent possible. If an area cannot be avoided, the NFS will be responsible for cleanup. No lands will be acquired until environmental clearances are received.

Armourdale Levee Unit

For the HTRW Site Assessment, the area evaluated was based on the July 2007 figures for the 500 year +3 feet alternative with an additional 40 feet beyond the proposed construction limits to account for temporary easements. Within the Kaw Valley existing right-of-way, no contamination is expected to be encountered. However, there are many areas where the alternative extends outside the existing right-of-way and impacts properties in close proximity to the levee. The areas adjacent to the Armourdale Levee are highly industrialized. There are sites which have soil or groundwater contamination that directly impacts the Feasibility Study alternative. The following summarizes the evaluation and conclusions relating to specific properties within the Armourdale Unit.

Procter & Gamble Manufacturing Company (P&G)

Based on the groundwater data collected by KDHE, the installation of relief wells as underseepage controls between station 45+00 and 75+00 may result in the discharge or migration of VOC contaminated groundwater and should be avoided. Since the plume has not been clearly identified, a smaller restrictive area could be established with additional investigations if relief wells are required. Since the 500+3 alternative does not require relief wells, no additional testing is recommended at this time. A few ASTs on the southern portion of the property are very close to the 40-foot construction limits. They may require removal and, depending on their contents, tank closure procedures may be required before construction commences. Also, if construction occurs on the riverward side near the SWMU 7 at approximately Station 50+00, the debris and dumped solid waste should be removed and replaced with suitable fill material. All waste should be sampled and characterized prior to disposal. The material is likely either construction or solid waste and must be taken to a solid waste landfill or appropriate disposal locations based on sampling results.

Inland Container Corporation

The Reconnaissance Report stated that a LUST was removed from Inland Container Corporation property in 1989. In an interview with personnel of the KDHE Storage Tank Section on August 2, 2005, it was stated that two 20,000 gallons tank full of heating oil were removed in 1989. Inland Container has previously used 1,1,1-TCA in their past processes, according to a KDHE information request. No information on current contamination was found for this site. Neither the Follow-Up Study nor the KDHE LUST database mentions any other hazardous waste issues for the property. No levee construction would occur on Inland Container property. The property would only be encroached on if there was a need for a temporary easement. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. No further investigation is necessary.

Kaw Power Station

From aerial photographs, the power plant has been at this location since 1970, a large coal pile used to fill the exterior yard of the site. Currently, coal is no longer required to generate power at the facility. From the site visit, a few ASTs are present as well as a large transformer,

but are outside the construction limits. The facility also has a small structure on the levee, which is believed to be a wastewater discharge point. Records indicate no concerns regarding soil or groundwater contamination, which would impact levee improvements. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. No further investigation is required.

KC Hardwood Corporation (a.k.a. American Walnut)

KC Hardwoods is adjacent to the levee. It was also known as American Walnut Company. The Reconnaissance Report, Follow-Up Report and KDHE LUST database list three LUSTs, which were removed in 1991. The tanks were removed with no contamination present. It lists four inactive USTs that were removed from the site. The Follow-Up Report also lists the site for No Further Action. This site is no longer located on the CERCLIS database on the EPA website. It is a permitted site for minor air releases.

There is no longer evidence of contamination associated with this property. The property would only be encroached on if there was a need for a temporary easement. Therefore no further investigation is necessary.

Auto Salvage Yards

The auto salvage yards lie next to the levee toe. A portion of the property will be used for construction of a new levee section. The Reconnaissance Report and Follow-Up Investigation did not contain any information on these sites and they were not listed as areas of concern in any EPA or KDHE databases. Aerial photographs show the auto salvage yards have been at their current location since 1970. Property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

Kansas City Railcar Services

A portion of Kansas City Railcar Services property will be used for construction of a new levee section. This property had numerous railroad cars parked in close proximity to the levee toe. Kansas City Railcar repairs and cleans railroad cars. Kansas City Railcar is listed as a hazardous waste permitted facility in the EPA database RCRAInfo. Aerial photographs from 1970 and 1978 show that the property was either a storage yard or salvage yard. During the Flood of 1993, photos show that the property was cleared out, perhaps as a precaution in the event of a levee failure or overtopping. In the 1995 aerial photographs, railcars returned for storage. During the site visit, railroad cars were seen set back away from the levee toe. Their operation had not moved beyond the property boundaries. Due to past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

Auto Salvage Yard, Formerly A to Z Production Plating

This property is adjacent to the levee with a fence separating it from the levee toe by only a matter of a few feet. A portion of the property will be used for construction of a new levee section. In 1991, A to Z Production Plating ceased operations and the property owners entered into a cleanup agreement with the EPA to dispose of the plating process waste that had

accumulated in their warehouse and contaminated equipment. Under EPA direction, soil was excavated outside on the south side of the building to check for any heavy metals contamination. Tests showed no elevated levels of metals in the soils. The EPA designated the site to have no further action required and considered it clean in 1994.

A review of the aerial photographs from 1970 and the following years shows that the site had some form of industrial activity, with the exception of the 1993 photo when the property was vacant during the flood. Following the cleanup of A to Z plating operations, the site was used a salvage yard. During the site visit, the salvage yard was completely cleared and an asphalt and lot was placed over the top of the whole yard.

Due to past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

Trimodal (a.k.a Container Recycling Inc.)

This property is in very close proximity to the levee, with trailers stored at the levee toe. The property has gone by many names since it started in 1965 including: Great Lakes Container Corporation, Container Recycling, Inc., and Trimodal. The facility was used for drum storage and cleaning. Past practices have contaminated the site. It is recommended that any type of excavation or installation of relief wells for subsurface underseepage control be avoided. The current 500+3 alternative has no construction activity occurring outside the existing right of way. KDHE would allow a construction easement through the property provided no excavation would occur and any damage to the existing cap is repaired. Coordination with the KDHE and EPA is recommended for any work on site.

Midwest Cold Storage (formerly Williams Meat Company)

Midwest Cold Storage property lies adjacent to the levee toe and is partially within the construction limit. This property operates a food storage facility. From aerial photos, the site has been industrialized since the 1960s at the latest. Prior to Midwest Cold Storage, the facility was a meat packing plant called Williams Meat Company. According to KDHE, one UST was known to occur on this property. According to Midwest Cold Storage, the UST was removed in the early 1990s.

During the site visit, a large propane tank and a transformer were seen on the landside of the levee. If removal of the transformer is required, the area around the transformer may require further inspection before construction to ensure that PCB contamination is not present. No other concerns were identified with the site.

Schock Truck & Leasing

Schock Truck & Leasing is located outside the construction limits but may be impacted by temporary construction easements. Aerial photographs show the site was industrialized after 1970. The 1983 photographs show trailers stored on the property in close proximity to the levee. Photos from 1995 show the trailer storage area is still present and appears to be very clean. The floods of 1951 and 1993 did not impact the property. During the site visit, two ASTs were seen from the levee road. The tanks were in good condition. A pile of debris lies between the tanks and jersey barriers that denote the edge of the property. It appears to be random construction

debris. No other soil or groundwater contamination appears to be present at the site; therefore no further investigation is planned.

Sambol Packing Co.

Sambol Packing Company operations include meatpacking and packaging, and their property lies partially within the construction limits of the proposed alternative. There is no mention of this property in either the Reconnaissance Report or the Follow-Up Investigation. According to KDHE, Sambol was listed as having two USTs removed after the previous investigations were published. These were removed in 2000. KDHE found no evidence of contamination and considered the site closed. During the site visit, no contamination issues were seen. No HTRW concerns exist for the site; therefore, no further investigation is required.

SELCO (formerly Chromium, Inc)

Chromium Inc. operated on this property from 1952 to 1972 and was listed as a KDHE Identified Site. Based on information provided by KDHE, there appears to be no immediate threat to soil or groundwater from the site's past practices.

SELCO has occupied the former Chromium, Inc. property since 1983. The property lies partially within the construction limits of the proposed alternative. SELCO manufactures car parts. It is not a permitted RCRA facility. No known contamination is associated with the current company.

The site visit did not indicate any current hazardous waste issues occurring at the site. As only very low concentrations of metals and VOCs below KDHE action levels exist at the site, no further investigation is required at this location. The previous buried construction debris, if excavated, may be taken to a solid waste landfill.

PBI Gordon Corporation

PBI Gordon is located adjacent to the current floodway. The facility manufactures various pesticides and other agricultural chemicals, as wells as clay and brick products. EPA databases list it as permitted RCRA hazardous waste treatment, storage and disposal facility. This company has a history of toxic air releases and spills. Although groundwater contamination has not been identified at this location, based on past activities, the potential for such contamination exists.

It is recommended that ground disturbance in this area be avoided if possible. The current proposed alternative does not involve excavation at PBI Gordon. However, if soil excavation is required from this area during the construction phase, soil testing will be required prior to disposal. Some of the soil may require disposal of hazardous waste. Additional groundwater investigation is warranted prior to the any expansion of the existing subsurface relief well system.

APAC - Wilkerson

A portion of the APAC – Wilkerson properties is located within the construction limit. The property is mainly used for equipment storage. From the aerial photographs, the properties currently owned by APAC have historically been industrial. Older buildings were present on the various properties. The 1983 photographs did not cover this section. By 1993, APAC had equipment stored on the site. Trailers were stored all the way to the levee. The remainder of the land was vacant.

During the site visit, APAC was still using the properties as a storage yard for their construction equipment. No contamination is known to be present at this site. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. Therefore, no further investigation is necessary.

Union Pacific Railroad

Union Pacific Railroad (UPRR) operates many rail yards in the Armourdale area, of which lie within the 40-foot construction limit. During a review of EPA documents, a permit for discharging wastewater and sludges was requested by UPRR. A 1982 document states that test results of the wastewater classified it as non-hazardous. The UPRR withdrew its permit application. While a potential exists for previous contamination resulting from spills along the railroad lines, all known contamination has been remediated from various UPRR sites. Therefore, no further investigation is warranted for the railroad yards.

Central Industrial District Unit:

The Central Industrial District (CID) Unit is being reevaluated as part of the Feasibility Study for overtopping and underseepage concerns based on observation made during the 1993 flood. The 500 year + 3 feet alternative was evaluated for the entire levee unit for the hazardous waste study. This alternative includes a combination of alternatives such as levee and floodwall raises, underseepage control and relief wells, and a new floodwall tieback. Currently, there is one existing system of relief wells located at stations 79+00 to 97+00.

For the HTRW Site Assessment, the area evaluated was based on the July 2007 figures for the 500 year +3 feet alternative with an additional 40 feet beyond the proposed construction limits to account for temporary easements. Within the Kaw Valley existing right-of-way, no contamination is expected to be encountered. However, there are many areas where the alternative extends outside the existing right-of-way and impacts properties in close proximity to the levee. The areas adjacent to the CID Unit are highly industrialized. There are sites which have soil or groundwater contamination that directly impacts the Feasibility Study alternative. The following summarizes the evaluation and conclusions relating to specific properties within the CID Unit. The single property of concern within CID is River View Properties, Inc.

River View Properties, Inc.

Potential HTRW concerns have been identified within the study area between station 40+31 and 51+00. Potential encroachment into this area associated with the levee raise is proposed to be avoided by steepening the landside levee slope rather than extending the landside toe. No other locations of HTRW concerns have been identified at this time.

20. Opposition/Support of Project by Local Landowners:

The Kansas Cities Flood Damage Reduction Project is in the feasibility phase so limited contact with the landowners has been initiated at this point. The Project Delivery Team and NFS will initiate increased public input during the latter stages of Feasibility, and during Planning, Engineering and Design (PED) Phase.

21. Notification to Non-Federal Sponsor of Early Acquisition of LERRD:

As project alternatives are clearly selected, the NFS will be issued a risk letter explaining the risk of acquiring lands prior the execution of the PCA and advised to wait on coordination of the acquisition plan and notice to proceed with acquisition.

The NFSs have been advised in writing of the risks associated with advance land acquisition activities. A risk letter was mailed to the NFSs 12 December 2008. The risks outlined in the letter include but are not limited to:

- Congress may not appropriate funds to construct the proposed project;
- The proposed project may otherwise not be funded or approved for construction;
- A PPA mutually agreeable to the NFS and the government may not be executed and implemented;
- The NFSs may incur liability and expense by virtue of their ownership of contaminated lands, or interests therein, whether such liability arising out of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), as amended;
- The NFSs may acquire interests or estates that are later determined by the government to be inappropriate, insufficient, or otherwise not required for the project;
- The NFSs may initially acquire insufficient or excessive real property acreage, which may result in additional negotiations and/or benefit payments under public law (P.L.) 91-646, as well as the payment of additional fair market value to affected landowners which could have been avoided by delaying acquisition until after PPA execution and the government's notice to commence acquisition and performance of LERRD; and
- The NFSs may incur costs or expenses in connection with their decision to acquire or perform LERRD in advance of the executed PPA and the government's notice to proceed, which may not be creditable under the provisions of P.L. 99-662 or the PPA.

22. All other Real Estate Issues:

Cultural Resources, District Archeologists have researched and performed field surveys of the areas of interest. Coordination with the Missouri State Historic Preservation Office (SHPO) is ongoing. It has been determined that the project will result in no known adverse effects on historic properties, but appropriate measures will be taken to avoid, minimize or mitigate any effects.

Unidentified public utilities could be an issue as most of the areas of interest are heavily developed and have been for close to one hundred years. Project Delivery Team members are working closely with sponsors to identify possible problem areas and avoid or address any utilities in question.

US Army Corps
of Engineers
Kansas City District

Appendix C

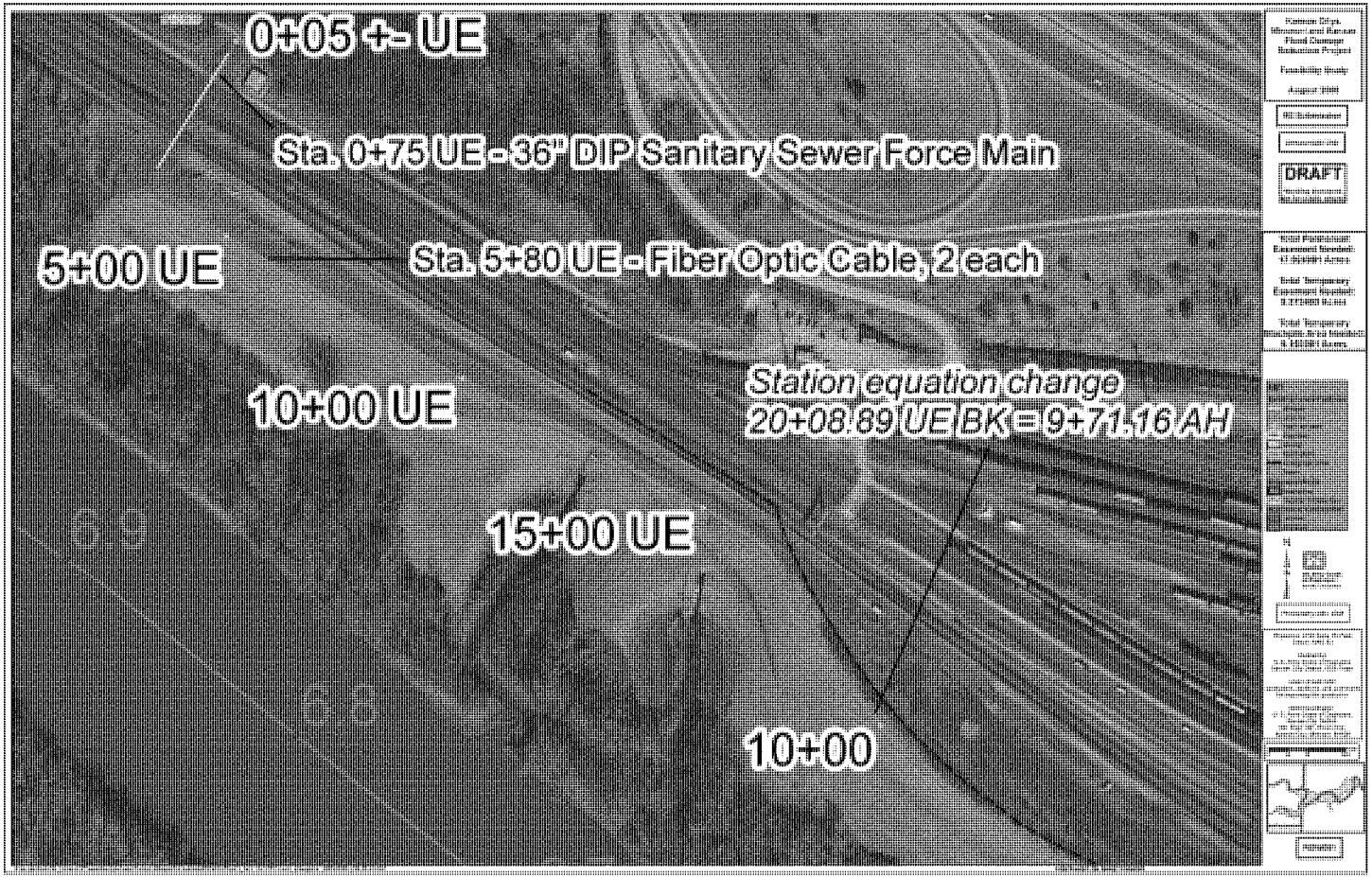
Real Estate Plan

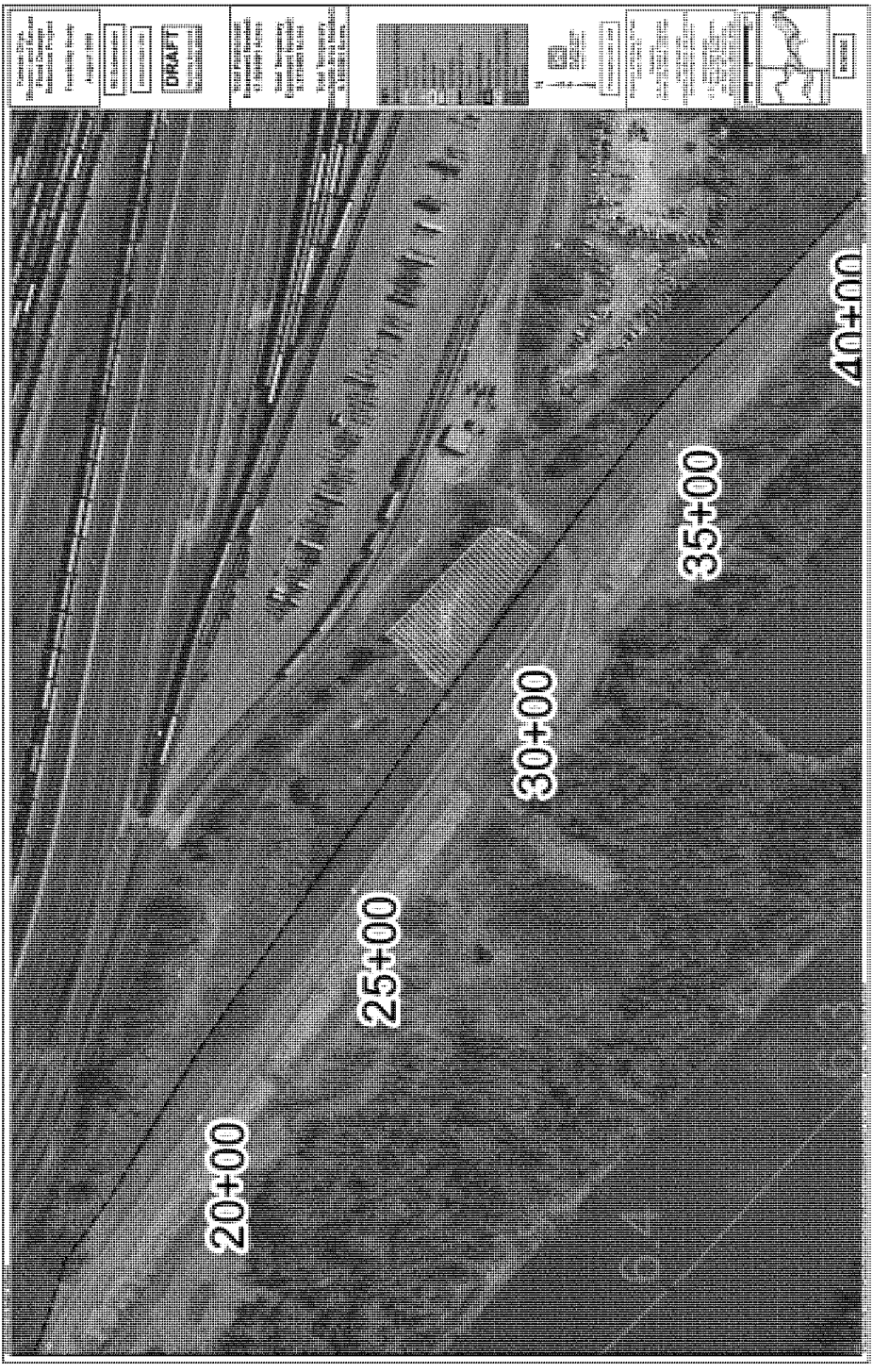
Armourdale Levee Unit
Central Industrial District Unit

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

PROJECT MAPS

EXHIBIT A


0+05 +- UE
Sta. 0+75 UE - 36" DIP Sanitary Sewer Force Main
5+00 UE
Sta. 5+80 UE - Fiber Optic Cable, 2 each
10+00 UE
Station equation change
20+08.89 UE BK = 9+71.16 AH
15+00 UE
10+00
DRAFT


20+00
25+00
30+00
35+00
40+00
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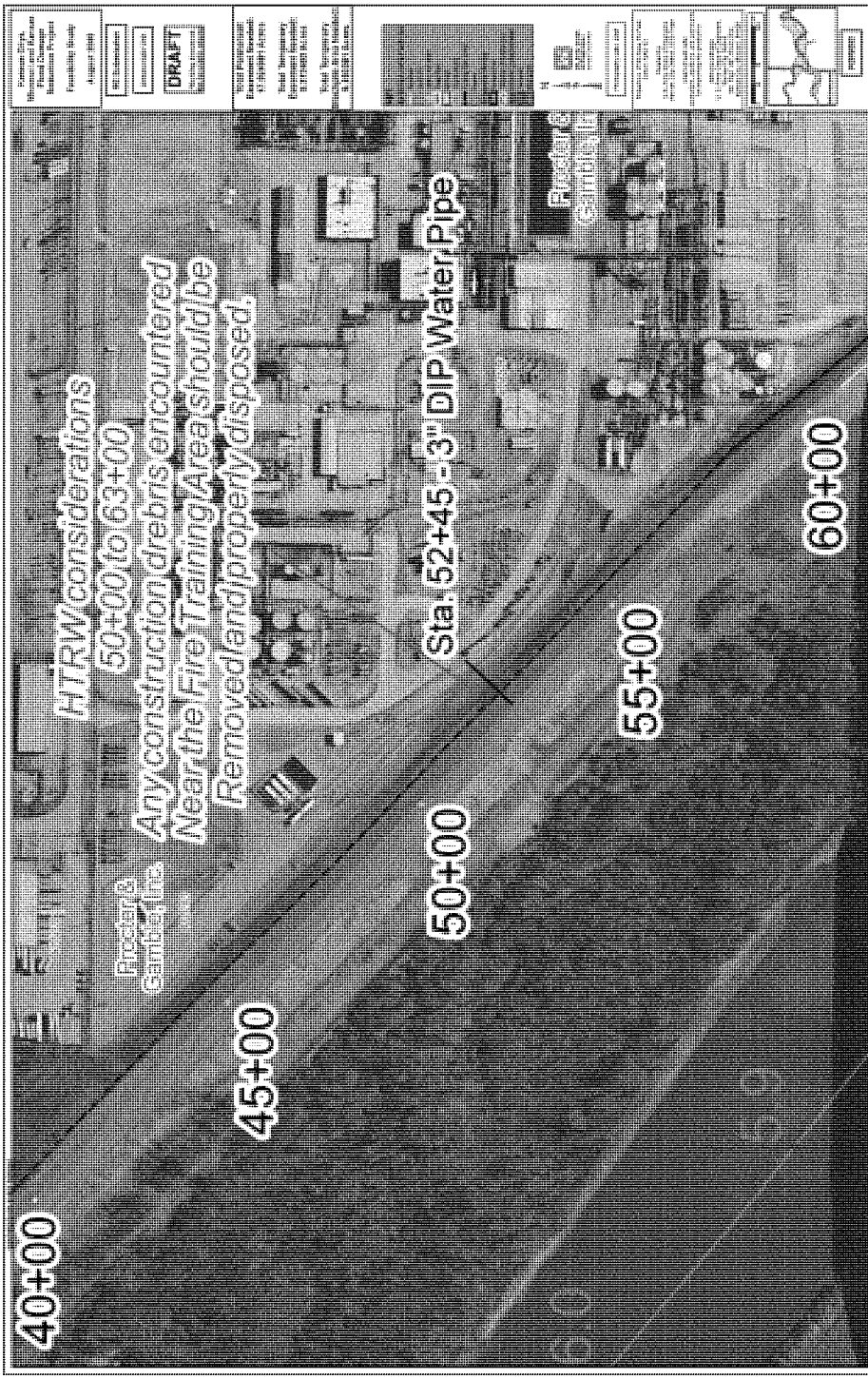
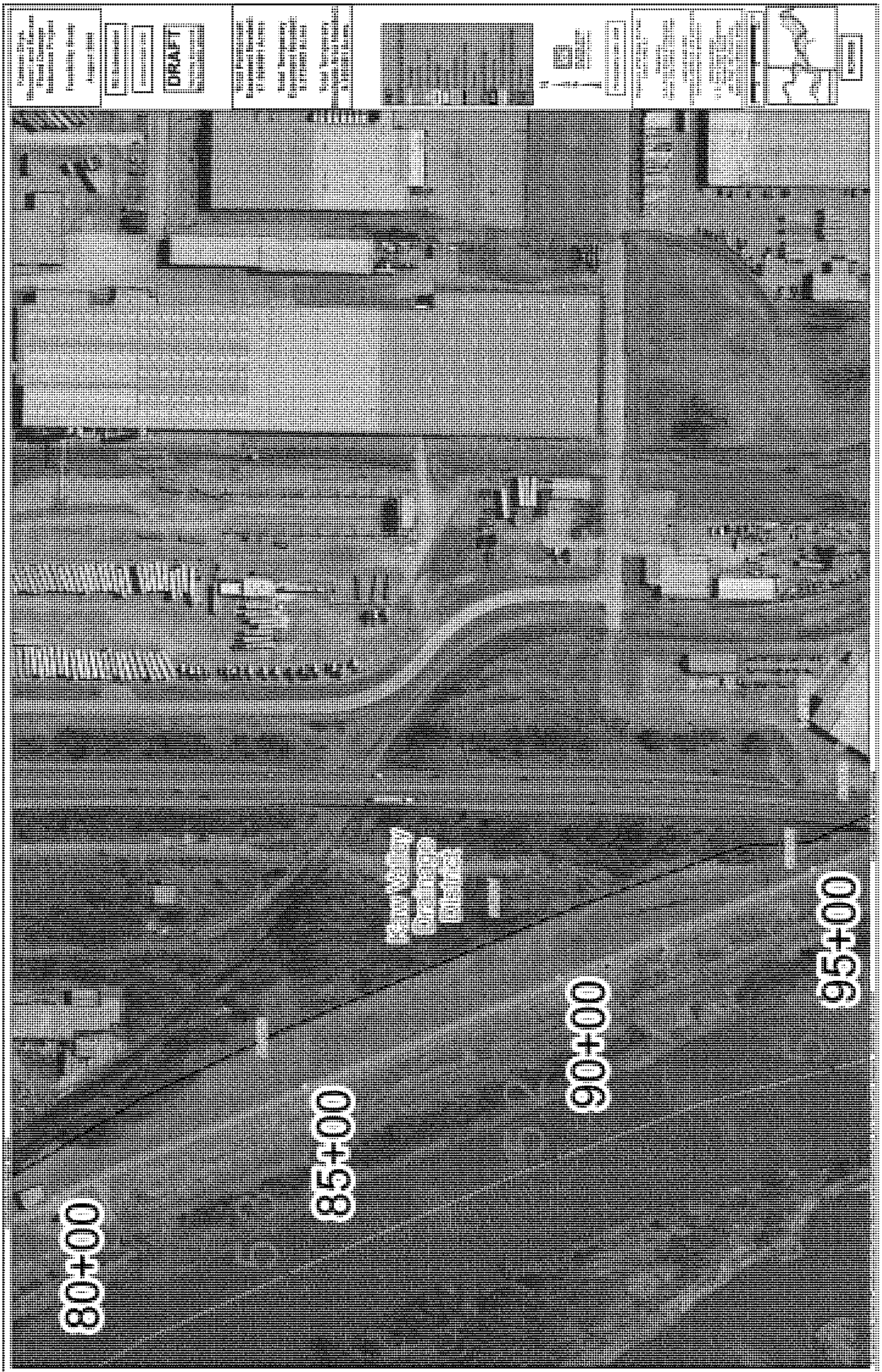
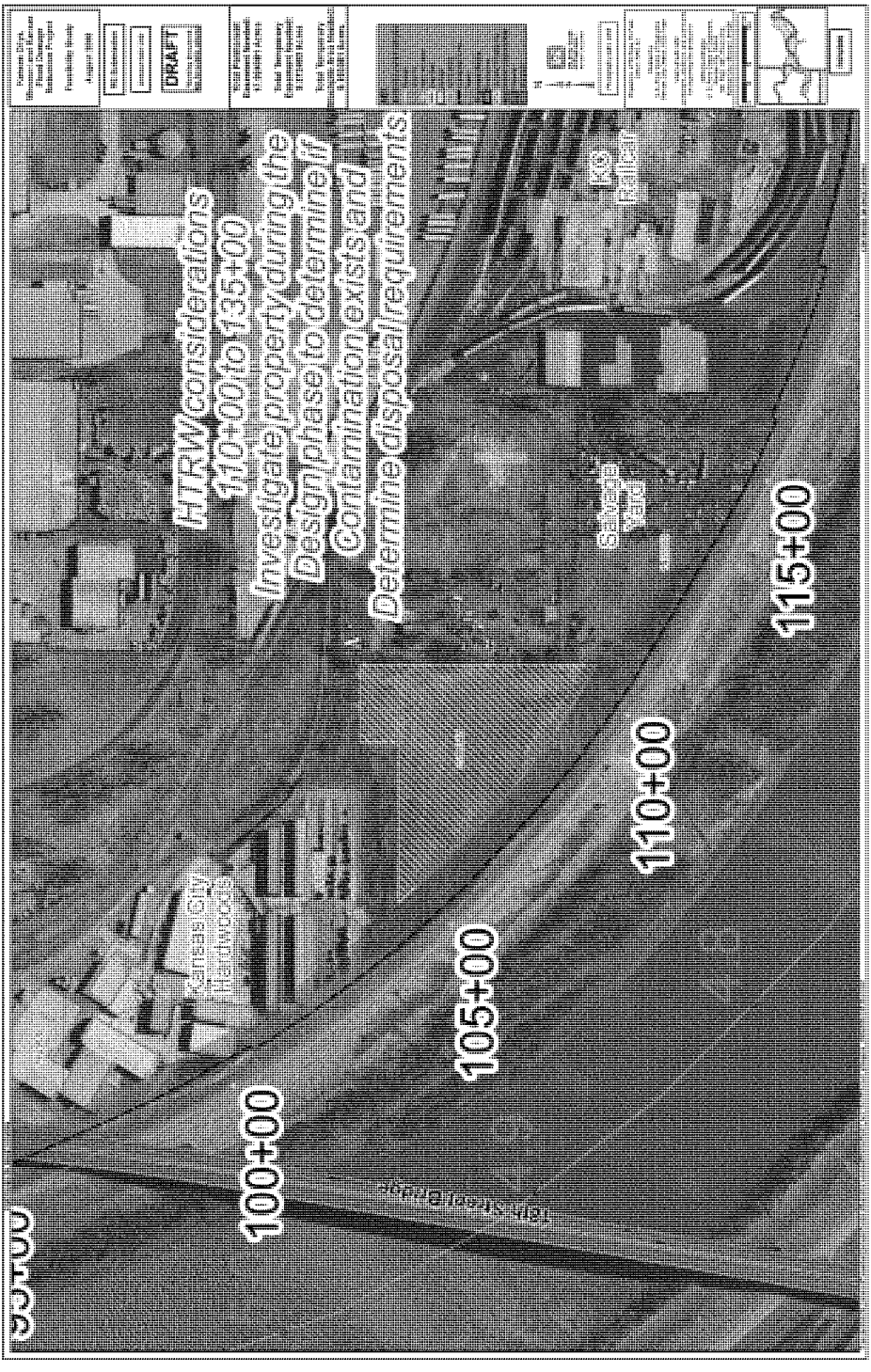
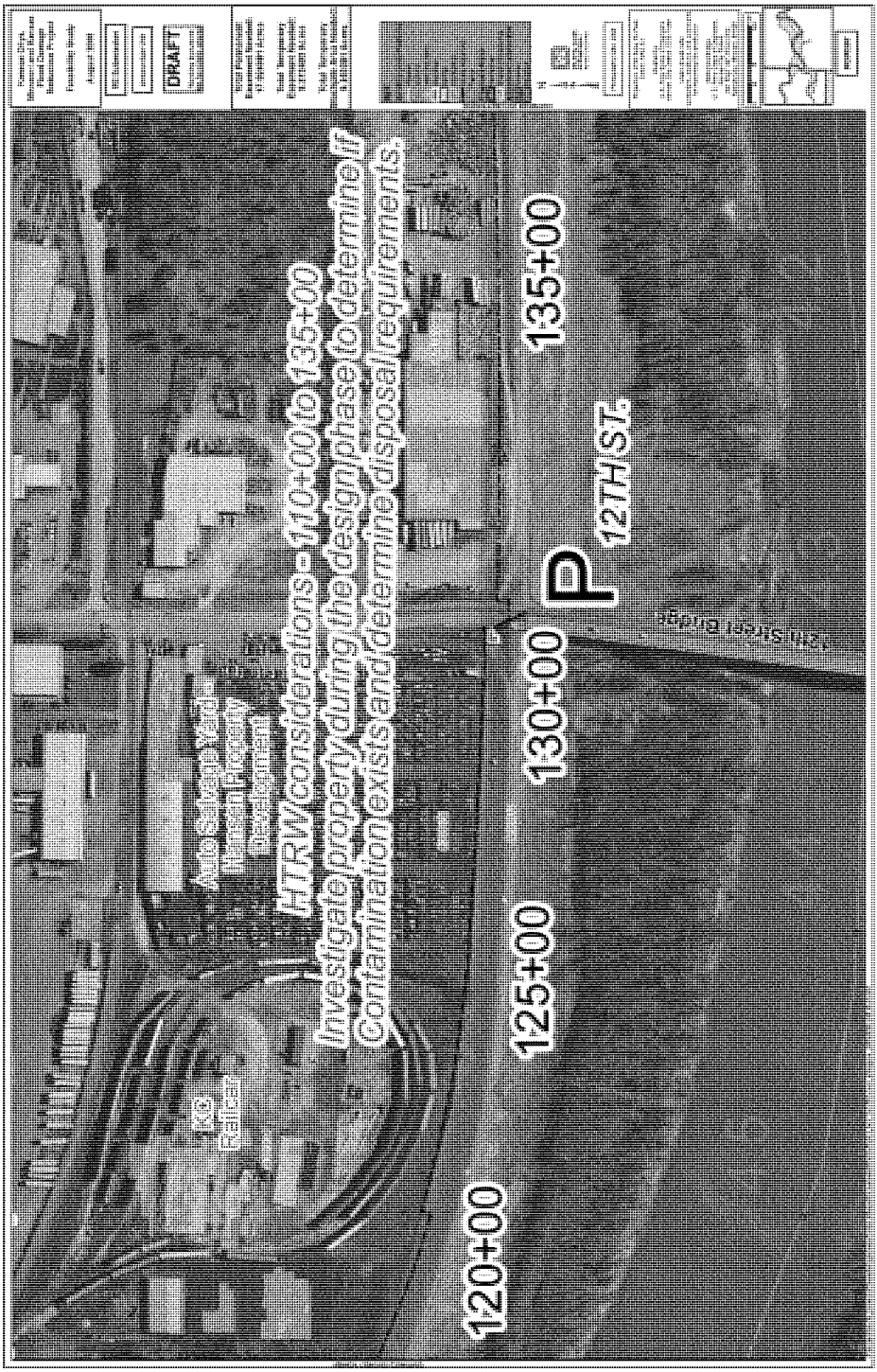
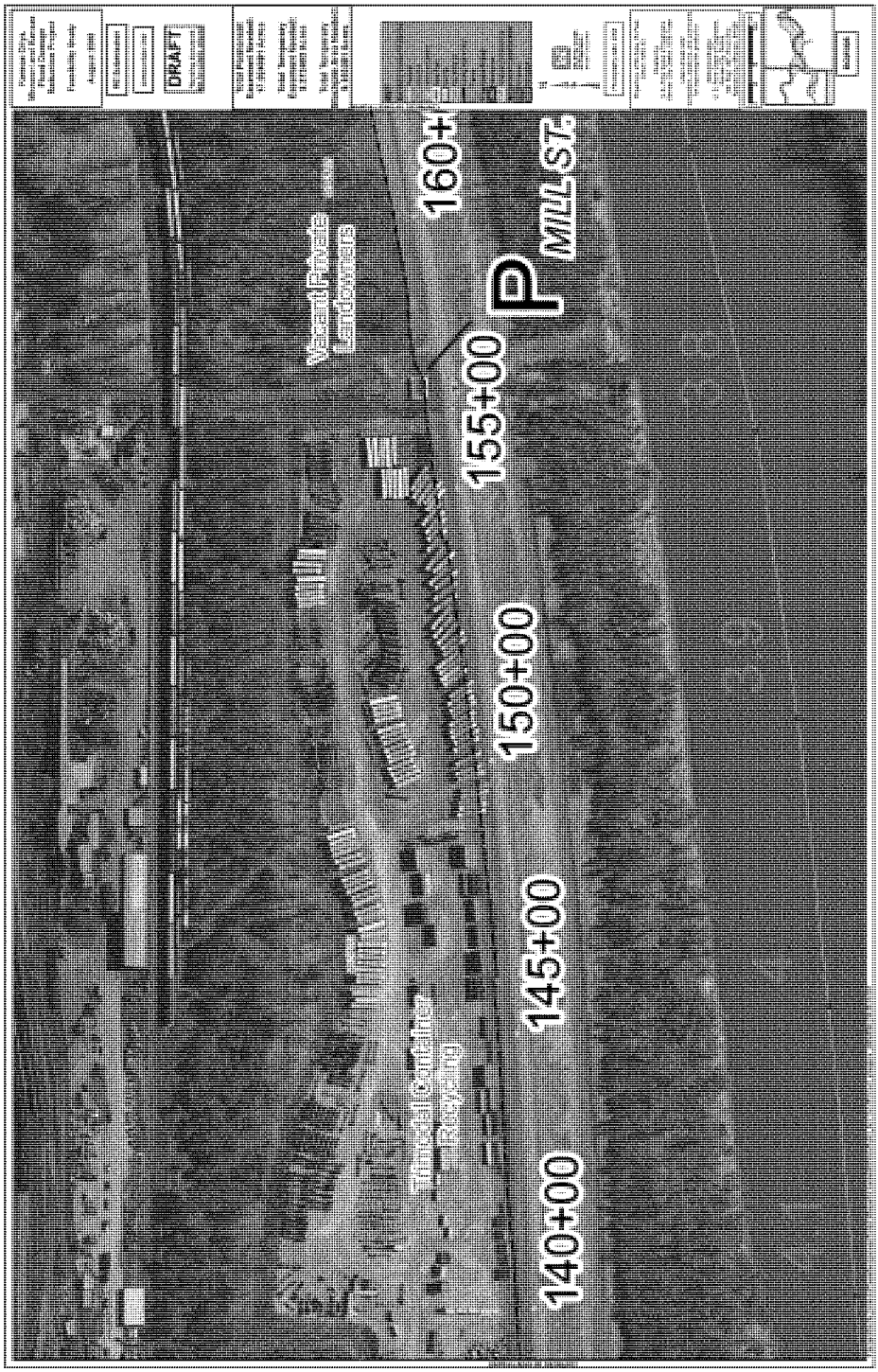

HTRW considerations
50+00 to 63+00
Any construction drebris encountered
Near the Fire Training Area should be
Removed and properly disposed.
Sta. 52+45 - 3" DIP Water Pipe
Procter & Gamble, Inc.
40+00
45+00
50+00
55+00
60+00
DRAFT

Sta. 62+70 - 6" Duct Fiber Optic Cable
65+00
70+00
75+00
PLOSAGE
DRAFT

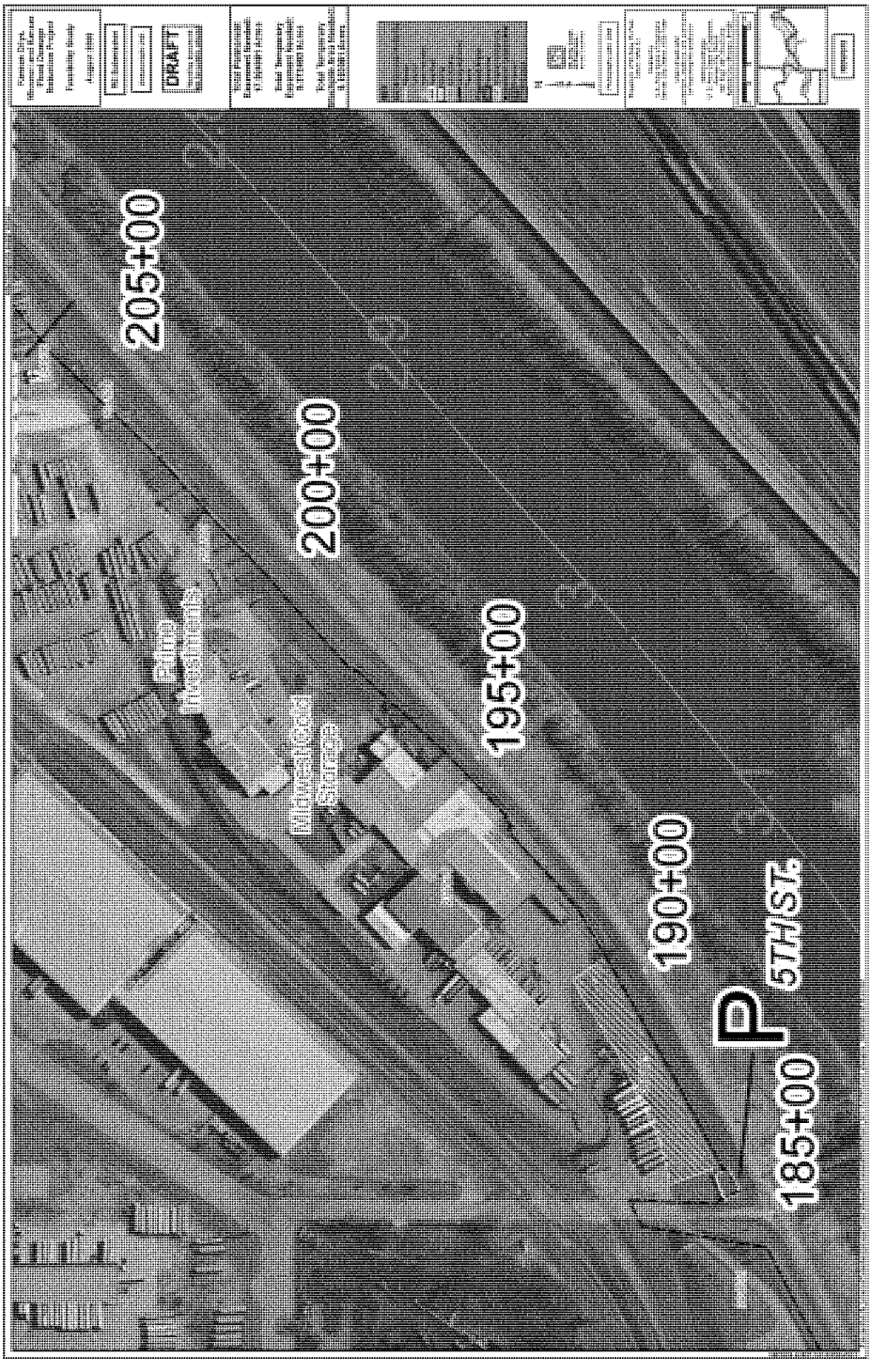

80+00
85+00
90+00
95+00


HTRW considerations
110+00 to 135+00
Investigate property during the
Design phase to determine if
Contamination exists and
Determine disposal requirements
Kansas City Hardwoods
Salvage Yard
KC Railcar
100+00
105+00
110+00
115+00
16th Street Bridge
DRAFT

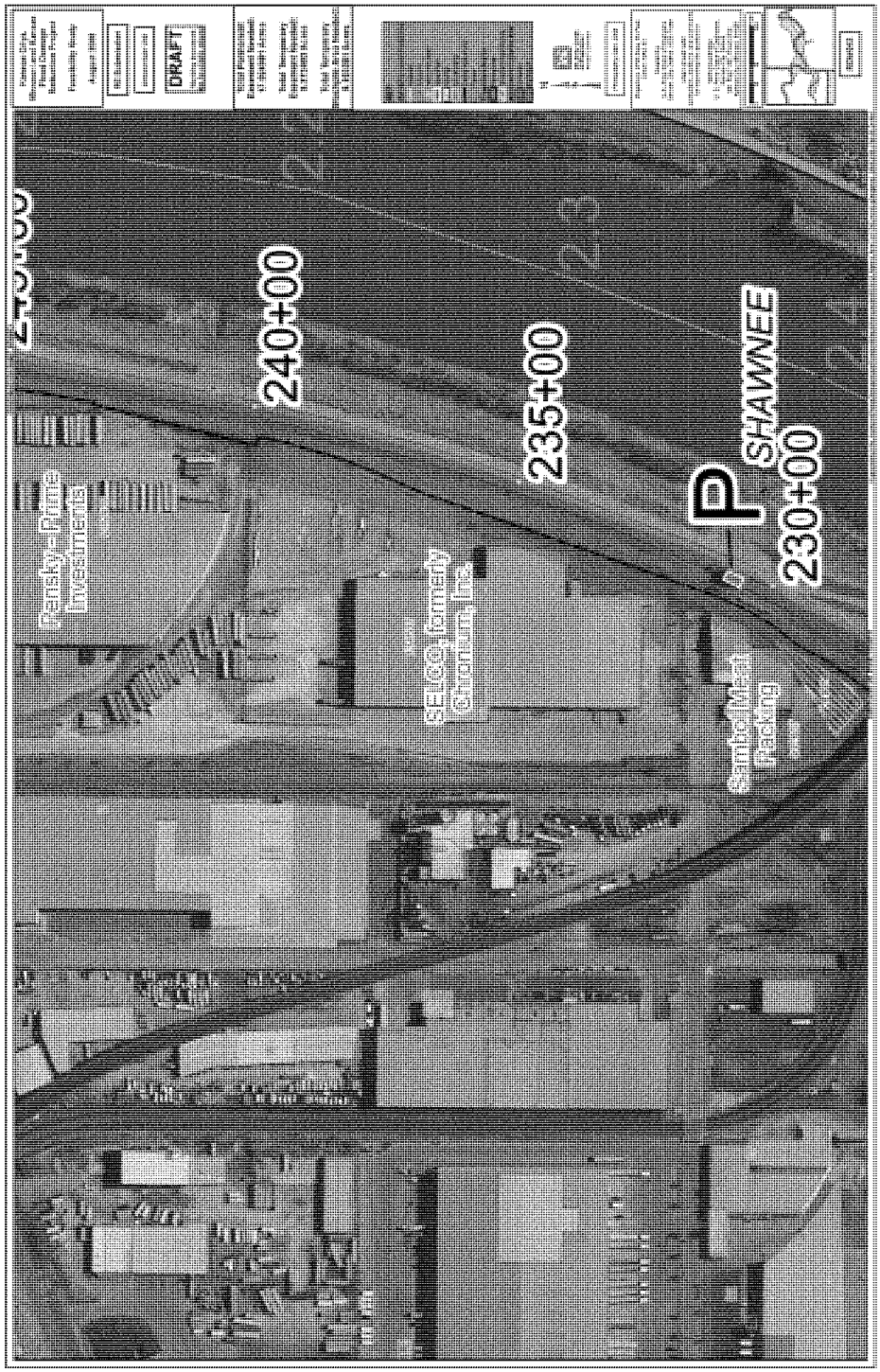

HTRW considerations - 110+00 to 135+00
Investigate property during the design phase to determine if
Contamination exists and determine disposal requirements.
120+00
125+00
130+00
135+00
P
12TH ST.
12th Street Bridge
Auto Salvage Yard - Hansen Property Development
KC Railcar
DRAFT


DRAFT
Vacant Private Landowners
Trimodal Container Recycling
160+
155+00
150+00
145+00
140+00
P
MILL ST.

DRAFT
180+00
175+00
170+00
165+00
160+00
7th Street Bridge
7L ST.


205+00
200+00
195+00
190+00
185+00
P
5TH ST.
Palmo Investments
Midwest Good Storage
DRAFT

230+00
Dean Co. - Overland Park
State Bank & Trust
225+00
Glass Co. - Prima Investments
220+00
Schock Truck &
Leasing
Station
equation change
206+12.43 BK =
212+00 AH
215+00
205+00
DRAFT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240+00
235+00
230+00
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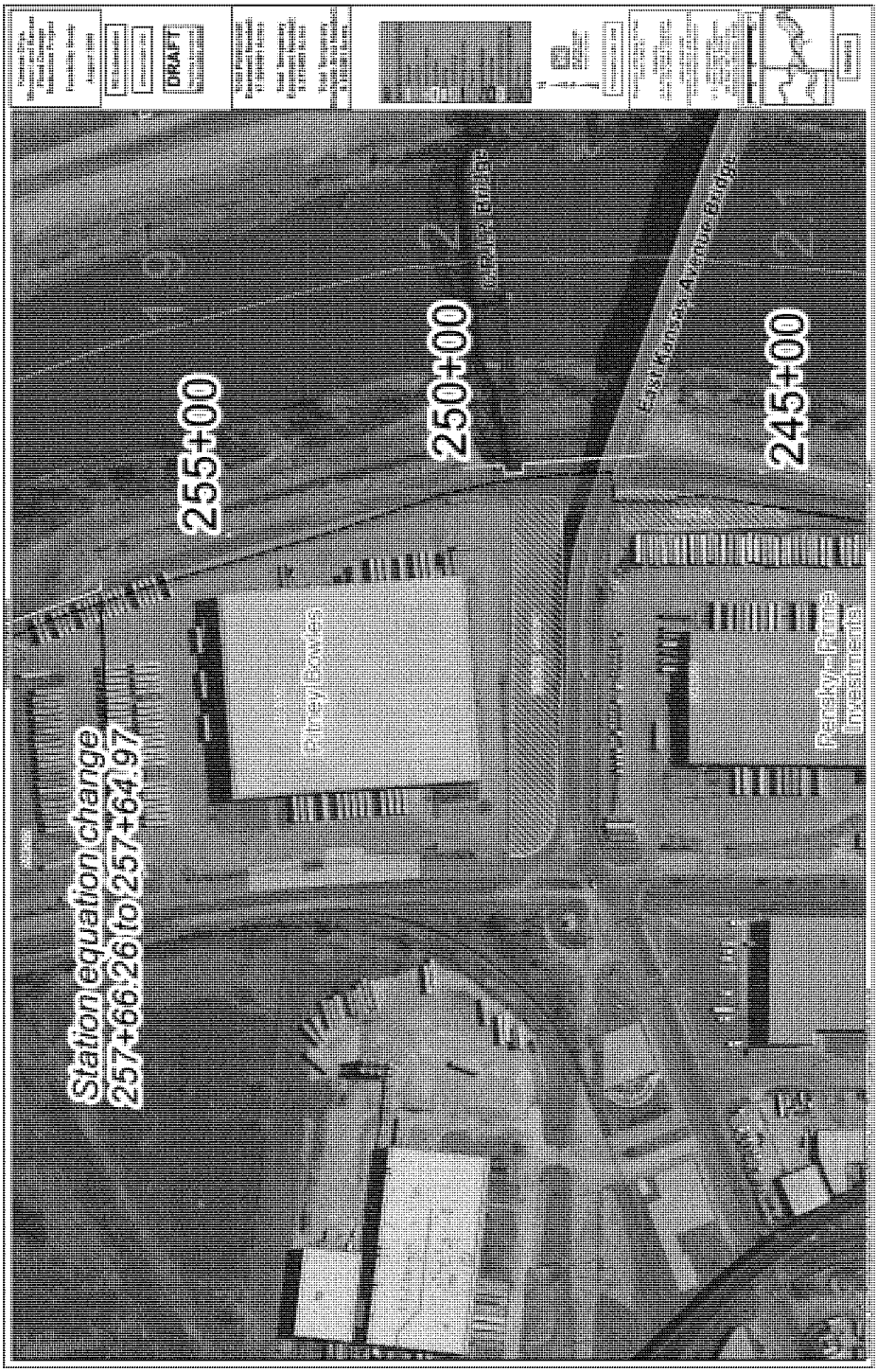
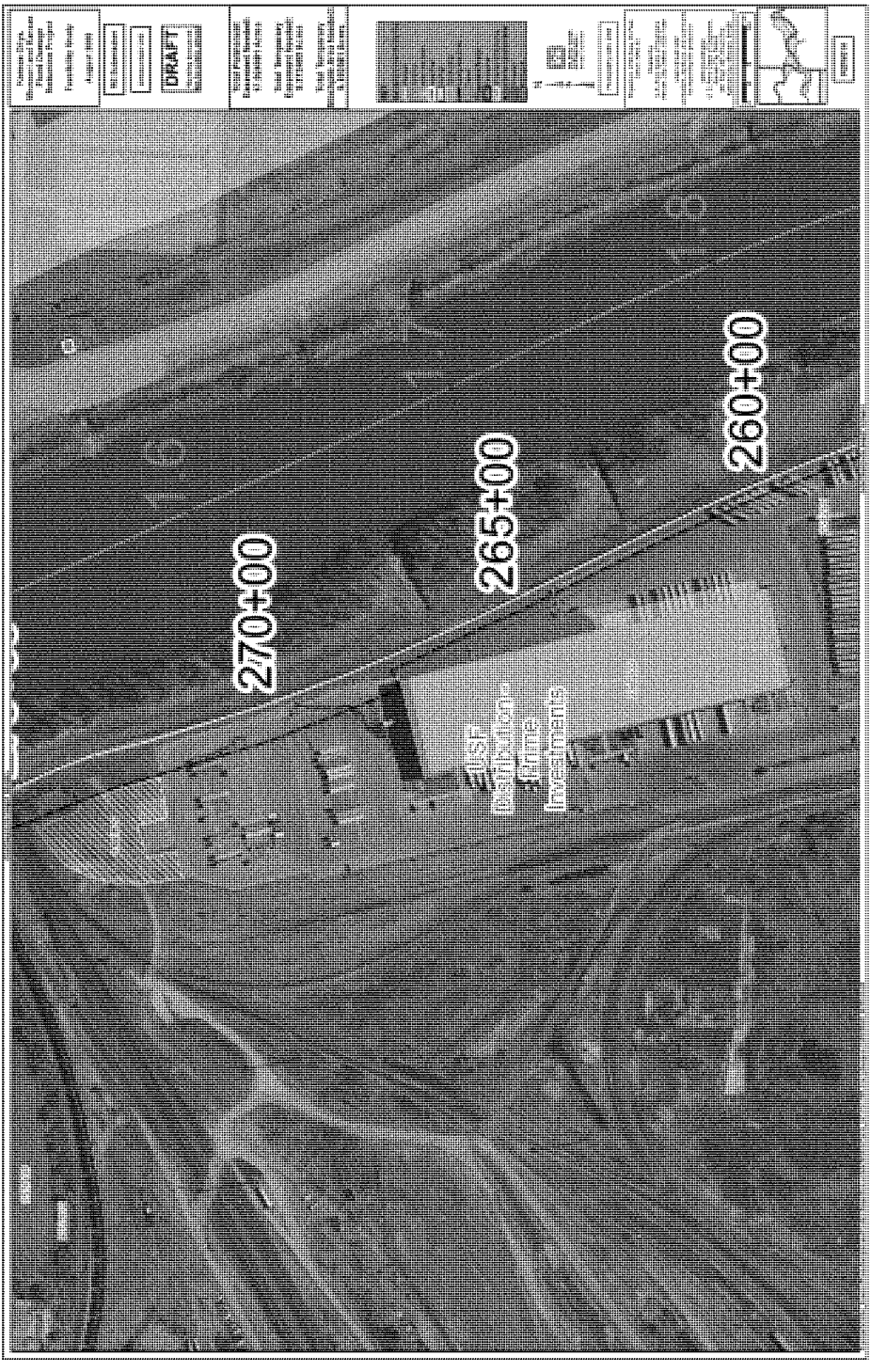
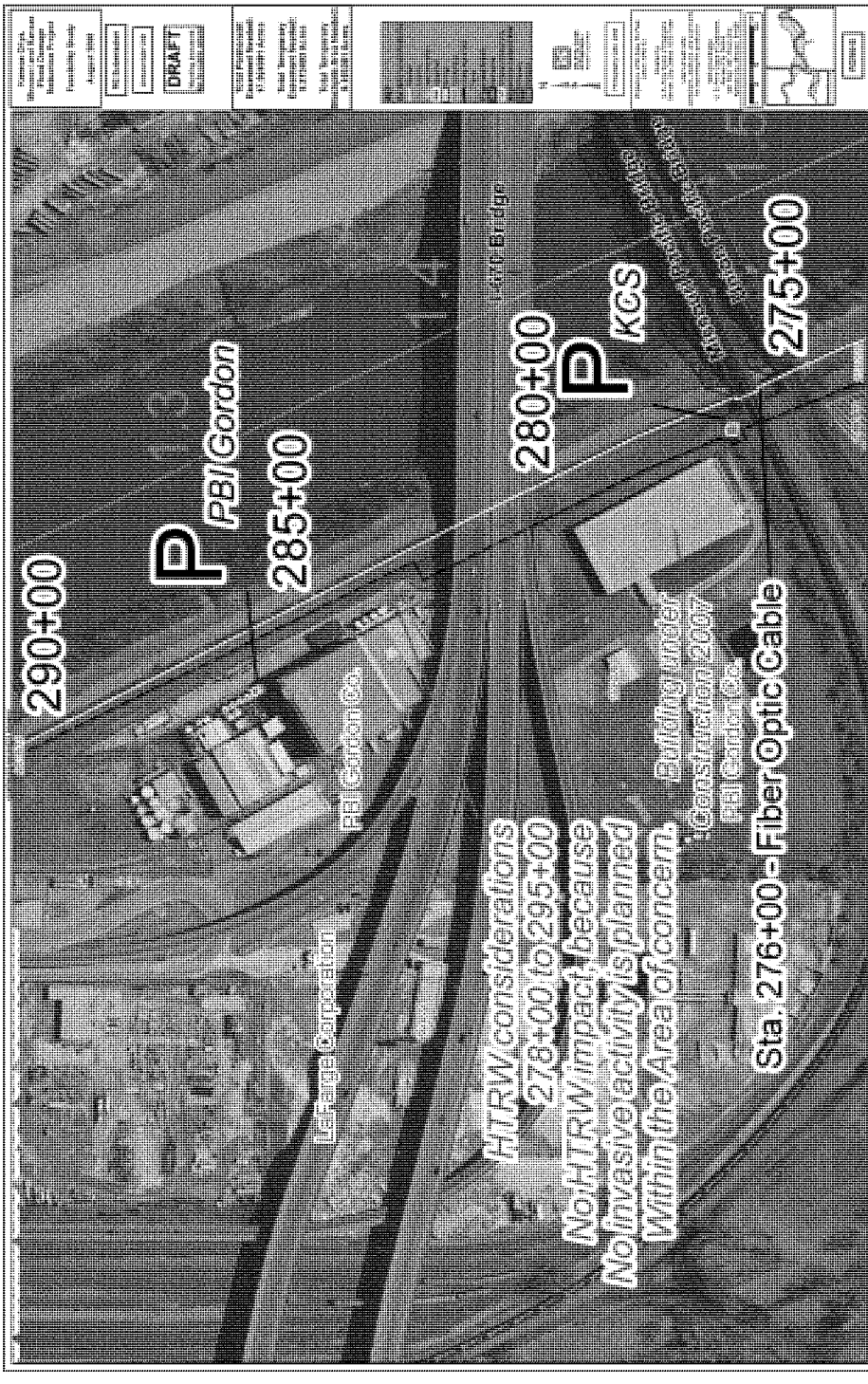
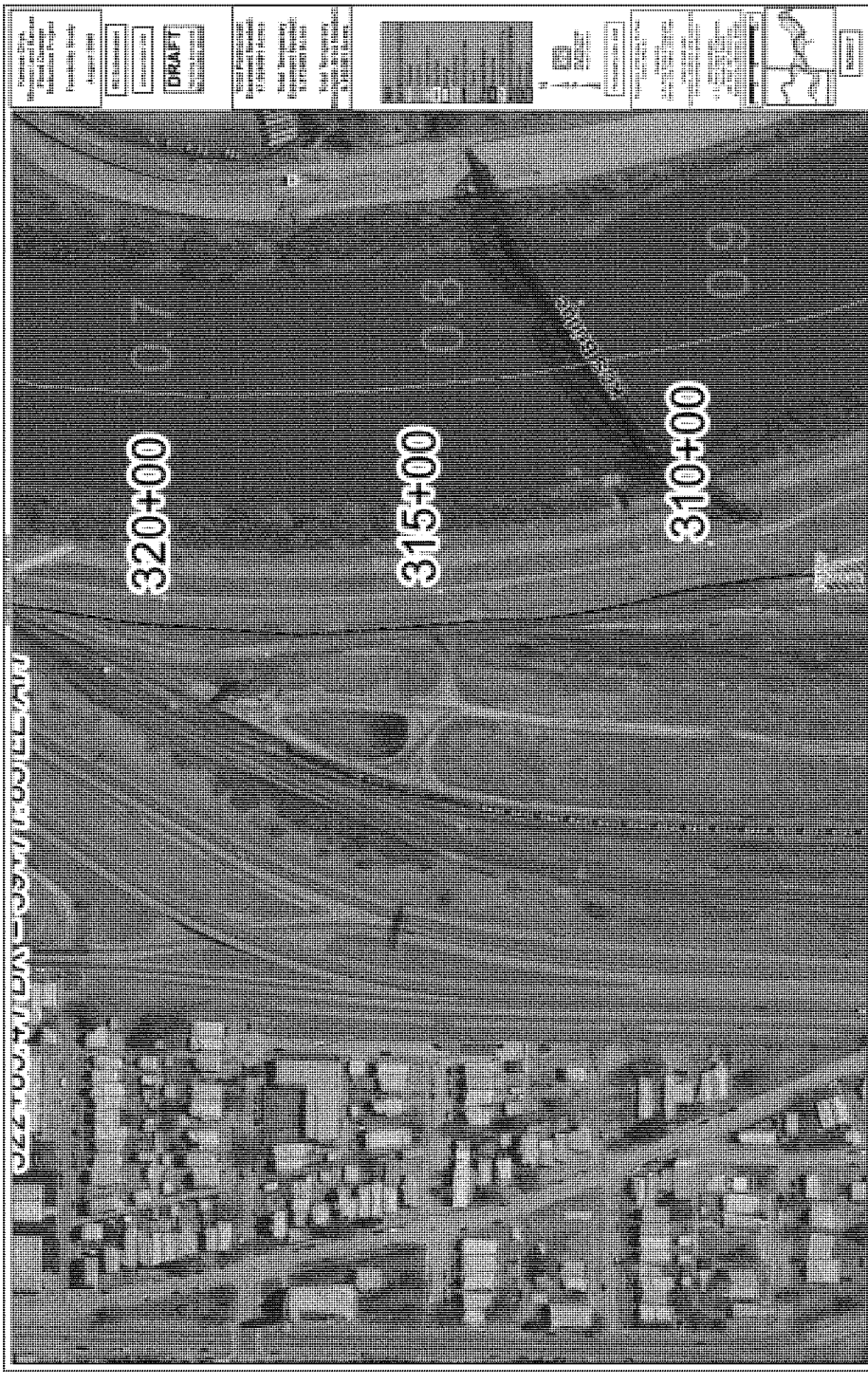
SHAWNEE
Pensky - Prime Investments
SELCO, formerly Chronium, Inc.
Sambo Meat Packing


Station equation change
257+66.26 to 257+64.97
255+00
250+00
245+00
Pitney Bowles
East Kansas Avenue Bridge
Pensky-Prime Investments
DRAFT

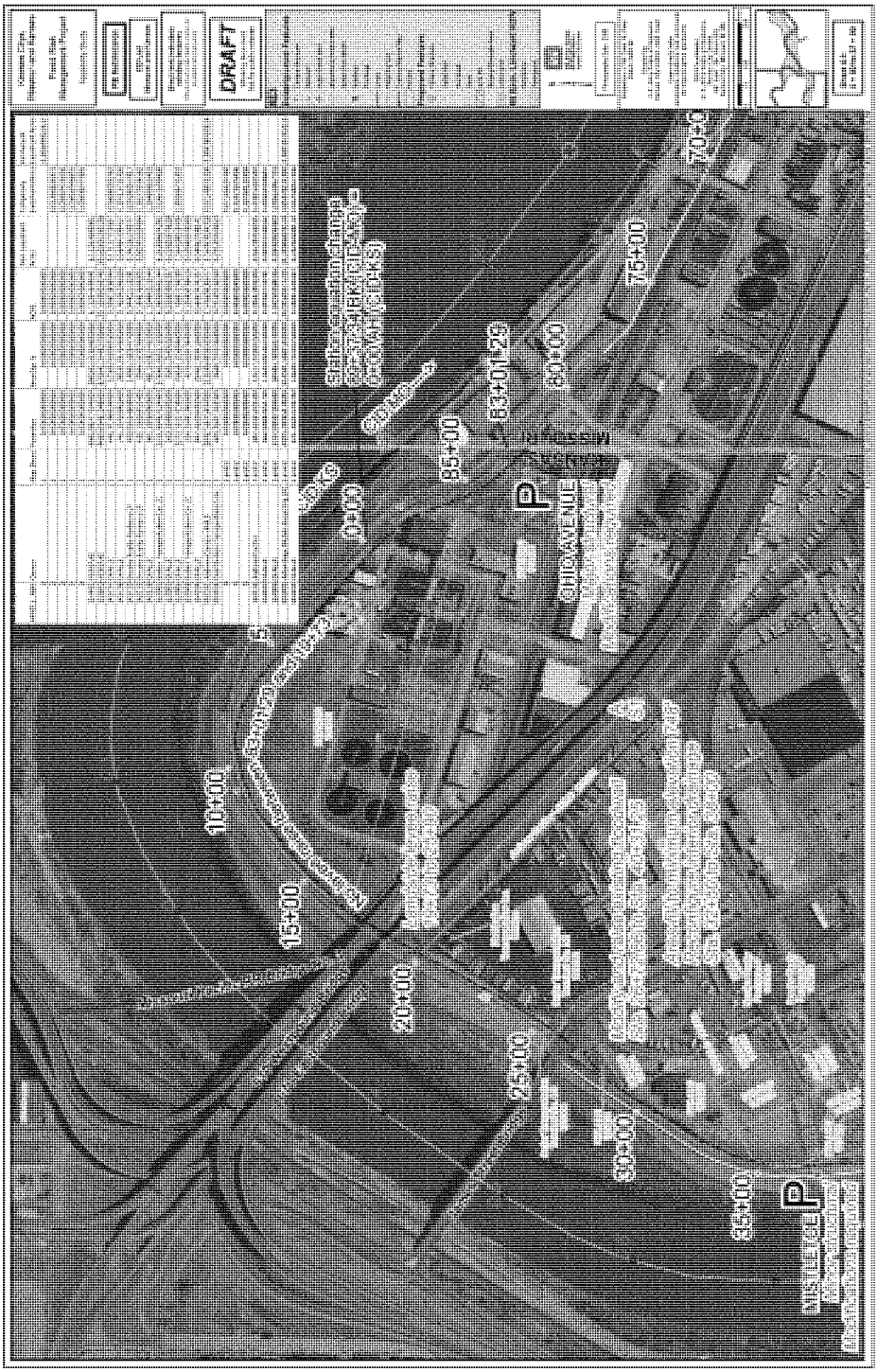

270+00
265+00
260+00
USF
Distribution
Prime
Investments
DRAFT


290+00
P PBI Gordon
285+00
280+00
P KCS
275+00
PBI Gordon Co.
LaFarge Corporation
HTRW considerations
278+00 to 295+00
No HTRW impact, because
No invasive activity is planned
Within the Area of concern.
Building under
Construction 2007
PBI Gordon Co.
Sta. 276+00 - Fiber Optic Cable
DRAFT

DRAFT
305+00
300+00
CENTRAL
BEEF
295+00
290+00
HTRW considerations - 278+00 to 295+00
No HTRW impact because no invasive activity is
Planned within the area of concern


320+00
315+00
310+00
DRAFT

55+00 LE
50+00 LE
45+00 LE
40+00 LE
James Street Bridge
New Sandpad Cap
Station equation change
322+85.41 BK = 39+71.83 LE AH
DRAFT


DRAFT
10+00
15+00
20+00
25+00
30+00
35+00
0+00
85+00
83+01.29
80+00
75+00
70+0
OHIO AVENUE
MISSILE CELL

DRAFT
40+00
45+00
50+00
55+00
60+00
65+00
70+00

DRAFT
70+00
75+00
80+00
85+00
90+00
95+00
100+00
STOCKYARDS #2
GATEWAY 2000
FEED PUMP HOUSE

DRAFT
KEMPER ARENA
100+00
105+00
110+00
115+00
120+00
125+00
130+00
Turkey Creek Outfall Sewer

DRAFT
130+00
135+00
140+00
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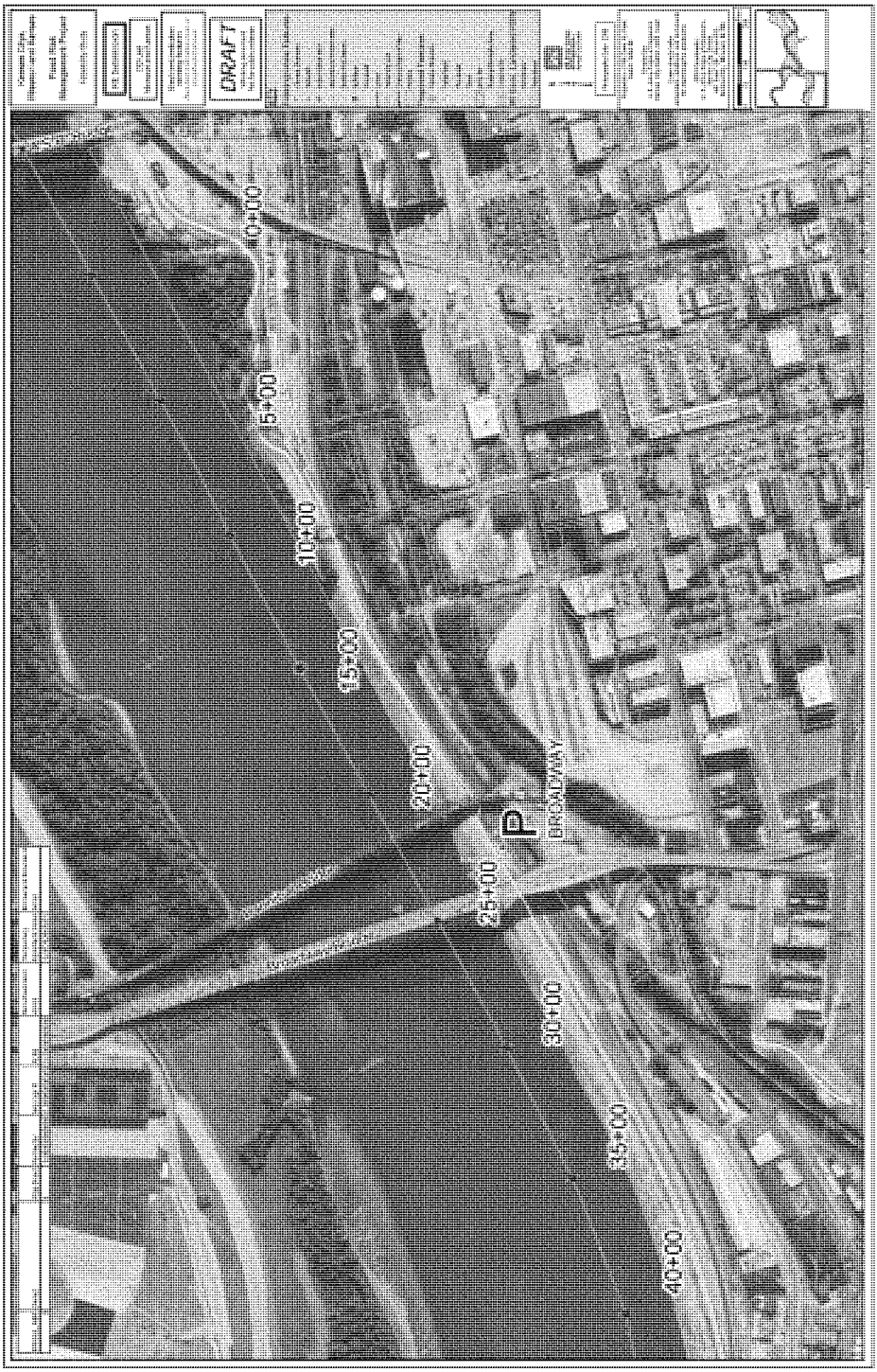
150+00
155+00
160+00
165+00


DRAFT
0+00
5+00
10+00
15+00
20+00
25+00
30+00
35+00
40+00
P
BROADWAY

US Army Corps
of Engineers
Kansas City District

Appendix C

Real Estate Plan

Armourdale Levee Unit
Central Industrial District Unit

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

LISTS OF PROPERTY OWNERS

EXHIBIT B

Landowners - Armourdale

Parcel #	Owner	Description	Street Address	City	State	Zip Code	Subdivision
134500	Glass Co. - Prime Investments, Inc.	PT L3: BEG 215FT E OF NW COR; S 467.52FT TO S LN, E 91.84FT, NELY CUR LT- 563.3FT, N55W-77FT TO N LN, W 399.8FT TO POB 4016-0837	35 Osage Ave 41 Osage Ave	Kansas City	Kansas	66105	Schwarzschild & Sulzberger Co.
134503		PART OF L4 AS FOLL: BEG SE COR L4 NELY ALG E LN 150 FT M/L, SWLY ALG R/R 295FT M/L, E ALG S LN L4 158FT M/L TO POB CONTG .2AC M/L ASSESSED WITH 2/3A/4109-3-2 3014-0491	No Address	Kansas City	Kansas		Schwarzschild & Sulzberger Co.
134506	Schock Truck and Leasing	PART OF L4 AS FOLL: BEG SE COR L4 NELY ALG E LN 150 FT M/L, SWLY ALG R/R 295FT M/L, E ALG S LN L4 158FT M/L TO POB CONTG .2AC M/L ASSESSED WITH 2/3A/4109-3-2 3014-0491		Kansas City	Kansas		
134601	SELCO (Formerly Chronium, Inc.)	PT LOT B TO LOT H: BEG 40FT W & 256FT S OF SW COR LOT H; N 897.59FT, E 641.63FT, S13W-627.33FT, CUR RT- 81.5FT, N73W-43.21FT, S11W-10.02FT, W 25.75FT, S16W- 222.47FT, W 335.04FT TO POB CONTG 10.57AC M/L 3497-0552		Kansas City	Kansas		
142500	APAC - Wilkerson-Maxwell Co.	L1 TO L13 SW OF RR, ALL L14 TO L18	300 Central Ave	Kansas City	Kansas	66118	Loomis Riverside Add
142818	Salvage Yard	BLOCK B, ALSO 1/2 VACATED STREET ADJACENT ON NORTH 2044-0255	1162 S 14th St	Kansas City	Kansas	66105	Bairds, F J Add
142900		L2 TO L5 L13 TO L32 LESS PARTS TO TURNPIKE; L33 TO L36 LESS E 30FT (ASSESSED WITH 2/3B/3542)		Kansas City	Kansas		
153500	K.C. Southern RY	B2 L1 TO L10 ALSO N 1/2 VACATED STREET ADJACENT ON S		Kansas City	Kansas		
153501	Wilkerson, J. Ted Inc.	B3 L1 TO L8 ALSO S 1/2 VACATED STREET ADJACENT ON N L1, E 119FT L9 TO L29 1691-0488	402 Central Ave	Kansas City	Kansas	66118	Riverside
153502	Bunker Properties Inc.	B3 L1 TO L8, N 13.8FT L9, ALSO 20FT ADJACENT ON E		Kansas City	Kansas		

Landowners - Armourdale

Parcel #	Owner	Description	Street Address	City	State	Zip Code	Subdivision
153700	PBI Gordon Co.	L1 TO L2, L3 LESS N PART, ALL L4 TO L6; ALSO RIVERSIDE B7 L1 TO L4	300 S 3rd St. 233 Bunker Ave	Kansas City	Kansas		Morris & Co Lands Plat
153701	Bunker Properties Inc.	N PART L3 NORTH OF VIADUCT APPROACH	201 Viaduct Appr	Kansas City	Kansas	66118	Morris & Co Lands Plat
249400	Prime Investments, Inc.	L1 3893-0191	1 Kansas Ave	Kansas City	Kansas	66105	Prime Industrial Development II
258200	Prime Investments, Inc.	LOT 1A	557 River Park Dr.	Kansas City	Kansas	66105	River Park
258201	Pitney Bowles	L1 4218-0478		Kansas City	Kansas		River Park
258202	USF Distribution - Prime Investments	L2 4182-0207	433 River Park Dr.	Kansas City	Kansas	66105	River Park
258500	Midwest Cold Storage	L1 5356-0869	1101 S 5th St	Kansas City	Kansas	66105	Prime Industrial Development II
258501		L2 4034-0636	555 River Park Dr.	Kansas City	Kansas	66105	
902607	Sambol Meat Packing	75A-1 A TR IN NE1/4TH S OF SHAWNEE AVE & E OF KC BELT RY N & W OF KC BELT RY .80A L.10A LS 0.07AC TO SAMBOL PKG CO TR75A2 BEG AT PT ON S LI SHAWNEE AVE 507.8FT E 1509-0127	13 Shawnee Ave	Kansas City	Kansas	66105	None
902608	Door Co. - Overland Park State Bank & Trust	75C BEG SE COR OSAGE & ADAMS; E 201.5FT, S54E-77FT TO HARBOR LINE, NELY ALONG HARBOR LINE-305.4FT TO S LINE RR, N66W-30FT, CUR LT- 447.17FT, S 280FT TO POB 3606-0357	30 Osage Ave	Kansas City	Kansas	66105	None
902609		BEG INT S LN OSAGE AV & NW LN ARMOURDALE PKWY, E 1193.6FT, S 467.52FT, W 23.87FT T0 TPOB; SWLY ALG CUR 166.75FT, N47E-43.35FT CUR LT 184.67FT, W 117.5FT TO POB 4146-0346	35R Osage Ave	Kansas City	Kansas	66105	None
902802	LaFarge Corporation	L-N BEG 20FT E OF SE COR B7 L4: RIVERSIDE ADD; S 1118.26FT, NWLY CUR RT- 1160FT M/L, N 273.18FT, E 478.95FT, N 50FT, E 200FT TO POB CONTG 14.077AC M/L 3606-0309	350 S 3rd St. 317 S 3rd St.	Kansas City	Kansas		None

Landowners - Armourdale

Parcel #	Owner	Description	Street Address	City	State	Zip Code	Subdivision
902803	Kansas City Stock Yards Company	O BEG 130 W & 150 S NW COR NE1/4, NE1/4; S85, SELY 680, NWLY 770 TO POB CONTG .680AC	151 Viaduct Appr	Kansas City	Kansas	66118	None
902906	K.C. Southern RY	136J COM 550FT N NE COR CENTRAL & 4TH ST E 103.7FT NW TO E LI 4TH ST S TO BEG ALSO E 1/2 VAC 4TH ST ADJ ON W	314R Central Ave	Kansas City	Kansas	66118	None
902911	Wilkerson-Maxwell Co.	1361 BEG 60.1FT E OF SE COR B3-RIVERSIDE; ELY 79FT, N 330FT, E TO WLY LINE OF RR R/W, NWLY ALONG R/W TO E LINE OF 4TH ST, S TO POB; & E 1/2 OF VACATED 4TH ST LESS N 195FT 0957-0021	314 Central Ave	Kansas City	Kansas	66118	None
904812	Private Landowners	95C-2&96A-11 COM 1492.65FT S OF NW COR SW1/4TH E410.56FT NELY 840.28FT TO PT IN K C S RY RT OF WAY NE AL RT OF WY 52FT S TO PT 230FT N OF HARBOR LI E134.5FT S40FT 1363-0615	1200R S 7th Trfy	Kansas City	Kansas	66115	None
904813		95C-1 BEG 1339.36FT E & 1101.55FT S OF NW COR SW1/4; SWLY- 206FT, S 375FT M/L, NELY-206FT, N TO POB LS 0.074AC M/L 3004-0264	1200 S 7th Trfy	Kansas City	Kansas	66115	None
904816	Schock Co.	88A1,92A,92B TRACT 93,95B1 BEG 778.25 S & 30.01 W OF NE COR SW1/4;S 83.81, S61W-729.28,SWLY-CUR RT- 338.38,SWLY-CUR RT-169.22 TO E LN 7TH ST, N19W-46.87, NWLY-CUR RT-302.10, N 27.33, N 73E-175.23,S10.22, N 73E-1075.89 S82.89 TO POB 3587-0510	1210 S 5th Street 1200 S 5th Street	Kansas City	Kansas	66105	None
905702	Trimodal Container Recycling	66AR-67 BEG 1657.14FT S & 40FT E OF NW COR SE1/4; E 900FT, N 218.56FT, S81E-429.65FT, E 1266.94FT, S 258.74FT, S81W-82.4FT, WLY CUR RT- 173.66FT, SLY-10FT, WLY CUR RT-796.64FT, NLY-10FT, WLY CUR RT-1533.36FT, N47W-13.97FT, N 95.21FT, W 8.2FT, N 194.32FT TO POB 3146-0021	1161 S 12th St.	Kansas City	Kansas	66105	None

Landowners - Armourdale

Parcel #	Owner	Description	Street Address	City	State	Zip Code	Subdivision
908002	Kansas City Hardwoods	62,63-364,64-1 BEG 152.65FT E OF NW COR SW1/4; E 100FT, S 117.50FT, W 7.35FT, S 316.68FT TO S LN RR, SE, S & SELY TO W LN BAIRDS ADD, S 558.55FT TO HARBOR LN, NWLY-1012.90FT, N57E-10FT, CUR RT-235.24FT TO E LN 18TH ST, N 119.58FT E 47.65FT, N 30FT,W 47.65FT N 182.50FT, E 7.64FT,CUR RT 38.66FT, N49E-37.89FT, N02E 87.61FT, E 45.51FT, N 182.5 TO POB ALSO CLIFTON PLACE ADD: L2-5;L13-32, LS TURNPK 3727-0017	1675 Argentine Blvd	Kansas City	Kansas	66105	None
908004	KC Railcar	65A2,A3 BEG 1339.86FT S & 599.88FT W OF NE COR SW1/4; S 576.02FT, CUR RT-473.34FT, S15W-15FT, CUR RT-271.95FT, N 694.51FT, S64E-99.87FT, CUR LT-539.29FT, CUR LT-139.84FT TO POB 3465-0737	1147 S 14th St	Kansas City	Kansas	66105	None
908005	Auto Salvage Yard - HansenProperty Development	65A2-1 BEG 1305.73FT S & 40FT W OF NE COR SW1/4; W 560.39FT, S 651.6FT, ELY-560.41FT, N 672FT TO POB CONTG 8.51AC M/L 3362-0231	1142 S 12th St	Kansas City	Kansas	66105	None
908011		65-A2-1-2 PART OF SW1/4 AS FOLL: BEG N BANK OF KANSAS RIVER @ W LINE OF 12TH ST; N 40FT, W 560FT, S 40FT, E 560FT TO POB CONTG 0.527AC M/L 1370-0571	1150 S 12th St	Kansas City	Kansas	66115	None
908901	Kaw Valley Drainage District	TR 92B TR IN NE COR SE1/4 BEING W OF 18TH & S KS TURNPIKE	940 S 18th St	Kansas City	Kansas	66105	None
908912		92A1-1 20FT STRIP IN SE1/4, C/L DESC: BEG W LN 18TH ST & 301.36FT S OF N LN SE1/4; N85W-93.94FT TO LEVEE R/W 3184-0721	1010 S 18th St	Kansas City	Kansas	66105	None
909000	Kaw Valley Power	83,83A,B,C A TR IN NE1/4NE1/4 CONTG 13ACM/L	2105 Kansas Ave	Kansas City	Kansas	66105	None

Landowners - Armourdale

Parcel #	Owner	Description	Street Address	City	State	Zip Code	Subdivision
909001	Inland Container	84-B BEG 191FT S & 878.02FT W OF NE COR; S 956FT, N32W- 910.42FT, N47W- 281.83FT, E 699.3FT TO POB CONTG 6.9487AC M/L LS 0.07AC M/L FOR R/W BAL CONTG 6.8787AC M/L LS 0.14AC M/L FOR R/W BAL CONTG 6.7387AC M/L	2101 Kansas Ave	Kansas City	Kansas	66105	None
909006		89-1 TRACT IN PT NE1/4 BEG W LI 18TH ST 439.3FT N S LI NE1/4 SEC CONTG 1.16AC	1900 Miami Ave	Kansas City	Kansas	66105	None
909007	Unknown	92C COM NW COR OF OSAGE AVE & KCOB & E RY NWLY 180FT TO RIVER SELY ALG RIVER TO OSAGE E36FT TO BEG .062A	2100 Osage Ave	Kansas City	Kansas	66105	None
909008	Unknown	BEG 191FT S & 1577.32FT W OF NE COR NE1/4; S47E- 281.83FT, S32E-910.42FT, S 85.70FT, CUR RT-382.52FT, N32W-292.21FT, CUR LT-292.21FT, N36W-114.39FT, CUR LT-161.64FT, CUR RT- 59.11FT, CUR LT-44.18FT, E 48.58FT TO POB CONTG 1.41AC M/L 3555-0244	2107 Kansas Ave	Kansas City	Kansas	66105	None
909104	Procter and Gamble	78A,78B,79A,80B,81; A TR IN SE1/4; BORDERED ON N BY BAYARD AV, ON W BY W/LINE 1/4 SEC & RR, ON S BY KANSAS AV, ON E BY C.R.I.& RR LS .354AC, LS 0.371AC M/L 0329-0159	1900 Kansas Ave 2077 Bayard Ave	Kansas City	Kansas	66105	None
911203	Procter and Gamble	218,219,220 TR222,223; BEG NE COR SE1/4 SW1/4; W378, S TO N LI K & MTRY, SELY ALNG RY TO W LI 26TH ST R/W, N TO POB CONTG 2.34AC M/L	484 S 26th St	Kansas City	Kansas	66105	None
912600	Richard H. Nickloy, Trust Etal	601-1 BEG 459.24FT S OF NW COR NE1/4; S59E-1000.40FT, N30E-14FT, N24E-369.29FT, NWLY ALG S LN LN K-32 TO INTER N LN UPRR, CUR LT- 536.78FT, S59E-431.08FT TO POB CONTG 11.5203 AC M/L 3372-0704	4001 Kaw Dr.	Kansas City	Kansas	66102	None

Landowners
CID Kansas/Missouri

Parcel Number	Owner
114401	KCKS/UG
131600	KCKS/UG
141727	DFALLC
141728	DFALLC
141729	BROOKS REALTY PARTNERS LP
142213	BROOKS REALTY PARTNERS LP
142218	DFALLC
153002	BUTLER TRANSPORTATION CO, LC
153003	DFALLC
153005	DFALLC
153008	BUTLER TRANSPORTATION CO, LC
154204	DFALLC
901101	ANDREW CARRIER - AMLS
903014	DISTRIBUTION TRANSPROTATION INC.
142214	ANDREW CARRIER
142400	DFALLC
153006	DFALLC
902903	ADVANTAGE METALS RECYCLING LLC
902913	DFALLC
142207	25 SOUTH WATER LLC
224804	KCKS/UG
142214	ANDREW CARRIER
142400	DFALLC
153006	DFALLC
902903	ADVANTAGE METALS RECYCLING LLC
902913	DFALLC
224800	MID SOUTH MILLING CO. INC.
224803	235 CENTRAL LLC
252800	BO PROPERTIES KC LLC
224800	MID SOUTH MILLING CO. INC.
224803	235 CENTRAL LLC
901400	LIVESTOCK EXCHANGE LLC
901401	AMERICAN CRANE & TRACTOR PARTS INC.
901402	AMERICAN CRANE & TRACTOR PARTS INC.
901500	AMERICAN CRANE & TRACTOR PARTS INC.
901400	LIVESTOCK EXCHANGE LLC
152900	C & H MIDWEST LLC
902500	K. C. TERMINAL RAILWAY CO.
902509	ATSF RAILROAD
152900	C & H MIDWEST LLC
0	ROW
0	RAILROAD
TOTALS	

US Army Corps
of Engineers
Kansas City District

Appendix C

Real Estate Plan

Armourdale Levee Unit
Central Industrial District Unit

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

NON-FEDERAL SPONSOR CAPABILITY CHECKLIST

EXHIBIT C

ASSESSMENT OF NON-FEDERAL SPONSOR'S REAL ESTATE ACQUISITION CAPABILITY

Kaw Valley Drainage District
Kansas City, Kansas
Kansas City's 7 Levee Flood Reduction Project, Phase II

I. Legal Authority:

a. Does the sponsor have the legal authority to acquire and hold title to real property for project purposes? Yes

b. Does the sponsor have the power of eminent domain for this project? Yes

c. Does the sponsor have "quick take" authority for this project? Yes

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? No

e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? No

II. Human Resource Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirement of Federal projects including P.L. 91-646, as amended? No

b. If the answer to II.a. is "yes", has a reasonable plan been developed to provide such training? N/A

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? The Non-federal Sponsor has sufficient real estate acquisition experience and also has an attorney to assist with the acquisitions. The sponsor will contract for additional assistance if necessary.

d. Is the sponsor's projected in-house staffing level sufficient considering its other work load, if any, and the project schedule? The Non-federal Sponsor has an attorney to assist with the acquisitions and the sponsor will contract for additional assistance if necessary.

e. Can the sponsor obtain contractor support, if required, in a timely fashion? Yes

f. Will the sponsor likely request USACE assistance in acquiring real estate? No

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? Yes

b. Has the sponsor approved the project/real estate schedule/milestones? Yes

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? Yes

b. With regard to this project, the sponsor is anticipated to be: Fully capable

V. Coordination:

a. Has this assessment been coordinated with the sponsor? Yes

b. Does the sponsor concur with this assessment? Yes

Prepared by:

Carla J. Buatte
Real Estate Specialist

Reviewed by:

RE-CM Team Leader

Approved by:

Gregory G. Wilson
Chief, Real Estate Division

Date: 8/26/13

ASSESSMENT OF NON-FEDERAL SPONSOR'S REAL ESTATE ACQUISITION CAPABILITY

City of Kansas City, Missouri
Kansas City's 7 Levee Flood Reduction Project, Phase II

I. Legal Authority:

a. Does the sponsor have the legal authority to acquire and hold title to real property for project purposes? **Yes.**

b. Does the sponsor have the power of eminent domain for this project? **Yes.**

c. Does the sponsor have "quick take" authority for this project? **No.**

d. Are any of the lands/interests in land required for the project located outside the sponsor's political boundary? **No.**

e. Are any of the lands/interests in land required for the project owned by an entity whose property the sponsor cannot condemn? **Yes.**

II. Human Resource Requirements:

a. Will the sponsor's in-house staff require training to become familiar with the real estate requirement of Federal projects including P.L. 91-646, as amended? **No.**

b. If the answer to II.a. is "yes", has a reasonable plan been developed to provide such training? **N/A.**

c. Does the sponsor's in-house staff have sufficient real estate acquisition experience to meet its responsibilities for the project? **Yes.**

d. Is the sponsor's projected in-house staffing level sufficient considering its other work load, if any, and the project schedule? **Yes.**

e. Can the sponsor obtain contractor support, if required, in a timely fashion? **Yes.**

f. Will the sponsor likely request USACE assistance in acquiring real estate? **There is potential that they may request assistance, however at this point in time it cannot be said for sure.**

III. Other Project Variables:

a. Will the sponsor's staff be located within reasonable proximity to the project site? **Yes.**

b. Has the sponsor approved the project/real estate schedule/milestones? **Yes.**

IV. Overall Assessment:

a. Has the sponsor performed satisfactorily on other USACE projects? **Yes.**

b. With regard to this project, the sponsor is anticipated to be: **Fully capable.**

V. Coordination:

a. Has this assessment been coordinated with the sponsor? **Yes.**

b. Does the sponsor concur with this assessment? **Yes.**

Prepared by:

Carla J. Buatte
Real Estate Specialist

Reviewed by:

RE-CM Team Leader

Approved by:

Gregory G. Wilson
Chief, Real Estate Division

Date: 8/26/13

U.S. Army Corps of Engineers, Kansas City District

Final Feasibility Report

APPENDIX D

HAZARDOUS, TOXIC, AND RADIOLOGICAL WASTE

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

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LIST OF ACRONYMS

AIRS - Aerometric Information Retrieval System
AKA – also known as
AST – Aboveground Storage Tank
BPU – Board of Public Utilities
CERCLIS – Comprehensive Environmental Response, Compensation, and Liability Information System
CID – Central Industrial District
EPA – Environmental Protection Agency
ERIIS - Environmental Risk Information and Imaging Services
HTRW – Hazardous, Toxic, and Radioactive Waste
K.C. – Kansas City
KCD – Kansas City District
KDHE – Kansas Department of Health and the Environment
LNAPL – Light Non Aqueous Phase Liquid
LUST –Leaky Underground Storage Tank
MCL – Maximum Contaminant Limit
NPDES – National Pollutant Discharge Elimination System
PA – Preliminary Assessment
PAH – polyaromatic hydrocarbons
PCBs – polychlorinated biphenyl
P&G – Procter & Gamble
RCRA – Resource Conservation Recovery Act
RFA – RCRA Field Assessment
ROD – Record of Decision
ROW – Right of Way
RSK – Risk-based Standards
SVOC – semivolatile organic compounds
SWMU – Solid Waste Management Units
TRIS –Toxic Release Inventory System
TSS – Total Suspended Solids
UPRR – Union Pacific Railroad
USACE – United States Army Corps of Engineers
UST - Underground Storage Tank
VOC - volatile organic compounds

1.0 INTRODUCTION

The Kansas City District performed a Hazardous, Toxic, and Radiological Waste (HTRW) site assessment as part of the Feasibility Study for the Kansas City Levees Project in accordance with ER 1165-2-132 "Hazardous, Toxic and Radioactive Waste Guidance For Civil Works Projects". The Project includes seven existing levees in the Kansas City area. A map of the Study Area is shown in Figure 1.0. The levee system has been divided into two phases for the Feasibility Report, as follows:

Phase I

- Argentine Levee Unit
- Fairfax-Jersey Creek Levee Unit
- North Kansas City Levee Unit
- East Bottoms Levee Unit

Phase II

- Armourdale Levee Unit
- Central Industrial District (CID) Levee Unit

Some areas adjacent to the existing levee units are industrialized and potentially have contamination in the soil or groundwater resulting from past industrial activities. The primary purpose of this assessment is to identify any hazardous and toxic waste areas of concern within the Armourdale Unit and to determine how these areas may impact the proposed levee construction alternatives described in the Feasibility Study.

In accordance with ER 1165-2-132, known HTRW contamination will be avoided where possible. If avoidance is not feasible, the removal of contaminated soil or materials required for implementation of the Recommended Plan will be the sole responsibility of the local project sponsor, including all costs. All removal actions on a contaminated property must take place before any construction can begin on that property.

2. 0 PREVIOUS STUDIES

A Reconnaissance Report - HTRW Assessment and HTRW Site Follow-up Investigation were performed on the Kansas City Levee Units prior to the Feasibility Study.

2.1 Reconnaissance Report - HTRW Assessment

The Reconnaissance Report for the Flood Reduction Study was completed in August of 1999. The HTRW Assessment can be found in Appendix I of the Reconnaissance Report. During the reconnaissance phase, a preliminary HTRW Assessment was performed to identify any hazardous and toxic waste sites that could directly impact construction. The preliminary HTRW Assessment included a database search and site visits. The database search was performed by Environmental Risk Information and Imaging Services (ERIIS). It identified all industries whose addresses fell within 500 feet of the levee units that were listed in either the federal or state databases. Based on

the database search, the Reconnaissance Report – HTRW Assessment identified only Great Lakes Container Corporation, a.k.a. Container Recycling Inc. as an area of concern which needed further investigations.

2.2 HTRW Follow-up Investigation Report

In November 1999, Hydrogeologic, Inc. performed a follow-up investigation of the areas of concern identified in the Reconnaissance Report – HTRW Assessment. The Follow-up Investigation consisted of the following:

1) Performing additional database searches
2) Reviewing aerial photographs
3) Reviewing Sanborn Fire Insurance Maps
4) Reviewing site documents from Environmental Protection Agency (EPA) and the Kansas Department of Health and the Environment (KDHE)
5) Conducting personal interviews.

The database searches, which were performed by VISTA Information Solutions, Inc, included the area within a 1- mile radius of each area of concern identified during the Reconnaissance Report. The Follow-up Investigation Report summarizes the investigations and findings for each area of concern, as identified in the Reconnaissance Report.

3.0 HTRW SITE ASSESSMENT

This HTRW Site Assessment performed as part of the Feasibility Study evaluated the HTRW areas of concern with respect to the proposed Feasibility Study alternatives. This assessment identifies the nature and extent of contamination, which could directly impact the proposed Feasibility Study alternatives. The assessment includes a document search, a site evaluation, and a field investigation for select areas.

In order to identify all the HTRW areas of concerns, the following activities were performed as part of the document search and site evaluation:

- Reviewed the Reconnaissance Report – HTRW Assessment dated August 1999 and the corresponding database search dated January 22, 1999.

- Reviewed the HTRW Follow-up Investigation dated November 1999 and the corresponding database search dated September 1999.

- Reviewed information obtained from an EPA and KDHE website database searches.

- Reviewed documents obtained from EPA – Region VII files and conducted personal interviews with EPA representatives concerning Proctor & Gamble and Container Recycling.

- Reviewed documents obtained from KDHE files and conducted personal interviews with the KDHE representatives concerning underground storage tanks found in the levee area, Proctor and Gamble and PBI Gordon Corporation.

- Reviewed the following aerial photographs:
  1951 and 1993 Flood photos for Armourdale
  Armourdale Levee - 1960, 1970, 1978, 1983, 1995

- Reviewed Corps of Engineer correspondence files for the Armourdale Levee Unit. The files included correspondence from adjacent landowners on work performed within 500 feet of the levee centerline.

- Performed a site visit to the Armourdale Levee Unit

4.0 SITE EVALUATIONS

Site evaluations have been performed on all properties adjacent to the levee units to determine if there are HTRW concerns that could directly impact proposed levee improvement alternative evaluated in the Feasibility Study.

4.1 Armourdale Unit

The Armourdale Levee Unit is being reevaluated as part of the Feasibility Study for overtopping and underseepage concerns based on observation made during the 1993 flood. The 500 year + 3 feet alternative was evaluated for the entire levee unit for the hazardous waste study. This alternative includes a combination of alternatives such as levee raises, T-walls, floodwalls, underseepage control and relief wells for the various reaches along the levee. Currently, three existing systems of relief wells are located at stations 190+75 to 246+35, 268+11 to 282+29, and 296+23 to 302+57.

For the HTRW Site Assessment, the area evaluated was based on the July 2007 figures for the 500 year +3 feet alternative with an additional 40 feet beyond the proposed construction limits to account for temporary easements. Within the Kaw Valley existing right-of-way, no contamination is expected to be encountered. However, there are many areas where the alternative extends outside the existing right-of-way and impacts properties in close proximity to the levee. The areas adjacent to the Armourdale Levee are highly industrialized. There are sites which have soil or groundwater contamination that directly impacts the Feasibility Study alternative. Figure 2.0 shows the overall levee unit with the adjacent properties identified. The following summarizes the evaluation and conclusions relating to specific properties within the Armourdale Unit.

4.1.1 Procter & Gamble Manufacturing Company (P&G)

Address: 1900 Kansas Ave.
Kansas City, KS 66105
Stationing: 43+00 to 63+00
Figure No.: 2.0, 3.0

The Procter & Gamble facility has been in operation since 1911. A portion of the western property boundary lies within the construction limit established for this assessment for the Armourdale levee. Proctor & Gamble manufactures various types of soaps, detergents, cleaning products, and industrial organic chemicals. Proctor & Gamble is a RCRA permitted hazardous waste treatment, storage, and disposal facility. A RCRA Facility Assessment completed in May 1998 identified 13 Solid Waste Management Unit (SWMUs) and eight Areas of Concern (AOCs) and identified evidence of releases of hazardous waste and hazardous constituents. Only one SWMU and one AOC are located within the area affected by the levee improvements. A fire-fighting practice area (see Figure 3.0) is located on the riverward side of the levee and is listed as SWMU 7. One spill from a storm drain into the Kansas River is identified as AOC 6 and is located at an outfall to the River. No other SWMUs are located within the area required to complete the levee improvements. No RCRA corrective action has occurred at the facility at this time.

Groundwater beneath P&G travels to the south-southwest and discharges to the Kansas River. Beginning in 1919, P&G drilled a number of wells for industrial use for the facility. Two wells, #11 and #12, were located within the levee easement, (see Figure 3.0). During an investigation of the Kansas Groundwater Quality Monitoring Network in 1988 by KDHE, both of these wells were found to contain concentrations of volatile organic chemicals (VOCs) significantly above Kansas Action Levels (KALs) and MCLs. Both wells had a large turbine pump head on a concrete pump base. Small containers of lubricant or solvents were located next to the wellhead turbine and spilled oil was present in the cracks of the concrete pad and this was considered as one but not the sole source of the groundwater contamination. These wells were sampled again in 1990, 1991, and 1992 as part of Preliminary Assessment of Well #11 (1990), Screening Site Assessment (1991) and Site Inspection Prioritization (1994) by KDHE. On-site monitoring wells were found to have much lower concentrations. The chemical concentrations above action levels are found in the below table.

Table 1.0: Procter & Gamble Supply Well #11 Groundwater Concentrations

Chemical	**Action Level KAL/MCL in ug/L**	**1988 ug/L**	**1990 ug/L**	**1991 ug/L**	**1992 ug/L**
Vinyl Chloride	2	36.6	10	30.9	117
1,1 DCE	5/7	705	412.0	469	386

Table 2.0: Procter & Gamble Supply Well #12 Groundwater Concentrations

Chemical	**Action Level KAL/MCL in ug/L**	**1990 ug/L**	**1991 ug/L**
Vinyl Chloride	2	ND	2.3
1,1 DCE	5/7	383.0	578

In 1994's Site Inspection Prioritization, KDHE recommend No Further Remedial Action for the groundwater issues at these wells. KDHE planned negotiations with the potentially responsible parties for further characterization and remediation of the site. Since that time, the two wells were removed. The wells extended to a depth of 98 feet below ground surface. No remedial action has occurred to address the groundwater; therefore contamination potentially still is present beneath this area.

The KDHE LUST database lists two underground storage tanks that were considered closed. One tank was closed in 1988 by filling the tank in place after siphoning out any remaining gasoline. The other tank was removed in 1997, and no contamination was found.

During the site visit on August 12, 2005 with representatives of the Kaw Valley Drainage District (KVDD), numerous above ground storage tanks were seen near the property fence line. According to Corps of Engineers letter of correspondence with Kaw Valley Drainage District, the construction of the fire training area (SWMU 7) occurred in the summer of 1991. According to KVDD, a large amount of debris and solid waste was dumped and buried in this area as well, prior to the training area construction.

Based on the groundwater data collected by KDHE, the installation of relief wells as underseepage controls between station 45+00 and 75+00 may result in the discharge or migration of VOC contaminated groundwater and should be avoided. Since the plume has not been clearly identified, a smaller restrictive area could be established with additional investigations if relief wells are required. Since the 500+3 alternative does not require relief wells, no additional testing is recommended at this time. A few ASTs on the southern portion of the property are very close to the 40-foot construction limits. They may require removal and, depending on their contents, tank closure procedures may be required before construction commences. Also, if construction occurs on the riverward side near the SWMU 7 at approximately Station 50+00, the debris and dumped solid waste should be removed and replaced with suitable fill material. All waste should be sampled and characterized prior to disposal. The material is likely either construction or solid waste and must be taken to a solid waste landfill or appropriate disposal locations based on sampling results.

4.1.2 Inland Container Corporation

Address:	2107 Kansas Ave. Kansas City, KS 66105
Stationing:	63+00 to 75+50
Figure No.:	2.0, 3.0

Inland Container Corporation processes and handles paper, boxes, and large cardboard drums, according the KDHE Site Screening Evaluation for Proctor & Gamble. The Reconnaissance Report stated that a LUST was removed in 1989. In an interview with personnel of the KDHE Storage Tank Section on August 2, 2005, it was stated that two 20,000 gallons tank full of heating oil were removed in 1989. Inland Container has previously used 1,1,1-TCA in their past processes, according to a KDHE information request. No information on current contamination was found for

this site. Neither the Follow-Up Study nor the KDHE LUST database mentions any other hazardous waste issues for the property. No levee construction would occur on Inland Container property. The property would only be encroached on if there was a need for a temporary easement. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. No further investigation is necessary.

4.1.3 Kaw Power Station

Address: 2015 Kansas Ave.
Kansas City, KS 66105
Stationing: 75+50 to 76+00
Figure No.: 2.0, 3.0

Kaw Power Station is a fossil fuel powered station that is currently inactive. It is run by the Board of Public Utilities to provide electricity for Kansas City, Kansas. Only the southwest portion of the site is within the area of concern for the levee improvements. The Reconnaissance Study lists 6 ASTs as currently in use. Capacities for the tanks range from 12,000 gallons for diesel to 564 gallons for used oil. A NPDES permit for wastewater established discharge limits for total suspended solids (TSS), bacteria, oils and grease. According to the AIRS database for air releases, it was listed as a major air discharger. It is a Kansas Sub-Class 1 Generator of hazardous waste. It also has a permitted UST onsite.

From aerial photographs, the power plant has been at this location since 1970, a large coal pile used to fill the exterior yard of the site. Currently, coal is no longer required to generate power at the facility. From the site visit, a few ASTs are present as well as a large transformer, but are outside the construction limits. The facility also has a small structure on the levee, which is believed to be a wastewater discharge point. Records indicate no concerns regarding soil or groundwater contamination, which would impact levee improvements. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. No further investigation is required.

4.1.4 KC Hardwood Corporation (a.k.a. American Walnut)

Address: 1021 S. 18th St.
Kansas City, KS 66105
Stationing: 98+00 to 111+00
Figure No.: 2.0

KC Hardwoods is adjacent to the levee. It was also known as American Walnut Company. The Reconnaissance Report, Follow-Up Report and KDHE LUST database list three LUSTs, which were removed in 1991. The tanks were removed with no contamination present. It lists four inactive USTs that were removed from the site. The Follow-Up Report also lists the site for No

Further Action. This site is no longer located on the CERCLIS database on the EPA website. It is a permitted site for minor air releases.

There is no longer evidence of contamination associated with this property. The property would only be encroached on if there was a need for a temporary easement. Therefore no further investigation is necessary.

4.1.5 Auto Salvage Yards

Address: 1162 S. 14th St.
Kansas City, KS 66105
Stationing: 111+00 to 116+50
Figure No.: 2.0, 4.0

The auto salvage yards lie next to the levee toe. A portion of the property will be used for construction of a new levee section. The Reconnaissance Report and Follow-Up Investigation did not contain any information on these sites. They were not listed as areas of concern in any EPA or KDHE databases. Aerial photographs show the auto salvage yards have been at their current location since 1970. Their operation had not moved beyond the property boundaries. Property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

4.1.6 Kansas City Railcar Services

Address: 1147 S. 14th St.
Kansas City, KS 66105
Stationing: 116+50 to 124+00
Figure No.: 2.0

A portion of Kansas City Railcar Services will be used for construction of a new levee section. This property had numerous railroad cars parked in close proximity to the levee toe. Kansas City Railcar repairs and cleans railroad cars. Kansas City Railcar is listed as a hazardous waste permitted facility in the EPA database RCRAInfo or in KDHE files. Aerial photographs from 1970 and 1978 show that the property was either a storage yard or salvage yard. During the Flood of 1993, photos show that the property was cleared out, perhaps as a precaution in the event of a levee failure or overtopping. In the 1995 aerial photographs, railcars returned for storage. During the site visit, railroad cars were seen set back away from the levee toe. Their operation had not moved beyond the property boundaries. Due to past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

4.1.7 Auto Salvage Yard, Formerly A to Z Production Plating

Address: 1142 S. 12th St.
Kansas City, KS 66105
Stationing: 124+00 to 130+00
Figure No.: 2.0, 4.0

This property is adjacent to the levee with a fence separating it from the levee toe by only a matter of a few feet. A portion of the property will be used for construction of a new levee section. The current owner is Hansen Property Development. The address was found in the Follow-Up Investigation (Hydrogeologic, 1999) database search under the name A to Z Production Plating. In 1980, the facility started operations as a electroplating and polishing facility. It plated chrome, copper, nickel, zinc and cadmium onto brass and steel parts. The production process involves chemicals listed as hazardous waste and generates sludge containing high levels of metals. It was a Kansas small quantity generator and had numerous RCRA violations. The site was cited by the EPA for not having a permit for storage of hazardous waste or for having a closure plan in place. By 1991, A to Z Production Plating had ceased operations. The property owners entered into a cleanup agreement with the EPA to dispose of the plating process waste that had accumulated in their warehouse and contaminated equipment. Under EPA direction, soil was excavated outside on the south side of the building to check for any heavy metals contamination. Tests showed no elevated levels of metals in the soils. The EPA designated the site to have no further action required and considered it clean in 1994.

A review of the aerial photographs from 1970 and the following years shows that the site had some form of industrial activity, with the exception of the 1993 photo when the property was vacant during the flood. Following the cleanup of A to Z plating operations, the site was used a salvage yard. During the site visit, the salvage yard was completely cleared and an asphalt and lot was placed over the top of the whole yard.

Due to past use as a salvage yard, the property which falls into the limits of disturbance for the selected alternative should be more fully investigated during the Design Stage to ensure that the surface and subsurface soil are not contaminated and to determine how to dispose of any contaminated soil.

4.1.8 Trimodal (a.k.a Container Recycling Inc.)

Address: 1161 S. 12th St.
Kansas City, KS 66105
Stationing: 130+00 to 157+00
Figure No.: 2.0, 4.0

This property is in very close proximity to the levee, with trailers stored at the levee toe. The property has gone by many names since it started in 1965 including: Great Lakes Container Corporation, Container Recycling, Inc., and Trimodal. The facility was used for drum storage and cleaning. Past practices have contaminated the site. In the Reconnaissance Report, the site was

listed as an area of concern with a recommendation for further investigation. It was also listed in the CERCLIS database as a Superfund site, but is currently listed as no further action planned. In the Follow-Up Investigation, the company was listed in many EPA databases for LUSTs, RCRA violations, and USTs. Two USTs that stored recovered oil and diesel fuel were removed from the site in 1990. One LUST was removed in 1990 and ground remediated. The facility was a RCRA EPA generator of hazardous waste, generating over 1,000 kg/month. It also had numerous RCRA violations from 1986 to 1994. Prior to Great Lakes Container, the property had a steel mill on the site from the 1950s to 1960.

Aerial photographs give an excellent representation of the widespread placement of drums and trailers over the site. Buildings present during 1960 photographs have been removed. By 1970 photographs the Great Lakes Container Corporation is active. Drums were seen covering the entire site, right up to the levee embankment. In the 1978 photos, some drums were seen near the 7th St. Bridge. By the 1983 photos, they are gone. Drums and trailers were still seen in the 1995 photographs.

As previously stated, the property is a Superfund site. When Container Recycling shut down operations in 1996, drums and trailers were left in place. The EPA led clean up actions at the site. The initial investigation showed elevated lead concentrations in the soil. Additional investigations of the drums were performed. From the KDHE database for the identified sites list, abandoned drums were found out in the elements as wells as stored in trailers. According to an EPA action memorandum from 1999, over 4,100 buried drums were found on-site. An estimate by the owner in 1997 had over 30,000 drums being stored on the property. Another 90 semi trailers were on site as well. The conditions of the drums varied from rusty and partially crushed to excellent. Surface soils were visibly stained with a petroleum-like substance. Inside the process building, process waste was present in floor drains. Caustic solids from the drum cleaning process were found on the floor in piles. Previous attempts to remove contaminated soils were visible at the site. From the Phase I assessment, concrete chunks were also found on the north side of the site. In 2000, a time-critical removal action was performed to remove all the drums and to remediate the site. Due to the larger than expected number of buried drums, plans to remove the soil was halted. Instead, the site was capped to prevent exposure to the contaminants remaining onsite.

According to the KDHE documents and conversations with EPA personnel, automobile fluff, which is a mixture of shredded plastic, foam, textiles, rubber, and glass, was placed as the sub grade fill over the contaminated natural soil. Subsurface drainage piping and stormwater drains were installed throughout the site to collect surface water and channel it off-site. On top of the autofluff sub grade, nonwoven geotextile fabric was placed. The cap extends up to the levee itself. A foot of crushed limestone gravel with a large percentage of fines was installed over the geotextile. In 2004, a restrictive covenant agreement was placed prohibiting excavations at the property. It states: " Except for minor excavations necessary to install, maintain or repair utility poles, fence posts, sidewalks, paving, and other comparable activities, as well as minor excavations necessary to maintain or repair existing underground utilities, the Property owners shall not engage in any excavation activities on the cap area that may damage the cap, without the previous written permission of KDHE such permission not to be unreasonably withheld or delayed." The property owner also agreed to restrict use to industrial or commercial use, notify the appropriate agency if

construction occurs, to place no wells for any purpose on the site, and to have inspections of the cap. In August of 2004, an inspection found only minor problems with erosion and siltation at the drains of the drainage system. The site is currently used to store shipping containers and semi trailers.

From EPA reports and letters, it is evident that some soil contamination from lead and PCBs still exists beneath the Container Recycling Corp. property. Any excavation on the property would require excavation of the cap and into the fill and would require KDHE approval due to the restrictive covenant. It is recommended any type of excavation or installation of relief wells for subsurface underseepage control be avoided. The current 500+3 alternative has no construction activity occurring outside the existing right of way. Rick Bean, a section chief in KDHE Bureau of Environmental Remediation stated KDHE would allow a construction easement through the property provided no excavation would occur and any damage to the existing cap is repaired. Coordination with the KDHE and EPA is recommended for any work on site.

4.1.9 Midwest Cold Storage (formerly Williams Meat Company)

Address: 1101 S. 5th St.
Kansas City, KS 66105
Stationing: 185+00 to 200+00
Figure No.: 2.0

Midwest Cold Storage property lies adjacent to the levee toe and is partially within the construction limit. This property operates a food storage facility. Prior to Midwest Cold Storage, the facility was a meat packing plant called Williams Meat Company. The Reconnaissance Report and the Follow-Up Investigation did not have any information listed for either property. During a visit to KDHE for a records search, a temporary-out-of-service UST was found at this location. KDHE's compliance section stated that according to KDHE's records, one UST was present on site and was out of compliance. Numerous letters to Williams Meat Co. asked for compliance fees as well as line and tank checks. These were all returned. It was later found that as of 1989, Williams Meat Company was no longer in business. A call was placed to Farley Hill with Midwest Cold Storage. He indicated that the UST, located on the north side of the levee, had been removed by the early 1990s.

From aerial photos, the site has been industrialized since the 1960s at the latest. During the site visit, a large propane tank and a transformer were seen on the landside of the levee. If removal of the transformer is required, the area around the transformer may require further inspection before construction to ensure that PCB contamination is not present. No other concerns were identified with the site.

4.1.10 Schock Truck & Leasing

Address: 51 Osage Ave.
Kansas City, KS 66105
Stationing: 205+10 to 215+25
Figure No.: 2.0, 5.0

Schock Truck & Leasing is located outside the construction limits but may be impacted by temporary construction easements. The facility repairs semi-trailer trucks. While no information was found for this property in the Reconnaissance Report or the Follow-Up Investigation, the KDHE LUST database did have a listing. KDHE also corroborated the database information in an interview. A diesel fuel spill in November of 2004 brought the tank to KDHE's attention. After the lines failed a tightness test for a fuel tank, the pipes and tank were removed. Stained soil was found to have a concentration of 5500 mg/kg and subsequently removed. The tank was considered closed by KDHE in 2004.

Aerial photographs show the site was industrialized after 1970. The 1983 photographs show trailers stored on the property in close proximity to the levee. Photos from 1995 show the trailer storage area is still present and appears to be very clean. The floods of 1951 and 1993 did not impact the property. During the site visit, two ASTs were seen from the levee road. The tanks were in good condition. A pile of debris lies between the tanks and jersey barriers that denote the edge of the property. It appears to be random construction debris. No other soil or groundwater contamination appears to be present at the site; therefore no further investigation is planned.

4.1.11 Sambol Packing Co.

Address: 13 Shawnee Ave.
Kansas City, KS 66105
Stationing: 227+00 to 230+00
Figure No.: 2.0, 5.0

Sambol Packing Company lies partially within the construction limits of the proposed alternative. Their operations include meatpacking and packaging. There is no mention of this property in either the Reconnaissance Report or the Follow-Up Investigation. According to KDHE, Sambol was listed as having two USTs removed after the previous investigations were published. These were removed in 2000. KDHE found no evidence of contamination and considered the site closed.

During the site visit, no contamination issues were seen. No HTRW concerns exist for the site; therefore, no further investigation is required.

4.1.12 SELCO (formerly Chromium, Inc)

Address 10 Shawnee Ave.
Kansas City, KS 66105
Stationing: 232+00 to 242+00
Figure No.: 2.0, 5.0

SELCO currently occupies the former Chromium, Inc. property. The property lies partially within the construction limits of the proposed alternative. SELCO manufactures car parts. It is not a permitted RCRA facility. No known contamination is associated with the current company. Chromium Inc. was listed as a KDHE Identified Site. A Preliminary Assessment (PA) was performed in 1994. Chromium Inc. operated from 1952 to 1972. The company allegedly disposed

of chromium wastes in on-site lagoons and directly discharged wastes into the Kansas River. Before the Flood of 1951, the site was the Cudahy Packing Plant. The construction debris from both Chromium, Inc. and Cudahy demolition was buried approximately 10 to 15 feet below the surface. Past disposal practices at Chromium, Inc include the pumping of lagoons into the Kansas River. By the late 1960s, the City of Kansas City ordered Chromium Inc. to cease the discharge of its industrial wastes into the sewer system and to close the lagoons with backfill. Subsequently, Chromium Inc. began discharging the industrial waste into Cudahy's former lard pit. It too was backfilled with demolition debris and soil before the facility closed. In 1990, an investigation of the subsurface soils found detectable levels of lead and chromium. Eight borings were drilled. Levels were below Kansas' action levels. These results did not warrant further soil investigation. A groundwater sample was taken from a monitoring well on-site and showed only low concentrations of barium and cadmium. Again in 1994, KDHE collected another groundwater sample. In this sample, chromium, 1,2-DCE and 1,1-DCA were all detected below the MCLs. Geoprobe samples were collected near the former surface impoundments to a depth of approximately 33 feet. These soil samples were also below the action levels. KDHE's recommendation was no further action was warranted at this site. There appears to be no immediate threat to the soil or groundwater from the site's past practices.

The aerial photographs show the site was industrialize before 1951, as KDHE records had said. The current building for SELCO was in existence by 1983. The site visit did not indicate any current hazardous waste issues occurring at the site. As only very low concentrations of metals and VOCs below KDHE action levels exist at the site, no further investigation is required at this location. The previous buried construction debris, if excavated, may be taken to a solid waste landfill.

4.1.13 PBI Gordon Corporation

Address:	300 S. 3rd St. Kansas City, KS 66118
Stationing:	278+00 to 293+00
Figure No.:	1.0, 6.0

PBI Gordon is adjacent to the current floodway. The facility manufactures various pesticides and other agricultural chemicals, as wells as clay and brick products. Neither the Reconnaissance Report nor the Follow-Up Investigation had any information regarding the property. The EPA databases list it as permitted RCRA hazardous waste treatment, storage and disposal facility. Numerous air releases occurred between 1987 and 2003, according to the Toxic Release Inventory System (TRIS).

In 1990, approximately 250 pounds of xylene spilled into the Kansas River. No further information was given in the EPA database. Another spill listed in the EPA RCRA Facility Assessment Final Sampling Summary Report occurred in 1980. Over 11,000 gallons of dimethylamine (DMA) were spilled. The Kaw Valley Drainage District was asked to shut their floodgates to prevent migration to the Kansas River. The Kansas City, Kansas Fire Department flushed the chemical into a deep ditch. A 7,000-gallon transport truck was used to pump the fluids into the collection lagoons. Some water did discharge through the storm water sewer system and into the river. The EPA attempted to strip

the effluent before it reached the river by placing 2 bags of activated carbon at the entrance to the storm drain. It was estimated that 500 gallons per minute of diluted chemicals were discharged over a 3-hour period.

The State of Kansas also has the facility listed on its Identified Sites list. The EPA conducted a site investigation and a RCRA Facility Assessment (RFA) in 1989 and 1997, respectively. Fourteen SWMUs were identified during this process. Of these, SWMU #8, former surface impoundments, is no longer in use. These impoundments, which consisted of partial cement and impervious soil floors, were closed in 1981. All residual soils were removed and tested for toxicity. The soils were found to be below action levels. The impoundments were then backfilled with soil. The RFA did not classify a 11,000 gallon DMA spill as a SWMU. As result of inadequate sampling for releases from SWMUs and the DMA spill, EPA collected soil samples from 5 locations around the site. Three samples were taken from drainage ditches to determine if DMA still was present from the spill. Samples were grabbed from a depth of 6 inches to1 foot below ground surface. Only two samples contained elevated levels of pesticides above Region 9 Preliminary Remedial Goals (PRGs). The results are found in the table below.

Table 3.0: PBI Gordon RFA Soil Investigation Results

Chemical	Sample 100 (μg/kg)	Sample 101 (μg/kg)	Region 9 PRGs (μg/kg)
Alpha BHC	500	ND	300
Aldrin	4,100	43,000	110
Dieldrin	30	260,000	120
4,4-DDE	6,000	28,000	5,600
4,4-DDD	12,000	ND	7,900
Heptachlor Epoxide	2,400	ND	210

Sample 100 and 101 are located close to a set of railroad tracks that run adjacent to the facility. The status of corrective action for the soil contamination has not progressed past the completion of the RFA. Also, the RFA recommended a groundwater investigation be conducted at the site. No groundwater sampling has occurred. Although groundwater contamination has not been identified at this location, based on past activities, the potential for such contamination exists.

It is recommended that ground disturbance in this area be avoided if possible. The current proposed alternative does not involve excavation at PBI Gordon. However, if soil excavation is required from this area during the construction phase, soil testing will be required prior to disposal. Some of the soil may require disposal of hazardous waste. Additional groundwater investigation is warranted prior to the any expansion of the existing subsurface relief well system.

4.1.14 APAC - Wilkerson

Address: Central Ave.
Kansas City, KS 66118
Stationing: 296+00 to 310+00
Figure No.: 2.0

A portion of the APAC – Wilkerson properties lie the construction limit. The property is mainly used of equipment storage. There are few structures on the site. These properties were listed in the Reconnaissance Report as a small quantity generator in the RCRIS database. Four inactive underground storage tanks were also listed for the site, each having a capacity between 3000 to 6000 gallons. The Follow-Up Investigation did not list APAC, as it was located outside the radius given for the database. A search of the EPA Envirofacts database only lists APAC as a small quantity generator. There was no listing for any of the tanks listed in either the EPA databases or the KDHE LUST database.

From the aerial photographs, the properties currently owned by APAC have historically been industrial. Older buildings were present on the various properties. The 1983 photographs did not cover this section. By 1993, APAC had equipment stored on the site. Trailers were stored all the way to the levee. The remainder of the land was vacant.

During the site visit, APAC was still using the properties as a storage yard for their construction equipment. No contamination is known to be present at this site. Based on information evaluated, no impacts to levee improvements resulting from HTRW concerns were identified. Therefore, no further investigation is necessary.

4.1.15 Union Pacific Railroad

Address: Numerous rail yards and spurs in Armourdale
Stationing: 0+00 to 75+00, 274+00 to 277+00, and 305+00 to 320+00
Figure No.: 2.0

Union Pacific Railroad (UPRR) operates many rail yards in the Armourdale area, of which lie within the 40-foot construction limit. Around Proctor & Gamble to the edge of the Kaw Power Plant and PBI Gordon to APAC, rail spurs run the length of the levee. The Reconnaissance Report and Follow-Up Investigation did not have any information concerning the rail yards in the Armourdale area. Numerous LUSTs were reported in KDHE LUST Database and have been remediated and closed. The following chart lists the closed LUSTs for the Union Pacific Railroad.

Table 4.0: UPRR Leaky Underground Storage Tanks

Location	Year	KDHE status	Contamination present
UPRR 18th St. Yard	1994	Closed	Yes, soil removed to roadway
UPRR 334 S. 5th St.	1994	Closed	Yes, soil removed
UPRR Wastewater Treatment Plant	1994	Closed	Yes, soil removed
UPRR Mill St. Shanty	1993	Closed	None above action levels, UST abandoned in place
UPRR Armstrong Yard	1993	Closed	None above action levels, UST abandoned in place

Another tank is listed for the Armstrong Yard. This tank was removed in 1992 with no leaks or contamination seen. Two other tanks were not found on the LUST database, but were also found to

have been closed in 1995 with no impacts to the soil or groundwater. These tanks were most likely not near the levee.

During a review of EPA documents, a permit for discharging wastewater and sludges was requested by UPRR. A 1982 document states that test results of the wastewater classified it as non-hazardous. The UPRR withdrew its permit application. While a potential exists for previous contamination resulting from spills along the railroad lines, all known contamination has been remediated from various UPRR sites. Therefore, no further investigation is warranted for the railroad yards.

4.2 Central Industrial District Unit

The Central Industrial District (CID) Unit is being reevaluated as part of the Feasibility Study for overtopping and underseepage concerns based on observation made during the 1993 flood. The 500 year + 3 feet alternative was evaluated for the entire levee unit for the hazardous waste study. This alternative includes a combination of alternatives such as levee and floodwall raises, underseepage control and relief wells, and a new floodwall tieback. Currently, there is one existing system of relief wells located at stations 79+00 to 97+00.

For the HTRW Site Assessment, the area evaluated was based on the July 2007 figures for the 500 year +3 feet alternative with an additional 40 feet beyond the proposed construction limits to account for temporary easements. Within the Kaw Valley existing right-of-way, no contamination is expected to be encountered. However, there are many areas where the alternative extends outside the existing right-of-way and impacts properties in close proximity to the levee. The areas adjacent to the CID Unit are highly industrialized. There are sites which have soil or groundwater contamination that directly impacts the Feasibility Study alternative. Figure X.0 shows the overall levee unit with the adjacent properties identified. The following summarizes the evaluation and conclusions relating to specific properties within the CID Unit.

4.2.1 River View Properties Inc.

Address: James Street, Kansas City, Kansas
Stationing: 40+31.25 to 51+00
Figure No.:

Potential HTRW concerns have been identified within the study area between station 40+31 and 51+00. Potential encroachment into this area associated with the levee raise is proposed to be avoided by steepening the landside levee slope rather than extending the landside toe. No other locations of HTRW concerns have been identified at this time.

5.0 REFERENCES

Reconnaissance Report, Kansas Citys, Missouri and Kansas Flood Reduction Project; Appendix I: Hazardous, Toxic and Radioactive Waste Assessment; USACE Kansas City District; August 1999.

Final Report HTRW Site Follow-Up Investigations, Cities of Kansas City, Missouri and Kansas Flood Damage Reduction Study; Hydrogeologic; November 1999.

KDHE LUST database; http://www.kdhe.state.ks.us/tanks/; Storage Tank Section updated December 2004.

EPA ENVIROFACTs database; http://www.epa.gov; updated February 25, 2005

USACE aerial photographs, years 1951, 1960, 1970,1978, 1983, 1993, and 1995

Preliminary Assessment, Chromium, Inc. Site; Kansas Department of Health and the Environment; December 1994.

Preliminary Assessment, Proctor & Gamble Well #11 Site; Kansas Department of Health and the Environment; July 1990.

RCRA Facility Assessment, PBI Gordon Corporation; B & V Waste Science, December 1990.

Screening Site Investigation, Proctor & Gamble Well #11 Site; Kansas Department of Health and Environment; September 1991.

Phase I Assessment Report, Container Recycling, Inc. Property; Barr Engineering; April 1997.

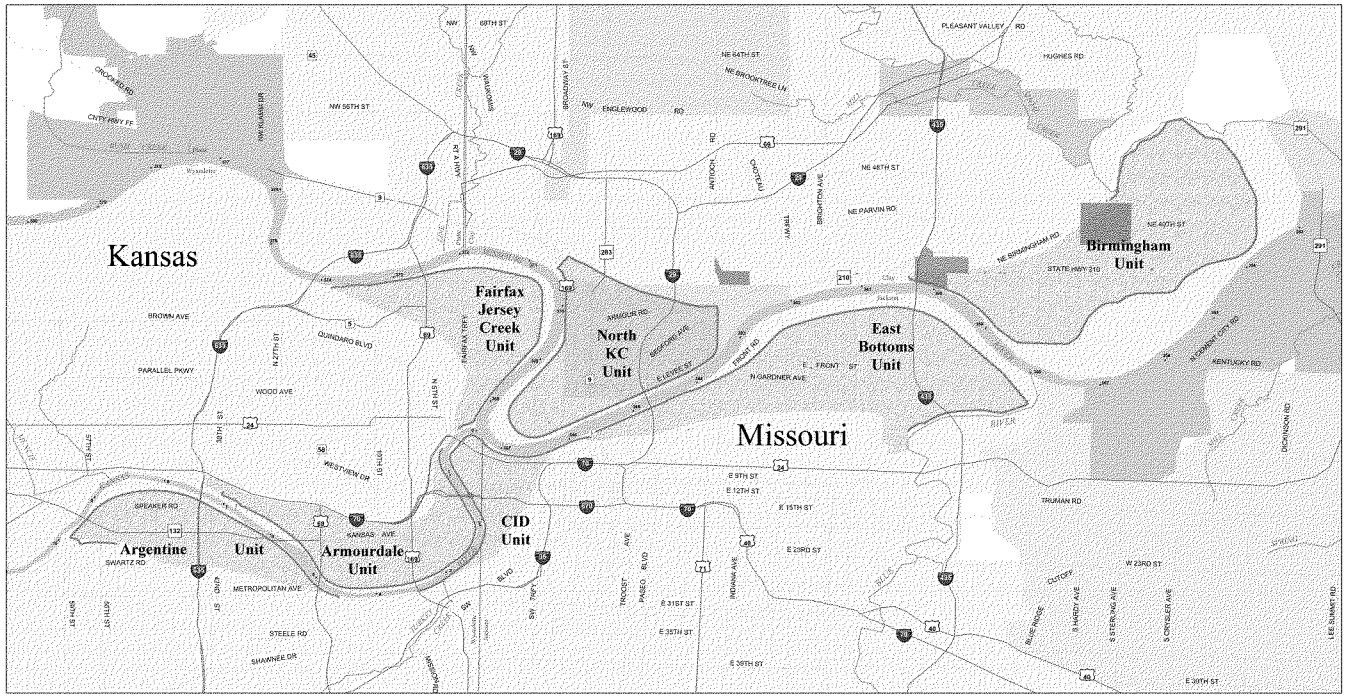
RCRA Facility Assessment Sampling Visit, PBI Gordon Corporation; PRC Environmental Management Company; January 1997.

RCRA Facility Assessment, Procter & Gamble; Tetra Tech EM Inc., May 1998

USACE correspondence from Kaw Valley Drainage District, various dates

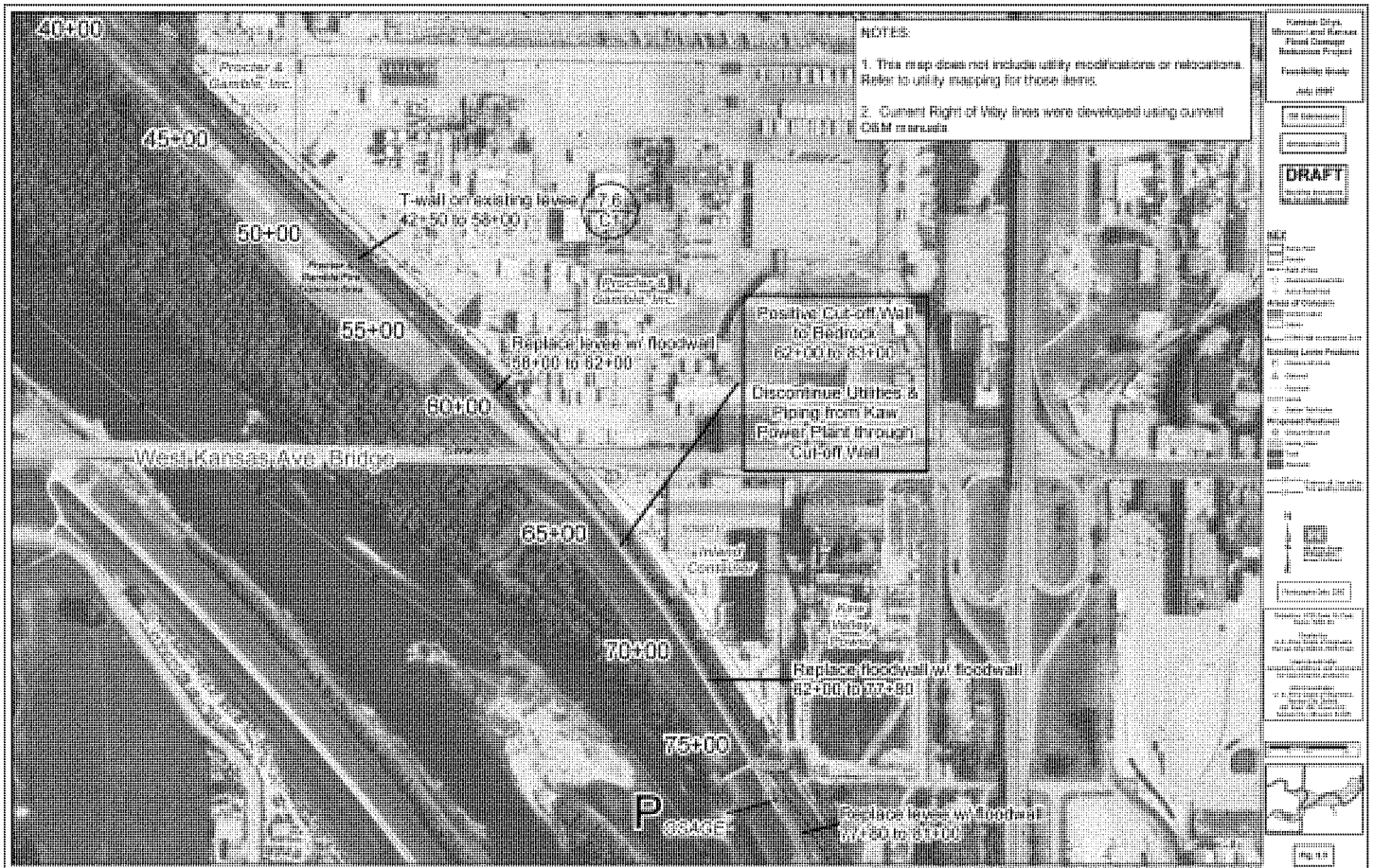
KDHE correspondence with property owners for Container Recycling and A to Z Plating, various dates

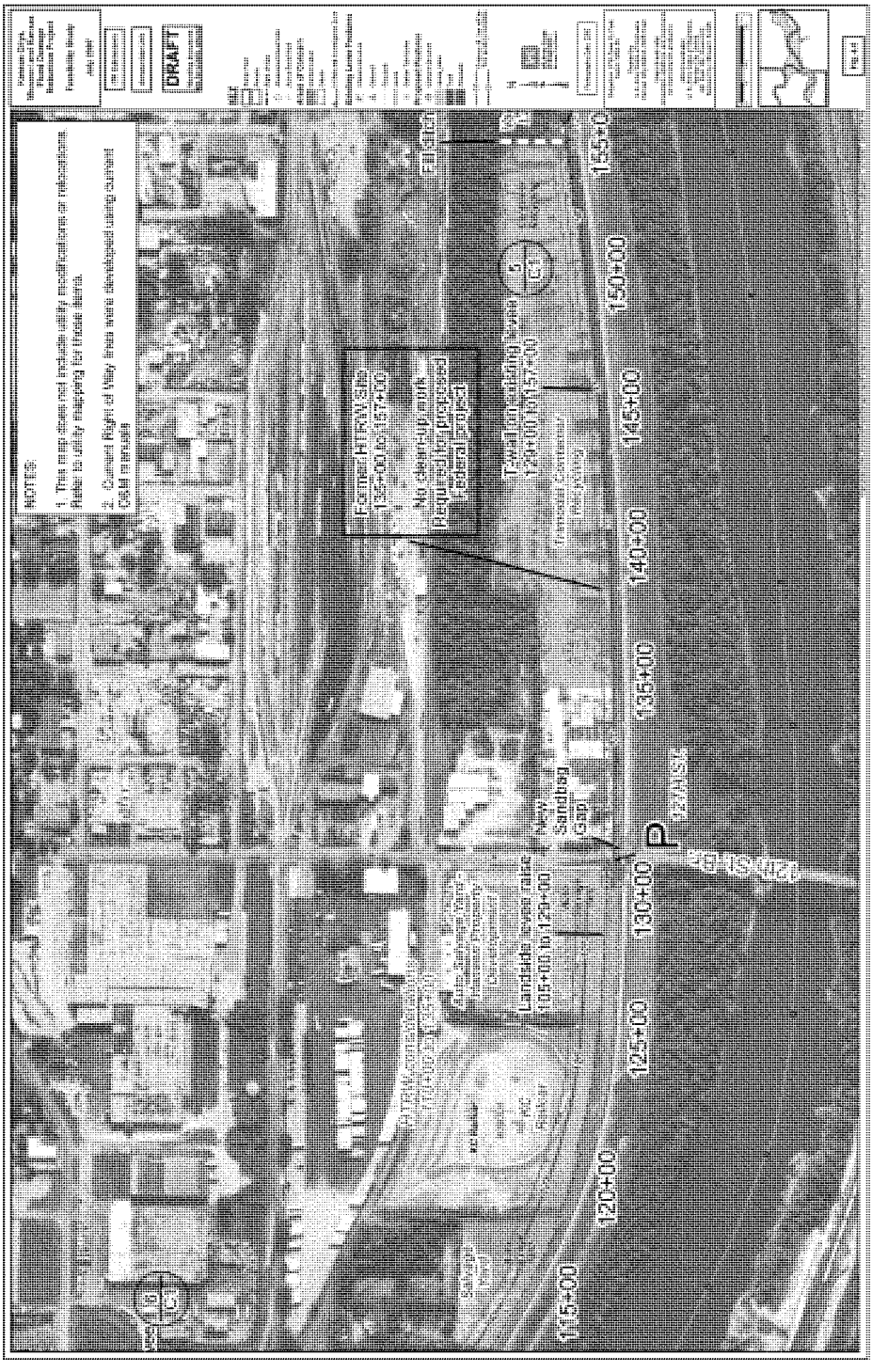
HTRW FIGURES



Appendix D - Figure 1

DRAFT


NOTES:
1. This map does not include utility modifications or relocations. Refer to utility mapping for those items.
2. Current Right of Way lines were developed using current GIS maps.
DRAFT
40+00
45+00
50+00
55+00
60+00
65+00
70+00
75+00
T-wall on existing levee
42+50 to 58+00
Replace levee w/ floodwall
58+00 to 62+00
Positive Cut-off Wall to Bedrock
62+00 to 83+00
Discontinue Utilities & Piping from Kaw Power Plant through Cut-off Wall
West Kansas Ave. Bridge
Replace floodwall w/ floodwall
62+00 to 77+80
Replace levee w/ floodwall
77+80 to 81+00
P


NOTES:
1. This map does not include utility modifications or relocations. Refer to utility mapping for those items.
2. Current Right of Way lines were developed using current O&M manuals
DRAFT
Former HTRW Site
135+00 to 157+00
No clean-up work
Required for proposed
Federal project
Landside levee raise
105+00 to 129+00
New
Sandbag
Gap
P
12th St Ga
115+00
120+00
125+00
130+00
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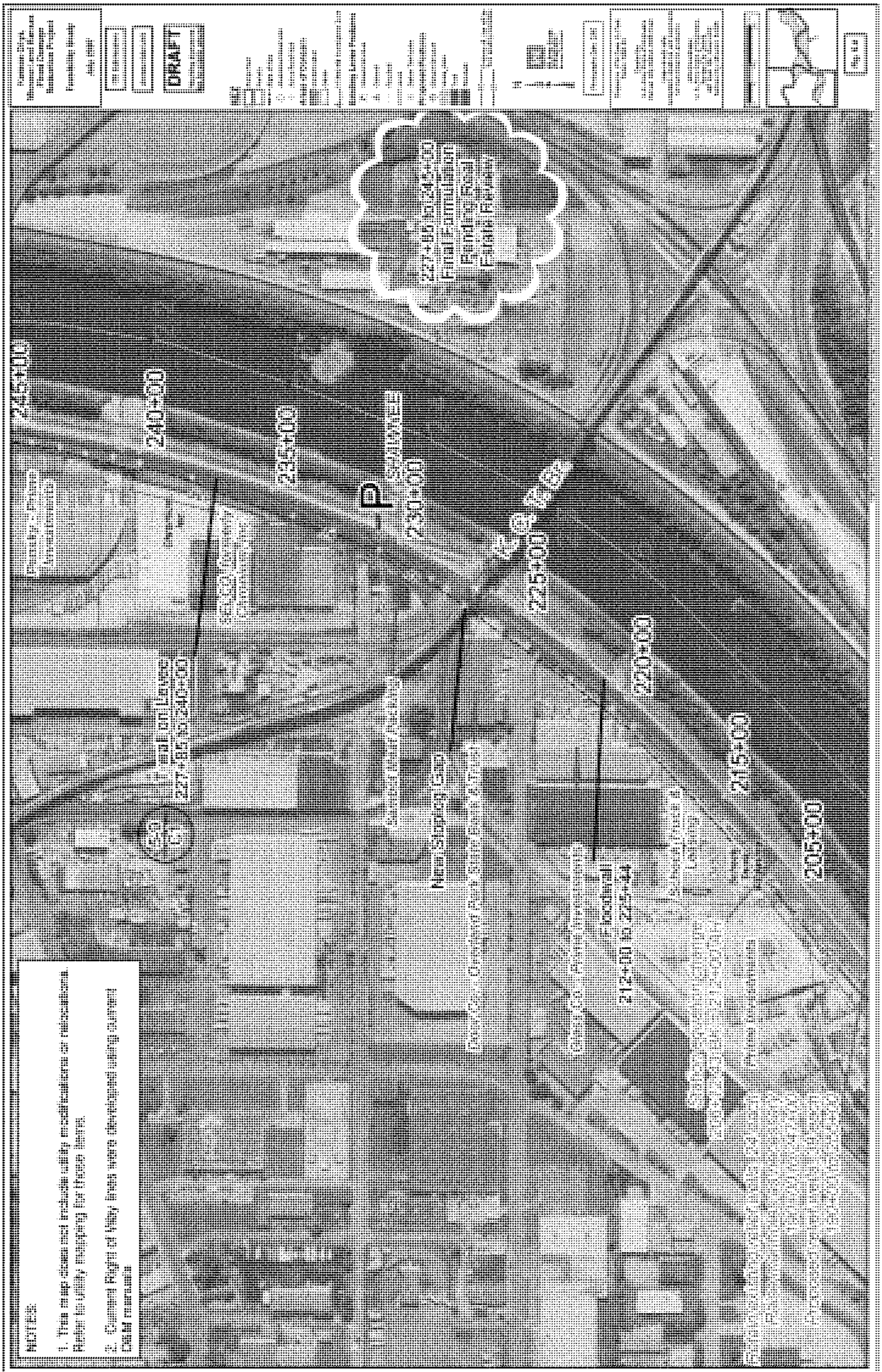
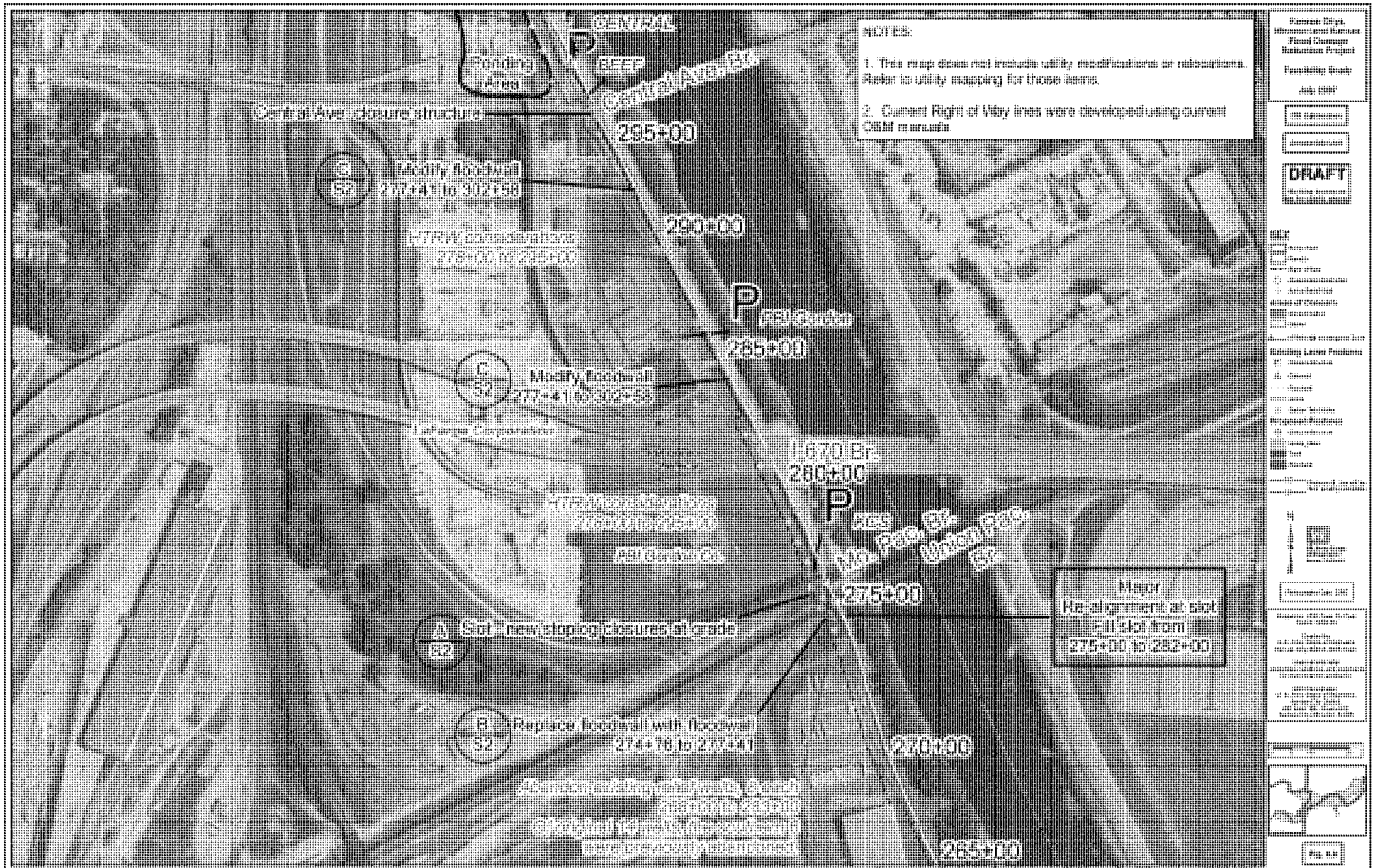
140+00
145+00
150+00
155+00


DRAFT
NOTES:
240+00
235+00
230+00
225+00
220+00
215+00
205+00


NOTES:
1. This map does not include utility modifications or relocations. Refer to utility mapping for those items.
2. Current Right of Way lines were developed using current O&M manuals.
CENTRAL
BEEF
Ponding Area
Central Ave. Br.
Central Ave. closure structure
295+00
Modify floodwall
277+41 to 302+58
290+00
HTRW considerations
278+00 to 295+00
RBI Garden
285+00
Modify floodwall
277+41 to 302+58
LaFarge Corporation
I-670 Br.
280+00
KCS
Mo. Pac. Br.
Union Pac. Br.
275+00
Major
Re-alignment at slot
Fill slot from
275+00 to 282+00
Slot - new stoplog closures at grade
Replace floodwall with floodwall
274+78 to 277+41
270+00
265+00
DRAFT

U.S. Army Corps of Engineers, Kansas City District

Final Feasibility Report

APPENDIX E

COST ESTIMATING

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

KANSAS CITYS, MO & KS
FLOOD RISK MANAGEMENT STUDY

APPENDIX E
COST ESTIMATING

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WALLA WALLA COST ENGINEERING MANDATORY CENTER OF EXPERTISE

COST AGENCY TECHNICAL REVIEW

CERTIFICATION STATEMENT

For Project No. P2 106927

NWK – Kansas City Levees Phase II
Armourdale Reach 500yr+3ft
Central Industrial Districts 500yr+3ft

The Kansas City Levees Phase II project, Armourdale and Central Industrial Districts, as presented by Kansas City District, has undergone a successful Cost Agency Technical Review (Cost ATR), performed by the Walla Walla District Cost Engineering Mandatory Center of Expertise (Cost MCX) team. The Cost ATR included study of the project scope, report, cost estimates, schedules, escalation, and risk-based contingencies. This certification signifies the products meet the quality standards as prescribed in ER 1110-2-1150 Engineering and Design for Civil Works Projects and ER 1110-2-1302 Civil Works Cost Engineering.

As of April 9, 2014, the Cost MCX certifies the estimated total project cost:

FY 2015	Price Level:	$318,517,000
Fully Funded Amount:		$399,395,000

It remains the responsibility of the District to correctly reflect these cost values within the Final Report and to implement effective project management controls and implementation procedures including risk management throughout the life of the project.

US Army Corps of Engineers®

JACOBS.MICHAEL.PIERRE.1160569537

Digitally signed by JACOBS.MICHAEL.PIERRE.1160569537
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=JACOBS.MICHAEL.PIERRE.1160569537
Date: 2014.04.09 13:24:11 -07'00'

Michael P. Jacobs, PE, CCE
Acting Chief, Cost Engineering MCX
Walla Walla District

****** TOTAL PROJECT COST SUMMARY ******

PROJECT: Kansas City Levees Phase II
PROJECT NO: 106927
LOCATION: Kansas City, KS/MO

DISTRICT: Kansas City District
PREPARED: 4/8/2014
POC: CHIEF, COST ENGINEERING, John D

This Estimate reflects the scope and schedule in report; Feasibility Report Dated April 2014

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)						TOTAL PROJECT COST (FULLY FUNDED)			
						Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14									
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Spent Thru: 10/1/2013 ($K)	FIRST COST ($K)	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	B	C	D	E	F	G	H	I	J				M	N	O
02	RELOCATIONS	$1,635	$504	31%	$2,139	1.6%	$1,661	$512	$2,173	$0	$2,173	24.1%	$2,061	$636	$2,697
11	LEVEES & FLOODWALLS	$195,698	$60,395	31%	$256,093	1.6%	$198,742	$61,334	$260,076	$0	$260,076	24.1%	$246,707	$76,137	$322,844
13	PUMPING PLANT	$7,914	$2,442	31%	$10,356	1.6%	$8,037	$2,480	$10,517	$0	$10,517	24.1%	$9,977	$3,079	$13,056
	CONSTRUCTION ESTIMATE TOTALS:	$205,247	$63,341		$268,589	1.6%	$208,439	$64,326	$272,766	$0	$272,766	24.1%	$258,745	$79,851	$338,596
01	LANDS AND DAMAGES	$3,754	$1,161	31%	$4,915	1.6%	$3,812	$1,179	$4,991	$0	$4,991	10.2%	$4,202	$1,299	$5,502
30	PLANNING, ENGINEERING & DESIGN	$16,122	$4,976	31%	$21,098	2.2%	$16,470	$5,083	$21,553	$0	$21,553	19.8%	$19,737	$6,091	$25,828
31	CONSTRUCTION MANAGEMENT	$14,367	$4,434	31%	$18,801	2.2%	$14,677	$4,530	$19,207	$0	$19,207	53.4%	$22,519	$6,950	$29,469
	PROJECT COST TOTALS:	$239,490	$73,912	31%	$313,402		$243,399	$75,118	**$318,517**	$0	**$318,517**	25.4%	$305,203	$94,192	$399,395

Mandatory by Regulation	CHIEF, COST ENGINEERING, John Dillon			
Mandatory by Regulation	PROJECT MANAGER, Eric Lynn	ESTIMATED FEDERAL COST:	**65%**	**$259,607**
Mandatory by Regulation	CHIEF, REAL ESTATE, Greg Wilson	ESTIMATED NON-FEDERAL COST:	**35%**	**$139,788**
	CHIEF, PLANNING, Jennifer Switzer	**ESTIMATED TOTAL PROJECT COST:**		**$399,395**
	CHIEF, ENGINEERING, David Mathews			
	CHIEF, OPERATIONS, Stuart Cook			
	CHIEF, CONSTRUCTION, Chris Prinslow			
	CHIEF, CONTRACTING, Theresa McCarthy			
	CHIEF, PM-C, John Holm			
	CHIEF, DPM, Steven Iverson			

Filename: Kansas City Levees Phase II TPCS dated 9 April 2014.xlsx
TPCS

****** TOTAL PROJECT COST SUMMARY ******

****** CONTRACT COST SUMMARY ******

PROJECT: Kansas City Levees Phase II
LOCATION: Kansas City, KS/MO
This Estimate reflects the scope and schedule in report: Feasibility Report Dated April 2014

DISTRICT: Kansas City District
POC: CHIEF, COST ENGINEERING, John Dillon
PREPARED: 4/8/2014

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 3/15/2014 Effective Price Level: 10/1/2013				Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	RISK BASED CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Armourdale - 500 Yr + 3ft													
02	RELOCATIONS	$1,389	$428	31%	$1,816	1.6%	$1,410	$434	$1,845	2026Q1	24.1%	$1,751	$539	$2,290
11	LEVEES & FLOODWALLS	$145,867	$44,912	31%	$190,779	1.6%	$148,135	$45,611	$193,746	2026Q1	24.1%	$183,887	$56,619	$240,506
13	PUMPING PLANT	$5,943	$1,830	31%	$7,772	1.6%	$6,035	$1,858	$7,893	2026Q1	24.1%	$7,491	$2,307	$9,798
	CONSTRUCTION ESTIMATE TOTALS:	$153,198	$47,170	31%	**$200,368**		$155,581	$47,903	$203,484			$193,129	$59,464	$252,594
01	LANDS AND DAMAGES	$2,024	$623	31%	$2,647	1.6%	$2,055	$633	$2,688	2020Q1	10.2%	$2,266	$698	$2,963
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
1.0%	Planning & Environmental Compliance	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
1.0%	Engineering & Design	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
1.0%	Reviews, ATRs, IEPRs, VE	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
0.3%	Life Cycle Updates (cost, schedule, risks)	$383	$118	31%	$501	2.2%	$391	$120	$512	2018Q1	11.6%	$437	$134	$571
1.0%	Contracting & Reprographics	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
1.0%	Engineering During Construction	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2026Q1	53.4%	$2,401	$739	$3,141
0.5%	Planning During Construction	$827	$255	31%	$1,082	2.2%	$845	$260	$1,105	2026Q1	53.4%	$1,296	$399	$1,695
1.0%	Project Operations	$1,532	$472	31%	$2,004	2.2%	$1,565	$482	$2,047	2018Q1	11.6%	$1,747	$538	$2,285
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$7,660	$2,359	31%	$10,019	2.2%	$7,825	$2,409	$10,235	2026Q1	53.4%	$12,006	$3,697	$15,703
0.0%	Project Operation:	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
2.0%	Project Management	$3,064	$943	31%	$4,007	2.2%	$3,130	$964	$4,094	2026Q1	53.4%	$4,803	$1,479	$6,281
	CONTRACT COST TOTALS:	$177,880	$54,769		**$232,650**		$180,783	$55,663	$236,447			$226,822	$69,838	**$296,660**

****** TOTAL PROJECT COST SUMMARY ******

****** CONTRACT COST SUMMARY ******

PROJECT: Kansas City Levees Phase II
LOCATION: Kansas City, KS/MO
This Estimate reflects the scope and schedule in report; Feasibility Report Dated April 2014

DISTRICT: Kansas City District
POC: CHIEF, COST ENGINEERING, John Dillon

PREPARED: 4/8/2014

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 3/15/2014 Effective Price Level: 10/1/2013				Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Central Industrial District KS - 500 Yr + 3ft													
02	RELOCATIONS	$246	$77	31%	$323	1.6%	$250	$78	$328	2026Q1	24.1%	$311	$97	$407
11	LEVEES & FLOODWALLS	$49,451	$15,364	31%	$64,816	1.6%	$50,220	$15,603	$65,824	2026Q1	24.1%	$62,341	$19,369	$81,710
13	PUMPING PLANT	$1,971	$613	31%	$2,584	1.6%	$2,002	$622	$2,624	2026Q1	24.1%	$2,485	$772	$3,257
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$51,669	$16,054	31%	**$67,723**		$52,473	$16,303	$68,776			$65,137	$20,238	$85,374
01	LANDS AND DAMAGES	$1,730	$538	31%	$2,268	1.6%	$1,757	$546	$2,303	2020Q1	10.2%	$1,937	$602	$2,538
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
1.0%	Planning & Environmental Compliance	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
1.0%	Engineering & Design	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
1.0%	Reviews, ATRs, IEPRs, VE	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
0.5%	Life Cycle Updates (cost, schedule, risks)	$258	$80	31%	$338	2.2%	$264	$82	$345	2018Q1	11.6%	$294	$91	$386
1.0%	Contracting & Reprographics	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
1.0%	Engineering During Construction	$517	$161	31%	$678	2.2%	$528	$164	$692	2026Q1	53.4%	$810	$252	$1,062
0.5%	Planning During Construction	$279	$87	31%	$366	2.2%	$285	$89	$374	2026Q1	53.4%	$437	$136	$573
1.0%	Project Operations	$517	$161	31%	$678	2.2%	$528	$164	$692	2018Q1	11.6%	$590	$183	$773
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$2,583	$803	31%	$3,386	2.2%	$2,639	$820	$3,459	2026Q1	53.4%	$4,049	$1,258	$5,307
0.0%	Project Operation:	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
2.0%	Project Management	$1,033	$321	31%	$1,354	2.2%	$1,055	$328	$1,383	2026Q1	53.4%	$1,619	$503	$2,122
	CONTRACT COST TOTALS:	$61,171	$19,006		**$80,177**		$62,169	$19,316	$81,485			$77,821	$24,179	**$102,000**

****** TOTAL PROJECT COST SUMMARY ******

****** CONTRACT COST SUMMARY ******

PROJECT: Kansas City Levees Phase II
LOCATION: Kansas City, KS/MO
This Estimate reflects the scope and schedule in report; Feasibility Report Dated April 2014

DISTRICT: Kansas City District
POC: CHIEF, COST ENGINEERING, John Dillon

PREPARED: 4/8/2014

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 3/15/2014 Effective Price Level: 10/1/2013				Program Year (Budget EC): 2015 Effective Price Level Date: 1 OCT 14								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	Central Industrial District MO - 500 Yr + 3ft													
02	RELOCATIONS	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
11	LEVEES & FLOODWALLS	$380	$118	31%	$498	1.6%	$386	$120	$506	2026Q1	24.1%	$479	$149	$628
13	PUMPING PLANT	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
							$0							
	CONSTRUCTION ESTIMATE TOTALS:	$380	$118	31%	**$498**		$386	$120	$506			$479	$149	$628
01	LANDS AND DAMAGES	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
30	PLANNING, ENGINEERING & DESIGN													
1.0%	Project Management	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
1.0%	Planning & Environmental Compliance	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
1.0%	Engineering & Design	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
1.0%	Reviews, ATRs, IEPRs, VE	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
0.5%	Life Cycle Updates (cost, schedule, risks)	$2	$1	31%	$3	2.2%	$2	$1	$3	2018Q1	11.6%	$2	$1	$3
1.0%	Contracting & Reprographics	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
1.0%	Engineering During Construction	$4	$1	31%	$5	2.2%	$4	$1	$5	2026Q1	53.4%	$6	$2	$8
0.5%	Planning During Construction	$2	$1	31%	$3	2.2%	$2	$1	$3	2026Q1	53.4%	$3	$1	$4
1.0%	Project Operations	$4	$1	31%	$5	2.2%	$4	$1	$5	2018Q1	11.6%	$5	$1	$6
31	CONSTRUCTION MANAGEMENT													
5.0%	Construction Management	$19	$6	31%	$25	2.2%	$19	$6	$25	2026Q1	53.4%	$30	$9	$39
0.0%	Project Operation:	$0	$0	31%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
2.0%	Project Management	$8	$2	31%	$10	2.2%	$8	$3	$11	2026Q1	53.4%	$13	$4	$16
	CONTRACT COST TOTALS:	$439	$136		**$575**		$446	$139	$585			$560	$174	**$735**

US Army Corps
of Engineers®

KANSAS CITYS FLOOD RISK MANAGEMENT PROJECT – ARMOURDALE UNIT FEASIBILITY STUDY

Project Cost and Schedule Risk Analysis Report

Prepared for:

U.S. Army Corps of Engineers,
Kansas City District

Prepared by:

U.S. Army Corps of Engineers
Kansas City District

October 11, 2013

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EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), Kansas City District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Kansas City's Flood Risk Management Project – Armourdale Unit Feasibility Study. In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a formal risk analysis, *Monte-Carlo* based-study was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommend 80% confidence level of successful execution to project completion.

The Armourdale Unit is located in Wyandotte County Kansas, along the left bank of the Kansas River from mile 7 (Mattoon Creek) to mile 0.3, near the confluence of the Kansas and Missouri Rivers. Prior to the Federal project, levees and floodwalls were constructed by the Kaw Valley Drainage District. These original works were modified and expanded in the initial Federal projects. Construction of the Federal project began in 1949 and was completed in 1951. More recent improvements, separately authorized under the 1962 Modification, were completed in 1976. The levees and floodwalls of the Armourdale Unit are currently authorized to pass a maximum Kansas River flow of 390,000 cfs coincident with a Missouri River flow of up to 220,000 cfs.

The primary components of the unit consist of earthen levees, floodwalls, riprap and toe protection on riverward slopes of levees, toe drains along the concrete floodwalls, sandbag gaps, stoplog gaps, drainage structures, relief wells and pumping plants. The floodwalls, in two reaches, vary from 11 to 17 feet high and total approximately 6,200 feet. The levees, in three reaches, vary from 4 to 17 feet high and total about 5.3 miles.

The unit begins with a stoplog gap across the Union Pacific (UP) Railroad which creates a tieback from high ground west of Mattoon Creek. The first levee section heads downstream approximately 1.28 miles along the left bank of the Kansas River, incorporating a portion of the UP embankment near the mouth of Mattoon Creek, and ends just north of the West Kansas Avenue Bridge. The first section of floodwall then extends downstream approximately 1,740 feet, ending just south of the Osage Pump Station. The second section of levee continues downstream approximately 3.3 miles to a point downstream (north) of the Chicago, Rock Island and Pacific (CRI&P) railroad bridge. This section contains one stoplog gap at the Kansas City Terminal (KCT) railroad bridge, five pumping stations, and a short reach of floodwall at the East Kansas Avenue Bridge. The second major reach of floodwall continues downstream another 4,493 feet to connect with the final levee section downstream of the Central Avenue

Bridge. This section contains two sandbag gaps at the UP and Missouri Pacific (MO Pac) railroad bridges, and two pumping stations. The final levee section extends another 4,156 feet and ties back into high ground at the embankment of the Lewis and Clark Viaduct.

Specific to the Armourdale Flood Risk Management Project, the current fully funded estimate approximates $298M. This CSRA study is expressed in FY 2014 dollars. Real Estate office provided a separate 25% contingency for its real estate requirements, which in turn was used in the Cost Risk Model. The Cost Engineering Section performed study on the total estimated project costs. Based on the results of the analysis, the Cost Engineering Section (located in Kansas City District) recommends a contingency value of approximately $55M or approximately 31% of base project cost. This contingency includes a separate $800K for Real Estate, another $47.2M for the construction costs, and $7M for design and construction management.

The Kansas City District Cost Engineering Section performed risk analysis using the Monte Carlo technique for the estimated construction costs, supported by the district PDT input. The following table ES-1 portrays the development of the construction contingencies (approx. 31%). The contingency is based on an 80% confidence level, as per USACE Civil Works guidance. It should be noted cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based in cost and percent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Case Construction Cost Estimate	**$178,439,000**	
Confidence Level	**Construction Value ($$)**	**Contingency (%)**
5%	$197,759,174	10.83%
50%	$221,077,594	23.90%
80%	**$233,385,329**	**30.79%**
90%	$239,776,832	34.37%

The following table ES-2 portrays the full costs of the cost recommendations, combining all remaining costs. The costs are intended to address the congressional request of estimates to implement the project. The contingency is based on an 80% confidence level, as per accepted USACE Civil Works guidance. The contingency has been rounded to 31%.

Table ES-2. Cost Summary of Remaining Costs (FY2014 dollars)

ARMOURDALE FRM FEATURE ACCOUNTS		COST ($1,000)	CNTG ($1,000)	CNTG %	TOTAL ($1,000)
01	LANDS AND DAMAGES	2,579	794	30.79	3,373
02	RELOCATIONS	5,731	1,765	30.79	7,496
06	FISH & WILDLIFE FACILITIES	0	0	0	0
11	LEVEES & FLOODWALLS	141,797	43,659	30.79	185,456
13	PUMPING PLANT	5,943	1,830	30.79	7,773
30	PLANNING, ENGINEERING, AND DESIGN	11,610	3,575	30.79	15,185
31	CONSTRUCTION MANAGEMENT	10,779	3,319	30.79	14,097
	FY 2013 PROJECT COSTS	**178,439**	**54,941**	**30.79**	**233,380**
Schedule Completion with Contingency		**120 mo**	**104 mo**	**86.57**	**224**

Notes:
1) Cost and Time contingencies presented w/ an 80% confidence level.
2) Costs exclude O&M and Life Cycle Cost estimates.
3) Differences between ES-1 and ES-2 are due to rounding

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on 15 and 16 March 2012. Additional PDT members met again on 10 July 2013 to provide additional experiences outside the original PDT team. During this timeframe the PDT discussed project scope definition, investigations, design and cost information, and determined risks in certain project areas. The key risk drivers identified through sensitivity analysis suggest a cost contingency of approximately $55M and schedule risks adding another potential of 104 months, both at an 80% confidence level.

Cost Risks: From the CSRA, the key or greater Cost Risk items of include:

- PR-1: Adequacy of Project Funding – Incremental congressional appropriations, and the sponsors ability to cost share. Congressional appropriations will most likely be incrementally funded with minimal appropriations per year assigned to this project. The risk of the schedule slipping is assumed and therefore a 1.5% annual escalation rate compounded has been included in the cost model.

- PR-3: Market Conditions/Bidding Conditions – The economy is currently in a downturn with signs of improvement. A range was given to account for the variance of possibilities. The estimate assumes a normal bidding climate.
- TL-1 : Confidence in Scope – Not enough information to adequately formulate a design. Designers did not always use a conservative approach but rather used design intuition for assumptions. In cases where no information was available, extrapolation techniques were used. Additional subsurface investigations may be required.

Moderate Risks

- CA-1 – Undefined Acquisition Strategy – Large business competitive pricing could be eliminated if other acquisition strategies are used.

- CON-4 – Contract Modifications – There are possible areas with HTRW concerns, unknown utilities not currently captured in the costs.

- EST-4 – Prime/Subcontractor structure matches likely acquisition strategy – Additional layers of markup may be required.

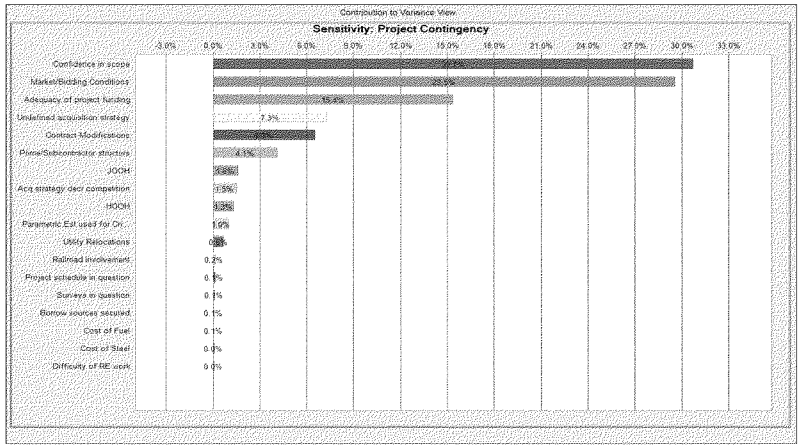
Schedule Risks: The high value of schedule risk indicates a significant uncertainty of key risk items, time duration growth that can translate into added costs. Over time, risks increase on those out-year contracts where there is greater potential for change in new scope requirements, uncertain market conditions, and unexpected high inflation. The greatest risk is:

- PR-1: Adequacy of Project Funding – Congressional appropriation will most likely be incrementally funded with minimal appropriation per year assigned to this project. The sponsor will most likely be unable to afford their cost share portion on a timely manner.

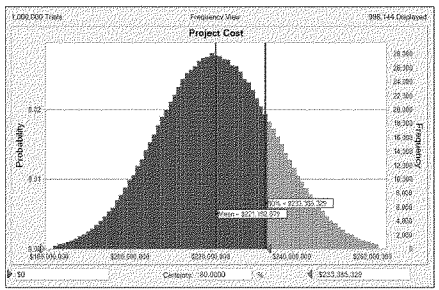
Recommendations: Timely coordination and risk resolution between the Sponsor, and USACE is needed in areas of funding needs and updates as applicable. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of the remaining project work within an approved budget and appropriation.

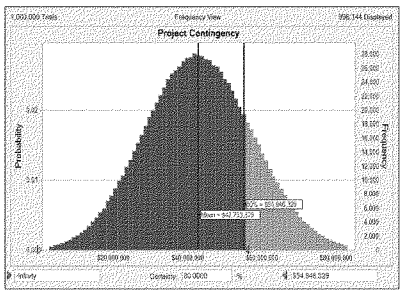
Armourdale n500+3 ft. Recommended Alternative - Cost Risk Analysis Model

Risk No.	Risk/Opportunity Event	Project Cost: Likelihood*	Project Cost: Impact*	Project Cost: Risk Level*	Variance Distribution	Correlation to Other(s)	Crystal Ball Simulation: Expected Values ($M): Low	Most Likely	High
Project Risks: Internal Risk Items (no Price Risk or Uncertainty, Unless an Issue Exists within the PDT's Scope of Influence)									
PROJECT & PROGRAM MGMT									
PPM-1	Project schedule in question	Very Likely	Critical	[illegible]	Triangular	Schedule Risk Items for PPM	[illegible]	[illegible]	$1,200,000
CONTRACT ACQUISITION RISKS									
CA-1	Undefined acquisition strategy	Very Likely	Marginal	Negligible	Triangular		[illegible]	[illegible]	$1,800,000
CA-3	Acq strategy decr competition	Very Likely	Negligible	Low	Triangular		$0	[illegible]	$1,800,000
TECHNICAL RISKS									
TL-1	Confidence in scope	Likely	Significant	[illegible]	Triangular	Duplicate with Risks 2 and 3 (not included)	[illegible]	$0	$28,200,000
TL-2	Surveys in question	Likely	Negligible	Low	Triangular		[illegible]	$0	$1,100,000
TL-3	Borrow sources secured	Likely	Negligible	Low	Triangular		$0	$0	$1,277,000
LANDS AND DAMAGES RISKS									
LD-5	Difficulty of RE work	Likely	Negligible	Low	Triangular		$0	$0	$1,250,000
REGULATORY AND ENVIRONMENTAL RISKS									
CONSTRUCTION RISKS									
CON-4	Contract Modifications	Likely	Marginal	Significant	Triangular		[illegible]	$0	$19,000,000
CON-5	Railroad involvement	Likely	Negligible	Low	Triangular		$0	$0	$2,400,000
ESTIMATE AND SCHEDULE RISKS									
EST-1	Cost of Fuel	Likely	Negligible	Low	Triangular		$0	$0	$1,900,000
EST-2	Cost of Steel	Likely	Negligible	Low	Triangular		$0	$0	$1,700,000
EST-3	Parametric Est used for Critical Items	Likely	Negligible	Low	Triangular		[illegible]	$0	$4,250,000
EST-4	Prime/Subcontractor Structure	Likely	Marginal	Significant	Triangular		$0	$0	$12,400,000
EST-5	Utility Relocations	Likely	Negligible	Low	Triangular		$0	$0	$4,700,000
EST-6	JOOH	Likely	Negligible	Low	Triangular		$0	$0	$2,700,000
EST-7	HOOH	Likely	Negligible	Low	Triangular		$0	$0	$1,900,000
Programmatic Risks: External Risk Items are Those Not Under Direct Control or Influence Directly by the PDT's Scope of Influence									
PR-1	Adequacy of project funding	Very Likely	Significant	[illegible]	Triangular	Duplicate with Risks 2, Risks 3 and Risk 5 (not included)	$0	$29,000,000	$34,000,000
PR-3	Market/Bidding Conditions	Likely	Significant	[illegible]	Triangular		[illegible]	$0	$10,000,000



PROJECT CONTINGENCY (BASELINE)	Percentile	Baseline TPC	Contingency Amt	Baseline w/ Contingency	Contingency %
	0%	$179,439,000	($15,919,322)	$163,519,678	-10.37%
	5%	$179,439,000	$15,808,174	$195,799,174	19.81%
	10%	$179,439,000	$24,301,499	$203,740,499	13.83%
	15%	$179,439,000	$27,764,893	$207,203,893	15.46%
	20%	$179,439,000	$29,368,697	$208,807,697	17.15%
	25%	$179,439,000	$32,497,698	$211,936,698	18.44%
	30%	$179,439,000	$35,002,589	$214,441,589	19.64%
	35%	$179,439,000	$37,956,497	$217,395,497	20.77%
	40%	$179,439,000	$38,909,681	$218,348,681	21.46%
	45%	$179,439,000	$40,818,674	$220,257,674	22.83%
	50%	$179,439,000	$42,488,394	$221,927,394	23.98%
	55%	$179,439,000	$44,521,434	$223,960,434	24.97%
	60%	$179,439,000	$46,525,766	$225,964,766	25.92%
	65%	$179,439,000	$48,365,098	$227,804,098	26.89%
	70%	$179,439,000	$50,310,689	$229,749,689	28.01%
	75%	$179,439,000	$52,708,317	$232,147,317	29.34%
	80%	$179,439,000	$53,848,329	$233,285,329	30.02%
	85%	$179,439,000	$57,793,215	$237,232,215	31.69%
	90%	$179,439,000	$61,777,830	$241,216,830	34.21%
	95%	$179,439,000	$66,317,751	$245,756,751	37.05%
	100%	$179,439,000	$69,139,313	$248,578,313	37.41%





COST RISK ANALYSIS

US Army Corps
of Engineers®

KANSAS CITYS FLOOD RISK MANAGEMENT PROJECT – CENTRAL INDUSTRIAL DISTRICT UNIT FEASIBILITY STUDY

Project Cost and Schedule Risk Analysis Report

Prepared for:

U.S. Army Corps of Engineers,
Kansas City District

Prepared by:

U.S. Army Corps of Engineers
Kansas City District

October 11, 2013

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EXECUTIVE SUMMARY

The US Army Corps of Engineers (USACE), Kansas City District, presents this cost and schedule risk analysis (CSRA) report regarding the risk findings and recommended contingencies for the Kansas City's Flood Risk Management Project – Central Industrial District Unit Feasibility Study. In compliance with Engineer Regulation (ER) 1110-2-1302 CIVIL WORKS COST ENGINEERING, dated September 15, 2008, a formal risk analysis, *Monte-Carlo* based-study was conducted by the Project Development Team (PDT) on remaining costs. The purpose of this risk analysis study is to present the cost and schedule risks considered, those determined and respective project contingencies at a recommend 80% confidence level of successful execution to project completion.

The Central Industrial District – Kansas flood protection unit is located in Wyandotte County, Kansas, and extends from the Kansas/Missouri state line along the right bank of the Missouri River to the mouth of the Kansas River. It then continues upstream along the right bank of the Kansas River to mile 3.4. The Kaw Valley Drainage District is the local agency responsible for operation and maintenance. The original unit was constructed by the Kaw Valley Drainage District prior to May, 1948, when initial improvements began. The bulk of the improvements were completed by November, 1955. The most recent improvements were completed in December, 1979. The unit consists of a system of levees and floodwalls, underseepage control including 17 relief wells, a stoplog gap, a sandbag gap, 10 pump plants, and 23 drainage structures. The levee is approximately 1.8 miles long and the floodwalls total about 7,900 feet.

The Central Industrial District – Missouri flood protection unit is located in Kansas City, Missouri within Jackson County. The unit extends along the right bank of the Missouri River, upstream from the Grand Avenue Viaduct (river mile 365.7); to the Kansas/Missouri state line (river mile 367.2). The City Council passed four resolutions between 1941 and 1947 to provide the required assurances of local cooperation. The initial construction began in March, 1946 and was completed in September, 1947. Significant improvements and repair of 1951 flood damage followed the initial construction and were completed in November, 1955.

The unit consists of a system of levees, floodwalls, underseepage control, 1 sandbag and 7 stoplog gaps, 7 pump plants, and 5 conduits. The levees total about 430 feet in length and the floodwalls are about 1.45 miles long.

Specific to the Central Industrial District Flood Risk Management Project, the current fully funded estimate approximates $104M. This CSRA study is expressed in FY 2014 dollars. Real Estate office provided a separate 25% contingency for its real estate requirements, which in turn was used in the Cost Risk Model. The Cost Engineering Section performed study on the total estimated project costs. Based on the results of the analysis, the Cost Engineering Section (located in Kansas City District) recommends a contingency value of approximately $19M or approximately 31% of base project cost. This contingency includes a separate $700K for Real Estate, another $16M for the construction costs, and $2.3M for design and construction management.

The Kansas City District Cost Engineering Section performed risk analysis using the *Monte Carlo* technique for the estimated construction costs, supported by the district PDT input. The following table ES-1 portrays the development of the construction contingencies (31.07%). The contingency is based on an 80% confidence level, as per USACE Civil Works guidance. Cost estimates fluctuate over time. During this period of study, minor cost fluctuations can and have occurred. For this reason, contingency reporting is based in cost and percent values. Should cost vary to a slight degree with similar scope and risks, contingency percent values will be reported, cost values rounded.

Table ES-1. Construction Contingency Results

Base Case Construction Cost Estimate	$62,079,416	
Confidence Level	**Construction Value ($$)**	**Contingency (%)**
5%	$70,323,663	13.28%
50%	$77,658,057	25.09%
80%	**$81,367,278**	**31.07%**
90%	$83,272,942	34.14%

The following table ES-2 portrays the full costs of the cost recommendations, combining all remaining costs. The costs are intended to address the congressional request of estimates to implement the project. The contingency is based on an 80% confidence level, as per accepted USACE Civil Works guidance.

Table ES-2. Cost Summary of Remaining Costs (FY2014 dollars)

CENTRAL INDUSTRIAL DISTRICT FORM FEATURE ACCOUNTS		COST ($1,000)	CNTG ($1,000)	CNTG %	TOTAL ($1,000)
01	LANDS AND DAMAGES	2,297	714	31.07	3,011
02	RELOCATIONS	4,814	1,496	31.07	6,310
06	FISH & WILDLIFE FACILITIES	0	0	0	0
11	LEVEES & FLOODWALLS	45,343	14,088	31.07	59,431
13	PUMPING PLANT	1,971	612	31.07	2,583
30	PLANNING, ENGINEERING, AND DESIGN	3,969	1,233	31.07	5,202
31	CONSTRUCTION MANAGEMENT	3,685	1,145	31.07	4,830
	FY 2014 PROJECT COSTS	**62,079**	**19,288**	**31.07**	**81,366**
	Schedule Completion with Contingency	**120 mo**	**108 mo**	**90.00**	**228 mo**

Notes:
1) Cost and Time contingencies presented w/ an 80% confidence level.
2) Costs exclude O&M and Life Cycle Cost estimates.

3) Differences between ES-1 and ES-2 are due to rounding.

KEY FINDINGS/OBSERVATIONS RECOMMENDATIONS

The PDT worked through the risk register on September 17, 2012 and July 10, 2013. During this timeframe the PDT discussed project scope definition, investigations, design and cost information, and determined risks in certain project areas. The key risk drivers identified through sensitivity analysis suggest a cost contingency of $19M and schedule risks adding another potential of 108 months, both at an 80% confidence level.

Cost Risks: From the CSRA, the key or greater Cost Risk items of include:

High Risks

- PR-3: Market Conditions/Bidding Conditions – The economy is currently in a downturn with signs of improvement. A range was given to account for the variance of possibilities. The estimate assumes a normal bidding climate.
- PR-1: Adequacy of Project Funding – Incremental congressional appropriations, and the sponsors ability to cost share. Congressional appropriations will most likely be incrementally funded with minimal appropriations per year assigned to this project. The risk of the schedule slipping is assumed and therefore a 1.5% annual escalation rate compounded has been included in the cost model.
- TL-3: Confidence in Floodwalls Design – Not enough information to adequately formulate a design. Designers did not always use a conservative approach but rather used design intuition for assumptions. In cases where no information was available, extrapolation techniques were used. Additional subsurface investigations may be required resulting in the floodwalls needing to be replaced instead of raised.

Moderate Risks

- CA-1: Undefined Acquisition Strategy – Large business competitive pricing could be eliminated if other acquisition strategies are used.
- CON-4: Contract Modifications – Large project could have several large modifications.
- TL-4: Pump Plants Design – Not enough information to adequately formulate a design. Designers did not always use a conservative approach but rather used design intuition for assumptions. In cases where no information was available, extrapolation techniques were used. Additional pump plant investigations may be required resulting in more extensive modifications than originally anticipated.
- TL-1: Relief Wells Design – Not enough information to adequately formulate a design. Designers did not always use a conservative approach but rather used design intuition for assumptions. In cases where no information was available, extrapolation techniques

were used. Additional subsurface investigations may be required resulting in more relief wells being added than originally anticipated.

Low risks were not modeled.

Schedule Risks: The high value of schedule risk indicates a significant uncertainty of key risk items, time duration growth that can translate into added costs. Over time, risks increase on those out-year contracts where there is greater potential for change in new scope requirements, uncertain market conditions, and unexpected high inflation. The greatest risk is:

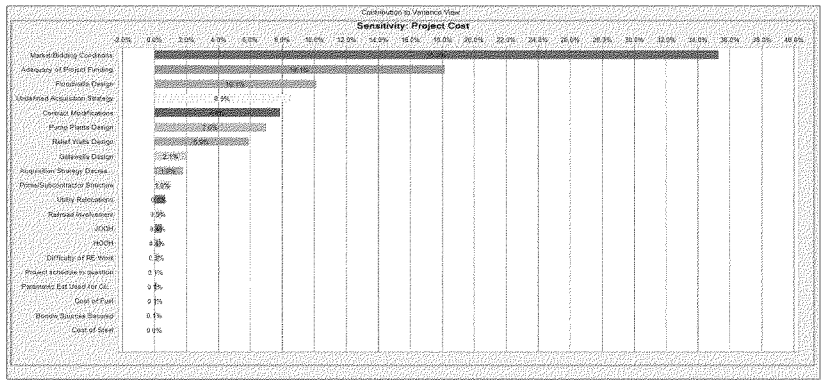
- PR-1: Adequacy of Project Funding – Congressional appropriation will most likely be incrementally funded with minimal appropriation per year assigned to this project. The sponsor will most likely be unable to afford their cost share portion on a timely manner.

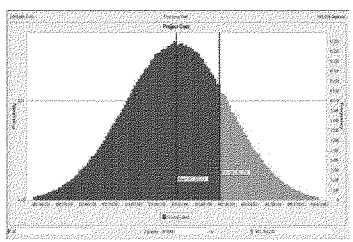
Recommendations: Timely coordination and risk resolution between the Sponsor and USACE is needed in areas of funding needs and updates as applicable. The PDT must include the recommended cost and schedule contingencies and incorporate risk monitoring and mitigation on those identified risks. Further iterative study and update of the risk analysis throughout the project life-cycle is important in support of the remaining project work within an approved budget and appropriation.

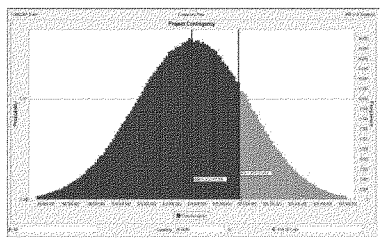
Central Industrial District n500+3 ft. Recommended Alternative - Cost Risk Analysis Model

Risk No.	Risk/Opportunity Event	Project Cost: Likelihood*	Project Cost: Impact*	Project Cost: Risk Level*	Variance Distribution	Correlation to Other(s)	Crystal Ball Simulation Expected Values ($$$): Low	Most Likely	High
General Risks (General Risk items not specified or particular, which is controlled within the PDT's sphere of influence)									
PROJECT & PROGRAM MGMT									
PPM-1	Project schedule in question	Very Likely	Negligible	LOW	Triangular	Schedule Recommended by PDT	$0	$50,000	$750,000
CONTRACT ACQUISITION RISKS									
CA-1	Undefined Acquisition Strategy	Very Likely	Marginal	MODERATE	Triangular		$0	$0	$3,500,000
CA-3	Acquisition Strategy Decreasing Competition	Likely	Negligible	LOW	Triangular		$0	$0	$3,500,000
TECHNICAL RISKS									
TL-1	Relief Wells Design	Unlikely	Significant	MODERATE	Triangular	Duplicate with PPM-2. PPM-2 not modeled	$0	$0	$4,000,000
TL-2	Gatewells Design	Unlikely	Marginal	LOW	Triangular	Duplicate with PPM-2. PPM-2 not modeled	$0	$0	$1,250,000
TL-3	Floodwalls Design	Likely	Marginal	MODERATE	Triangular	Duplicate with PPM-2. PPM-2 not modeled	$0	$0	$3,750,000
TL-4	Pump Plants Design	Unlikely	Significant	MODERATE	Triangular	Duplicate with PPM-2. PPM-2 not modeled	$0	$0	$2,500,000
TL-6	Borrow Sources Secured	Likely	Negligible	LOW	Uniform		$0	$0	$500,000
LANDS AND DAMAGES RISKS									
LD-5	Difficulty of RE Work	Likely	Negligible	LOW	Triangular		$0	$0	$1,000,000
REGULATORY AND ENVIRONMENTAL RISKS									
CONSTRUCTION RISKS									
CON-4	Contract Modifications	Likely	Marginal	MODERATE	Triangular		$0	$0	$1,250,000
CON-5	Railroad Involvement	Likely	Negligible	LOW	Triangular		$0	$0	$1,250,000
ESTIMATE AND SCHEDULE RISKS									
EST-1	Cost of Fuel	Likely	Negligible	LOW	Triangular		$0	$0	$500,000
EST-2	Cost of Steel	Likely	Negligible	LOW	Triangular		$0	$0	$500,000
EST-3	Parametric Est Used for Critical Items	Likely	Negligible	LOW	Triangular		$0	$0	$750,000
EST-4	Firms/Subcontractor Structure	Likely	Negligible	LOW	Triangular		$0	$0	$1,000,000
EST-5	Utility Relocations	Likely	Negligible	LOW	Triangular		$0	$0	$1,000,000
EST-6	JOCH	Likely	Negligible	LOW	Triangular		$0	$0	$1,250,000
EST-7	HOCH	Likely	Negligible	LOW	Triangular		$0	$0	$1,500,000
Programmatic Risks (Global Risk items are those that are generated, caused, or controlled outside control of PDT's sphere of influence)									
PR-1	Adequacy of Project Funding	Likely	Significant	HIGH	Triangular	Duplicate with PPM-1. Problems of PPM-1 not modeled	$0	$10,000,000	$30,000,000
PR-3	Market/Bidding Conditions	Likely	Significant	HIGH	Triangular		$0	$0	$8,000,000

PROJECT CONTINGENCY (BASELINE)	Percentile	Baseline TPC	Contingency Not Included	Baseline w/ Contingency	Contingency %
	0%	$62,079,416	$54,231,583	$57,311,833	-8.97%
	5%	$62,079,416	$9,244,247	$76,323,662	13.28%
	10%	$62,079,416	$9,349,424	$71,928,840	15.87%
	15%	$62,079,416	$10,040,739	$73,020,183	17.62%
	20%	$62,079,416	$11,012,432	$73,891,848	19.00%
	25%	$62,079,416	$12,155,394	$74,834,810	20.22%
	30%	$62,079,416	$13,230,857	$75,311,073	21.41%
	35%	$62,079,416	$13,555,075	$75,892,491	22.32%
	40%	$62,079,416	$14,464,059	$76,025,486	23.27%
	45%	$62,079,416	$15,038,073	$77,160,689	24.28%
	50%	$62,079,416	$15,574,681	$77,654,097	25.09%
	55%	$62,079,416	$16,139,863	$78,219,279	25.99%
	60%	$62,079,416	$16,705,364	$78,787,980	26.81%
	65%	$62,079,416	$17,297,708	$79,373,124	27.66%
	70%	$62,079,416	$17,901,994	$79,981,410	28.54%
	75%	$62,079,416	$18,581,973	$80,661,391	29.59%
	80%	$62,079,416	$19,287,882	$81,367,279	31.07%
	85%	$62,079,416	$20,131,938	$82,211,336	32.42%
	90%	$62,079,416	$21,193,928	$83,273,342	34.14%
	95%	$62,079,416	$22,797,664	$84,877,080	36.83%
	100%	$62,079,416	$35,238,355	$97,317,771	56.75%







Final Feasibility Report

APPENDIX F

VALUE ENGINEERING

Kansas Citys, Missouri and Kansas, Flood Risk Management Project
Final Feasibility Report

Value Engineering Certification

I, Eric Lynn, Project Manager, certify that the Value Engineering process as required by ER 11-1-321 (Change 1), Army Programs Value Engineering has been completed for this Feasibility Study. I certify compliance with Public Law 99-662 (33 USC 2288) and OMB Circular A-131. The VE study was completed on 25 October 2013 by the appropriate authority. All VE proposals indicating potential savings over $1,000,000 have been resolved with approval of the MSC Commander.

LYNN.ERIC.S.1270505518
Digitally signed by LYNN.ERIC.S.1270505518
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=LYNN.ERIC.S.1270505518
Date: 2014.04.10 15:09:14 -05'00'

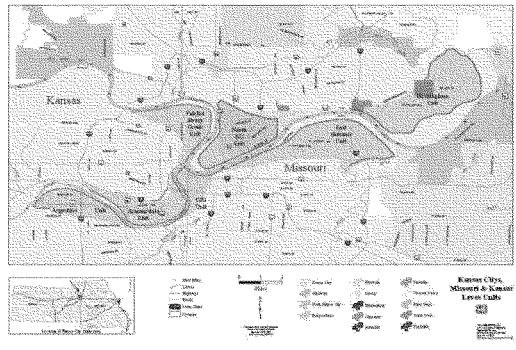
Project Manager

Digitally signed by BENSON.JOHN.MICHAEL.1123224406
DN: c=US, o=U.S. Government, ou=DoD, ou=PKI, ou=USA, cn=BENSON.JOHN.MICHAEL.1123224406
Date: 2014.04.10 15:17:38 -05'00'

District Value Engineering Officer

US Army Corps of Engineers®

Kansas Citys, Missouri and Kansas Flood Risk Management Project



Value Engineering Study

Final Report

October 21 – October 25, 2013

PREPARED FOR
USACE (NWK)
601 E. 12th Street
Kansas City, Missouri
816-389-2000

PREPARED BY
HNTB Corporation
715 Kirk Drive
Kansas City, MO 64105
816-472-1201

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HNTB would like to extend its appreciation to the US Army Corps of Engineers for being selected to assist the Project Delivery Team in verifying current design concepts, offering potential improvements, and reducing risks to the project by conducting this value engineering workshop. The enthusiasm of the VE participants provided an abundance of information for USACE (NWK) to consider throughout the remainder of the project delivery process. Thank you all for your insight and effort.

EXECUTIVE SUMMARY

Introduction

USACE (NWK) requested HNTB to conduct a value engineering study of the Kansas Citys, Missouri and Kansas Flood Risk Management Project located in Kansas City, Kansas and Kansas City, Missouri. The value engineering study was conducted according to USACE regulations and consisted of the following phases:

- Pre-workshop planning
- Information:
  - Review of available information gathered or generated by Project Delivery team
  - Validation of constraints
  - Validation of project goals, objectives, purpose and need
- Function Analysis:
  - Component functions, performance and operations
  - Risk identification
  - Cost analysis
- Creative: brainstorming ideas open throughout the workshop on all components and elements
- Evaluation of ideas
- Development of ideas and recommendations
- Presentation of conclusions and recommendations
- Resolution: documented decision on the implementation of each recommendation

The value engineering study occurred early in the project development near completion of feasibility study and before preliminary plans. The Project Delivery Team is able to implement recommendations as part of the continuing concept evaluation process. The recommendations focused on operations, design concepts, utility coordination, right of way acquisition, both the design and construction schedule, and the entire project delivery process.

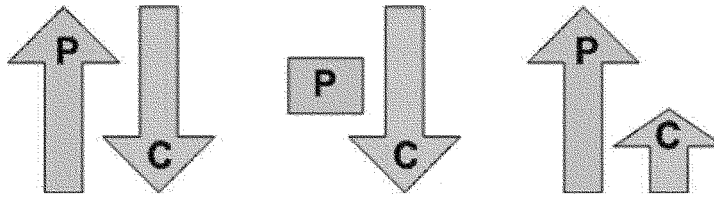
The VE report for the Kansas Citys, Missouri and Kansas Flood Risk Management Project summarizes the analyses, recommendations, and suggestions of the VE team and presents their conclusions in several tables. More detailed information for each idea is provided in the Appendices. Most importantly, the decision and implementation worksheet is provided following the summary. Completing this worksheet by the management team complies with USACE (NWK) regulations, and will enable written directions to be provided to the Project Delivery team.

Next Actions

Conclusion of workshop and Presentation of Recommendations: October 25, 2013	Draft idea worksheets provided by VE team to Project Manager and Project Delivery team.
Five days after workshop: October 30, 2013	Worksheet comments returned from USACE (NWK) to HNTB.
Seven days after workshop: November 1, 2013	Draft report submitted to USACE (NWK) management with Recommendation Approval Worksheet.
PDT review of draft report submittal: January, 2014	USACE (NWK) management completes the Recommendation Approval Worksheet with decisions on each recommendation; and returns comments on draft report.
Final Report Submittal: March, 2014	Final report provided to USACE (NWK) incorporating comment resolution.

Value Engineering Recommendations

Value engineering seeks to improve performance (P) and reduce costs (C). Some examples and a graphic illustration of positive value engineering analyses are:



Increased Performance	Same Performance	Increased Performance
Decreased Cost	Decreased Cost	Small Increased Cost

The VE team identified three ideas, six design suggestions, and three recommendations for implementation with a potential cost savings of approximately $15 million to $18 million based on evaluating performance and cost. These ideas originated from analyzing the construction cost estimates, identifying potential areas of risk regarding the schedule (performance) and the construction cost; and discussing the expectations of the VE study.

The report utilizes a Change of Cost methodology to properly account for the estimated savings or added costs. Savings are subtracted from the current cost estimate; while added costs would increase the current cost estimate. Therefore, please note that throughout the report cost changes are indicated as:

- (Savings) – the parentheses and red font indicate a potential cost reduction
- **Increase** – no parentheses and the black font indicate a potential added cost

The Summary of Recommendations is noted in the table below. In some cases, construction time was reduced; or situations that could increase construction time were avoided. In some cases, two recommendations are mutually exclusive; only one can be implemented and the other(s) is therefore excluded. Each recommendation identifies those others that are mutually exclusive. The Appendix includes more detailed worksheets and calculations for each of the recommendations.

	Idea	Change in Cost (Savings) Increase	VE Team Recommended Action
1	Consider Portable Pumps and Relief Wells for Armourdale Sta. 62+00 to 82+00 in lieu of Slurry Wall. (Mutually exclusive with Recommendation 2)	($18.0 M)	The VE team recommends a collected relief well system with portable pumps be considered during PED provided the extent of the potential groundwater contamination between 45+00 and 75+00 is determined by testing performed during PED. While a relief well system option requires maintenance, it has a much lower upfront cost than a slurry wall and does not disrupt existing utilities.
2	Consider Permanent Pumps and Relief Wells for Armourdale Sta. 62+00 to 82+00 in lieu of Slurry Wall. (Mutually exclusive with Recommendation 1)	($13.6 M)	The VE team recommends a collected relief well system with a permanent pump station be considered during PED provided the extent of the potential groundwater contamination between 45+00 and 75+00 is determined by testing performed during PED. While a relief well system option requires maintenance, it has a much lower upfront cost than a slurry wall and does not disrupt existing utilities.
3	The CID Levee terminus at the south end, approximate station 138+00, is proposed to use tieback Alt. #3, a new floodwall and two stoplog structures from the earthen levee to high ground. Proposed change is to replace one of these stoplog structures with a continuation of the floodwall leaving only one stoplog structure.	($2.1 M)	Replace the proposed stoplog structure to be constructed over the rail yard track with a continuation of the concrete floodwall and only construct one stoplog structure over the mainline tracks. This will require the acquisition of the land through the end of the rail yard and removal of the rail necessary to construct the floodwall however, the elimination of the stoplog alleviates the local sponsor's tasks necessary to maintain the stoplog structure, store stoplogs, install them at times of high water, etc.
	Preliminary Estimated Change in Cost	($ 15.7 to $18.0 Million)	

Design Suggestions

The VE team identified a number of ideas that because of time limitations, development of conceptual plans, or lack of cost data could not be further developed as recommendations. Nonetheless, the idea has been investigated and suggested for the Project Delivery team to consider as the project continues to develop. Please refer to the Idea Development Worksheets in the Appendices for possible further details.

Idea	DESIGN SUGGESTIONS
DS-1	Review and clarify design criteria as identified in the Report Inconsistencies.
DS-2	Review and expand on the CID-KS and ARM Construction Phasing Suggestions.
DS-3	As stated in Appendix D of the draft Feasibility Report for the Armourdale Unit, "the installation of relief wells as underseepage controls between station 45+00 and 75+00 may result in the discharge or migration of VOC contaminated groundwater and should be avoided." But it goes on to state that "since the plume has not been clearly identified, a smaller restrictive area could be established with additional investigations if relief wells are required." Because a relief well system may prove to be more economical than the currently selected slurry wall (see Ideas 1 & 2), the VE team suggest performing environmental screening during PED on soil and water samples obtained along the landside toe between station 66+00 and 75+00. In addition, an argument can be made that groundwater will flow to relief wells from the river during a flood event, rather than from a landward area of contamination.
DS-4	Based on our review of the draft Feasibility Report, it appears that the landside blanket thickness from station 66+00 to station 79+00 along the Armourdale Unit was estimated to be locally thin (12 to 15 feet) based on historic borings performed near centerline of levee. Significant project costs associated with the underseepage control are tied to this assumption. During PED, the VE team suggests performing relatively shallow borings on the landside toe along this reach to verify the landside blanket's character and thickness.
DS-5	If a potential zone of groundwater contamination is confirmed to be present along the Armourdale Unit between station 66+00 and 75+00 and relief wells cannot be used (see DS-3), the VE team suggests evaluating partial implementation of Ideas 1 or 2 during PED. In other words, the life cycle cost of a relief well system installed between Station 75+00 and 79+00 should be compared to savings associated with shortening the slurry wall 700 feet (by deleting the portion between Station 75+00 and 82+00). This shorter relief well system might make the use of portable pumps or a reconfiguration of the existing Osage Avenue Pump Plant attractive options. We also note that the vast majority of existing utilities that conflict with the proposed slurry wall are located between station 75+00 and 82+00, another reason to carefully consider a mixture of underseepage control methods during PED.
DS-6	The cost of the slurry cutoff wall selected for the Armourdale Unit between Station 62+00 and 82+00 is highly dependent on the top of bedrock elevation and, if a key is required, the character of the bedrock. Therefore, the VE team suggests performing exploratory borings along the alignment of the wall during PED. The VE team also suggests that hydraulic analysis be performed during PED to determine if the slurry wall can be terminated directly above top of rock or if it must be keyed into the rock (the rock key is the most expensive portion of the slurry wall).

Implementation

A Value Engineering Recommendation Approval Form is included below. A separate working copy of the Recommendation Approval Worksheet has been furnished for distribution to the management team for their review and documentation of their deliberations and decisions on each recommendation. When completed the final worksheet with the documented decisions will be included in this section of the report.

Value Engineering Recommendation Approval Form

Project: Kansas Citys, Missouri and Kansas Flood Risk Management Project

VE Workshop Dates: October 21 - October 25, 2013

	Recommendation	Accept, Reject, or Accept for Further Review (Select cell and use dropdown arrow)	Reason for Acceptance or Rejection	Functional Benefit (use capital X)					VE Team Estimated Change of Cost (Savings) Increase
				Safety	Operations	Environment	Construction	Other	
1	*Consider Portable Pumps and Relief Wells for Armourdale Sta. 62+00 to 82+00 in liue of Slurry Wall (mutually exclusive with Recommendation 2).* The VE team recommends a collected relief well system with portable pumps be considered during PED provided the extent of the potential groundwater contamination between 45+00 and 75+00 is determined by testing performed during PED. While a relief well system option requires maintenance, it has a much lower upfront cost than a slurry wall and does not disrupt existing utilities.	Accept for Further Review	potential cost savings and reduced utility disruption						($18.0 M)
2	*Consider Permanent Pumps and Relief Wells for Armourdale Sta. 62+00 to 82+00 in lieu of Slurry Wall (mutually exclusive with Reommendation 1).* The VE team recommends a collected relief well system with a permanent pump station be considered during PED provided the extent of the potential groundwater contamination between 45+00 and 75+00 is determined by testing performed during PED. While a relief well system option requires maintenance, it has a much lower upfront cost than a slurry wall and does not disrupt existing utilities.	Accept for Further Review	potential cost savings and reduced utility disruption						($13.6 M)

Value Engineering Recommendation Approval Form

Project: Kansas Citys, Missouri and Kansas Flood Risk Management Project

VE Workshop Dates: October 21 - October 25, 2013

	Recommendation	Accept, Reject, or Accept for Further Review (Select cell and use dropdown arrow)	Reason for Acceptance or Rejection	Functional Benefit (use capital X)					VE Team Estimated Change of Cost (Savings) Increase
				Safety	Operations	Environment	Construction	Other	
3	*The CID Levee terminus at the south end, approximate station 138+00, is proposed to use tieback Alt. #3, a new floodwall and two stoplog structures from the earthen levee to high ground. Proposed change is to replace one of these stoplog structures with a continuation of the floodwall, leaving only one stoplog structure.* Replace the proposed stoplog structure to be constructed over the rail yard track with a continuation of the concrete floodwall and only construct one stoplog structure over the mainline tracks. This will require the acquisition of the land through the end of the rail yard and removal of the rail necessary to construct the floodwall however, the elimination of the stoplog alleviates the local sponsor's tasks necessary to maintain the stoplog structue, store stoplogs, install them at times of high watr, etc.	Accept for Further Review	potential cost savings and reduced need for sponsor maintenance						($2.1 M)
			All Recommendations and Total Change in Cost:	**0**	**0**	**0**	**0**	**0**	**($15.7 to $18.0 M)**
			Total for # Accepted Recommendations:	**0**	**0**	**0**	**0**	**0**	**$0**
								Total Project Cost	**$217,818,409**
								Number of Design Suggestions	**6**

U.S. Army Corps of Engineers, Kansas City District

Final Feasibility Report

APPENDIX G

PUBLIC INVOLVEMENT AND COORDINATION

Kansas Citys, Missouri and Kansas
Flood Risk Management Project
Final Feasibility Report

APPENDIX G

PUBLIC INVOLVEMENT

The Draft Final Feasibility Report was released on November 22, 2013, for a thirty (30) day public review and comment period via the Kansas City District website.

Notice of the report availability and public comment period was posted on the Kansas City District website and provided to local Kansas City area media outlets. Additionally, notice was mailed to Kansas and Missouri congressional offices; state and local elected officials; Federal, State, County, and City agencies; community and environmental interest groups; Indian tribes; and businesses and property owners within the project area.

Written comments were requested to be submitted by mail or through the project website. The mailing list, public notice, and press release are included in this appendix as Exhibits 1, 2, and 3 respectively. The notification list includes contacts obtained from the Environmental Protection Agency, Region VII, to ensure compliance with Environmental Justice requirements.

In response to the Public Notice and public comment period, comments were received from the following entities:

Kansas State Historic Preservation Office
U.S. Fish & Wildlife Service
Missouri Department of Conservation
Missouri Department of Natural Resources
U.S. Environmental Protection Agency

The comment letters received are included in this appendix as Exhibit 4. There was no response received from media outlets. Responses to comments received are included in Exhibit 5. Please note that the EPA provided thirteen separate comments which are each presented separately with a response in the attached exhibit.

Public meetings and workshops were previously held in the Kansas City area in conjunction with the Reconnaissance Study and first phase of the Feasibility Study. Information gathered at those meetings was included in the previously published Interim Feasibility Report. Comments previously received on the Interim Feasibility Report and Environmental Impact Statement are included and responded to within those documents. If a previous comment addressed an issue or aspect of the second phase of the study it was considered in the development of the alternatives discussed in this Final Feasibility Report.

Regular contact and coordination has been maintained throughout the Feasibility Study with the local sponsors to provide updates on the status and findings of the study. With the sponsor's assistance, project status information has been shared with multiple stakeholder groups representing businesses and industries in the project area. Continually throughout this process, the local sponsors have expressed their desire to see their levee system restored to acceptable reliability. The local sponsors have initiated and maintained contact with local governments and their Congressional representatives to share project status information and urge continued support for the project.

The views of several State and Federal resource agencies were considered in the study, including the following:

U.S. Fish & Wildlife Service
Natural Resources Conservation Service
U.S. Environmental Protection Agency
Kansas Department of Wildlife and Parks
Kansas State Historic Preservation Office
Missouri Department of Conservation
Missouri Department of Natural Resources

Exhibit 1 – Public Notice Mailing List

Congressional Offices
Senator Pat Roberts – KS
Senator Jerry Moran – KS
Senator Claire McCaskill – MO
Senator Roy Blunt – MO
Representative Kevin Yoder – KS
Representative Emmanuel Cleaver II – MO
Representative Sam Graves – MO

Local Elected Officials
Governor Sam Brownback – KS
Governor Jay Nixon – MO
Mayor Mark Holland, Kansas City, KS
Mayor Sly James, Kansas City, MO
Mike Sanders, Jackson County Executive

Federal Agencies
Environmental Protection Agency, Region 7
Federal Aviation Administration
Federal Highway Administration
Federal Railroad Administration
Federal Transit Authority
FEMA, Region 7
National Park Service
Natural Resources Conservation Service
U.S. Coast Guard
U.S. Fish and Wildlife Service
U.S. Geological Survey
U.S. Department of the Interior

State Agencies
KS Biological Survey
KS Department of Agriculture
KS Department of Health and Environment
KS Department of Transportation
KS Department of Wildlife, Parks & Tourism
KS Division of Emergency Management
KS Geological Survey
KS State Conservation Commission
KS State Historical Society
KS Water Office
MO Dept of Natural Resources
MO Dept. of Transportation
MO Dept. of Conservation
MO Dept. of Public Safety

Local Government Agencies
City of Kansas City, Missouri:
- Water Services Department
- Planning and Development Dept.
- Parks and Recreation Department
- Public Works Department

Unified Government of Wyandotte County and Kansas City, Kansas:
- Public Works Department
- Urban Planning and Land Use Dept.
- Parks and Recreation Department

Board of Public Utilities, Kansas City, KS
Wyandotte County Emergency Management
Jackson County, Missouri:
- Public Works Department
- Stormwater Commission

Planning and Zoning Dept., Clay County, MO

Business and Community Organizations
Port Authority of Kansas City, MO
Mid-America Regional Council
Missouri and Associated Rivers Coalition
Kansas City Industrial Council
CID Association
FM Global
Riverside-Quindaro Bend Levee District
Birmingham Drainage District
North Kansas City Levee District
Fairfax Drainage District
Kansas Corporation Commission
Greater Kansas City Chamber of Commerce
Kansas Chamber of Commerce
Kansas City, KS, Chamber of Commerce
Water District No. 1 of Johnson County, KS
Greater Kansas City Federal Executive Board
F.W. Dodge Company
Kansas City Public Library
Kansas City, KS, Public Library
Armourdale Renewal Association
Rosedale Development Association
Guadalupe Centers, Inc.

Exhibit 1 – Public Notice Mailing List

NeighborWorks America Midwest Region
Westside Community Action Network
Greater KC LINC, Inc.
Bridging the Gap
Heartland Habitat For Humanity
Kansas City Neighborhood Alliance

Environmental and Recreation Interest Groups
Audubon of Kansas
Sierra Club – Kansas Chapter
Friends of the Kaw
Friends of Kaw Point Park
Friends of KCMO Riverfront Park
Conservation Federation of Missouri
Missouri Coalition for the Environment
Kansas Canoe & Kayak Association
KC River Trails
Kansas City Bicycle Club
Earthriders Mountain Bike Club
Missouri Bicycle Federation
Riverfront Utilization and Development Foundation
Johnson County Bicycle Club

Project Area Property Owners
See property owner listing in Real Estate Appendix

Indian Tribes
Cheyenne River Sioux Tribe
Crow Creek Sioux Tribal Council
Ho-Chunk Nation
Iowa Tribe of Kansas and Nebraska
Iowa Tribe of Oklahoma
Kickapoo Traditional Tribe of Texas
Kickapoo Tribe in Kansas
Kickapoo Tribe of Oklahoma
Miami Tribe
Northern Cheyenne Tribal Council
Ogallala Sioux Tribal Council
Omaha Tribe of Nebraska
Osage Tribe
Pawnee Nation of Oklahoma
Ponca Tribe of Nebraska
Prairie Band Potawatomi Nation
Rosebud Sioux Tribe
Sac and Fox Nation of Missouri
Sac and Fox Nation of Oklahoma
Sac and Fox Tribe of the Mississippi in Iowa
Santee Sioux Tribe
Shawnee Tribe
Spirit Lake Tribe
Three Affiliated Tribes
Wichita and Affiliated Tribes
Winnebago Tribe of Nebraska
Wyandotte Tribe of Oklahoma
Yankton Sioux Tribe

Media Outlets receiving Press Release
The Kansas City Star
Reuters
The Kansas City Globe
The Examiner
The Platte County Citizen
The Kearney Courier
Gladstone Dispatch
Fort Leavenworth Lamp
The Olathe News
Atchison Daily Globe
The Landmark
Kansas City Business Journal
The Pitch
The Platte County Citizen
Northeast News
Gardner News
The Call
Wyandotte Daily News
Lee's Summit Journal
Smithville Herald
Dos Mundos
Liberty Tribune
Leavenworth Times
The Weston Chronicle
KSMO-TV
WFTX-TV [Fox 4]
KCTV-TV [KCTV 5]
KSHB-TV [41 Action News]
WDAF-TV [Fox 4]
KMBC-TV [KMBC Channel 9]
KCWE-TV [KCWE Kansas City's CW]
WBAP-AM [WBAP News/Talk 820 AM]

Exhibit 1 – Public Notice Mailing List

KMMO-FM; KMMO-AM
KCUR-FM
KBEQ-FM [Q-104]
KFKF-FM [Country 94.1 KFKF]
KQRC-FM [98.9 The Rock!]
KMBZ-AM [98.1 FM, KMBZ]
WFIU-FM [WFIU]
KANU-FM
KDKD-AM [Classic 1280]; KDKD-FM
KXEA-FM [104.9 The Eagle]
KAIR-FM
KCHI-FM; KCHI-AM

Exhibit 2 – Public Notice Letter

DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
635 FEDERAL BUILDING
601 E 12TH STREET
KANSAS CITY, MISSOURI 64106-2896

REPLY TO
ATTENTION OF:

Planning, Programs and
Project Management
Planning Branch

NOTICE OF REPORT AVAILABILITY

Dear Interested Party:

The Draft Final Feasibility Report for the Kansas Citys, Missouri and Kansas, Flood Risk Management Study is available for public review at the website location below.

http://www.nwk.usace.army.mil/Missions/CivilWorks/CivilWorksProgramsandProjects/KansasCitys.FloodRiskManagement.aspx

This Final Feasibility Report is prepared as a complement to the Interim Feasibility Report published in 2006. This Final Report presents the evaluation of alternatives for improving the level of flood risk management for portions of the existing Kansas City metropolitan levee system not previously presented in the Interim Report. Specifically, the Final Report addresses the Armourdale and Central Industrial District Levee Units and examines projected future impacts and expected risks with and without the proposed alternatives.

Written comments on this Final Feasibility Report should be directed to the address below no later than December 21, 2013.

Corps of Engineers, Kansas City District
Kansas Citys Levees Project Manager
Room 529, PM-PF
601 E. 12th Street
Kansas City, Missouri 64106-2896

Written comments can also be submitted via electronic mail through the project website. Digital copies of the report on CD are available upon request at the address above or via the website e-mail address.

Army Corps of Engineers responses to written comments received in response to this notice will be published in the completed Final Feasibility Report.

Exhibit 3 – Press Release

NEWS RELEASE

U.S. ARMY CORPS OF ENGINEERS

BUILDING STRONG ®

For Immediate Release:
Release #PA-2013-72
November 22, 2013

Contact:
U.S. Army Corps of Engineers
Public Affairs Office
Kansas City, Mo. 64106-2896
Phone: (816) 389-3486
Fax: (816) 389-3434

Corps seeks public comments for seven levees report

KANSAS CITY, Mo.— The U.S. Army Corps of Engineers Kansas City District has studied flood risk management and reliability improvements for the seven levee units in the Kansas City metropolitan area and is seeking review and public comment on the information presented within the Draft Final Feasibility Report (DFFR).

The study was conducted at the request and with cooperation of the sponsors of the seven levee units under the authority of Section 216 of the 1970 Flood Control Act. Any comments received will be considered by the Corps of Engineers to determine whether to modify the recommendations within the report.

The purpose of the overall study of the seven levee units was to determine whether one or more plans for improvements to the existing levee system to reduce flood risk and improve levee reliability is technically viable, economically feasible, and environmentally acceptable, or if no action is warranted. Failure of any part of the existing flood risk management system during a major flood would have significant adverse impacts on the human environment including property damage and potential loss of human life.

The recommendations for the reliability improvements in the Armourdale and Central Industrial District (CID) units are addressed and available for review in the DFFR, which presents the completed feasibility analysis of alternatives for the Armourdale and CID levee units.

Proposed alternatives identified to improve flood risk management system reliability include, but are not limited to earthen levee raise, pump station modifications, floodwall replacement, stoplog and sandbag closures, pressure relief wells, floodwall modification using buttresses, and the no action alternative. DFFR analysis concluded that the nominal 500-year+3ft projected water surface elevation levee raise and underseepage improvements is the preferred alternative for both the Armourdale and CID units. The DFFR identifies a combination of measures as the Corps' overall recommended plan for these two levee units and presents an analysis of the costs and impacts associated with the alternatives listed above.

NATIONAL ENVIRONMENTAL POLICY ACT (NEPA) OF 1969, as amended: Considering potential significant impacts on the human environment, and in accordance with the National Environmental Policy Act, the Corps prepared and released the Interim Feasibility Report (IFR) and Draft Environmental Impact Statement (DEIS) in June 2006 for a 45-day public review (Public Notice 200601672). The IFR and DEIS presented the feasibility analysis, no action, action alternatives, preferred alternatives and associated environmental impacts for the Argentine, East Bottoms, Fairfax-Jersey Creek, Birmingham, and North Kansas City levee units. Preliminary feasibility analysis, the no

action alternative, action alternatives, tentatively preferred alternatives and associated environmental impacts for the Armourdale and CID units were also addressed in the IFR and DEIS. The tentatively preferred alternatives for Armourdale and CID were documented within the IFR and DEIS as the nominal 500-year+3 levee raise and underseepage improvements. A Section 404 of the Clean Water Act (33 USC 1344) compliance evaluation was included as Appendix H of the DEIS and Appendix J of the Final Environmental Impact Statement (FEIS) completed in August 2006.

PUBLIC INTEREST REVIEW: The Corps of Engineers is soliciting comments on the DFFR from the public, federal, state, and local agencies and officials, Indian Tribes and other interested parties in order to consider and evaluate the impacts of this proposed activity. Any comments received will be considered by the Corps of Engineers to determine whether to modify the recommendations within the report.

COMMENTS: Any interested party is invited to submit written comments relative to the DFFR on or before the public notice expiration date. Comments both favorable and unfavorable will be accepted, included within the project record and will receive full consideration in determining whether to modify report recommendations. Comments should be mailed to:

Kansas City District, Corps of Engineers
Kansas City Levees Project Manager
Room 529, PM-PF
601 E. 12th Street
Kansas City, MO 64106-2896

ADDITIONAL INFORMATION: The DFFR, previous IFR and FEIS may be viewed at the following website:
www.nwk.usace.army.mil/Missions/CivilWorks/CivilWorksProgramsandProjects/KansasCitys,FloodRiskManagement, or may be obtained by writing to the address above, or by contacting the project manager at KC.LeveesComments@usace.army.mil.

A copy of this public notice may also be viewed at the following website:
www.nwk.usace.army.mil/Missions/RegulatoryBranch.aspx. For more information, please contact the Public Affairs Office at (816) 389-3486.

-30-

Exhibit 4 – Comments Received

Written Comments Received In Response to Public Notice

Written comments were received from the following organizations on the dates indicated:

KS State Historic Preservation Office: letter dated December 6, 2013
Missouri Department of Conservation: letter dated December 20, 2013
U.S. Fish and Wildlife Service: electronic mail dated December 24, 2013
Missouri Department of Natural Resources: letter dated December 27, 2013
U.S. Environmental Protection Agency: e-mail dated December 31, 2013

6425 SW 6th Avenue
Topeka, KS 66615

Kansas

phone: 785-272-8681
fax: 785-272-8682
email@kshs.org

Kansas Historical Society

Sam Brownback, Governor
Jennie Chinn, Executive Director

KSR&C No. 13-12-036

December 6, 2013

Levees Program Manager
Kansas City District, Corps of Engineers
Room 529, PM-PF
601 E. 12th Street
Kansas City, MO 64106

Via E-mail

RE: Kansas City Levee Improvements
Armourdale and Central Industrial District (CID) Units
Wyandotte County

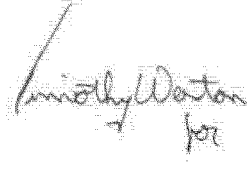
Dear Levees Program Manager:

In accordance with 36 CFR 800, the Kansas State Historic Preservation Office has reviewed your public notice dated November 22, 2013 regarding the above-referenced project. According to our records, we reviewed proposed improvements to both the Armourdale and Central Industrial District (CID) levee units in 2006. That review is noted in the Final Feasibility Report, dated November 2013. Since we see no significant changes in the current plans, our original clearance can stand. Our office continues to have no objection to the proposed levee improvement projects.

This information is provided at your request to assist you in identifying historic properties, as specified in 36 CFR 800 for Section 106 consultation procedures. If you have questions or need additional information regarding these comments, please contact Tim Weston at 785-272-8681 (ext. 214) or Kim Gant at 785-272-8681 (ext. 225).

Sincerely,

Jennie Chinn, Executive Director and
State Historic Preservation Officer



Patrick Zollner
Deputy SHPO

MISSOURI DEPARTMENT OF CONSERVATION

Headquarters
2901 West Truman Boulevard, P.O. Box 180, Jefferson City, Missouri 65102-0180
Telephone: 573-751-4115 ▲ www.MissouriConservation.org

ROBERT L. ZIEHMER, Director

December 20, 2013

Kansas City Levees Project Manager
Kansas City District, Corps of Engineers
Room 529, PM-PF
601 E. 12th Street
Kansas City, MO 64106-2896

Re: Public Notice, Kansas City's, Missouri and Kansas Flood Risk Management Study Missouri and Kansas Rivers Draft Final Feasibility Report

Dear Project Manager,

Thank you for the opportunity to comment on the Draft Final Feasibility Report for the Flood Risk Management Study. The existing Kansas City metropolitan flood risk management system includes seven levee units that provide benefits to the Cities of Kansas City, North Kansas City, Birmingham, Jackson, and Clay Counties, Missouri, and Wyandotte County, Kansas. The two levee units addressed by this public notice include the Armourdale and Central Industrial District (CID) levee units. The Armourdale levee unit is located within Wyandotte County, Kansas. The CID levee unit is located primarily within Wyandotte County Kansas (1.5 miles), with a relatively small portion of the levee (about 0.1 miles) located within Jackson County, Missouri.

The Missouri Department of Conservation (Department) is the agency responsible for forest, fish, and wildlife resources in Missouri. As such, we actively participate in project review when projects might affect those resources. Our comments and recommendations are for your consideration and are offered to reduce impacts to the forest, fish, and wildlife resources in the project area.

ACTIVITY: The purpose of the overall study of the seven levee units was to determine whether one or more plans for improvements to the existing levee system to reduce flood risk and improve levee reliability is technically viable, economically feasible, and environmentally acceptable, or if no action is warranted. Failure of any part of the existing flood risk management system during a major flood would have significant adverse impacts on the human environment including property damage and potential loss of human life.

The recommendations for the reliability improvements in the Armourdale and CID units are addressed and available for review in the Draft Final Feasibility Report (DFFR). The DFFR presents the completed feasibility analysis of alternatives for the Armourdale and CID levee units. Proposed alternatives identified to improve flood risk management system reliability include, but are not limited to earthen levee raise, pump station modifications, floodwall

COMMISSION

DON C. BEDELL, Sikeston | JAMES T. BLAIR, IV, St. Louis | MARILYNN J. BRADFORD, Jefferson City | TIM E. DOLLAR, Blue Springs

replacement, stoplog and sandbag closures, pressure relief wells, floodwall modification using buttresses, and the no action alternative. DFFR analysis concluded that the nominal 500-year+3 ft projected water surface elevation levee raise and underseepage improvements is the preferred alternative for both the Armourdale and CID units. The DFFR identifies a combination of measures as the U.S. Army Corps of Engineers (USACE) overall Recommended Plan for these two levee units and presents an analysis of the costs and impacts associated with the alternatives listed above.

The Department is concerned with potential impacts of the preferred alternative to downstream interests. The Department realizes that this is an industrial area and a major flood could have significant adverse impacts, however, raising the levee an additional 3 feet in this area could increase flood issues downstream. The Department would like USACE to ensure that downstream interests have been adequately considered in the analysis.

Thank you for the opportunity to provide comments. If you have any questions about these comments, please contact me at (573) 522-4115, Extension 3346 or by email at alan.leary@mdc.mo.gov

Sincerely,

Alan W. Leary

ALAN W. LEARY
POLICY COORDINATOR

AWL/pb

cc: Jennifer Campbell-Allison, Pam Lanigan, Vince Travnichek

From: Skinker, Richard A NWK
To: Lynn, Eric S NWK
Subject: FW: [EXTERNAL] Re: Armourdale and CID Flood Risk Management (UNCLASSIFIED)
Date: Tuesday, December 24, 2013 12:21:49 PM

Classification: UNCLASSIFIED
Caveats: NONE

FYI

-----Original Message-----
From: Blackford, Susan [mailto:susan_blackford@fws.gov]
Sent: Tuesday, December 24, 2013 11:45 AM
To: Skinker, Richard A NWK
Subject: [EXTERNAL] Re: Armourdale and CID Flood Risk Management (UNCLASSIFIED)

Hi Richard,

We didn't submit any comments. One thing I noticed but didn't seem big enough to submit a comment was that the Draft FR said that FWS had provided a Draft Coordination Act Report. We have provided a Final CAR.

Have a Merry Christmas and Best Wishes for the New Year.

Susan

On Tue, Dec 24, 2013 at 11:02 AM, Skinker, Richard A NWK <Richard.A.Skinker@usace.army.mil> wrote:

Classification: UNCLASSIFIED
Caveats: NONE

Hi Susan,

Please let me know if you have any questions or comments regarding the final feasibility report for these levee units that was previously on public notice.

Thanks,
Rich

Richard A. Skinker
Project Manager, PM-PF
Kansas City District
U.S. Army Corps of Engineers
816-389-3134

Classification: UNCLASSIFIED
Caveats: NONE

--

Susan Blackford
Fish and Wildlife Biologist
U.S. Fish and Wildlife Service
2609 Anderson Ave.
Manhattan, KS 66502
785-539-3474 ext. 102
Susan_Blackford@fws.gov

Classification: UNCLASSIFIED
Caveats: NONE

STATE OF MISSOURI
Jeremiah W. (Jay) Nixon, Governor • Sara Parker Pauley, Director
DEPARTMENT OF NATURAL RESOURCES
www.dnr.mo.gov

DEC 27 2013

Corps of Engineers, Kansas City District
Kansas City Levees Project Manager
Room 529, PM-PF
601 E. 12th St.
Kansas City, MO 64106

Re: Draft Final Feasibility Report - Flood Risk Management Study for Kansas City, Missouri and Kansas City, Kansas

Dear Kansas City Levees Project Manager:

The Missouri Department of Natural Resources (department) appreciates the opportunity to review the information submitted and provide comments for the Draft Final Feasibility Report - Flood Risk Management Study for Kansas City, Missouri and Kansas City, Kansas. The department offers the following comments for consideration.

The Final Feasibility Report describes two of the seven units in the levee system that provides flood protection for metropolitan Kansas City, Missouri and Kansas City, Kansas along the Kansas and Missouri rivers. The two units are the Armourdale unit along the left bank of the Kansas River and the Central Industrial District along the right bank of the Missouri River in the state of Missouri. The other five units were evaluated in the Interim Feasibility Report published in 2006.

Consistent with the Interim Feasibility Report, the Final Feasibility Report recommends the seven unit flood protection system be modified to provide protection from the 0.2 percent chance (500-year) exceedance peak flow recurrence interval. The Final Report also indicates "none of the Missouri River Levee Units in this study physically overtop for the 0.2% (500-year), 0.133% (750-year), or even the 0.1% (1,000-year) chance of exceedance flood event."

Staff reviewers did not find much detail about the hydraulic analysis of the two units - Armourdale and Central Industrial District. More detail was available in the Hydrology/Hydraulics Appendix of the 2006 Interim Feasibility Study, which did not include these latter two units. More detailed documentation about the these two units would be needed before the department could provide comments on the Draft Final Feasibility Report - Flood Risk Management Study for Kansas City, Missouri and Kansas City, Kansas.

Recycled Paper

We appreciate the opportunity to provide comments for the Draft Final Feasibility Report - Flood Risk Management Study for Kansas City, Missouri and Kansas City, Kansas. If you have any questions or need clarification, please contact me, phone number (573) 751-3195. The address for correspondence is Department of Natural Resources, P.O. Box 176, Jefferson City, MO 65102. Thank you.

Sincerely,

DEPARTMENT OF NATURAL RESOURCES

Robert Stout
Chief of Policy

RS/jbk

From: Skinker, Richard A NWK
To: Lynn, Eric S NWK
Subject: FW: RE: Armourdale and CID Flood Risk Management (UNCLASSIFIED)
Date: Tuesday, December 31, 2013 2:36:10 PM

Classification: UNCLASSIFIED
Caveats: NONE

FYI

-----Original Message-----
From: Shepard, Larry [mailto:Shepard.Larry@epa.gov]
Sent: Tuesday, December 31, 2013 2:35 PM
To: Skinker, Richard A NWK
Cc: Robichaud, Jeffery; Horchem, Brad
Subject: [EXTERNAL] RE: Armourdale and CID Flood Risk Management

Thank you for the opportunity to review the draft Final Feasibility Report for the Kansas Citys, Missouri and Kansas, Flood Risk Management Study. The Final Feasibility Study was prepared by the Corps as a complement to the Interim Feasibility Report (2006) which evaluates alternatives for improving the level of flood risk management for portions of the existing Kansas City metropolitan levee system. This system includes seven levee units that provide flood risk management benefits to the Cities of Kansas City, North Kansas City, Birmingham, Jackson and Clay Counties in Missouri and Wyandotte County in Kansas. This Final Feasibility Report addresses the Armourdale and Central Industrial District Levee Units. The 2006 Interim Feasibility Report included recommendations for the Argentine, North Kansas City, East Bottoms, Birmingham and Fairfax-Jersey Creek levee units. Although hydrology and hydraulics modeling and analysis was complete for the entire seven-levee system, structural and geotechnical analysis and calculations were not complete for the Armourdale and CID units. This Final Feasibility Report documents the existing conditions, evaluation of alternatives and flood risk management improvement recommendations for these last two levee units. The Corps intends the Interim and Final Reports to be complementary efforts that view the Kansas Citys project as one complete system. Recommendations within the Interim Feasibility Report for five of the seven levee units were subsequently authorized by the Water Resources Development Act of 2007 and are being implemented. Recommendations within the Final Feasibility Report will be authorized and implemented separately following approval of this Report.

Please consider the following comments on the Final Feasibility Report.

II. Existing Project Conditions and Flood History

A. Existing Project Conditions, B. Construction History and Design Discharge, D. Authorized Project Design Hydraulics and H. Final Assessments of Existing Levee Integrity

The document would be improved with a characterization of the relationship between unit structures' "design discharges" and flood "design frequency" as they affect structure height, i.e., design discharge of 390,000 cfs and levee flood profile at flood frequency of 0.2%.

F. Overview of Existing Environmental and Cultural Resources

5.0 Hazardous, Toxic and Radioactive (HTRW) Sites

This section would be improved with more detail summarizing here what is provided in the 2006 EIS and the Final Feasibility Report appendices for the Armourdale and CID units. Appendix D is thorough, but the inclusion of a summary of this information in the body of the Report would improve its readability.

III. Future Without Project Condition

B. Hydrologic and Hydraulic Considerations of the Without Condition Scenario

2.0 Expected Future Condition Changes

This section does not address potential changes in precipitation patterns or intensity resulting from projected changes in regional climate. Recognizing that the basis for future hydrological predictions is the 2003 Upper Mississippi River System Flow Frequency Study, this report should at least address the possibility that climate changes are predicted and characterize the degree to which such changes would or would not affect the performance of the planned changes to the levee system, i.e., some form of sensitivity analysis.

IV. Plan Formulation

C. Step 1 – Identifying Problems and Opportunities

3.0 Planning Constraints

b. Systems Approach to Plan Formulation

The third paragraph on page 33 summarizes the measures recommended for the 3 Missouri River units and the Argentine unit of the three Kansas units. The Birmingham unit was addressed earlier as not requiring any modifications. Although I might have missed it, this document would be improved if there were a table which summarized the recommended actions for the five units addressed in the Interim Feasibility Report.

D. Step 3 – Formulating Alternative Plans

1.0 No Federal Action

The last paragraph on page 35 describes, abstractly, the threat to the environment from the potential release of contaminants into the river associated with levee failure. This section would be improved if it referenced chemicals, chemical classes or types of contaminants found within the areas protected by the two levee units. I would not expect a complete inventory of materials or contaminants, but it would be appropriate for the public to know the kinds of contamination expected, how these contaminants

'behave' in the floodplain and river environment once flushed from behind the levees and the nature of the threat to the human and natural environment. A general characterization, rather than repeating what was included in the EIS, would suffice.

4.0 Combining Management Measures into Alternative Plans

I recognize the limitations to using non-structural measures as part of alternatives design, particularly within these two levee units. However, the document would be strengthened if the rationale provided in this section, which supported not utilizing such measures, was more detailed and specific to the type and density of structures within these two units. Flood-proofing and structure removal are viable measures even within industrial and residential areas under certain circumstances. The current text suggests to the reader and the public that these non-structural measures are not suitable components of flood risk management in any area with "dense urban development" without qualification. Although these measures are not likely to be suitable in these specific areas, the Corps should be cautious about rejecting these measures without qualification or detailed explanation in specific instances.

E. Step 4 – Evaluating Alternative Plans

1.0 Unit-Raise Structural Alternatives

b. Armourdale Unit

Since the measures evaluated for this unit are for specific reaches of this unit, the section would be improved if it included a figure with a map of the discreet stations. Additionally, a figure identifying those reaches with HTRW concerns, rail interaction and the location of the groundwater contamination between stations 45+00 and 75+00 is essential to understanding the evaluation of structural measures in alternatives design.

3.0 Non-Structural Alternatives

c. Non-Structural Summary

See the above comments. Further, the explanation of this measure is vague and obtuse. In addition, the document states that these measures were eliminated from consideration "as potential stand-alone solutions." In my reading of alternative design, no structural measures were required to serve as "stand-alone solutions." If there are instances or locations where non-structural measures could be implemented in concert with structural measures to create the same protections as structural measures alone, these should receive appropriate consideration.

F. Step 5 - Comparing Alternative Plans ("Step 5" left out of header)

1.0 Central Industrial District

Again, a figure mapping the locations of the tieback configurations and the railroad tracks would

improve the readability of the text.

2.0 Armourdale Unit

No HTRW sites were identified in the selection of alternatives for specific reaches within this unit with multiple alternatives. Does that mean that there were no HTRW-driven limitations to alternatives selection?

G. Step 6 – Selecting a Plan

Again, references to HTRW-affected reaches and HTRW material disturbance as it might affect the evaluation of alternatives were not supported with any detail or meaningful description. If this component of site limitation was not germane to the evaluation, it should be so noted and left out of the process description.

V. Description of the Recommended Plan

I. Environmental and Cultural Considerations

5.0 Wetlands

The statement in this section that "Wetlands within the proposed project are limited in number, size and quality" conflicts with the characterization in II.F., 2.0 which states that there are "no wetlands within the Armourdale or CID Units." This should be corrected and, if there are any wetlands, a map showing their location should be provided.

9.0 Contaminated Areas

I understand that a detailed assessment of this component of 'existing environment' exists within the 2006 FEIS and the HTRW appendix of this document; however, a summary of that information within the body of this document is appropriate. The two paragraphs within this section addressing each unit is inadequate and obtuse, e.g., "Potential HTRW concerns have been identified within the study area…". These statements have no meaning and do not inform. Again, see my earlier comments on this topic. Particularly illustrative of this issue is the reference to the presence of a Superfund site within the Armourdale Unit without further description of site name and nature of contamination. Further, Table 26 includes unit reaches which were not listed as requiring further evaluation for multiple alternatives in Table 16. This aspect of 'existing condition' and alternatives analysis within the main body of this document is not adequately depicted, organized or explained.

If you have any questions regarding these comments, please contact me.

Sincerely,

Larry Shepard

NEPA Team

U.S. Environmental Protection Agency

Region 7

11201 Renner Blvd.

Lenexa, Kansas 66219

913-551-7441

shepard.larry@epa.gov

Classification: UNCLASSIFIED
Caveats: NONE

Exhibit 5 - Comment/Response Summary

Agency	Nature of Contact/Date	Comment Summary
Kansas State Historic Preservation Office	Submitted via letter dated 12/6/2013	No significant changes since previous review in 2006. No objections to the proposed project.
Response Summary		
Comment noted.		

Agency	Nature of Contact/Date	Comment Summary
Missouri Department of Conservation	Submitted via letter dated 12/20/13	Lack of hydraulic information contained in report.
Response Summary		
Hydrology and hydraulic information and evaluations for all units in the Kansas Citys levee system is documented in the Interim Feasibility Report, Engineering Appendix, Chapter A-2, published in 2006. Specifically, water surface profiles for the 0.2% chance flood versus top of levee at existing conditions for the CID and Armourdale Units are shown in Exhibits A-2.18 and A-2.19 of that Appendix. Water surface profiles for the 0.2% chance flood versus top of levee under the Future Without Project condition in the CID and Armourdale Units are shown in Exhibits A-2.96 and A-2.97. The H&H Chapter of the Interim Report Engineering Appendix is available for download from the Kansas City District website at: http://www.nwk.usace.army.mil/Missions/CivilWorks/CivilWorksProgramsandProjects/KansasCitys,FloodRiskManagement.aspx		

Agency	Nature of Contact/Date	Comment Summary
U.S. Department of the Interior, Fish and Wildlife Service	Submitted via e-mail 12/24/2013	No comments on the project. Noted that report references Draft Coordination Act Report that is actually a Final Coordination Act Report.
Response Summary		
Reference to Draft Coordination Act Report corrected.		

Agency	Nature of Contact/Date	Comment Summary
Missouri Department of Conservation	Submitted via letter dated 12/20/2013	Concerned with potential increase in downstream flood impacts due to levee raise.

Response Summary

At their existing heights the Armourdale and CID Kansas Levee units are currently able to pass the 0.5% (200 year) event. As noted in the Phase 2 report there is a proposed raise for the Armourdale and CID Levee units to an elevation equivalent to the 0.2% (500 year) event plus 3 feet (500 year +3).

Under existing conditions the Armourdale and CID-Kansas levee units would be overtopped by an average of 0.6 feet and 1.1 feet respectively during the 500 year event (341,000 cfs) primarily along the upstream segments of each levee. However, levee weir flow calculations show that the CID-Kansas levee unit would have a maximum overflow into the protected area of 3,800 cfs, which is only 1.1% of the total 500 year flow. In addition, weir flow calculations for the Armourdale levee unit show that the protected area would be inundated and filled prior to the 500 year peak overtopping elevation of 1.1 feet. As such any water overtopping the upstream portion of the levee would be returned to the Kansas River on the downstream end of the levee as the maximum flow and depth is approached. This would cancel out any flow reduction due to overtopping at the upstream end of the levee.

Based on this evaluation, raising the Armourdale and CID-Kansas Levee units would cause approximately 3,800 cfs in additional flow at the Kansas/Missouri River confluence for the 500 year event. This equates to a 0.7% increase in peak flow and a 1.3 inch increase in water surface elevation immediately downstream of the Kansas River / Missouri River confluence. These increases are considered to be negligible. The overtopping weir flow calculation depths and durations were based on the rate of rise observed during the 1993 flood event (1.7 feet/day)."

Agency	Nature of Contact/Date	Commeut Summary
U.S. Environmental Protection Agency	Submitted vie e-mail dated 12/31/13	1. Draft Report Section II.A. The document would be improved with a characterization of the relationship between unit structures' "design discharges" and flood "design frequency" as they affect structure height, i.e., design discharge of 390,000 cfs and levee flood profile at flood frequency of 0.2%

Response Summary

The Discharge-Frequency relationships summarized in Table 3-10 of the Final Feasibility Report indicate that the authorized design discharge of 390,000 cfs has an annual chance exceedance of less than 0.1%. Given the very low chance of occurrence of a flood of this magnitude it was determined to be neither practical nor desired to evaluate existing performance or develop alternatives to modify the existing project for the design discharge. For consistency with the desired benefits and uniformity of risk management within the levee system, evaluations of current and future project performance conducted for this study focused on the 0.2% chance flood. This response has been added to the report text after Table 3-10.

Comment Summary
2. Draft Report Section II.F.5.0. This section would be improved with more detail summarizing here what is provided in the 2006 EIS and the Final Feasibility Report appendices for the Armourdale and CID Units. Appendix D is thorough but the inclusion of a summary of this information in the body of the Report would improve its readability.
Response Summary
A summary of the HTRW findings from Appendix D can be found in the Final Feasibility Report Section 5.1.3.

Comment Summary
3. Draft Report Section III.B.2.0. This section does not address potential changes in precipitation patterns or intensity resulting from projected changes in regional climate change. Recognizing that the basis for future hydrological predictions is the 2003 Upper Mississippi River System Flow Frequency Study, this report should at least address the possibility that climate changes are predicted and characterize the degree to which such changes would or would not affect the performance of the planned changes to the levee system, i.e., some form of sensitivity analysis.
Response Summary
USACE guidance on climate change adaptation inputs for inland hydrology is at the draft final stage of production, and has not yet been officially released for use. As such, there was no guidance in place when the hydrologic analysis was conducted (finalized 2006) for the Kansas City Levees Feasibility Study. The proposed USACE guidance will initially recommend a qualitative approach. A summary of the qualitative approach as would be applied to the Kansas City Levees is provided below. The climate of northeast Kansas trends toward a continental weather pattern of cold winters and hot, humid summers. The average temperature in 2013 at Topeka, KS (which represents the northeast portion of Kansas) was 60 degrees. The average high temperature was 73 and average low temperature was 47. The average yearly precipitation was about 37 inches of moisture. A model of future conditions for the central plains of the United States was created by the NOAA National Environmental Satellite, Data and Information Service in a report issued in January 2013. This report is an assessment of Climate Trends and Scenarios into the next 50 to 100 years. The report cites that over the past period of record for the Kansas River basin, both temperature and precipitation has trended above normal, especially over the last 50 years. To account for climate change in the meteorological conditions, the future forecast of conditions in the region takes into consideration the past temperature and precipitation records, and then considers future modeled conditions in the area through 2070. According to the NESDIS report, a warming trend of about 3-5 degrees F and a precipitation trend very slightly toward wetter conditions can be expected through the next 50 years although significant uncertainty is expected with these estimates. Based on this slight trend toward wetter conditions frequency flows over the study basin may increase, but these increases are being treated in this evaluation to be retained within the bands of uncertainty in the Existing Condition Feasibility hydrologic analysis.

Comment Summary
4. Draft Report Section IV.C.3.0.b. The third paragraph on page 33 summarizes the measures recommended for the 3 Missouri River units and the Argentine unit of the three Kansas units. The Birmingham unit was addressed earlier as not requiring any modifications. Although I might have missed it, this document would be improved if there were a table which summarized the recommended actions for the five units addressed in the Interim Feasibility Report.
Response Summary
The recommendations of the Interim Feasibility Report are discussed in Section 7.0 of the Final Feasibility Report.

Comment Summary
5. Draft Report Section IV.D.1.0. The last paragraph on page 35 describes, abstractly, the threat to the environment from the potential release of contaminants into the river associated with levee failure. This section would be improved if it referenced chemicals, chemical classes or types of contaminants found within the areas protected by the two levee units. I would not expect a complete inventory of materials or contaminants, but it would be appropriate for the public to know the kinds of contamination expected, how these contaminants 'behave' in the floodplain and river environment once flushed from behind the levees and the nature of the threat to the human and natural environment. A general characterization, rather than repeating what was included in the EIS, would suffice.
Response Summary
The discussion of potential environmental impacts of the No Federal Action alternative, especially as relates to chemical releases, has been revised and expanded and is found in Section 4.3.1 of the Final Feasibility Report.

Comment Summary
6. Draft Report Section IV.D.4.0. I recognize the limitations to using non-structural measures as part of alternatives design, particularly within these two levee units. However, the document would be strengthened if the rationale provided in this section, which supported not utilizing such measures, was more detailed and specific to the type and density of structures within these two units. Flood-proofing and structure removal are viable measures even within industrial and residential areas under certain circumstances. The current text suggests to the reader and the public that these non-structural measures are not suitable components of flood risk management in any area with "dense urban development" without qualification. Although these measures are not likely to be suitable in these specific areas, the Corps should be cautious about rejecting these measures without qualification or detailed explanation in specific instances.
Response Summary
The discussion of Non-Structural measure evaluations has been expanded to provide additional details of the types of measures, expected performance, and typical costs. This discussion is found in Section 4.3.4.2 of the Final Feasibility Report.

Comment Summary
7. Draft Report Section IV.E.1.0.b. Since the measures evaluated for this unit are for specific reaches of this unit, the section would be improved if it included a figure with a map of the discreet stations. Additionally, a figure identifying those reaches with HTRW concerns, rail interaction and the location of the groundwater contamination between stations 45+00 and 75+00 is essential to understanding the evaluation of structural measures in alternatives design.
Response Summary
Maps are provided following the text of the report that show the locations and stationing of the recommended plan improvements relative to existing infrastructure, including railroads. Notations are provided on the maps indicating known HTRW locations. Since all measures evaluated were applied to the same existing levee locations, separate map sets were not created for each measure or alternative.

Comment Summary
8. Draft Report Section IV.E.3.0.c. See the above comments. Further, the explanation of this measure is vague and obtuse. In addition, the document states that these measures were eliminated from consideration "as potential stand-alone solutions." In my reading of alternative design, no structural measures were required to serve as "stand-alone solutions." If there are instances or locations where non-structural measures could be implemented in concert with structural measures to create the same protections as structural measures alone, these should receive appropriate consideration.
Response Summary
The term "stand-alone solutions" was meant to convey that non-structural alternatives would not provide the desired degree of system-wide flood risk management improvement without the addition of structural project components. The Non-Structural measures discussion has been revised. See response to previous comment on Non-Structural Alternatives.

Comment Summary
9. Draft Report Section IV.F.1.0. Again, a figure mapping the locations of the tieback configurations and the railroad tracks would improve the readability of the text.
Response Summary
Maps are provided following the text of the report that show the locations and stationing of the recommended levee improvements relative to existing infrastructure, including the multiple tieback configurations evaluated in the CID Unit.

Comment Summary
10. Draft Report Section IV.F.2.0. No HTRW sites were identified in the selection of alternatives for specific reaches within this unit with multiple alternatives. Does that mean that there were no HTRW-driven limitations to alternatives selection?
Response Summary
Avoidance of HTRW was one driver for alternative selection in reaches adjacent to known contaminated areas. Depending on the existing features, necessary improvements desired, and other adjacent infrastructure, it was often not the only driver. The reaches described in this section were not adjacent to HTRW areas and the selection of measures was driven by real estate relocation and access concerns as described.

Comment Summary
11. Draft Report Section IV.G. Again, references to HTRW-affected reaches and HTRW material disturbance as it might affect the evaluation of alternatives were not supported with any detail or meaningful description. If this component of site limitation was not germane to the evaluation, it should be so noted and left out of the process description.
Response Summary
Again, the primary use of HTRW in the evaluation of alternatives was only for identification and avoidance of measures that would require construction activity in those areas. The discussion in this section presented a summary of the concerns that drove alternative selection throughout the two units. This discussion is found in Section 4.6.3 of the Final Feasibility Report and has been edited to reflect the emphasis on avoidance of HTRW disturbances.

Comment Summary
12. Draft Report Section V.I.5.0. The statement in this section that "Wetlands within the proposed project are limited in number, size and quality" conflicts with the characterization in II.F., 2.0 which states that there are "no wetlands within the Armourdale or CID Units." This should be corrected and, if there are any wetlands, a map showing their location should be provided.
Response Summary
The Final Feasibility Report has been edited to eliminate the inconsistencies of the wetlands discussion. There are NWI mapped wetland areas in the Armourdale and CID units as discussed in Section 5.1.4 of the Final Feasibility Report. Wetland impacts were discussed in the 2006 EIS. There are no anticipated wetland impacts associated with proposed work within the Armourdale and CID units.

Comment Summary
13. Draft Report Section V.I.9. I understand that a detailed assessment of this component of 'existing environment' exists within the 2006 FEIS and the HTRW appendix of this document; however, a summary of that information within the body of this document is appropriate. The two paragraphs within this section addressing each unit is inadequate and obtuse, e.g., "Potential HTRW concerns have been identified within the study area…". These statements have no meaning and do not inform. Again, see my earlier comments on this topic. Particularly illustrative of this issue is the reference to the presence of a Superfund site within the Armourdale Unit without further description of site name and nature of contamination. Further, Table 26 includes unit reaches which were not listed as requiring further evaluation for multiple alternatives in Table 16. This aspect of 'existing condition' and alternatives analysis within the main body of this document is not adequately depicted, organized or explained.
Response Summary
A summary of the review of the 2006 EIS is included in Section 5.0 of the Final Feasibility Report, including a summary of the HTRW assessment from appendix D in Section 5.1.3. Tables 16 and 26 contain information intended for different purposes and decisions and are not meant to be compared. Table 16 lists those reaches of the Armourdale Unit where more than one technically feasible alternative was identified to implement the desired levee raise. As stated in a previous comment response, these reaches are not adjacent to HTRW sites and the selection of preferred measures was driven by other real estate concerns. Table 26 lists those reaches within the Armourdale that are adjacent to HTRW areas and the potential for impacts from the recommended plan. These tables are presented in the Final Feasibility Report as Tables 4-3 and 5-1, respectively.