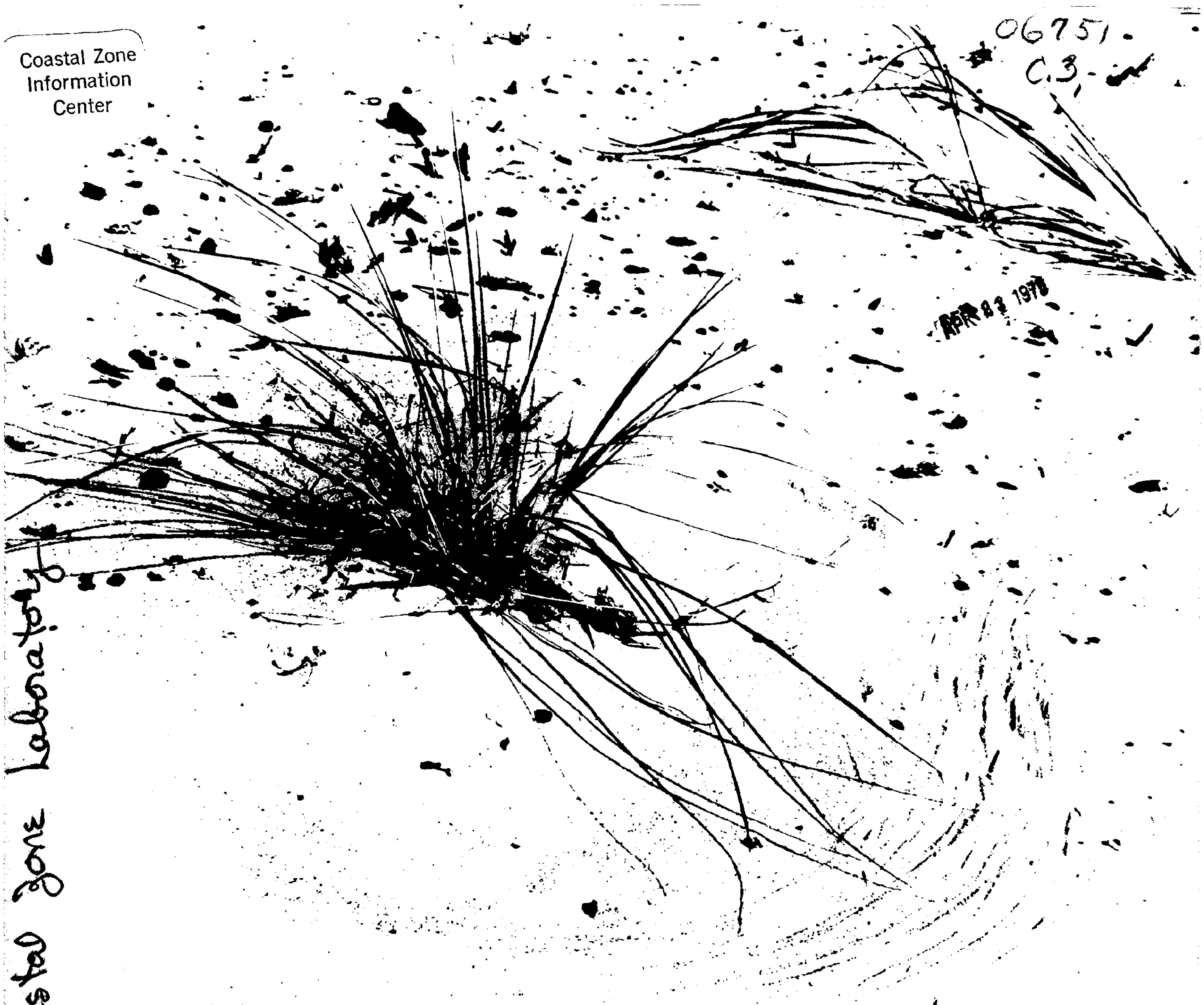

michigan's demonstration erosion control program

update evaluation report - august, 1975

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background & summary

The Michigan Department of Natural Resources, Bureau of Water Management was directed by Act 14 of the Public Acts of 1973 to take "action to avert catastrophic consequences" of severe shoreland erosion. The Demonstration Erosion Control Program was formulated and implemented under this directive.

The Department of Natural Resources contracted the University of Michigan's Coastal Zone Laboratory to be an integral part of the program, since Governor Milliken had charged it to act as the State's Coastal Zone Laboratory on January 12, 1973. This evaluation is included as a part of the Demonstration Erosion Control Program.

This research program focused on the selection, design, installation and evaluation of various demonstration projects around the state. The sites were selected on the basis of geographic distribution, and because they had experienced severe erosion problems. The series of installations were to demonstrate both innovative and conventional means of protection. In determining the overall effectiveness of each project, documentation and evaluation of factors such as the reduction of erosion rates, cost, construction difficulties and durability were taken into account.

In February 1974, a report entitled "Shore Erosion Engineering Demonstration Project Post-Construction-Season Progress: Interim Report" was issued, which described the selected test sites and the devices to be tested at these locations. "Michigan's Demonstration Erosion Control Program Evaluation Report," issued in November 1974, summarized the activities and results of the first year of study, representing completion of the first phase of this study as financed by Public Act 14 (1973). The Michigan Sea Grant Program has financed the continued study and monitoring of these project sites since the fall of 1974, with additional research support now being provided by the Coastal Engineering Research Center, Corps of Engineers, Department of the Army. This report is intended as an update and supplement to the November 1974 report.

Some of the impact of the first year of study was lost since an unusually quiet storm year was experienced. This meant that most project sites were not tested by storms of a magnitude common to the Great Lakes, thus making it difficult to draw fair conclusions about a device's general effectiveness. During the second year of study a more typical storm year was experienced, therefore, this report will contain a more complete analysis of structural performance. Table #1 (see Appendix) summarizes the major* storms experienced for the period of study for the project sites.

This report is written to be complete in itself, although much of the background information to be found in the November 1974 report has been omitted, thus, reading the two reports in conjunction with one another is advisable. Each site will be analyzed in a format containing the following elements: 1) a description of the condition of the structure, including one or two pictures taken in the late fall of 1974 and/or the spring of 1975, and engineering drawings of the site, 2) an analysis of the structure's failure, if such a failure has taken place, 3) an explanation of the amount and type of maintenance required for each structure, 4) a description of design modifications which might be required to increase the effectiveness of devices, and 5) general observations and conclusions about the structures.

* See report of November 1974 for definition of major storm.

Summary

Evaluation. An evaluation of these projects and an attempt to find basic principles from them must be undertaken with a full realization of the following three factors:

1. All of the installations were intended to be in the "low-cost" category. The costs varied from about $50 to $100 per foot. This constraint limited the design severely. For example, a design which would be expected to provide protection for all but very rare storms, such as 25 or 50 year frequency storms, might cost from $200 to $400 per foot. The designs used in the demonstration projects can be expected to suffer damage from much less severe storms, such as 5 or 10 year frequency storms.

2. It is too soon to judge the effectiveness and durability of the projects. Some of the projects might not be tested for 5 or 10 years, while, on the other hand, they might be damaged by a storm within a few months.

3. An important aspect of this program was to try new "low-cost" ideas in construction which showed promise and/or received considerable public interest. Such innovative projects are probably more likely to produce failures than would be expected from more tested procedures.

Some General Conclusions. In fully exposed areas appropriate "low-cost" shore protection will prevent bluff erosion during small storms and will reduce the rate of erosion during larger storms. During very large storms the protection may be ineffective or even destroyed, yet a well designed "low-cost procedure might prolong the life of the property.

The following comments are based on the experience gained from the demonstration projects, plus observations on many other private and public projects. They will be presented according to categories of protective procedures.

Artificial Beach Nourishment. This is the best method of protecting a shoreline from the point of view of retaining the recreational use of the beach. It is most effective when the sand is placed between groins, as at Tawas City. Without groins, the sand may be rapidly lost, especially if applied to only a short section of beach.

Groin Systems. If there is sufficient littoral drift, groin systems will capture sand and create a protective beach. This is well illustrated by the projects at Lincoln and Sanilac Townships. If there is little or no littoral drift, as at Mears and Ludington State Parks, the groins must have artificial nourishment. In such cases, nourished groin systems provide recreation and protection, but in non-recreational areas, a revetment might be more appropriate.

Well constructed wooden groins, such as the one at Lincoln Township, have proven to be very durable many times. The mastic covered stone groin at Sanilac has shown little sign of deterioration thus far. The gabion, sand bag and sand filled tube groins used in the Sanilac project have tended to be undermined and to sink at their outer ends. There has also been considerable loss of sand bags due to natural causes or vandalism. However, their defects have not generally been so severe as to preclude these types of construction if they have an appreciable cost advantage at any location. A gabion groin at Big Sable Point was poorly constructed and failed quite rapidly.

Revetments. When a shoreline is not being used for recreation, a revetment is often the most effective and economical protection. If constructed with a rubble foundation, and built to an elevation high enough to prevent overtopping, a properly desigmed armor stone revetment can provide complete protection. The disadvantage is the high cost. The mastic covered stone revetment at Michiana and the three sand filled tubes at Moran Township were attempts to try lower cost innovative methods. So far the tubes have held up, but the mastic revetment was greatly damaged and lost much of its effectiveness during a severe storm. Both of these types of construction have a disadvantage compared to rubble in that their smooth surfaces facilitate a higher run up as well as a more damaging rundown. This rundown causes erosion and undermining of the toe area.

The rock revetment at Tawas Lighthouse has been effective so far, but has not been tested by a severe storm. The rock size used at that location is too small to resist displacement under severe conditions, but it will be of great interest to determine how much maintenance will be required after a large storm. The rock revetments at Whitefish Township and in the Keweenaw Peninsula on Lake Superior, a less exposed area, have functioned very well to date. The 'Nami Ring" revetment at Little Girls Point has suffered some rearrangement of the rings due to sliding of the clay bluff. However, it has not yet been tested by large storm waves.

Seawalls. Since vertical seawalls accelerate erosion and are very expensive if properly built, none were installed in this project. The three sand filled tubes described under "Revetments" could be considered a seawall. However, the single, small sand filled tube at Empire proved inadequate shortly after installation.

Offshore Breakwaters. Conventional offshore breakwaters not only provide protection, but also reduce turbulence, permitting littoral drift to settle out and form a beach. Such construction would usually be far too expensive to be in the "low-cost" category. However, a pre-cast concrete type of wall was available, which when placed over only 50 per cent of the reach of shoreline, was in the acceptable cost range. Because of the interest in offshore breakwaters, this type of shore protection was installed in an exposed location at Buttersville, Michigan. This structure functioned well for over a year until a large storm not only overtopped the structure, but caused several units to settle. Due to the ineffectiveness of the structure during the storm, rapid bluff recession occurred. A laboratory investigation showed that a wind setup of one foot was sufficient to greatly reduce the effectiveness of the breakwater. It appears that offshore breakwaters have no place in the "low-cost" category.

revetments & seawalls

Michiana

Background. This site is exposed to long fetches to the West and Northwest, and had been suffering severe erosion for many years. The bluff recession was threatening a road and associated utility lines. It was decided to try a revetment consisting of an asphalt mastic placed on relatively small rock. The cost of this construction was in the "low-cost" range because the asphalt allowed the use of smaller, less costly rock. The revetment was constructed in the fall of 1973, and functioned very well during its first year of operation.

Condition. Diagram #1 shows the condition of the structure at this site on December 10, 1974. The first sign of problems with the structure appeared in the fall of 1974 when undermining of the toe of the revetment occurred in a few small areas. At that time it was also evident that surface runoff from the roads was concentrating at one or two locations, causing local undermining. A survey made after a fairly heavy November storm showed that the revetment had settled in some places. Photo #1 illustrates the condition of the revetment after the November storm. After the snow and ice disappeared in the spring of 1975, however, the revetment was still in quite good condition. Settlement had lowered the top edge about a foot in some places.

In April 1975, a storm occurred which produced breaking waves about 7 feet high, for a duration of approximately 24 hours. Photo #2 shows the site on April 5, 1975, 2 days after the storm. This storm undermined about 4/5 of the revetment in a manner sufficient to change the slope of the structure from a ratio of 2 to 1 to one of 4 to 1. Thus, the back side was about 3 feet lower after the storm, although the revetment was still mostly unbroken except in local areas where surface runoff had occurred. It is believed that the undermining was due partly to scour at the toe, and partly to overtopping. The wave heights were a foot higher than the design wave.

Analysis. It is difficult to place a frequency on this storm. While it was not a "super" storm as far as actual wave heights are concerned, it was devastating because of its exceptionally long duration. An indication of its strength is given by the fact that in the adjacent section of shoreline a steel wall was destroyed and bluff erosion was so severe that emergency measures were taken to save the road. Since the two adjacent areas are nearly identical in all respects, it is obvious that the mastic revetment was much more effective than the steel wall. Without this revetment, the road might have been damaged. An additional factor which may have had a considerable effect on the area is the erosion of sand in the underwater region immediately off-shore. The water is considerably deeper there than it had been during construction, which means that the run-up near the end of the storm could have exceeded that at the beginning.

There is evidence that the steel wall which projects out from the shore adjacent to the north end of this project may have caused reflections which increased the destructive force of the waves in that area. The 75 feet of revetment immediately adjacent to the steel wall has suffered severe slumping, whereas the next 75 feet is virtually undamaged.

It seems probable that the revetment could have handled lesser storms for many years if this storm had not occurred. Even in its present condition, it is providing some protection to the toe of the bluff.

Photo #1 December 10, 1974
This photograph was taken shortly after the November storm. Evidence of wave overtopping can be seen.

Maintenance. The revetment is beyond repair at this point in time; it would cost as much to repair the structure as it would to rebuild it entirely. After the damage experienced in the fall of 1974, the structure could have been repaired by replacing and regrading lost sand and by placing additional rock and asphalt mastic in those areas where the revetment had begun to collapse.

Design Modifications. A number of changes could be made in the design and construction of this type of revetment to improve its performance. First, the rocks should be larger and more uniform in size so that all rocks are in the 8 to 12 inch range. This would improve the depth of asphalt mastic penetration into the rock. The incorporation of a more massive section at the lower edge might reduce undermining at the toe. A deeper layer of rock with an underlayer of smaller rock (1 to 3 inch size) might provide a path for seepage under the revetment and, thus, reduce undermining due to overtopping. Points of surface runoff drainage and drains should be placed in or beneath the revetment to accommodate storm runoff. If a single layer of rock and asphalt mastic was added to the main portion of the revetment, the increased elevation could eliminate overtopping, but the costs would also be increased.

General Observations. This structure type still holds promise as a low-cost alternative for shore protection. Even with all of the aforementioned modifications, the structure could still be built for under $100 per linear foot of frontage. In spite of structural failure, the revetment did protect the land area behind it, and prevented loss of the road at this locale.

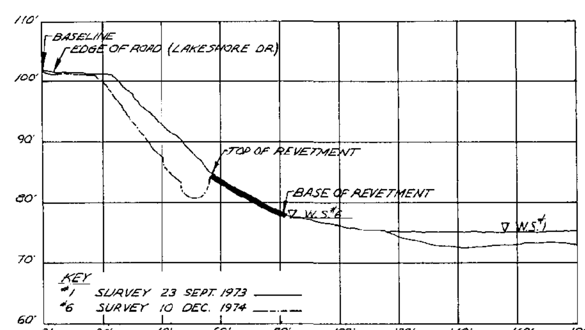


Diagram #1
Typical Cross Section

Photo #2 April 5, 1975
This photograph was taken shortly after the April 3, 1975 storm. The fill sand has been washed from behind the structure and the rock mastic revetment has collasped on the beach.

Big Sable Point

Background. Information about this state-owned site became available near the end of the site selection and construction phase of the project. A 200 foot long steel wall extending 4 feet above the water surface, which had been constructed to protect the lighthouse, was flanked, and the fill behind the wall was being rapidly eroded. The shore on both sides of the lighthouse had receded enough to create a peninsula. The steel wall was not supported by tie-backs and the water had become quite deep in front of the wall. The survival of the wall under such adverse circumstances seems to be attributable to the great span and strength of the steel piling and the curved path upon which the wall was installed.

The problems at the lighthouse created favorable conditions for a demonstration site. However, the constraints were severe because of the limited amount of funding available. The installation of tie-backs to strengthen the wall was decided upon, along with the addition of three groins from the wall to the bluff behind the steel piling. The area between the existing bluff and the wall would then be backfilled. The groins were installed to prevent erosion of the backfill material by water which came over the wall. In order to conserve funds and construction materials, the groins were constructed from gabions which were left over from another project. These were to be made relatively impermeable to sand movement by lining the cages with filter cloth.

Condition. Photo #3 shows the condition of the site with the gabions in place, but prior to the backfilling which occurred in the fall of 1974. Photos #4 and #5 show the condition of the site in the spring of 1975 after the winter's storms.

The gabion return wall in the south has failed. The backfill material has been washed out from behind the wall. The gabion cut-off walls have been flanked, and some further bluff recession appears to have taken place.

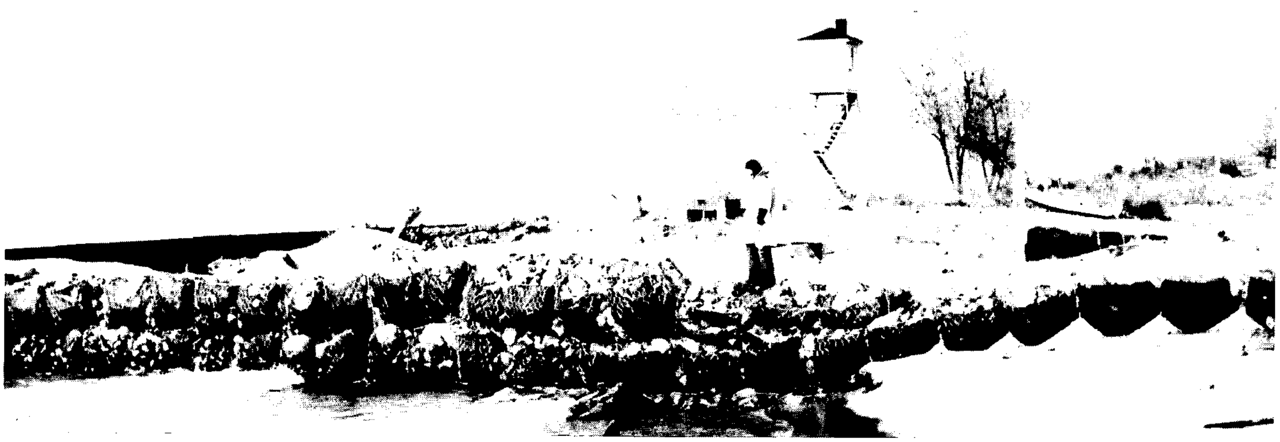


Photo #3 October 19, 1974
The structure prior to completion of the gabion return wall.

Analysis. The south groin was poorly constructed and failed, while the center groin, also poorly constructed, was too short to be functional. As a result, severe overtopping by waves has washed out about half of the backfill. Removal of the backfill also revealed that the tie-backs had not been completed properly. On June 5, 1975 the water in front of the steel wall was 8 feet deep, but the wall had not failed.

Maintenance. A number of repairs must be made at this site to enable these structures to prevent further damage. The center cut-off wall needs to be extended further back into the existing bluff, while the gabion return wall in the south needs to be replaced in its entirety. The tie-backs installed next to the existing seawall need to be secured to pilings, and the backfill material needs to be replaced.

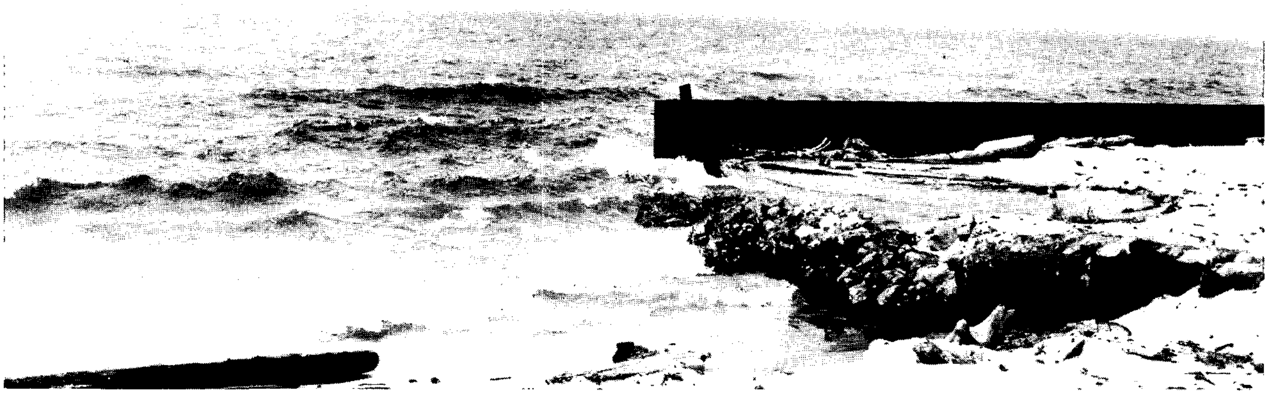


Photo #4 May 19, 1975
Condition of the gabion return wall after several months of service.

Photo #5 May 19, 1975
Damage to gabion return wall by winter storms. Gabion cut off walls were flanked and backfill washed out from behind wall. Improper tie-back usage is evident.

Design Modifications. The ideal type of groin for this site would be one made of wood or steel. If the groins are replaced by gabions, they should probably be placed on gabion mat foundations with additional emphasis on water-proofing of the groins. The violence and volume of the overtopping appear to be so great that the gabion groins may never be able to contain the sand backfill. The only way this overtopping can be reduced is by building a rubble toe onto the outside of the wall. An alternative to replacing the backfill would be to protect the area behind the wall by combining rocks of mixed size with larger stones weighing at least 50 pounds to form a 24 inch buffer zone which would decrease the intensity of wave attack directly on the backfill material.

General Observations. Had this installation been constructed properly, it would have reduced the rate of erosion of the bluff. At the present time, the site is nearly as vulnerable as it was before the project was started. This project has served to demonstrate the tremendous erosive power of water which overtops a seawall. It also shows the importance of finding a reliable contractor who has a good understanding of marine construction, especially in remote areas where constant inspections may be impossible.

Empire

Background. This project was completely described in the original report (November 1974). It demonstrated the inadequacy of a single sand-filled tube placed parallel to the shore on a filter cloth in such an exposed location.

Condition. Diagrams #2 and #3 and Photo #6 are included for reference and also to provide information obtained during the final survey of this site.

Photo #6 October 6, 1974
Only a few remaining fragments of the Longard tube structure can be seen. Bluff recession was 25 feet during the year of study.

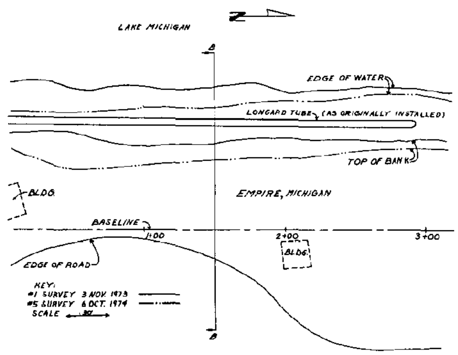


Diagram #2
Plan View of Site

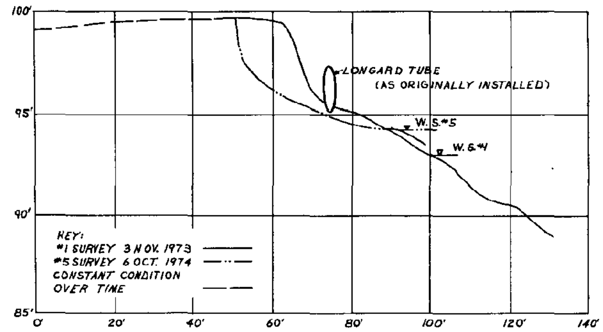


Diagram #3
Profiles at Section B-B

Moran Township

Background. Two structures were built at this site. The first, completed in the fall of 1973, consisted of three 40" diameter Longard tubes stacked in pyramid fashion. Later, in the spring of 1974, sand bags were placed in their final positions, east of the sand-filled tubes, using four different stacking patterns. During the first year of study, both structures settled somewhat, while portions of the bluff continued to recede. This recession was not caused by wave attack, rather, it occurred because the sand was in an unstable condition. It will continue to slide due to wind, rain and frost until a more stable condition is reached. This slumping of the sand pushed the tubes forward, out toward the lake a few feet. In addition, a number of the sand bags were vandalized (cut open), and subsequently, lost.

Condition. Diagrams #4, #5 and #6 and Photos #7 through #10 show the condition of the structures at this site. The rate of settlement in both structures has fallen off to a practically negligible level, and the slumping bank has stopped pushing the tubes forward from behind. Both protective devices, including the various sand bag stacking arrangements, have nearly retained their original effectiveness. Few if any additional sand bags have been vandalized during the second year.

Maintenance. The top Longard tube has been shimmed with wooden wedges to increase its stability. Some additional shims may be required to help hold the top tube in place. A number of missing sand bags should be replaced.

Design Modifications. The bluff should be stabilized before construction, either by cutting it back or by adding fill sand. Such action could not be taken in this case due to financial limitations.

General Observations. Both structures are functioning well. There is a wider expanse of beach in front of the Longard tubes than in front of the sand bags, but this seems to be more of a natural phenomenon than one that can be attributed to the structures.

None of the sand bag stacking patterns at this site is any better than the others. This is partly because a number of bags were lost, making comparison difficult. In general, the bags are performing well.

Photo #7 October 5, 1974
Shims were added to help hold the top tube in place against the back pressure of the slumping bank.

Photo #8 May 18, 1975
Lost shims have not prevented the tubes from remaining relatively stable.

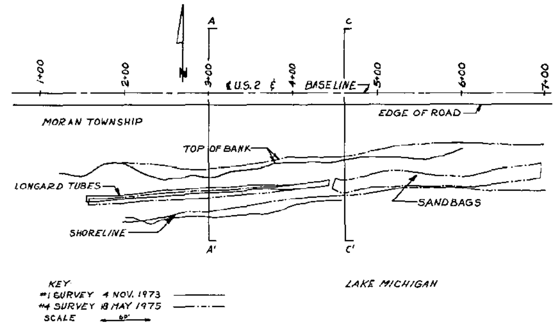


Diagram #4
Plan View of Site

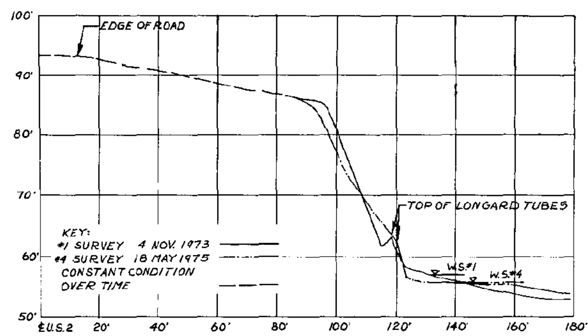


Diagram #5
Profile at Station 3+00 Section A-A

Photo #9 October 5, 1974
Vandalism and damage to sandbags from floating debris is evident here.

Photo #10 May 18, 1975
Winter storms caused some sand bags to shift and others to be lost.

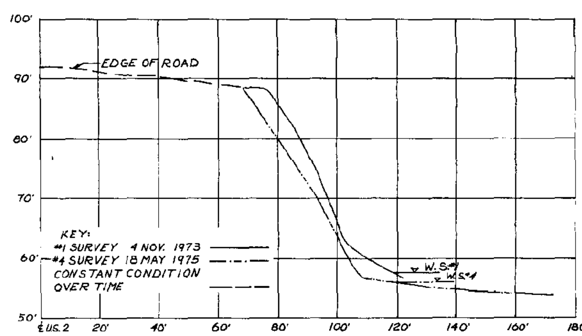


Diagram #6
Profile at Station 4+60 Section C-C'

Tawas Point Coast Guard Station

Background. In the summer of 1974 a rock revetment was built at this site. One half of this layered (graded rock) revetment was covered with armor stone. During the early period of the study this revetment performed well.

Condition. Diagrams #7 and #8 and Photos #11 and #12 show the condition of the site. No further erosion or recession of the bluff has occurred to date. Some of the rock has begun to slump and slide toward the lake, although this movement has been minimal to date. It should be noted that further storm activity may cause additional shifting.

Maintenance. No maintenance or repair work has been required to date.

Design Modifications. This structure has not needed any design changes.

General Observations. This structure has performed well during a period of minimal storm activity, with the rock acting as protection for the low bluff area. The one disadvantage to this type of structure is that use of the beach is impeded for recreational activities, such as swimming. This posed no problem at this site, however, since such activities are not allowed here.

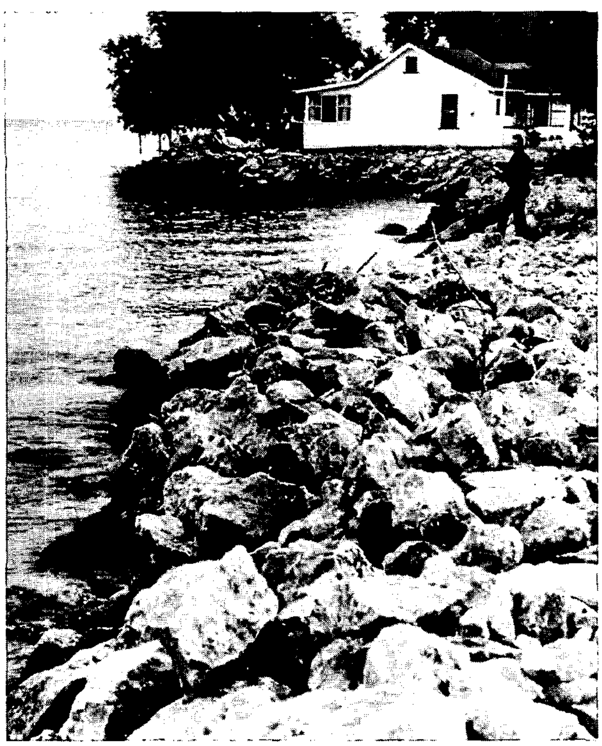


Photo #11 July 30, 1974
This photograph was taken shortly after completion of the project.

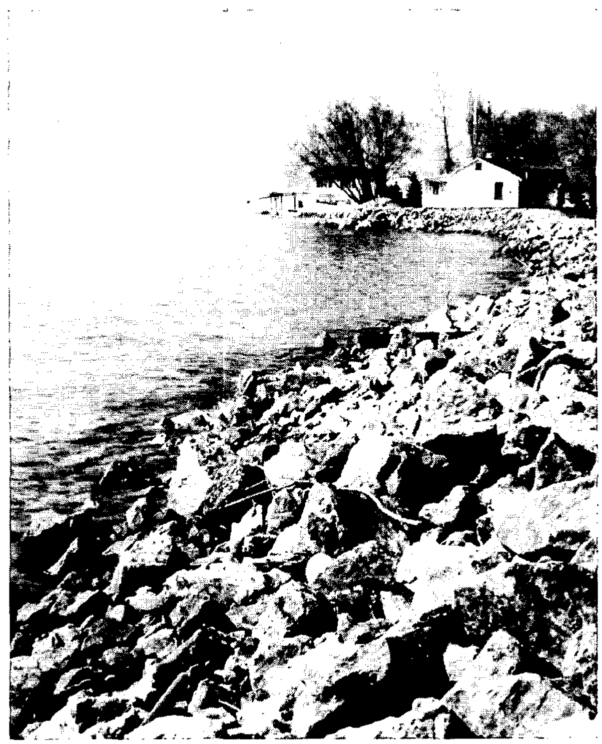


Photo #12 May 17, 1975
After the first storm season, minor rock shifts are noted.

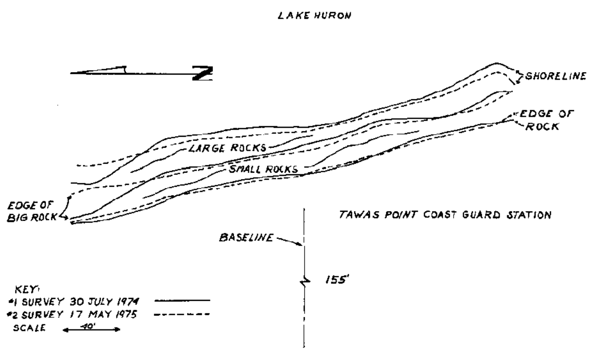


Diagram #7
Plan View of Site

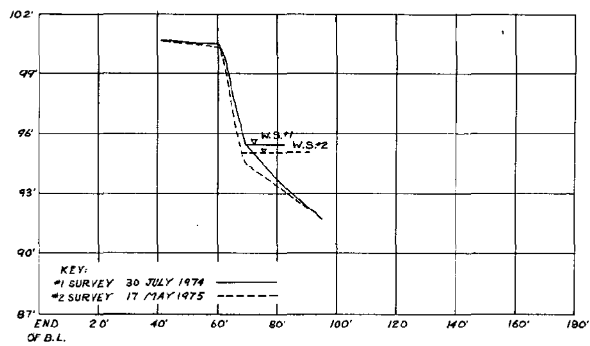


Diagram #8
Typical Cross Section

Sanilac - section 11

Background. Late in the summer of 1974 the Michigan Department of State Highways and Transportation installed two 69" diameter Longard tubes at this site. The installation took place at a roadside park (approximately 3 miles south of Port Sanilac in Sanilac County), where the tubes were secured in a seawall fashion, viz. at the toe of the bluff parallel to the shoreline. The total length of this "seawall" was approximately 400 feet.

Condition. Diagrams #9 and #10 and Photos #13 and #14 show the condition of the site. The field surveys show no substantial movement of the tubes in either a vertical or horizontal direction. Some minor slumping has been detected which can probably be attributed to terrestrial processes. One critical storm has been experienced at this site since installation, and the tubes appear to be performing satisfactorily.

Maintenance. No maintenance has been required to date.

Design Modifications. The need for design modifications is not evident at this time.

General Observations. The Longard tubes have worked well at this location to the present time. They appear stable at this locale which is characterized by a clay bottom, and have not reacted to the back pressure caused by the slumping bank. Only one critical storm has been experienced thus far, and the tubes resisted this attack.

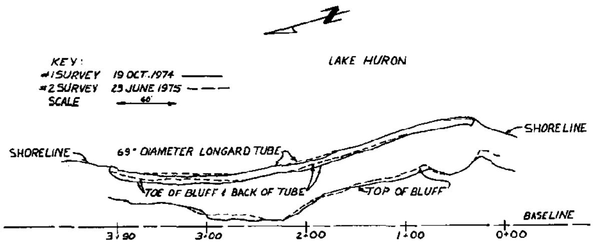


Diagram #9
Plan View of Site

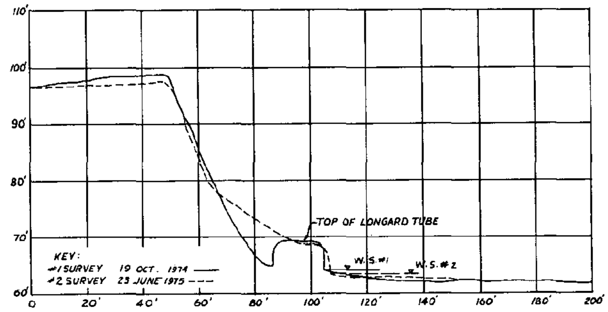


Diagram #10
Profile at Station 3+50

Photo #13 October 20, 1974
The Longard tubes shortly after installation. Note the steep clay bluff.

Photo #14 April 29, 1975
The tubes have remained stable although bluff slumping is evident.

Whitefish Township

Background. This project was located on Whitefish Bay to protect a small state-owned roadside park. The site offered two unusual features: 1) the land was very low and 2) the erosion had already reduced the size of the park to the point of limited usefulness. These factors made the reclamation of some land desirable. It was decided that filling with sand would obtain the necessary land area, and that protection of the sand could be achieved by installing a rock revetment with groins extending from the structure, through the sand and into the original land. Construction was completed in the fall of 1974.

Condition. Photo #15 shows the condition of the site. The use of the groins in conjunction with the rock revetment are shown in their present state in the park area.

General Observations. This installation has achieved its purpose by protecting the refilled park area. To the present time it has escaped storm damage, although there has been a minor loss of rock due to vandalism. It will be of great interest to observe this project for several years to determine how much maintenance will be required.

Photo #15 June 18, 1975
A groin was used in conjunction with rock revetment at this site.

Manistique

This installation has been covered with fill. Until the present time, it has not been exposed to a major storm.

Keweenaw Peninsula

There is a plentiful supply of mine rock available in this area. This material is smaller than that which is normally used for a revetment. However, because of its low cost, it was of great interest locally to determine whether its durability would be worth the effort of placing it. Until this time it has remained in place and provided protection, but it has not yet been tested by severe storm activity. Thus, conclusive statements cannot be made about its effectiveness.

Little Girls Point

Background. This site is in an exposed location on Lake Superior. An innovative type revetment was installed, consisting of "Nami Rings". Part of these concrete rings, which are 2 1/2 feet in diameter and one foot high, were placed on filter cloth, while the remainder were installed on a rock foundation. Portions of the revetment which were placed on the filter cloth were constructed by fastening rings together, while other portions were left unfastened.

Condition and Analysis. Some of the lower rows have been covered with beach sand and most of the other rings are nearly filled with sand. This appears to be due to wave action. The eastern portion of the revetment, which was placed on a rock foundation without tie rods, has been forced upward along its landward edge by clay sliding from the bluff. The slide appeared to be due to unstable conditions which occurred when the steep bluff was very wet during the spring. The upper rows of rings were moved about quite a bit, as shown in Photo #16. Wave action may have contributed somewhat to the movement of the rings. However, a local observer reported that no major northerly storm has occurred since the ice went out. This is borne out by the fact that there has been little toe erosion this spring at the unprotected bluffs to the east and west of the revetment. Some of the movement of the rings could have been caused by ice piling up on the shore.

There has been noticeable settlement near the center of the revetment. This is in the region where the rings are on filter cloth and are held together with steel rods. In this region there is considerable groundwater seepage. This seepage has been so severe that a gulley was formed behind the revetment in the bluff at this location.

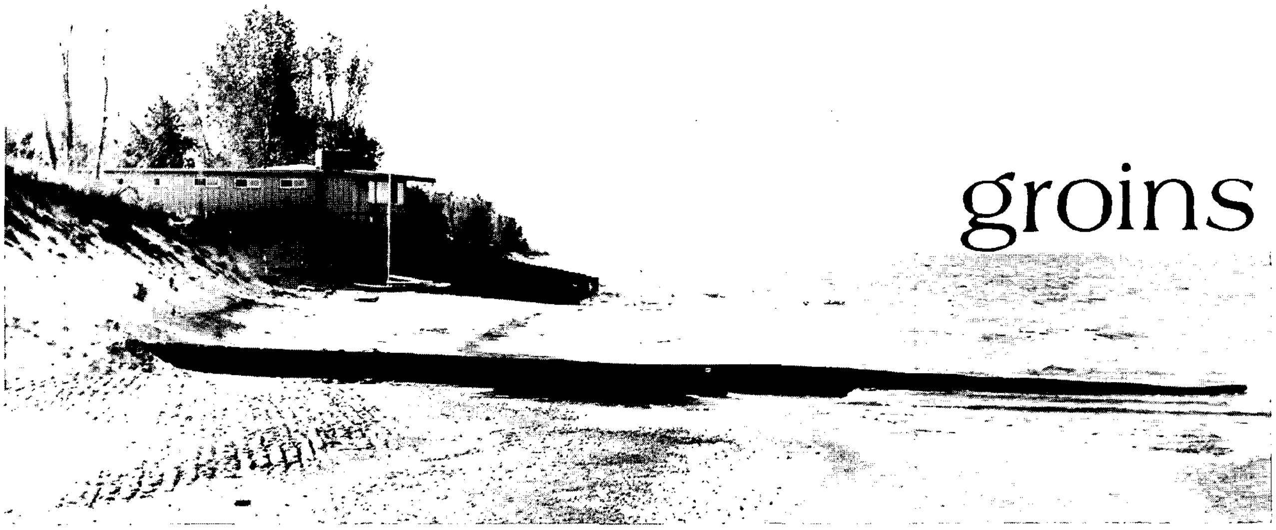
The west end of the revetment seems to nearly be in its original condition. In this area the rings were placed on filter cloth; some of them have tie rods and some do not. The clay slides which caused the uplift on the east end appear to have been milder on the west end.

Maintenance. With a small amount of excavation, those rings which have been displaced could be brought back into the general pattern.

Design Modifications. The revetment has not been tested sufficiently to indicate specific design changes. If the bluff behind the revetment had been cut back to a 2 to 1 slope, much of the sliding which caused the displacement could have been avoided.

General Observations. The capacity of this structure to protect the toe of the bluff from wave erosion has not yet been tested. Some severe storms will probably occur during the fall of 1975 which should provide valuable information on effectiveness and durability.

Photo #16 May 21, 1975
The Nami Rings have shifted and filled with sand.



groins

Lincoln Township

Background. Two groins were placed at this site during the fall of 1973. A 40" diameter Longard tube was the first groin to be installed. The second structure was a timber pile groin. During the first year of study, both groins trapped sand and successfully protected the bluff. However, the Longard tube was settling quite rapidly, and a portion of it had ripped open.

Type of Structure: Longard Tube Groin

Condition. Diagrams #11 and #12 and Photo #17 show the condition of the tube. It has continued to settle, but its rate of settlement has dropped off substantially. In spite of total settlement (which has amounted to as much as 3 feet at the outer end since installation) the tube is still trapping sand and offering protection. The rip in the structure which was observed earlier does not seem to have caused much loss of sand.

Maintenance. The rip in the tube needs to be repaired. An attempt was made previously to repair the tube by sewing the hole together, placing patching material over the entire area and sealing the repair work with roofing cement. This method proved to be ineffective as the hole reopened soon afterward. In all likelihood, grouting this tube with portland cement concrete would probably be the best way to repair it. This would not only seal the hole, but would also hold the remaining sand in place and offer substantial resistance to further damage.

Design Modifications. As with all Longard Tube installations, better provisions must be made to anchor the tube into the existing bluff. This is somewhat difficult to accomplish because of the mechanics involved in installation. A fairly large work area is necessary at the end of the tube nearest the bluff so that it may be filled with the sand that will hold it in place. However, the need for this large work space rules out the possibility of placing the end of the tube right next to the bank, thereby making secure anchoring of it difficult.

Another change would be to install the tube on some type of foundation, when used in areas where the lake bottom is sand. This should prevent settlement of the tube. The best method for providing some support for this tube would be to place it on a rubble foundation.

The last modification to be considered would be provision of an armored, protective coating for the tubes. Such coatings have been developed and should help prevent damage to the tube from flotsam.

General Observations. Despite settlement, the tube has still acted successfully as a groin. If the suggested modifications were incorporated into the design, this tube would probably be a good low-cost method of shore protection, though not as low-cost as the present installation.

Type of Structure: Timber Pile Groin

Condition. Diagrams #11 and #13 and Photo #17 show the condition of the structure. The groin has remained stable since its installation, with no deterioration or other adverse effects being detected. Substantial amounts of sand have been trapped and a protective beach has been built.

Maintenance. No maintenance work appears to be required at this time.

Design Modifications. The design of this pile groin does not appear to need any changes at present.

General Observations. Groins have been a favorite means of shore protection in areas where there has been sufficient littoral drift. This timber pile groin has performed very well to date, thus, providing additional evidence that wood is an excellent material for groin construction.

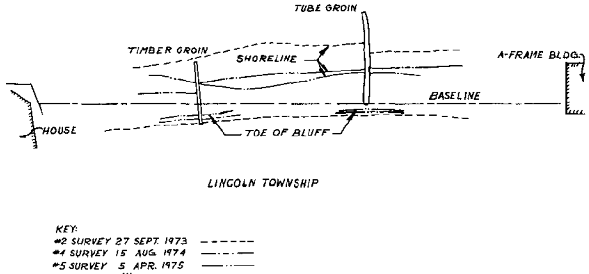


Diagram #11
Plan View of Site

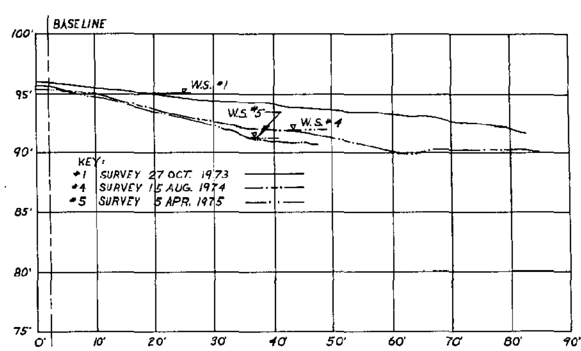


Diagram #12
Profiles Along Top Centerline of Longard Tube

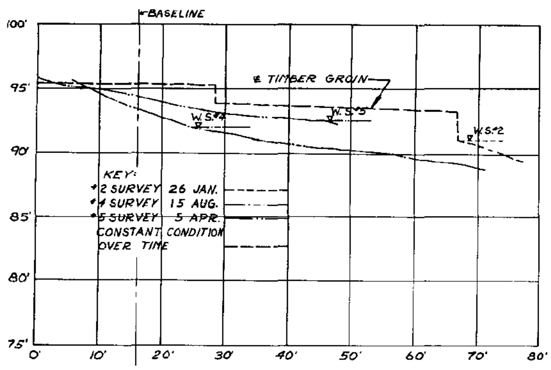


Diagram #13
Profile Along Centerline and
South Edge of Timber Groin

Photo #17 July 18, 1974
A typical example of conditions at this site. Both groins are providing shore protection.

Charles Mears State Park

Background. Originally, this site was selected as the testing ground for five different configurations of gabion groins. This was later changed (see November 1974 report for details) so that the system now under study consists of three groins. These groins are composed of gabions from their inner ends of the waterline. From that point on, out into the lake, the structures are made up of sand bags.

Condition. Diagrams #14 and #15, along with Photos #18 and #19 show the condition of this site. The groins seem relatively stable, although some shifting and settlement has been detected in the sand bags. Compared to the other sand bag installations, however, this movement has been insignificant. Each of the groins has lost one or two bags at the outer ends, yet considering the location of these structures on park land, which is open to the public where the potential for vandalism is high, it is amazing that most of the bags are still intact.

Maintenance. The beach area around these groins appears to have been kept more stable by the presence of the structures. There does not seem to have been any natural filling from littoral drift, but rather a gradual loss of sand during the summer. Each spring the groins are filled with the sand that is removed from the parking area.

At this point, only minimal maintenance seems to be in order. Those few bags which have shifted or have been lost should be replaced. Periodic filling will apparently always be required.

Design Modifications. The design of these groins was dictated in part by constraints in cost and by the direction offered by local interest groups. More time will be required to determine whether their effectiveness and durability are adequate, or whether a more conventional design should have been used.

General Observations. The sand bags seem to be working much more efficiently in this location than in other similar installations. The beach appears to be maintained by the presence of the groins and the annual filling.

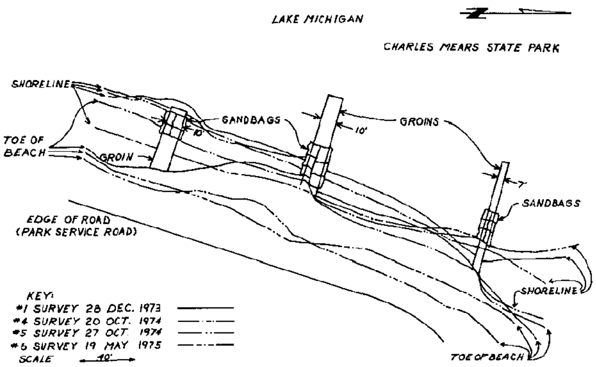


Diagram #14
Plan View of Site

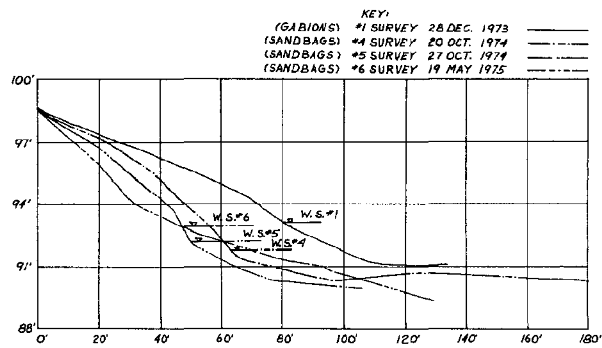


Diagram #15
Typical Profile 25' North and Parallel to Groin

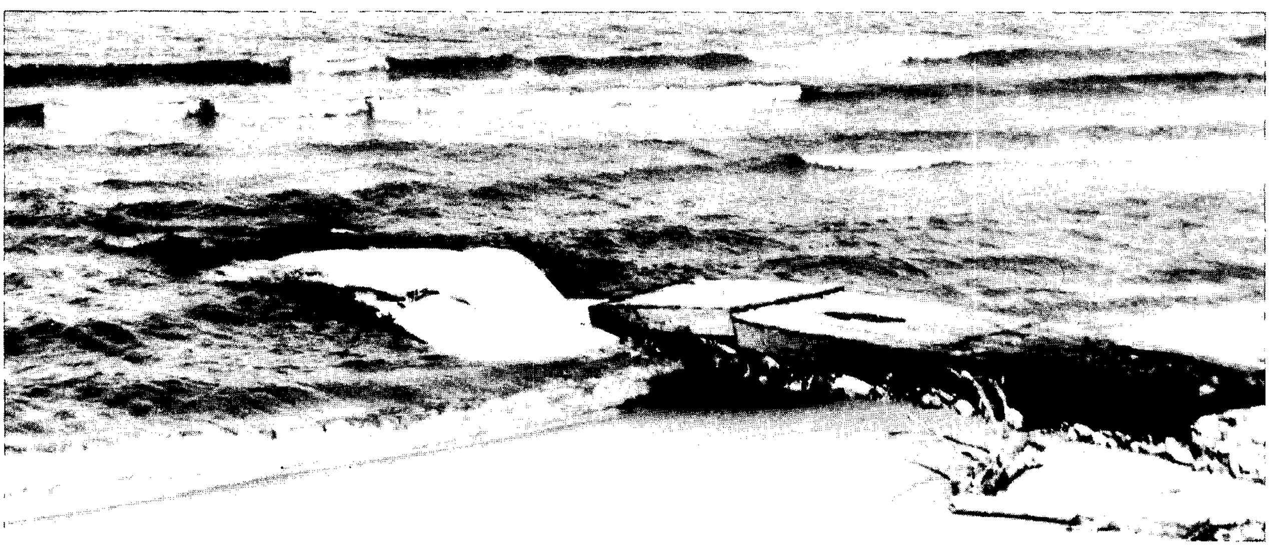


Photo #18 October 20, 1974
The sand bag-gabion arrangement is shown here. Some bags have lost sand and have shifted.

Photo #19 May 19, 1975
The shoreline has remained relatively stable at this location. Periodic sand replacement has been required.

Ludington State Park

Background. Two steel groins were built at this site in the fall of 1973. The groin system experienced flanking and scour problems during the first year of study, requiring maintenance and repair work (see November 1974 report).

Condition. Photo #20 shows the condition of the site in late October 1974. Photo #21 shows the same site as it appeared in mid-May 1975. Some of the field data recorded at this site is included in Diagrams #16 and #17.

Analysis. Presently, these structures are preforming moderately well. Some natural filling has been observed, but continued maintenance and sand replacement (nourishment) by park personnel has been required. It is evident that the sand would be removed much faster without the groins.

Maintenance. Since this site is located at a state park where the groins must be maintained to protect a parking lot, the structures are under continued inspection by the Parks Division. Periodically these groins have required work - sand has been replaced, piles have been redriven, and groins have been lengthened to prevent flanking. Normally, the need for some maintenance and sand nourishment might be expected with any groin system. This extreme level of required maintenance (and the difficulties encountered here) is not typical of steel groins, or of groin systems in general. It is expected that further maintenance, such as the addition of piles or repair and redriving of other piles will be required from time to time.

Design Modifications. Changes in the design and construction of these groins might include the addition of rubble along the ends of the groin to prevent scour, or the use of longer piles to help prevent scour damage to the groin. The use of longer groins to provide better "anchorage" into the existing back shore could also be implemented.

General Observations. With periodic maintenance, the groins at this site have not only helped to prevent erosion of the land area immediately behind the system, but also have protected the parking area. Required maintenance and sand replacement has been unusually heavy at this location.

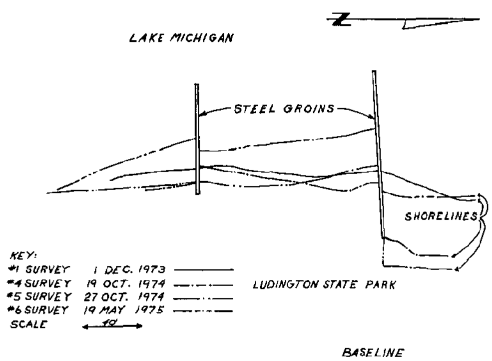


Diagram #16
Plan View of Site

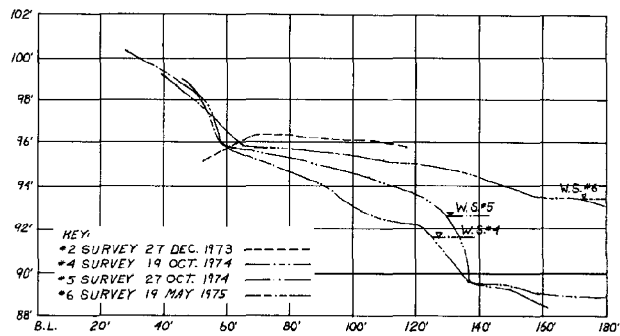


Diagram #17
Profiles Along Centerline
of North Groin

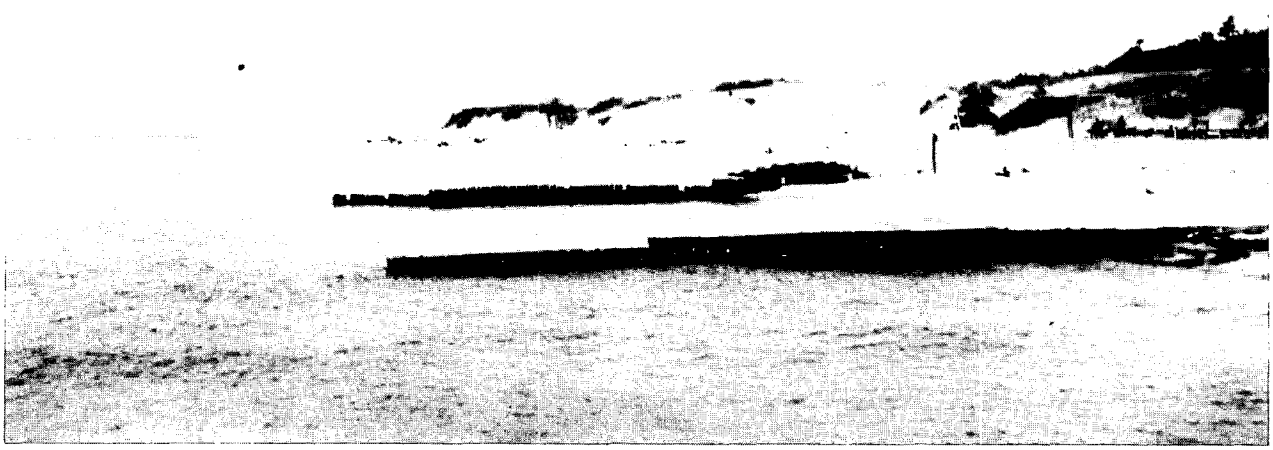


Photo #20 October 20, 1974
The groins at this site have required continual maintenance. A third groin (shown in background), which was built last and is not a part of the project, has fallen over.

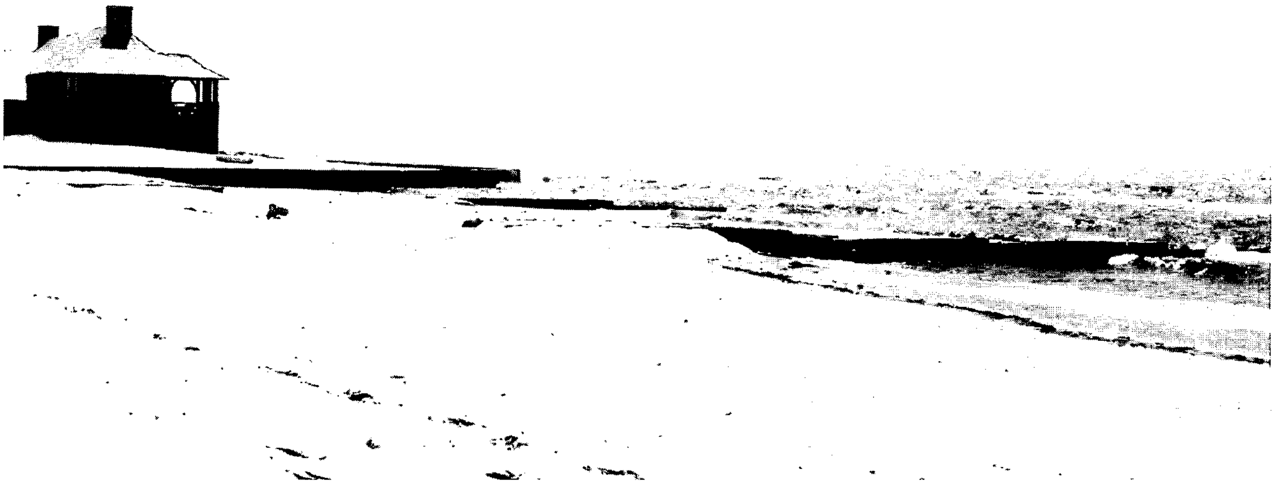


Photo #21 May 19, 1975
A view of the groins looking south.

Sanilac - section 26

Background. A park located approximately four miles south of Port Sanilac, under the jurisdiction of the Michigan Department of State Highways and Transportation, was selected for the testing of six different types of groins. An added feature in this study was unusually large spacing between groins. Normally, the spacing between any two groins is anywhere from one-to-two times the length of the groin that extends into the lake. In this study the groins were spaced at about three times the length. It is still too early in the study to draw specific and final conclusions as to the success of the system. Bluff recession has amounted to as much as six feet behind and between some of the groins in this system. However, in an area of till (clay) such as this, much of the recession could be attributed to clay slumping induced by other factors such as wind, rain, and frequency of thawing. Since a number of different structures are included in this site, each will be discussed separately.

Storm activity has been slight in this area throughout the two-year study period, with only one major storm experienced. Therefore, these structures have not been tested sufficiently to date.

Type of Structure: 40" Diameter Longard Tube Groin

Background. Two 40" diameter Longard tubes were installed side-by-side in the fall of 1973. Originally, installation of three tubes was intended, with the third tube to be placed on top of and between the base two. However, a storm interrupted construction, preventing the placement of the third tube (see November 1974 report for a more detailed discussion of this problem).

Condition. Both of the tubes are essentially intact and properly placed. Some settlement has occurred in both tubes (see Photo #22), especially in the southern (down drift) tube (see Photo #23), but this movement has been minor and has not hindered the performance of the groin. Diagrams #18 and #19 depict the conditions described above.

Maintenance. This structure requires no maintenance at this time.

Design Modifications. Although the use of two tubes side-by-side offers some added strength, it does not fundamentally offer cost-effective advantages as compared to a single tube structure. A third tube in place, as originally planned, would probably increase the structural effectiveness of this groin.

Future construction should try to achieve better anchorage of these tubes into the existing shore and bluff.

General Observations. These tubes have weathered well at this location, and have helped to build a protective beach by trapping sand.

There has been about a six-foot recession of the bank immediately behind this structure. At present, however, this should not be considered a detrimental factor with respect to the performance of these tubes since this land loss might be attributed to factors other than wave attack.

Photo #22 April 27, 1975
Settlement in the south tube is evident here. The groin is still effectively trapping sand.

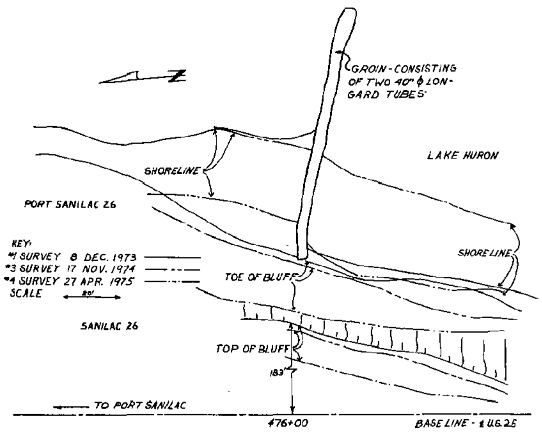


Diagram #18
Plan View of 40" Diameter Longard Tube

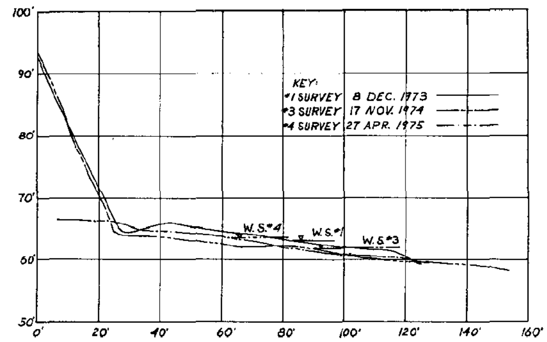


Diagram #19
Profiles Along Centerline of 40"
Diameter Longard Tubes

Photo #23 November 17, 1974
The condition of the 69" diameter Longard tube has remained relatively stable since installation. Note the other Longard tube in the background.

Type of Structure: 69" Diameter Longard Tube Groin

Background. A 69" diameter Longard tube was installed at this site during the spring of 1974. When the November 1974 report was being prepared, no conclusions could be made concerning the performance of this tube since it had not been under study for a sufficient period of time.

Condition. Diagrams #20 and #21 and Photo #23 show the condition of the structure. The tube has remained stable, with only minimal settling. Some bank recession has occurred in the area immediately behind this structure.

Maintenance. Maintenance or repair work is not required at this time.

Design Modifications. The only suggested change in the installation of this tube is to more securely tie and anchor it to the existing bluff. This would require that a portion of the bluff be cut into and replaced, so that the heel of the groin could be set back into the bank during construction.

General Observations. Upon installation of this tube, it was anticipated that is would perform more effectively than the 40" diameter tubes since it had more free board. To date, this has not been the case--both the double 40" diameter and the single 69" diameter Longard tubes have performed equally well.

Only nominal settlement has been experienced here. This seems to indicate that these tubes may be placed directly on the lake bottom without a supporting foundation in areas where the soil is clay.

The recession of the bank behind this tube cannot be attributed to poor performance of the structure at this time since factors other than wave attack may be responsible.

Diagram #20
Plan View of 69" Diameter Groin

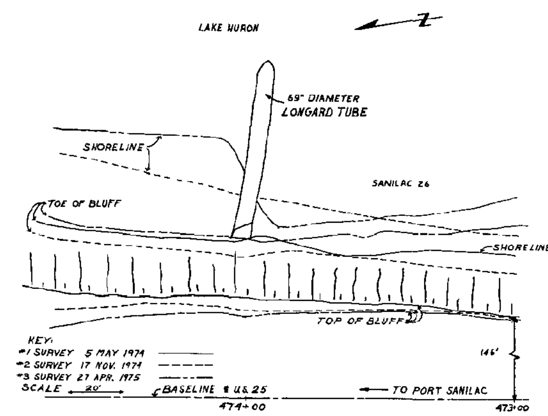
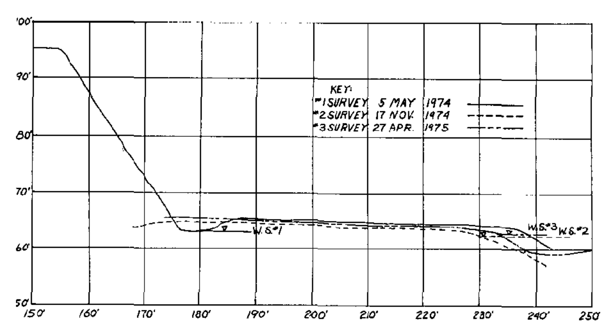


Diagram #21
Profiles Along Centerline
of 69" Diameter Longard Tube



Type of Structure: Sand Bag Groin

Background. The sand bag groin was installed at this location late in the fall of 1973. In the November 1974 publication it was reported that a number of bags had shifted and some were lost.

Condition. Diagrams #22 and #23, and Photos #24 and #25 show the condition of these bags. Additional bag loss has occurred at the lake end and along the top of the groin, destroying approximately 15 feet of this groin. Some bank recession has been recorded in the vicinity of this structure; however, this recession has been less than that recorded near each of the Longard tube installations.

Despite the damage incurred thus far by the structure, it still seems to be an effective device for trapping sand and building a beach.

Maintenance. Apparently provisions should be made for the yearly replacement of a number of bags in this type of installation. Armoring protection might possibly be installed at the lake end of the groin in order to protect the bags from ripping and tearing.

Design Modifications. Some provision should be made to hold the sand bags in the structure together in a more efficient manner. Presently, the bags are "held" in place because of their weight and by being tied together with the nylon leaders to be found at the corners of each bag. This does not appear to be an adequate method. Some means of armoring the structure or providing a tear (rip) resistant coating for the bags is also required.

General Observations. In spite of the loss of bags in this structure, it appears to be trapping substantial amounts of sand. In the November 1974 report it was recommended that this means of shore protection be considered temporary. Now, after further study (with mild storm activity), this statement should be revised to read that these bags should not be counted on for as much permanence as other structures. With provisions made to help eliminate the ripping and tearing of bags, and with some annual replacement of damaged or lost bags, this type of structure might be expected to trap sand for a period of about two to three years, under mild wave attack.

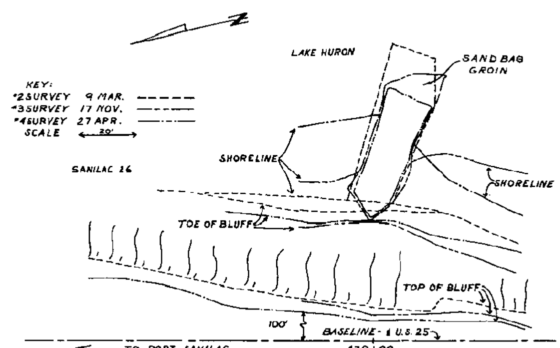


Diagram #22
Plan View of Sandbag Groin

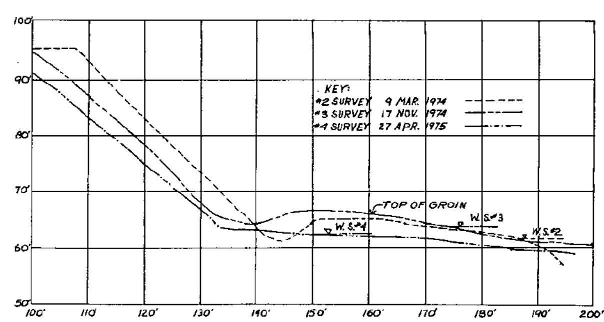


Diagram #23
Profiles Along Centerline
of Sandbag Groin

Photo #24 November 17, 1974
The groin at this site has lost a number of bags, but is still trapping sand and helping to build a beach.

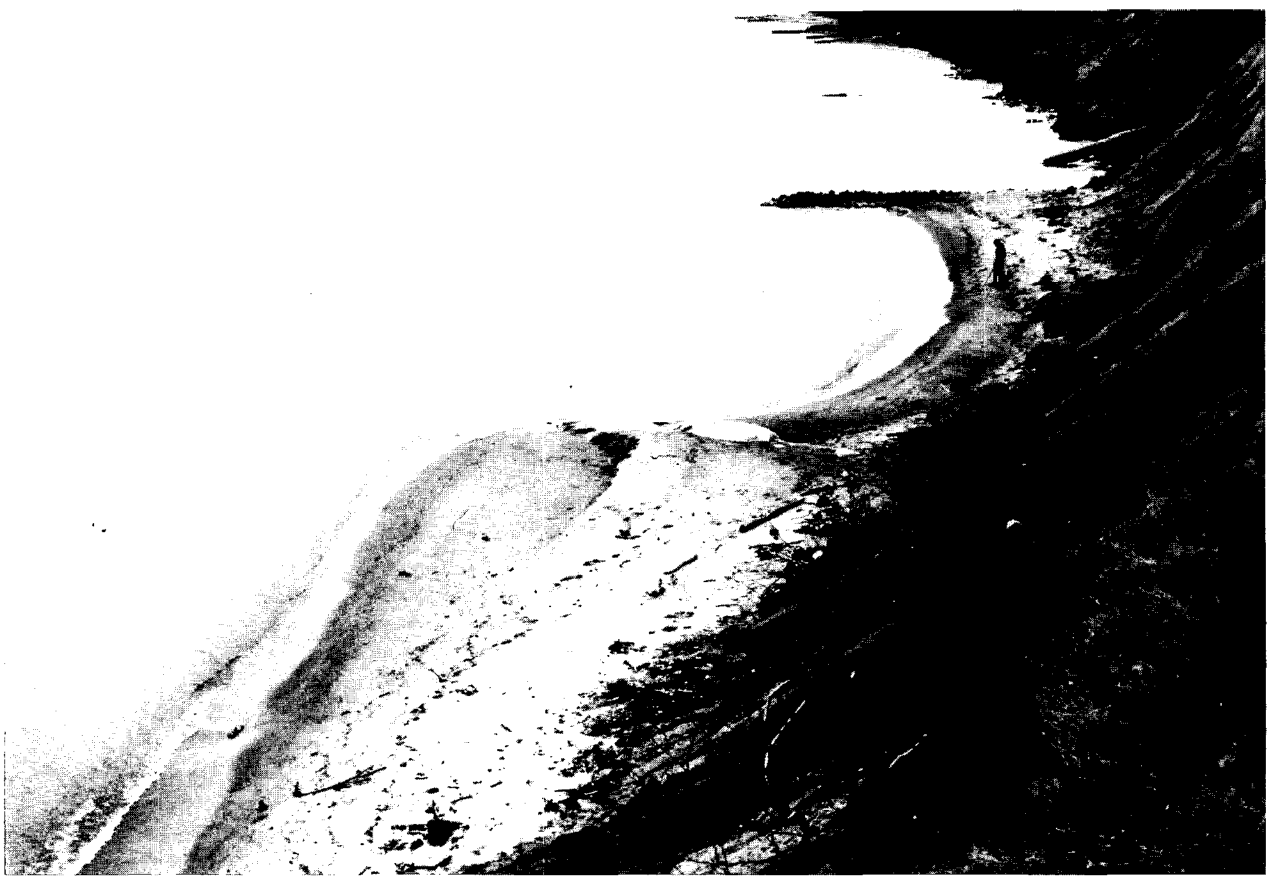


Photo #25 April 27, 1975
Both the sand bag (which has lost many bags) and the rock mastic groin are shown here.

Type of Structure: Rock Mastic Groin

Background. This structure was built in the fall of 1973. It was built to test the use of smaller, less expensive stone capped with asphalt mastic in a groin installation. During construction it was demonstrated that the available construction techniques were sufficient to build this type of installation and to place asphalt mastic through depths of water much greater than previously anticipated (see the November 1974 report for more detail).

Condition. Diagrams #24 and #25 and Photos #25 and #26 show the condition of the structure. Neither a vertical nor a horizontal movement of the structure has been detected. Substantial amounts of sand have been trapped, building a protective beach. About two feet of bluff recession aas been observed in the area immediately behind this structure. This cannot be attributed to the structure.

Maintenance. This structure requires no maintenance or repair work.

Design Modifications. No changes are being recommended in the design of this groin at the present time.

General Observations. This structure is performing very well. The lack of maintenance requirements and design modification allowances indicate that this structure will only involve an initial expenditure for building. It has performed about as well as the much more expensive, conventional groin structure (layered rock with armor stone) that it was intended to duplicate. Thus, the same successful shore protection can be produced much more economically.

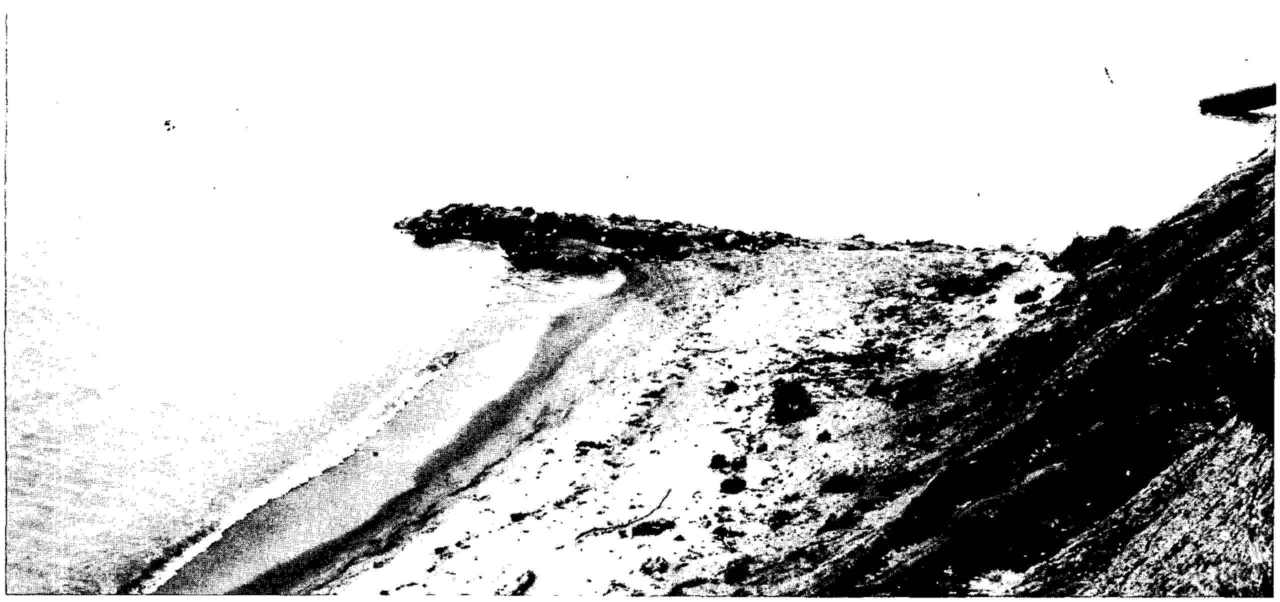


Photo #26 April 27, 1975
The rock mastic groin has remained relatively stable. Trapped sand has built a beach.

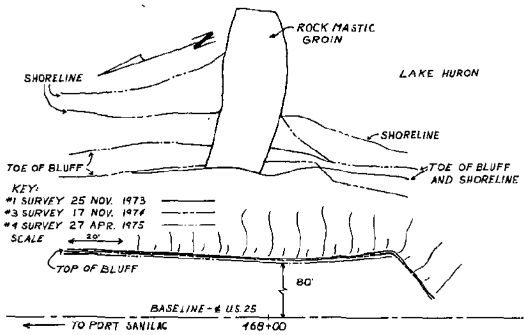


Diagram #24
Plan View of Rock Mastic Groin

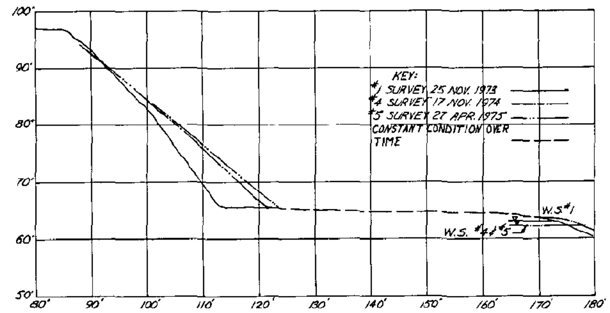


Diagram #25
Profiles Along Centerline of
Rock Mastic Groin

Type of Structure: Gabion Groin

Background. A gabion groin was built at this site after preparation of the November 1974 report. Data on this structure's performance has only been available since this spring (1975). Thus a discussion and analysis of this structure is impossible at this time.

Condition. Diagrams #26 and #27 and Photos #27 and #28 show the base reference condition of the structure. This limited observation indicated that the structure has begun to trap sand and perform as expected.

Photo #27 April 27, 1975
This gabion groin is already building a beach. Note the sand bag installation and the rock mastic groin in the background.

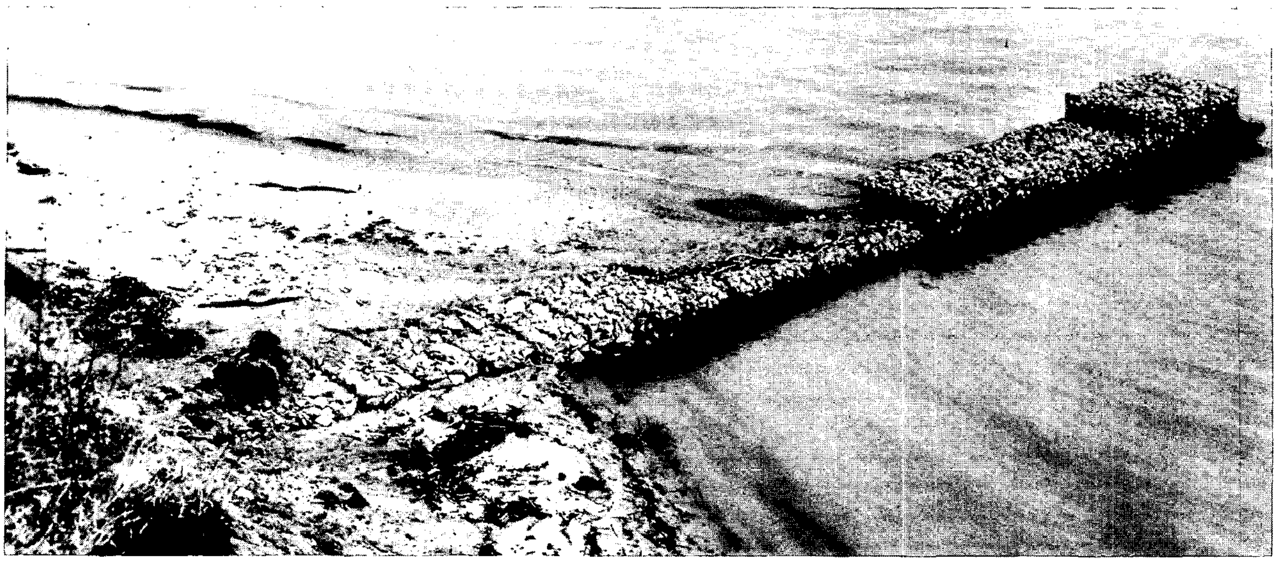


Photo #28 April 27, 1975
Another view of the gabion groin shows its "stepped" configuration.

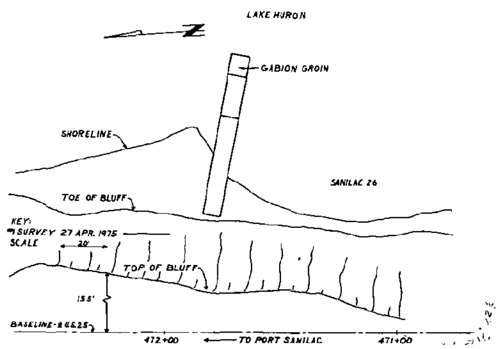


Diagram #26
Plan View of Gabion Groin

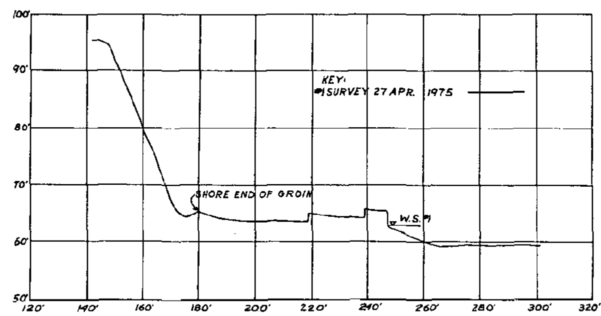


Diagram #27
Profile Along Centerline
of Gabion Groin

Summary of Sanilac Installations.

The wider spacing between the groins under study at this location appears to be performing adequately. Some bank recession has occurred in certain stretches of the bluff; however, as stated earlier, this might be completely attributed to terrestrial processes.

The rock mastic groin has remained, to date, the most stable of all the groins under study. The sand bag structure has suffered the most damage and will probably be completely destroyed within the next year. The Longard tubes are performing adequately. The gabion groin has not been under study long enough to make any general observations.

The last type of structure to be built is a timber crib. The construction of the structure is quite a bit behind schedule and may be dropped from the program entirely. The difficulties encountered in getting this structure built reflect the problems that homeowners face when arranging for the building of a structure by a contractor, even once the homeowner has decided on the type of shore protection required for his specific property.

Marquette

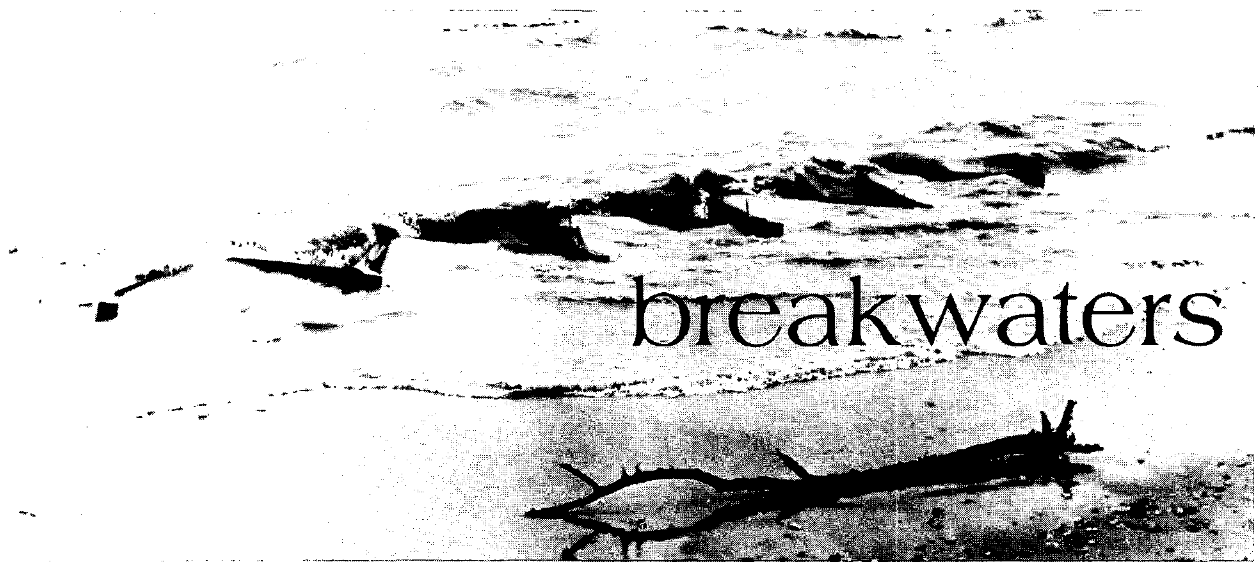
Background. Local concern over the gradual depletion of sand at Shiras Park led to the consideration of this area as a demonstration site. The large size of the park eliminated any possibility of covering even a reasonable fraction of the shoreline with a demonstration project in the form of nourishment. However, when a large supply of sand became available from a local industry, this site was included as a part of the program. The sand contained some very fine particles and was, therefore, not ideal for use as fill material, as the finer grains would be lost to the natural processes of wind and water. However, since the sand was available at no cost, and the coarser grains would remain, the project was considered worthwhile. Because it was felt that the life of the sand nourishment would be quite short without the installation of a grain system, a steel sheet piling grain was constructed within the sand fill as a part of the demonstration program.

Condition. The fill was placed with a very steep front surface in order to facilitate initial shifting which would enable the slope to be adjusted from about 1 1/2 to 1 to about 20 to 1 at the shoreline. The fill has provided protection to the eroding bluffs. It will be of great interest to observe how long it is effective. The erosion of the fill has exposed the groin, and the end of the groin has begun to fail.

Maintenance. If no other measures were taken, it would be necessary to add additional sand every few years.

Design Modifications. This area is sufficiently exposed to make the life of sand fill alone relatively short. The addition of a groin system would retain the sand much longer.

General Observations. This is a good example of how local resources can be used to help protect a shoreline. Even if nothing more is done at this location, the back shore will have been protected for a number of years.



Pere Marquette Township

Background. An offshore breakwater system was constructed at this site. In the fall of 1973 the first breakwater, a 70' long zig-zag structure, was installed approximately 50' offshore. A second 70' long breakwater was installed in the spring of 1974 on the same line, 50' south of the first breakwater. Fifty feet north of the first breakwater a third 56' long structure was installed, also on the same line. During the first year of study, this breakwater system did a good job of protecting the bluff and beach area.

Condition. Diagram #28 and Photos #29 through #34 show the condition of the site. Major recession of the bluff has occurred during this second year of study, with as much as 30 feet of bluff being lost. A bath house, located immediately behind this breakwater system, which the breakwater system was to protect, was lost when the bank eroded. At least one foot of settlement has been detected in the breakwater. The center structure (the first to be built) has lost one module(2 panels), and the north structure has settled at least two feet and has been severely damaged. At least one module has been damaged and lost during this period of study.

Analysis. During the first year of the study these structures provided a great deal of protection and helped create beach in the shadows of the breakwater sections. However, a major storm in January 1975, during which breaker heights were in the 6 to 10 foot range, caused extensive damage to the structures themselves as well as extensive bluff recession. Laboratory tests of models of these structures showed that with normal water levels, the structures provide protection. However, when water levels are higher (up to one foot higher) this effectiveness is greatly reduced. It appears, therefore, that a wind tide of one foot, combined with the settling of the two breakwaters (total settling amounted to one foot or more) accounts for their ineffectiveness during the storm.

Maintenance. The desired procedure to maintain these structures would be to remove them from the water, repair them and replace them at higher elevations on rubble foundations. However, the cost and field difficulties involved in such an operation make this recommendation somewhat unfeasible under the constraints of this program. One of the breakwaters is now so low that it is of virtually no benefit.

Using fill sand to rebuild a portion of the bluff would also be desirable. This sand would probably be lost in the next major storm, but it should help prevent further rapid recession of the bluff.

Photo #29 October 20, 1974
This photograph shows all three units of the breakwater system.

Design Modifications. In order to use these zig-zag walls as a suitable offshore breakwater system some design changes must be implemented. Based on this study, it would be desirable to shorten the gaps between structures. A 30' gap between 70' walls might be more successful.

The structures also need to be placed further offshore--at least 100' from the existing waterline. At this location they would require a minimum of three feet of freeboard.

Some type of foundation support must be provided to prevent the settling of these structures in areas with a sand lake bottom. A rubble foundation might provide a suitable base.

The north breakwater, which was only 56' long, seemed to experience the most damage. The longer sections (70' long) were not damaged as much, thus indicating that short sections may be more vulnerable.

Photo #30 May 19, 1975
The center and north panels are shown here; both have settled.

Photo #31 May 19, 1975
The north unit has settled to the point where it is completely submerged. Some of the shape of the submerged structure can be discerned in the water.

General Observations. These offshore breakwaters were not useful in preventing bluff recession during a major storm. It is highly possible that even more damage would have been done to the bank without these structures, but the protection that they offer is probably not worth the cost.

Offshore breakwater systems are the most expensive of all shore protection methods. With the implementation of the suggested design modifications, these structures may offer suitable means of protection. However, if these changes were made, they would no longer be in the "low-cost" category of shore protection devices, although this type of structure may still be relatively inexpensive as compared to other breakwater systems.

It should be also noted that this type of wall design was developed for on shore usage. This type of usage has not been studied in this program and no observations can be made regarding their effectiveness when used on shore.

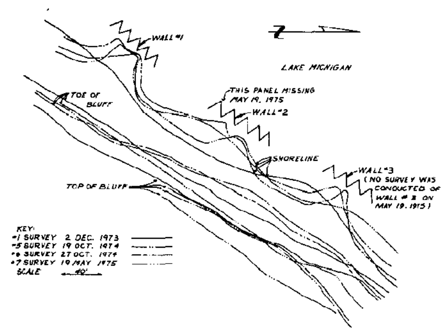


Diagram #28
Plan View of Site

Photo #32 May 19, 1975
The south and center units are shown here. Some of the blocks from the destroyed bathhouse can be seen on the beach.

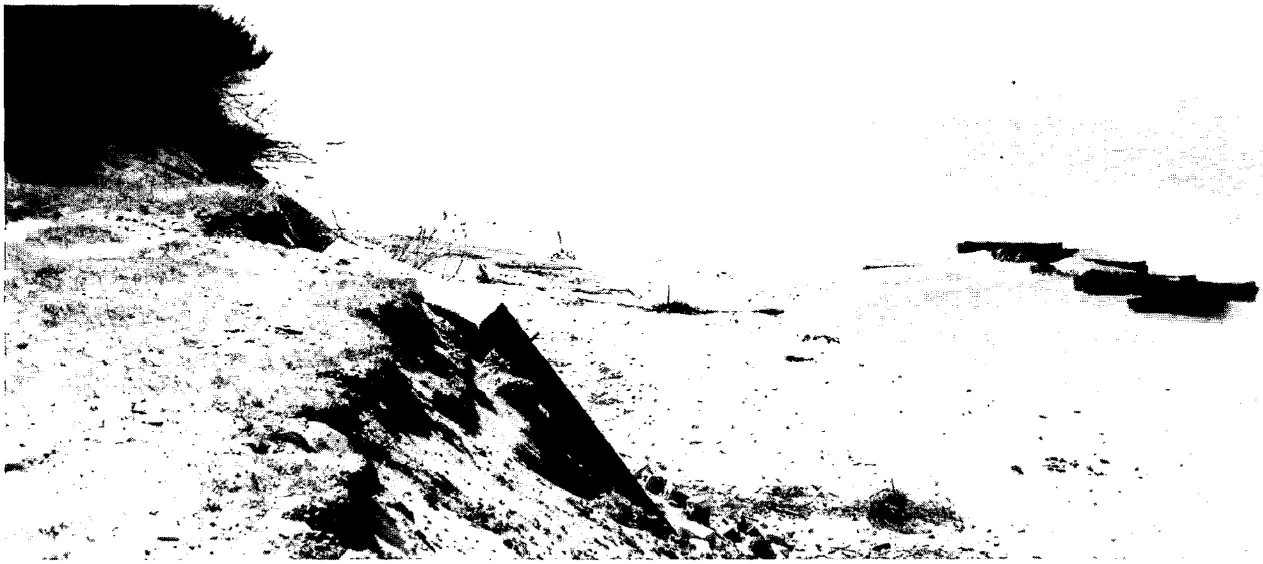


Photo #33 May 19, 1975
These are the remains of the bathhouse. The foundation slab is lying in a litter of blocks.

Photo #34 May 19, 1975
The force of the water was great enough to crack and break up portions of these concrete panels.

Lakeport State Park

Background. In the spring of 1974 a 40" diameter Longard tube was placed at this location on a partly submerged sand bar. This structure was originally intended to be an offshore breakwater, but changes in the local topography prevented this (see November 1974 report for further details).

Condition. Diagrams #29 and #30 and Photo #35 show the condition of the site. Changes (probably unrelated to the installation of this structure) continued to occur in the local topography after placement of the tube, to such an extent that the tube was no longer functioning as a breakwater. It was left lying on the beach with sand in front of it. In order to minimize the tube's interference with bathing on this "new" beach, it was covered with sand. In light of the changes on the beach, the tube no longer fills its primary role as an ongoing experimental shore protection device.

Only one major storm has been experienced at this site to date. Under the brunt of more severe storm action this tube might possibly be needed again for shore protection at this location. However, until such a time, analysis of the tube's performance is not possible.

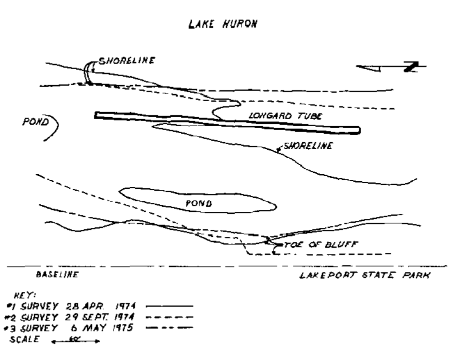


Diagram #29
Plan View of Site

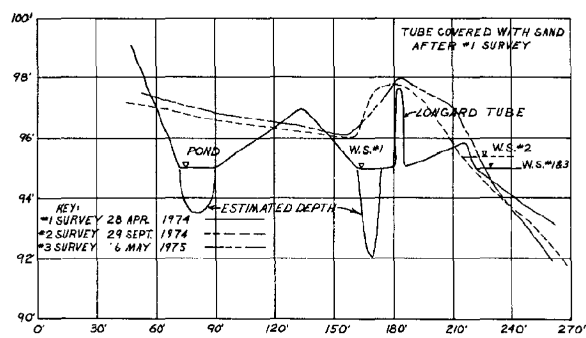
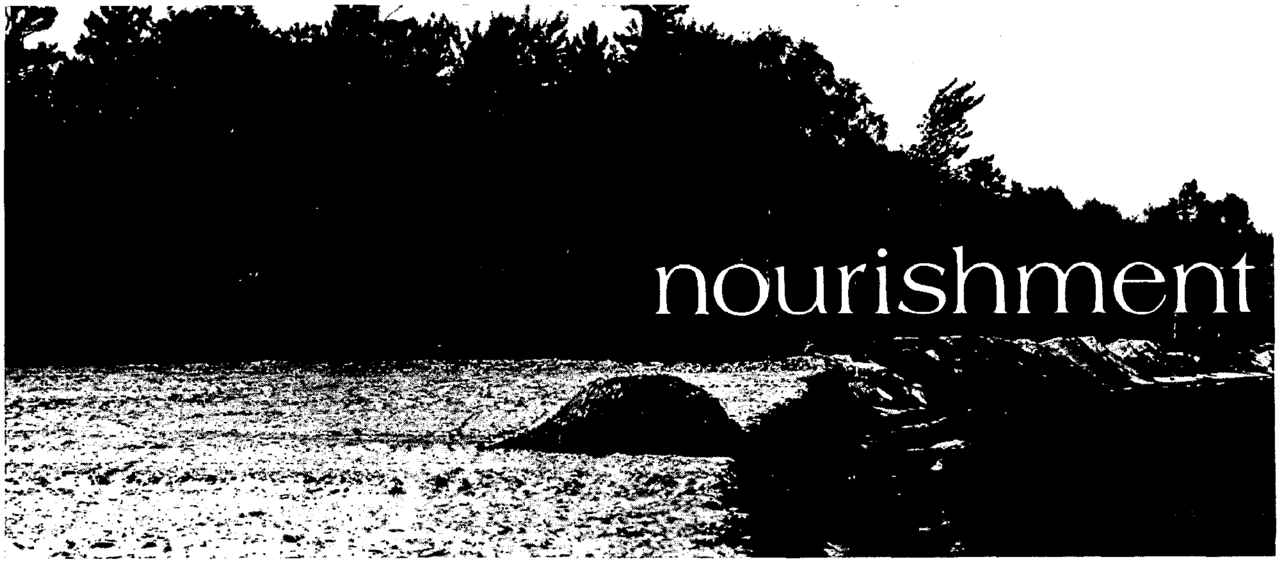


Diagram #30
Profiles 100' South of North End of Longard Tube Perpendicular to Baseline

Photo #35 May 6, 1975
Sand has covered all but small portions of the tube. The structure is no longer needed and is not functioning as a shore protection device.



Tawas City

Background. This site consists of a small public park and the bathing beach for Tawas City. The beach had receded over the years to the point where it was encroaching on the playground, and a tennis court was being threatened. The site is sheltered because of its location in Tawas Bay and also because of a 300 foot pier on the south end. Beach nourishment was the ideal form of shore protection because it would not interfere with the area's recreational use, and because of the sheltered location of the site. In a cooperative effort, Tawas City constructed a wooden groin at the north end of the site.

Condition. Diagrams #31, #32 and #33 and Photos #36 through #38 show the condition of the site. After some initial shifting, the sand used for nourishment seems to be relatively stable. A survey made in mid-May 1975 showed that no major changes had occurred since the survey of mid-October 1974.

The timber groin and pier has helped to hold this fill sand in place, and there is also some additional evidence that it is helping to trap additional sand.

Maintenace. This location has not required any maintenance to date.

Design Modifications. No changes are presently called for at this site.

General Observations. This nourishment project has been very successful thus far. However, only one major storm has been experienced in this area. Therefore, further judgement on the merits of this project must be reserved until additional storm activity is experienced at the site. With no maintenance required, the project is extremely low-cost, running about $10 per foot of shoreline per year for the sand fill. The cost of the groin was $2535, which adds an additional cost of approximately $60 per foot to the project, which is about 425 feet long. Some of the sand would probably have been displaced without the presence of this groin.

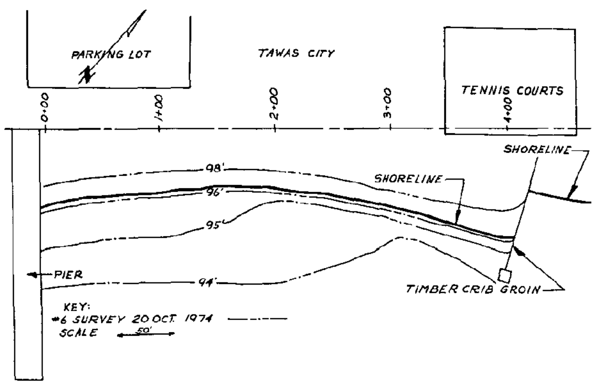


Diagram #31
Plan View of Site and Contours

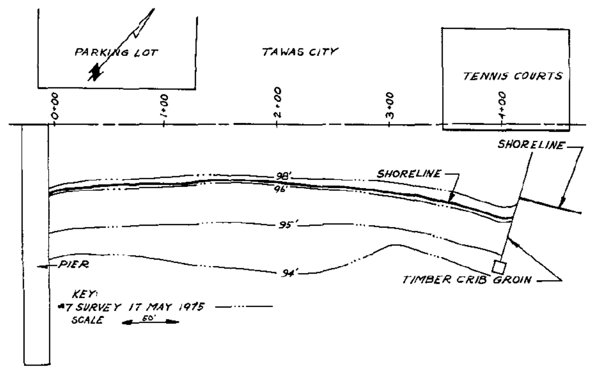


Diagram #32
Plan View of Site and Contours

Photo #36 October 20, 1974
Compare the relatively straight shoreline shown in this photograph with the curved appearance of the shoreline in Photo #37.

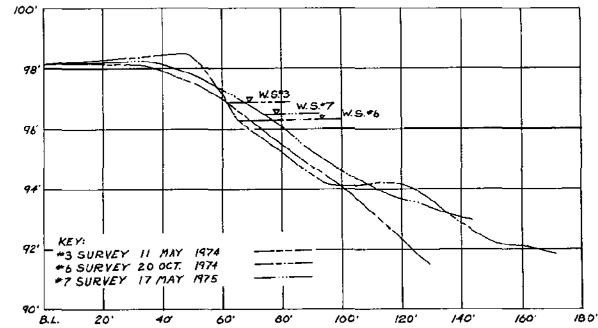


Diagram #33
Profile at Station 3+00

Photo #37 May 17, 1975
Only slight sand movement has been detected at this site.

Photo #38 May 17, 1975
Stability of the sand is helped by the groin shown here.

East Tawas

Background. This site was selected to test a beach nourishment project because of its sheltered location in Tawas Bay. In addition to the sand filling, a small rock revetment was built to protect a small building and a tree. Completion of both phases of this project occurred in the spring of 1974. During the initial monitoring phases of this project some shifting of sand was detected, as expected, and movement of some of the stones in the toe of the rock revetment was recorded.

Condition. Diagrams #34, #35 and #36 and Photos #39 through #41 show the condition of the site. Most of the sand has moved from its original location and appears to have spread multi-directionally in a layer which is almost too thin to detect. The rock revetment is still essentially intact, with limited further movement being detected in the area around the water-line.

Maintenance. No maintenance has been required on the rock revetment to date. The original sand fill remained in place for nearly a year.

Design Modifications. Changes in the design of this project site do not seem necessary.

General Observations. The sand nourishment ramained fairly stable at this site for nearly a year. During the winter of 1974-75 much of the sand moved from its original position and spread into an adjacent area. Its presence is undoubtedly beneficial to this portion of the Bay.

The small rock revetment is still essentially intact, but it has not been subjected to a major storm from the southeast.

Photo #39 October 20, 1975
Although the sand fill has shifted, it still offers shore protection.

Photo #40 May 17, 1975
During the winter some of the small rock revetment was displaced and more sand was moved.

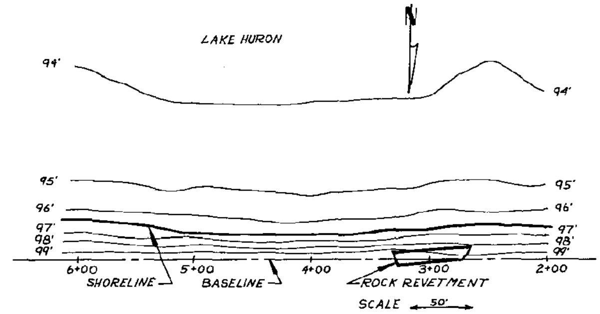


Diagram #34
Plan View of Site and Contours
#7 Survey 18 May 1975

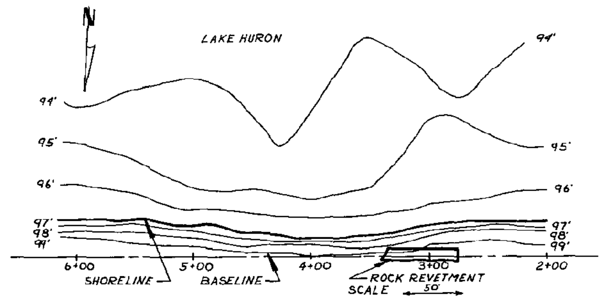


Diagram #35
Plan View of Site and Contours
#6 Survey 20 October 1974

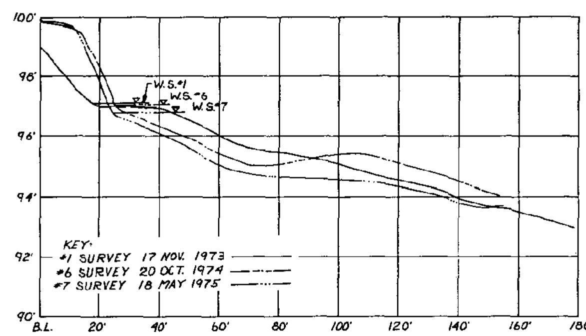
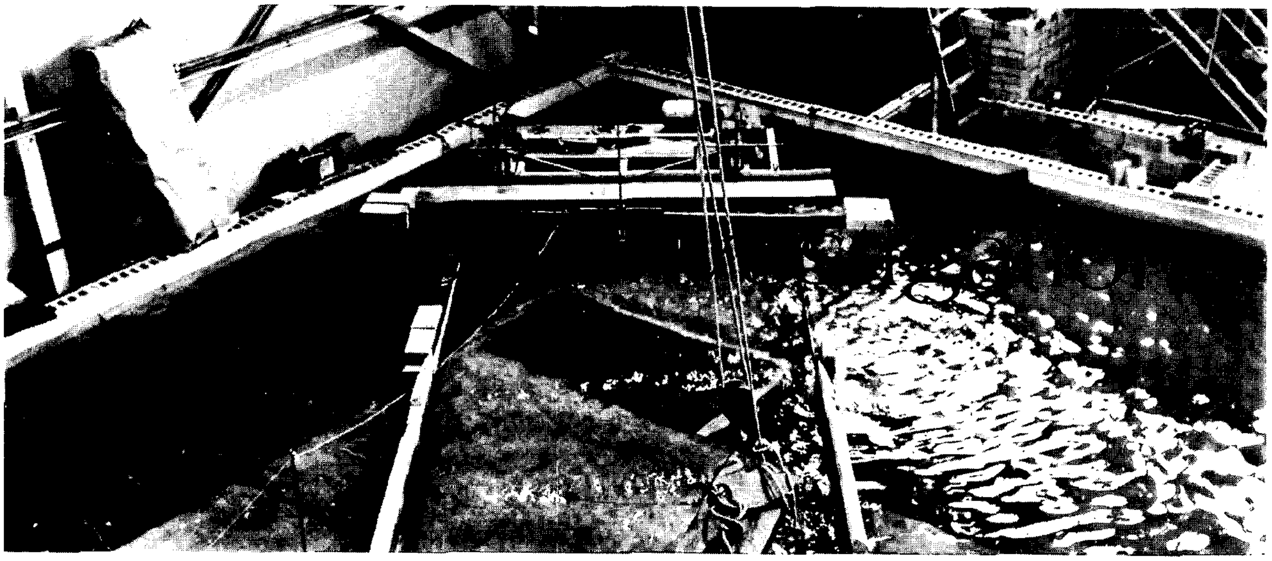


Diagram #36
Profile at Station 3+00

Photo #41 May 17, 1975
Another view of the sand fill and small rock revetment.

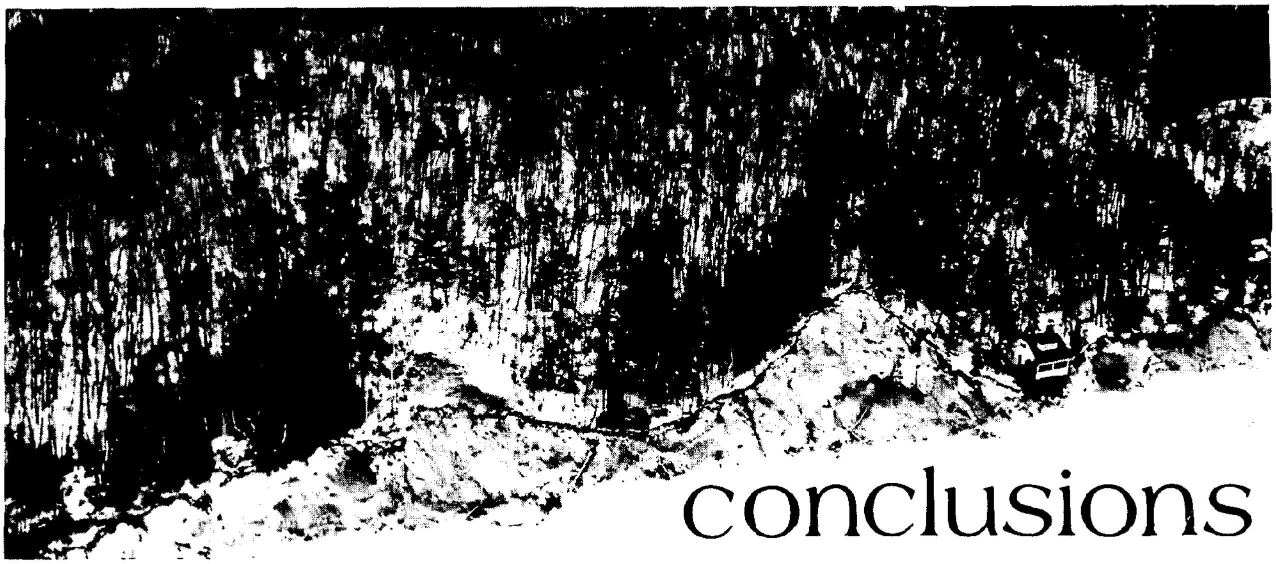


An important input from the laboratory portion of the program was additional insight into the failure of the offshore breakwater, provided by the model tests. This was described in the previous sections of this report.

Model studies were also made on conventional groin systems, primarily to test whether the model could produce results similar to natural processes. The model groins collected and held a beach where there was ample littoral drift, yet showed little benefit in a starved area. The tests showed the importance of keeping groin systems low enough to allow overtopping to occur during storms. Otherwise, waves reflecting from the groin tend to cause erosion in the area where sand accretion would normally occur.

A series of laboratory tests were made on a protective system which consisted of a permeable wall combined with impermeable groins extending out from the wall. This system has been installed by property owners in many locations on Lake Michigan. The wall is usually placed about 50 feet from shore, and the groins, which are about 35 feet long, are spaced from 50 to 100 feet apart. In the laboratory, the system was first tested with just the permeable wall. Two trials were performed on this wall using permeabilities of 40 and 30 per cent. The wall with the 30 per cent permeability was combined with the groins for the next test. The permeable wall alone provided only a small amount of protection, even when the smaller permeability was used. However, the combined system provided about as much protection as good conventional groin series. The tests on this system are not complete.

The model tests have demonstrated the over-riding importance of the water surface elevation on recession rates. Some of the original tests with groins and other devices using higher water surface elevations will be repeated.



conclusions

1. Beach and bluff recession is a natural process which is difficult to guard against.

2. Revetments or groin systems (possibly using sand nourishment) are the best means of shore protection available for areas where shore protection is mandatory.

3. No shore protection can be guaranteed to be 100 per cent effective. There is always the possibility of a very high magnitude storm which can destroy any means of shore protection.

4. Any shore protection device limits the use of the shoreline, and some may actually accelerate bluff recession. Seawalls are particular offenders in this area.

5. Some contractors may experience difficulty in performing the required work either because they underestimate the job, are inexperienced, or for a multitude of other reasons. It is important to be sure that a contract states exactly what will be built, how much material will be used, when it will be built and at what cost. If the services of a Consulting Engineer are not used, the contractor should provide drawings (plans) of the proposed structure, and these plans should be made part of the contract. It is strongly recommended, however, to include the services of a Consulting Engineer.

6. It is important to allow enough lead time (up to four months) so that permits can be obtained. A shore protection device should not be built without a permit. If beginning the work is urgent, the permit application process may be speeded up by using various emergency procedures which are available.

* Conclusions stated here are the result of this study and other related research.

7. Even with suitable means of shore protection and with minimum wave attack, some bluff recession may still continue. This is due strictly to terrestrial processes.

8. Sand is most vulnerable to wave attack; till (clay) is less vulnerable, but may slump strictly due to rain saturation of the clay.

9. Offshore topography and other on-site factors may "suddenly" change, thus altering the impact of a shore protection device on bluff recession.

10. Most means of shore protection which are simply "thrown together" by homeowners are ineffective and often detrimental in the fight against recession. Junk should not be placed on the bluff. Simply putting items such as tree stumps, trees, car bodies, hay or straw, or garbage on the toe of the bluff will not prevent bluff recession. Shore protection devices should be impermeable. Permeable devices offer the least chance for success.

11. Capping rock revetments or rock groins with armor stone, asphalt mastic, or concrete is absolutely essential.

12. It is easy to be fooled by a poor means of shore protection which appears to be "working." Thus far it has been lucky (i.e., storms have been mild or local conditions have changed so that it seems to be working) and it will likely fail in the long run, or it was never really needed in the first place.

13. Do not remove sand from the water to fill sand bags or Longard tubes, or to use for nourishment. This sand should come only from an inland source. Sand used for nourishment should have grain size distribution similar to that of natural lake sand to be most effective. Very fine sand may not be suitable for use in filling sand bags since it may wash out of the bag through the fabric.

14. Beach nourishment programs may be the best means of shore protection if adequate littoral drift is not available. Groins may be used to help hold the sand in place.

15. Sand bags may serve as excellent temporary means of shore protection or be used in other interim roles. However, a high rate of bag replacement may be required since the bags rip easily and can be readily vandalized.

16. Groins which settle may remain partially effective if enough (about one foot) freeboard remains.

17. Groin systems may have detrimental effects on adjacent shore reaches (as any shore protection device might), if not properly designed and constructed.

18. Filling groin systems with sand (artificial nourishment) makes the system immediately effective, possible mitigating damage to downdrift areas.

19. The longer spacing used between groins at Sanilac seemed to show no adverse effects on the groin system's performance.

20. Pile groins, either timber or steel, can be relatively effective in areas where a groin system can be used, but as with any groin, they may require toe protection to prevent scour, and they must be securely anchored into the bluff to prevent flanking.

21. At a minimum, groins either timber to steel, can be relatively effective in areas where a groin system can be used, but as with but groin, they may require toe protection to prevent scour, and they must be securely anchored into the bluff to prevent flanking.

22. Shore protection devices such as sand bags, Longard tubes, or other structures which rest on the lake bottom and depend on it for support, must have foundation support to prevent their settling into the sand.

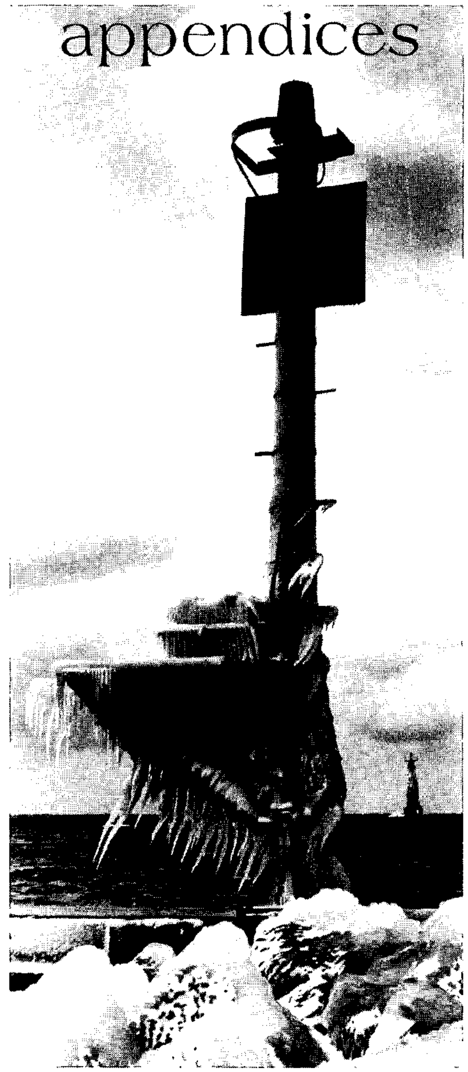
23. Gabions may be a good means of homeowner "do-it-yourself" shore protection. They require a sound foundation, and must be designed and built properly. If they are used in an area where swimmers may come into contact with the gabion, provisions must be made to prevent injury to the swimmer from the sharp wires which are associated with the gabion.

24. A shore protection structure may fail very suddenly in a given storm, even if it has been working well during previous periods of milder storm activity.

25. Revetments and seawalls, especially asphalt mastic revetments which rely on sand for foundation support, experience accelerated recession when overtopped by waves. These structures should have provisions made to prevent the lateral flow of water if the structure is overtopped. Cutoff and return walls, when properly designed and anchored to the bluff, may serve this purpose. Provisions must be made to release water which has overtopped the structure.

26. Asphalt mastic rock revetments may possess the more desirable features of the more expensive, conventional rock revetments, but at possibly half the cost.

27. Precast concrete zig-zag walls may serve as suitable offshore breakwaters, but in order for them to work properly, a suitable foundation must be provided which will increase their already relatively high cost.



Asphalt Mastic Design

Asphalt mastic (sometimes called sand mastic) is a voidless mix of asphalt cement, sand (fine aggregate), and mineral filler (micro-aggregate). These materials are mixed in a proportion of approximately 20% asphalt cement, 60% sand, and 20% mineral filler at a temperature of approximately 350° F. Two books by Baron W.F. VanAsbeck, Bitumen in Hydraulic Engineering, and Bitumen in Hydraulic Engineering, Volume 2, and a paper by R.E. Kerkhoven, "Recent Developments in Asphalt Techniques for Hydraulic Applications in the Netherlands," Proceedings of the Association of Asphalt Paving Technologists, 1965, provide excellent reference information. Fundamental design information is provided in these references; especially useful if you know the required viscosities is a proportioning procedure presented by Kerkhoven.

For the two projects which were part of the Michigan Shore Protection Demonstration Program, an asphalt mastic rock revetment at Michiana, Michigan, and an asphalt mastic rock groin near Port Sanilac, Michigan, a modified trial-and-error proportioning technique was used, Viscosity requirements were not known, therefore Kerkhoven's procedure could not be utilized. Mechanical sieve analysis was conducted on the sand to be used at each project and the grade of asphalt cement to be used was selected, then by comparing the materials presented in the referenced literature, a rough approximation of the required proportions could be made. Small trial batches were made in the laboratory with proportions varying around those suggested in the literature. Through visual inspection, handling, and some crude flow tests, final proportions were selected which seemed to offer the performance characteristics desired.

This procedure worked very well with the materials to be used in the rock revetment, yielding a mix with proportions of:

19% 85-100 penetration asphalt cement
65% sand
16% filler (limestone dust)

The trial-and-error proportioning procedure required more subjective adjustment for the materials used in the groin project. This was probably caused by the use of flyash as the mineral filler component of the mix instead of a natural aggregate material. (The particles of flyash are shaped differently than natural aggregate particles, usually used as mineral filler, and thus it appears that this caused the mix to behave differently). Final proportions were achieved after additional testing. These proportions were:

16% 60-70 penetration asphalt cement
67% sand
17% filler (flyash)

The asphalt mastic mixes behaved well at each site, both in application and service. Application was accomplished at both sites using a front-end loader--this method of application, though very simple, worked extremely well. Now after almost two years of service conditions no adjustment seems to be required in either asphalt mastic mix.

During construction of the groin project at Sanilac, it was demonstrated that asphalt mastic could be placed through greater depths of water than the reference material suggested. Asphalt mastic flowed through as much as seven feet of water and into its desired location without difficulty. The reference literature had suggested that one foot of free flow should be considered as a limit.

An extensive annotated bibliography has been compiled on asphalt mastic mixes and related materials; three of these references might be expecially useful to anyone working with asphalt mastic:

Coast Protection with Bitumen by W. Vissar, Shell Bitumen Reprint 20, Shell International Petroleum Company, Ltd., 1969;

Asphalt in Hydraulic Structures, The Asphalt Institute, Manual Series No. 12, 1965;

Asphalt Jetties Save the Worlds' Shorelines, The Asphalt Institute, Information Series No. 149, 1969.

Since the proportioning of asphalt mastic is essentially trial-and-error and since it appears that it requires some subjective review with the background work experienced here at the University of Michigan, the services of the Civil Engineering Department and the Coastal Zone Laboratory might be volunteered to help in design and construction of further projects.

Critical Storm Data

SITES	Oct 5, 23 Nov 20-21 Dec 1 1974	Jan 10-11, 1975	Jan 18-19, 1975	Jan 29, 1975	Feb 24-26, 1975	DATE OF STORM March 22, 1975 Critical Condition	March 29, 1975	March 30, 1975	April 2-4, 1975	April 19, 1975
Michiana	†††		SW 15-20 knts 18 hrs ** NW 20-25 knts **	WSW 23 knts 6 hrs **	S-SW 18 knts 8 hrs **				NNW 22 knts 4hrs NNW 23 knts 3 hrs NNW 22 knts 8 hrs NNW 21 knts 11 hrs**	
Stevensville (Lincoln Twp)	†††		SW 15-20 knts 18 hrs ** NW 20-25 knts 12 hrs **	WSW 23 knts 6 hrs **	S-SW 18 knts 8 hrs **	WNW (320°) 20 knts 18 hrs U = 20 BW = 6.2 U_1 = 23 BW_1 = 7.8 U_2 = 15 BW_2 = 4.1		WNW 23 knts 8 hrs U = 23 BW = 6.7 U_1 = 26.5 BW_1 = 8.0 U_2 = 17.3 BW_2 = 4.7	NNW 20 knts 38 hrs U = 20 BW = 6.5 U_1 = 23 BW_1 = 7.8 U_2 = 15 BW_2 = 4.5	
Charles Mears State Park	††	WSW 27 knts 18 hrs U = 27 BW = 8.4 U_1 = 31 BW_1 = 9.5 U_2 = 20 BW_2 = 6.4	SW 15-20 knts 24 hrs ** WNW 20-25 knts 10 hr **	WSW 23 knts 8 hrs **	S-SW 15 knts 8 hrs **					
Ludington (Pere Marquette)	††	WSW 27 knts 18 hrs U = 27 BW = 8.6 U_1 = 31 BW_1 = 10.2 U_2 = 20 BW_2 = 6	SW 15-20 knts 25 hrs ** WNW 20-25 knts 25 hr **	WSW 23 knts 7 hrs **	S-SW 15 knts 8 hrs **					SSW 20.5 knts 12 hrs U = 20.5 BW = 6.5 U_1 = 23.6 BW = 7.6 U_2 = 15.4 BW_2 = 4.5
Ludington State Park	††	SW 26 knots 6 hrs U = 28 BW = 8.2 U_1 = 32.2 BW_1 = 10.0 U_2 = 21 BW_2 = 7.7 WSW 27 knts 4 hrs U = 27 BW = 8.4 U_1 = 31 BW_1 = 10 U_2 = 20 BW_2 2.6	SW 15-20 knts 24 hrs **		WSW 20 knots 10 hrs U = 20 BW = 5.9 U_1 = 23 BW_1 = 7.1 U_2 = 15 BW_2 = 4.1					SSW 18.6 knts 25 hrs U = 18.5 BW = 6.0 U_1 = 21.3 BW_1 = 7.3 U_2 = 13.9 BW_2 = 4.0
Brevort (Moran Twp)	†	SSW 28 knts 4 hrs U = 28 BW = 6.6 U_1 = 32.2 BW_1 = 8.4 U_2 = 21 BW_2 = 4.6 WSW 28 knts 14 hrs U = 28 BW = 10.2 U_1 = 32.2 BW_1 = 12.1 U_2 = 21 BW_2 = 7.1								WSW 20.6 knts 14 hrs U = 20.6 BW = 4.8 U_1 = 23.7 BW_1 = 5.7 U_2 = 15.5 BW_2 = 3.4
East Tawas	††††	SE 20 knts 3 hrs ** SSW 25 knts 9 hrs								
Tawas City	††††									
Sanilac 11,26	††††									
Lakeport	††††									

U = wind velocity over the fetch (knots) (measured at nearest land station)
U_1 = average wind velocity over the fetch if 30% higher than U
U_2 = average wind velocity over the fetch if 50% lower than U
BW = height of breaking wave for given wind velocity (feet)
BW_1 = height of breaking wave for U_1
BW_2 = height of breaking wave for U_2

** no breaking wave height values computed for this site
† October 5, 1974, no storm data available
†† October 23, 1974, no storm data available
††† November 20-21, 1974, no storm data available
†††† December 1, 1974, no storm data available