Shore Protection Plan
National Gypsum Site
Alpena Michigan

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Approximately 4500 ft of shoreline at the National Gypsum site are being investigated for remediation from past disposal of cement kiln dust (CKD) in the immediate vicinity of Lake Huron in Thunder Bay near Alpena, Michigan. CKD exists in a large pile along the shoreline and along the lake bottom in some areas. In addition, loose CKD chips and dust exist in the nearshore zone due to past erosion of the pile. 1850 ft of the shoreline was covered by a revetment during the year 2000 to provide immediate protection of the toe of the CKD pile that was being undercut by wave activity. Three remaining areas are currently being evaluated for further remedial efforts. Area 1 is a section of shoreline approximately 650 ft long located to the southwest of the recently completed revetment. Area 2 starts at the opposite end of the revetment and extends approximately 1250 ft in a northerly direction along the shoreline while Area 3 extends another 800 ft along the shoreline beyond Area 2. The characteristics of the shoreline as well as the extent of the CKD are different in each area, potentially requiring different remedial efforts at each location. This report analyzes the wave climate that would be expected at each of the three areas and provides a description of potential shore protection systems that could be used to minimize further contacts between the CKD and Lake Huron. It is understood that it is desirable to minimize construction in the lake beyond the low water datum (LWD) of 577.5 ft (1985 IGLD). Current water levels are very close to the LWD.

Design Wave Analysis

Two sources of information are readily available from which design wave conditions can be estimated. These are:

Design Wave Information for the Great Lakes, Report 4, Lake Huron by D.T. Resio and C.L. Vincent, Technical Report H-76-1, September 1977, U.S. Army Engineers Waterways Experiment Station Hydraulics Laboratory

Wave Information Study Report 26, Hindcast Wave Information for the Great Lakes, Lake Huron by R.D. Reinhard, D.B. Driver and J.M. Hubertz, December 1991, U.S. Army Engineers Coastal Engineering Research Center.

In the discussion below, these two reports are referred as the WES and WIS reports, respectively. The basis for the two reports is basically the same numerical model, which forecasts waves from wind fields interpolated from meteorological records at weather stations located around the Great Lakes region. The WIS can be considered to be an update of the WES report with additional meteorological data, refinements to the numerical model, and apparently variations in the numerical grid employed.

Results of the wave hindcasting are presented as various statistical records for offshore sites at selected locations along the Lake Huron shoreline. The closest location to the National Gypsum site in the WES report is referred to as shoreline grid point 17 and is located just off North Point on the outside of Thunder Bay. A similar location in the WIS report is H18 that is located just to the south of North Point and at the same offshore location as Station 17. The waves forecast by the models are intended to be interpreted as offshore or deep-water waves. The hindcast waves from both model simulations are subject to various statistical analyses to define the wave climates according to season, direction etc. There are significant differences in the presentation between the two reports but both provide wave heights as a function of return period for the entire hindcast wave record. For comparison, the two reports provide the following common statistics:

Report		**Return Period (Years)**		
		5	**20**	**50**
WES	**Station 17**	20.0	22.3	24.0
WIS	**Station H18**	20.0	21.7	23.0

Presented in the table above are wave heights in feet. The two are sufficiently close to each other that for the purpose presented below; either set of wave heights may be considered in the analysis.

The peninsula at North Point from wave attack from northerly or easterly directions protects the National Gypsum Site location in Thunder Bay. Therefore, the wave heights provided above do not accurately represent the climate at the site. Both reports indicate that the most prevalent large waves at the indicated stations (17 and H18) are from an angle sector generally between north and east. Exposure at the National Gypsum Site is from waves arriving from about the southeast to south-southeast sector. The WES report breaks down the wave statistics in a variety of convenient ways including wave direction and season of the year. Station 17 provides statistics for what is referred to as angle class 1 waves; these would have an origin from a sector ranging from the south to a bearing 60 degrees towards the east. This angle class more than covers the possible bearings for wave that could impact the shoreline at the site.

Winter and Fall waves have the greatest heights for a given return period for angle class 1 waves at station 17, as is generally the case all over the Great Lakes. In the WES report, Fall waves are defined as those occurring during October to December, Winter waves between January and March, Spring between April and June, and Summer corresponds to the months of July to September. For example, the 5-year return period wave for angle class 1 wave is 13.1 ft for Winter waves and 11.5 feet for Fall Waves. By contrast, the 100-year return period wave height for Summer waves is only 5.9 ft. The significance of this is that high wave conditions are generally confined to the October to March time period and only occur infrequently during Spring (especially late Spring) and Summer periods.

Given the Fall and Winter wave heights for even moderate return periods such as five years, the design wave for any shore protection system at the National Gypsum site will be controlled by wave breaking. The near shore waters are relatively shallow (generally on the order of 2-3 ft) up to 100-200 feet offshore of the current shoreline. Although the Great Lakes are currently experiencing relatively low water levels (an elevation of 577.6 ft on August 7, 2001) even going to the maximum recorded monthly mean water level of 582.35 ft would only increase these depths less than five additional feet. Waves with heights on the order to six to eight feet or greater would break in waters of these depths. Therefore, the design wave should be the maximum breaking wave that is supported by a given water surface elevation and the selection of a design water level is a more critical factor in determination of shore protection systems.

Design Water Level

A statistical analysis was performed to estimate a design water level from which breaking heights can be computed. Monthly mean water levels were obtained from the U.S. Army Corps of Engineers website (http://huron.lre.usace.army.mil/) for the period of 1918-2000. These were analyzed in two different sets, one that only considered waves during the months from October to April (periods of high wave activity) and another that considered all months of the years. The maximum monthly water level was extracted from each year's record and sorted and plotted using standard frequency analysis procedures. Both data sets approximately follow a normal distribution. Fitting a normal probability distribution to the data, the set from all months of the year show a mean of 579.6 feet and a standard deviation of 1.31 ft while the partial month record indicates a mean of 579.1 feet and the same standard deviation. The difference between the two records is due to the fact that on an average basis, the highest water levels occur in July during periods of low wave activity.

The monthly water levels analysis could be used to establish design water levels, but storm surge can increase water levels, particularly at times of high wave activity. There are no storm surge data for Thunder Bay and the closest water level monitoring station is located at Harrisville to the south. The U.S. Army Corps of Engineers provides data on storm surge probabilities (broken out by month of the year) at this site. They provide results for exceedance frequencies of 20, 10, 3, 2, and 1 percent. There is a fundamental difference between the Harrisville site, exposed to waves from a wider range of directions including the north to east exposure that yields the largest waves (and presumably storm surge as well) and the National Gypsum site that is located in an enclosed bay that would experience more wind setup effects for a given wave from the southeast. Since these two factors oppose each other, it was decided to use the Harrisville data without adjustment to estimate the frequency distribution for storm surge. The surge for the month with the highest value at each given probability was combined and a curve fitted through the values. This curve was used to define the exceedance probability and combined with the monthly mean distribution to give a joint probability since storm surge should be nearly statistically independent of monthly mean water level. This joint probability was analyzed to provide estimates of water level exceedance due to the two factors. For the partial months distribution, the analysis provides water levels associated with the following return periods:

Return Period (years)	10	25	50	100
Water Surface Elevation (ft)	581.1	581.7	582.1	582.4

The same return periods yield water surface elevations approximately 0.5 feet higher if the distribution of maximum mean monthly level for all months of the year is used in the analysis. For purposes of comparison, the all-time monthly high water surface elevation for Lakes Huron/Michigan is 582.35 feet.

Since the purpose of the remediation is to prevent contact between Lake Huron and the on-shore CKD, a return period of 100 years as given above is recommended and thus a water surface elevation of 582.4 feet. Assuming that a revetment would protect to the LWD or roughly the existing shoreline, the bottom elevation at the toe of the structure would be 577.5 ft providing a water depth of 4.9 feet at the design high water level.

Breaking wave heights may be estimated from this depth at the toe of the structure given the offshore slope. The Appendix gives offshore profiles measured on August 7, 2001 at several locations in Areas 1 and 2 and one profile in Area 3. In Area 1, typical offshore slopes (for the first 100 ft offshore) are on the order of 0.03 to 0.05. Using the maximum slope of 0.05 and the breaking wave curves from Weggel as presented in the 1984 version of the Shore Protection Manual as Figure 7-4, the breaking wave height will be approximately 6.3 ft. This requires an iterative approach since the wave period is required in this analysis and wave period and height are correlated. This wave height will have a peak period on the order of 7 seconds according to the WES and WIS reports.

The above breaking wave heights are predicated on the assumption that a proposed shore protection system extends to the LWD. The shoreline in Area 1 currently consists of several feet of sand over buried CKD and the current width of the shoreline is between approximately 60 and 120 feet. The current location of the toe of the CKD pile is apparently determined by the limit of shoreward erosion during high water wave attack. Another alternative would be to place protection at the toe of the CKD bluff on the existing shoreline. Under these conditions, a much smaller design wave height would be appropriate as the protection could be placed above the normal high water level. Under this assumption, the design water level should be increased to the 582.9 ft level since the smaller waves could occur during the summer months when the lake level is normally higher. The exact water depth at the toe of a structure is would be greater than indicated by the current shoreline elevation due to two effects. Shore protection systems that reflect wave energy tend to experience erosion of the shoreline immediately offshore of the structure. This can be partially mitigated by designing a structure that reflects very little wave energy. In addition, during storm conditions, beach material is generally eroded from the foreshore and deposited in a bar offshore of the breaker zone. Although this material tends to be restored to the beach during a recovery period following the storm, the temporary shoreline erosion will increase the local depth at the toe of the structure. Therefore, shore protection system located above the high water level can be expected to experience breaking waves of a foot or so in height and potential runup from even larger waves.

The offshore slope in Areas 2 and 3 is much less than in Area 1. More importantly, waves arising from the south-southeast to southeast sectors will be propagating nearly parallel to shore in these areas. Refraction analyses were performed to determine whether the refraction process could reorient the waves more shore normal, but this effect was determined not to be important. Therefore, the breaking wave height for Areas 2 and 3 are estimated at the zero (relative to the direction of wave propagation) slope limit and provide a breaking wave height of 3.8 feet. Since this wave height is so low and could occur during summer months, the 100-year water surface elevation of 582.9 feet estimated for all months of the year was used to yield a final breaking wave height of 4.2 feet for Areas 2 and 3. This wave would have a period on the order of 6.5 seconds.

Design for Ice Forces

In addition to design for wave forces, considerations must also be given for ice forces in the Great Lakes environment. Design guidance for ice forces is much less well developed compared to wave forces. Indeed, conventional design approaches generally consider that if the structure is designed to withstand wave forces, it will also be capable of withstanding any forces due to ice. This approach would not necessarily be valid in a low energy wave environment such as is the case in Areas 2 and 3 or in the event that a shore protection system is placed above the high water level in Area 1. Forces associated with ice may be of several types including buoyant if ice forms on the armor units and then subsequently is lifted under rising water levels. The most important considerations are apparently associated with "ride-up" in which a sheet of ice is forced onshore by winds or currents. If the sheet of ice is sufficiently thick to avoid buckling failure, it may exert horizontal forces up to the crushing strength of the ice. The key to design is to create a buckling failure at lower horizontal forces so that the potential horizontal ice force cannot be mobilized. Buckling failure can be promoted by an irregular onshore profile such as is indicated schematically in Figure 1.

Alongshore Sediment transport

The direction of prevailing sediment transport along the general section of Lake Huron shoreline is south to north. This is also consistent with the observations on-site. Considering that the point just recently protected with a revetment has served as a primary source for CKD material, the location of the CKD chips along the shoreline is consistent with transport from this area. The area offshore of Area 1 is covered with sand that is apparently being transported from further updrift to the southwest. Although the limited fetch directions in Thunder Bay allow only waves with a fairly direct attack to approach Area 1, apparently residual currents set up lower in the bay support sediment transport from southwest to northeast along the shoreline. The peninsula formed from CKD and currently protected by the previously installed revetment at least partially serves to block transport through Area 1 and further to the east. Currently sand has accumulated to depths of approximately four feet on top of existing CKD. This sand will be relatively stable to changes in water level. The Bruun (Per Bruun, *Sea Level Rise as a Cause of Shore Erosion,* Journal of the Waterways and Harbors Division, ASCE, Vol. 88, February 1962, pp. 117-139) concept of adjustment of beach profiles to long term changes in mean water levels has been used to interpret shoreline changes along the Great Lakes. An increase in long term water level will be associated with an increase in the bottom elevation. In the current situation, the lakes are very close to their extreme lows and therefore there is little chance of erosion of the existing lake under conditions of rising water levels at some time in the future. A key issue is that the supply of sand necessary to provide the increase in the offshore wave profile is generally derived from the beach and the shoreline position at a given higher lake surface elevation will be located further onshore than the current elevation on the existing beach. The elevation of the transition from the existing beach to the CKD bluff is between 585 and 586 feet. This transition has been established by high wave conditions during previous high water periods with the existing beach established during the drop in lake levels since that time.

Further along the shoreline in Areas 2 and 3, the direction of prevailing wave attack from waves with long fetches is nearly shore parallel as discussed above and towards the north. The littoral currents created by this wave climate are capable of transporting material along the shoreline towards the north. There are a number of indicators of this trend. First of all, the shoreline and nearshore area in Area 2 has considerable amounts of CKD "cobbles" as indicated by digital images of the shoreline in Figures 2-5. This material is primarily derived from materially that was previously sloughed at the toe of the CKD bluff that is currently protected by the new revetment. Further along the shoreline (basically the dividing line between Areas 2 and 3), the CKD is partially blocked from alongshore transport by a small peninsula on the National Gypsum property. There is however some transport of CKD fines and smaller chips further to the north of the peninsula (Area 3) although in lesser volumes than in Area 2. Further to the north, off National Gypsum property, an additional peninsula towards the back of Whitefish Bay effectively halts the migration of CKD along the shoreline. Since the CKD consists of both fines as well as the cobble, there is currently a supply of material to be transported to the north along this entire stretch of shoreline although the ability to transport decreases towards the north.

Shore Protection Systems

Area 1

Since high water levels in Lake Huron will produce wave condition that can reach the toe of the CKD bluff, some level of shoreline protection will be required. One economical design approach would be to place a cover over the existing beach above the high water level. There is at least 25 feet of beach width above the high water level everywhere along Area 1. A graded riprap cover extending lakeward from the toe of the bluff would be adequate to protect the shoreline and prevent erosion to the toe of the existing bluff under high water conditions. A conservative approach to the sizing of the stone indicates that 50 pound stone would provide adequate protection against the broken waves running up the shoreline. Protection against ice damage may be necessary as a stone of this size would not be stable to ice riding up the shoreline.

The advantage to the second alternative is that much smaller stone volumes are required, making this a much less costly alternative. At smaller sizes, stone costs is not generally strongly related to size so that the riprap can be oversized to provide an additional factor of safety. Larger armor stone could also be provided at the lakeside of the armor layer to provide protection against ice damage if desired. Otherwise, provisions for repair of the armor layer due to ice damage should be factored into the analysis of this design alternative. Regardless, this alternative will be less costly to implement and is the recommended alternative on this basis.

The CKD bluff currently sits from a little over 50 ft to about 120 ft from the current shoreline in Area 1. This provides an adequate area to provide protection for the toe of the CKD bluff while maintaining construction above the high water levels defined by the State of Michigan to be 582 ft and 582.5 ft by the Corps of Engineers. An additional advantage to remaining above these levels is that no permitting is required above the high water levels. An additional advantage is that breaking waves of only very low height can be supported even at the extreme high water levels and much smaller stone can be used for wave protection. In this approach, a revetment would be placed against the toe of the CKD bluff, generally at an elevation of 586 feet. This revetment would be extended lakeward 20-25 ft and could be composed of stone as small as 50-100 pounds since it would not be subjected to breaking wave attack. One or two rows of larger stone on the order of one ton could be placed on the lakeward side of the revetment to provide additional protection against ice forces. This stone would serve both to resist horizontal forces as well as to prevent ice ride-up further up the shoreline. The addition of this feature should depend on an analysis of the relative costs of repairing damage to the smaller armor stone layer versus the additional initial cost of this large stone. A filter beneath the stone should be integrated into the design to prevent wave runup from washing sand at the toe of the armor layer. Figure 6 provides a conceptual sketch of this alternative. Care may have to be taken to provide an adequate transition between this alternative and the existing revetment.

Other less costly alternatives may be implemented at the site but these will require more maintenance or attention to construction detail. A single layer of armor stone similar to that indicated Figure 6 should be sufficient to prevent wave runup to the CKD bluff provided that the stone is placed above the high water level. That is, the protective layer behind the stone could be deleted provided that the stone is placed carefully to prevent significant amounts of wave runup past the stone. The underlayers described above would still be required to stabilize the stone. Past experience with large armor units placed on a sand beach indicates that the stone will gradually sink into the underlying sand under the action of incident waves on the base material. This condition would be expected in the Area 1 environment if an underlayer/filter fabric base is not provided. This problem would be expected at high water levels only since wave runup under the current water level would probably not reach the armor units. Another alternative would be to just install the armor stone defense without underlayers and perform maintenance by replacing armor stone where and when required.

Area 2

The near shore for Area 2 consists of a fairly flat area that is currently covered by chips of CKD both on- and offshore. Images of the shoreline are indicated in Figures 2-5. Onshore, the elevation only increases 2.5 ft in 100-200 ft of horizontal distance with similar offshore decreases. The definition of the shoreline location is even difficult due to the flat slope. The depth of the CKD exceeds ten feet in some locations. The choice of shore protection system will depend to some extent on decisions made as to any removal of the CKD; if a significant volume of CKD were to be excavated, the nature of the shoreline would be substantially altered unless fill were brought in to restore the current grade.

Assuming that the waterline is not materially changed and that the current relatively flat slopes remain, a riprap cover over the existing ground surface could provide adequate erosion protection. Using the Isbash equation as outline in the 1984 Shore Protection Manual with the design breaking wave height determined above yields a graded riprap with a median weight of 60 pounds. This assumes a uniform velocity under a gradually breaking wave, this is consistent with the incident waves that could impact this area. The stone size can be increased to 100 pounds or more to provide an additional factor of safety without materially affecting costs. Design concerns would default to ice forces and a lakeward barrier of one ton stone should provide the necessary protection against ice forces as well as to eliminate the possibility of ice ride-up onto the smaller stones behind. The barrier row would be more important in Area 2 than in Area 1 due to the fact that the cover layer would be underwater under moderate to high water levels. The top elevation of the barrier stone should be on the order of 582 ft such that it would only be just submerged at the extreme high water levels. This should avoid any navigation hazards. Figure 7 presents a conceptual sketch of this type of shore protection. It is understood that some sort of liner material will be required to limit contact between the CKD and lake water and this can be integrated into the design to provide the primary filter to prevent erosion of sediments beneath the armor stone.

Area 3

Area 3 apparently has only surface CKD that has been transported from updrift locations. Therefore it does not appear necessary to provide shoreline protection in this area. However, it may be appropriate to perform CKD removal activities in this area and it may also be desirable to provide protection from further transport into this area. From inspections at the site, the local wave climate and known prevailing directions for littoral drift on Lake Huron, the direction of prevailing sediment transport along the general section of Lake Huron shoreline is south to north. This is also consistent with the observations on-site. Considering that the point just recently protected with a revetment has served as a primary source for CKD material, the location of the CKD chips along the shoreline in Area 2 are consistent with transport from this area. Areas 2 and 3 should experience continued transport of the in-lake CKD even after stabilization of the existing shoreline. The area offshore of Area 2 is apparently composed of cemented CKD to a few hundred feet offshore so there does not appear to be much that can be done to prevent continued contact with the CKD short of dredging the entire offshore area. Area 3 however, has only surficial CKD and if removal activities are performed, could be protected by a rock groin extending offshore somewhere near the "peninsula" separating Area 2 and Area 3. The purpose of this groin would be twofold; first to prevent recontamination of Area 3 and second, to provide a CKD "trap" to capture CKD transported along the shoreline that can then be periodically removed on the updrift side of the groin. The groin should be permeable in order to prevent the development of a strong rip current on the updrift face, but that captures the bulk of the CKD material; a stone groin would serve this purpose. Figure 8 provides a conceptual sketch of the cross-section of a stone groin. Since loose CKD materials are currently observed offshore of the existing shoreline, the groin should be extended lakeward of the LWD in order to be effective. It is probably not practical to place large enough stone to be stable in all situations and keep the groin to a low profile. Therefore, it is recommended to utilize a smaller stone size, on the order of 100-200 pounds and to consider the prospect that occasional repair of the groin will be required. If dredging is also periodically performed to remove accumulated CKD, the groin inspection and repair, if necessary, can be incorporated into an ongoing maintenance program.

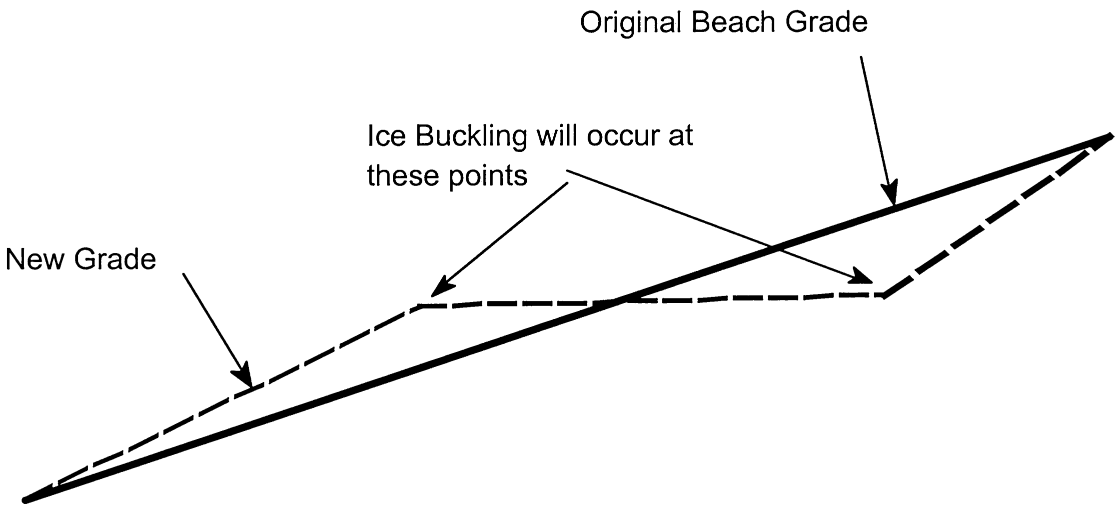


Figure 1. Alteration of plane beach profile to prevent ice ride-up by introduction of grade changes to promote buckling failure of ice.

Figure 2. Shoreline in Area 1 looking towards the southwest from near the existing revetment.

Figure 3. Northeast end of existing revetment with CKD chips on shoreline in front of structure.

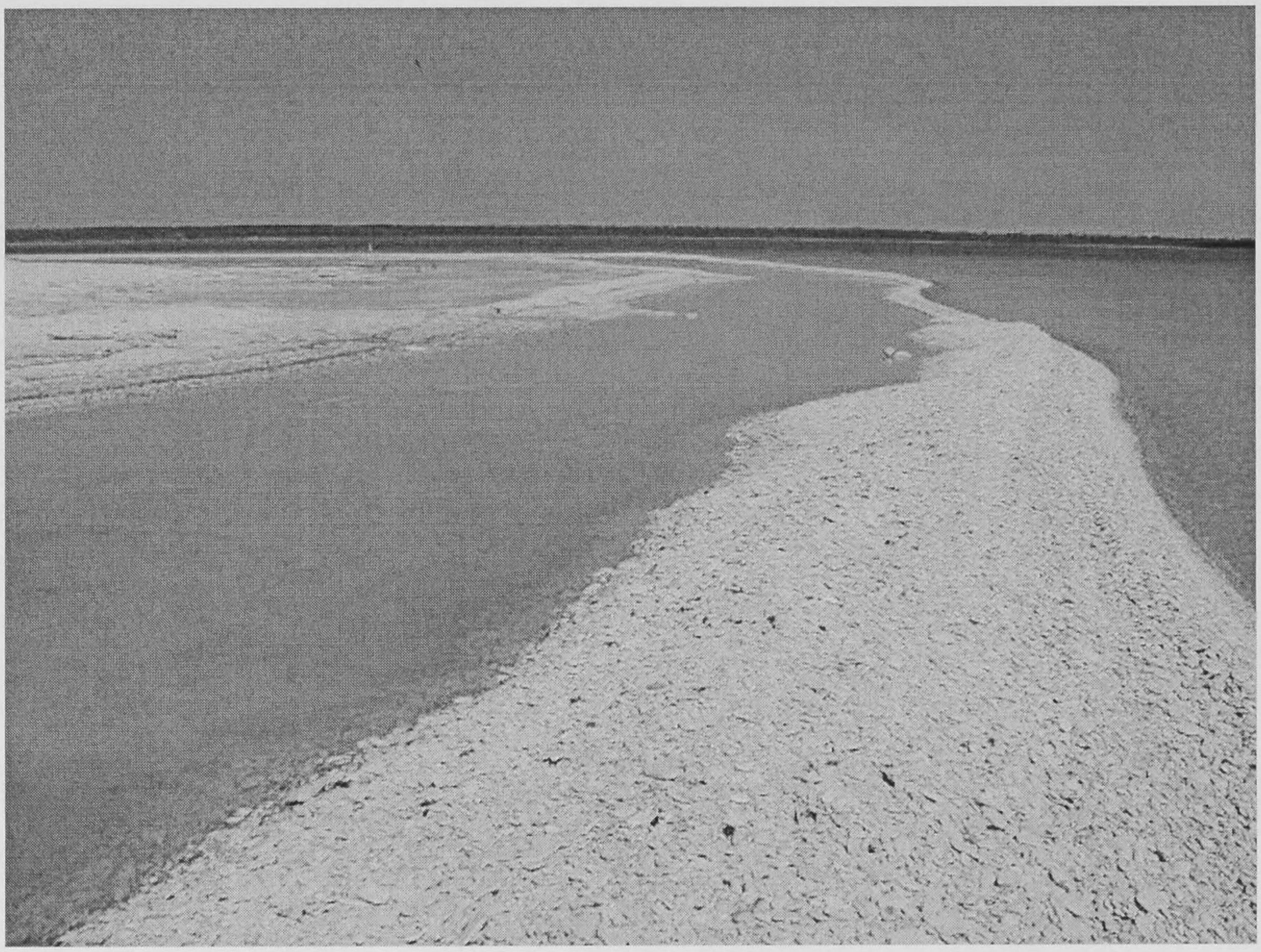


Figure 4. Shoreline in Area 2 showing surfical loose CKD deposits.

Figure 5. Underwater CKD deposits offshore in Area 2.

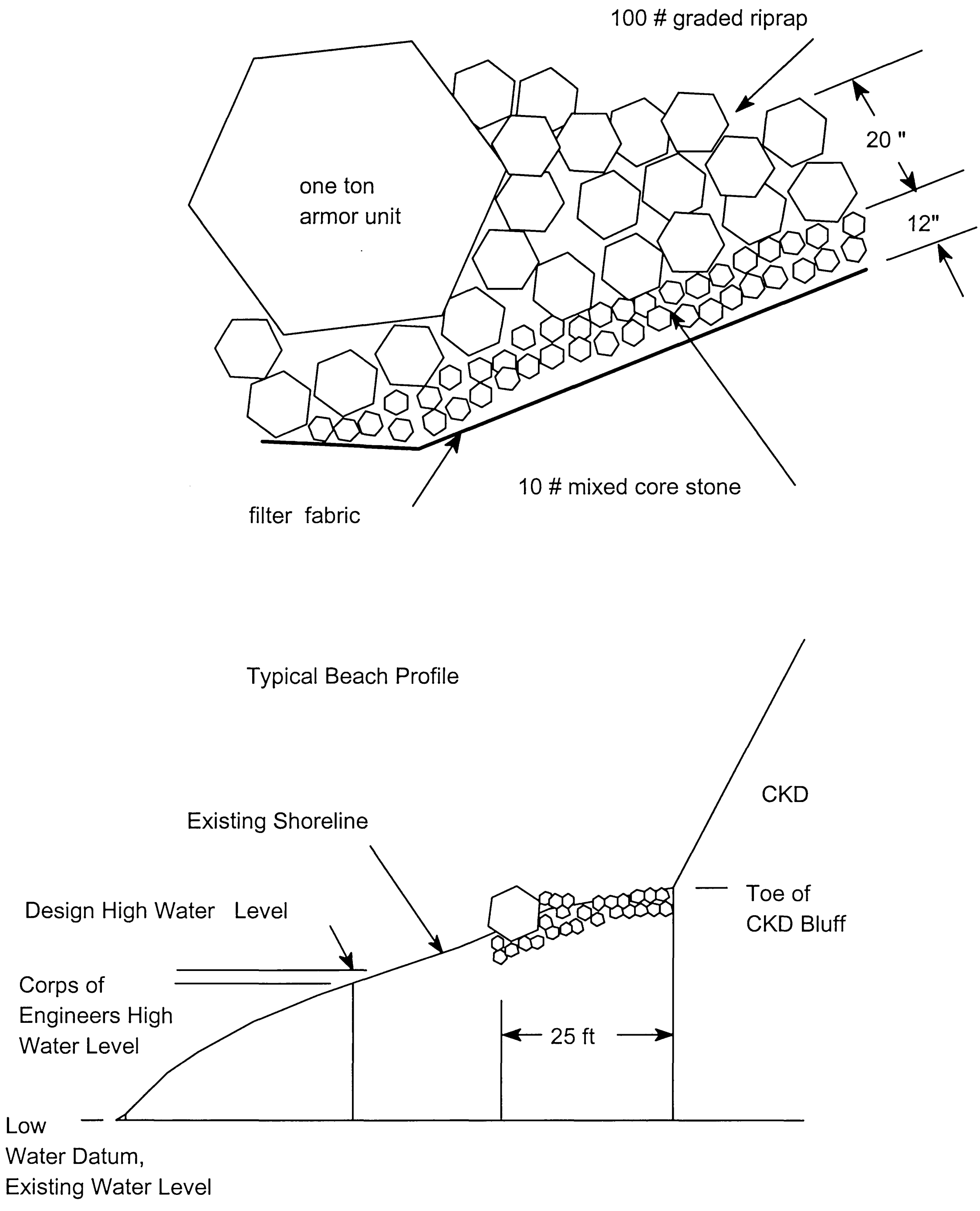


Figure 6. Schematic of recommended shore protection system, Area 1.

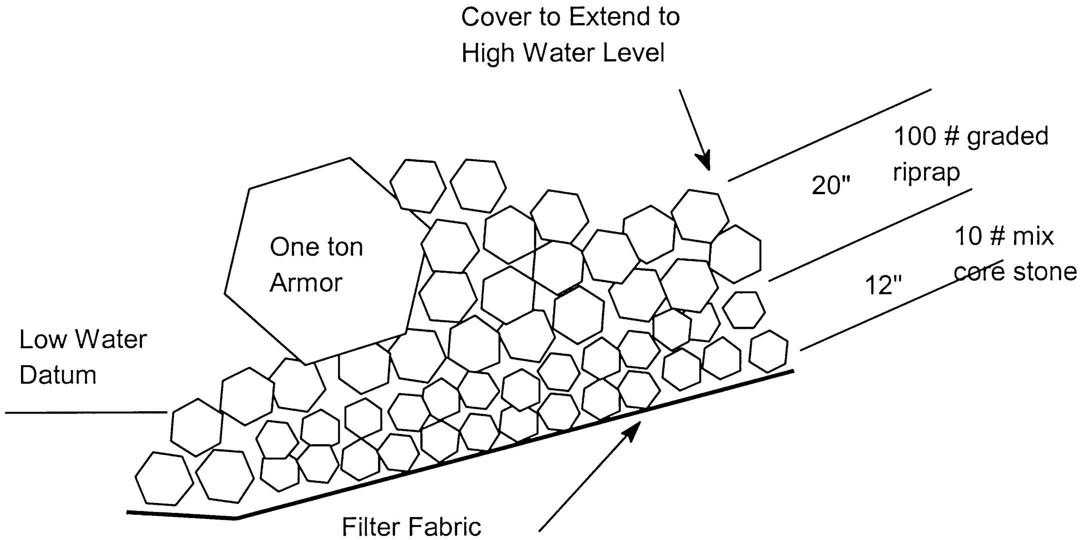


Figure 7. Protective layer to cover shoreline in Area 2 from low water datum to high water level.

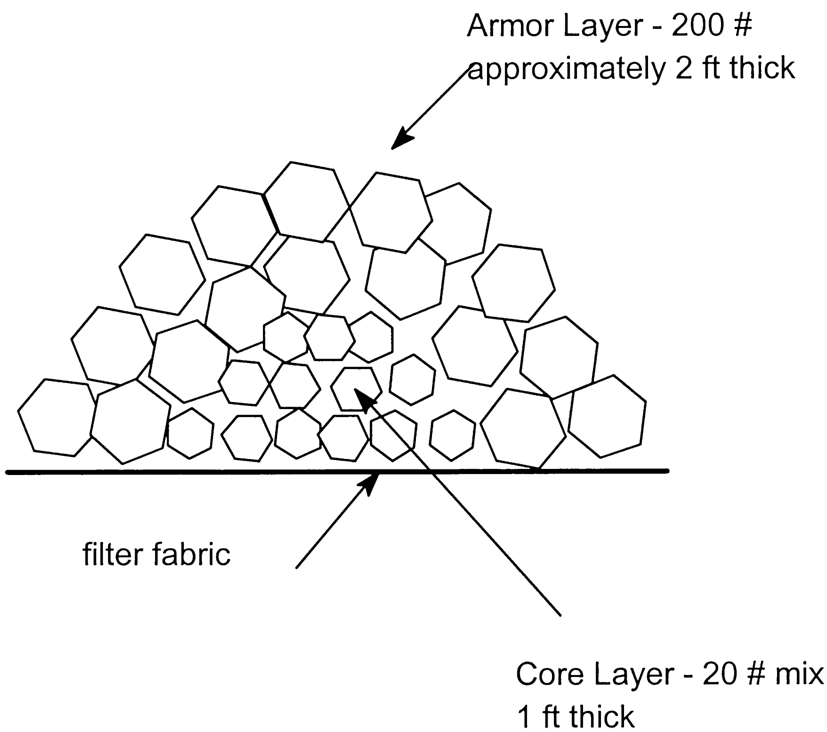
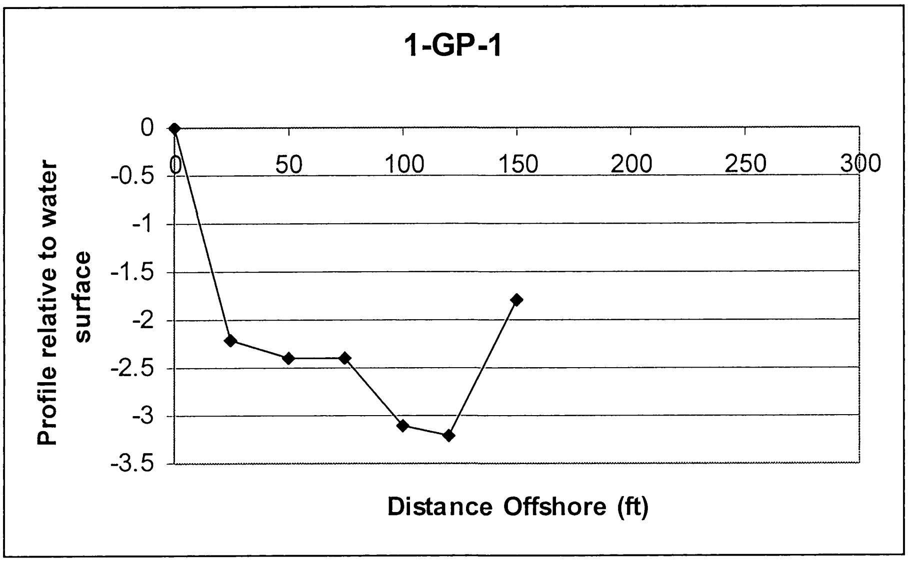
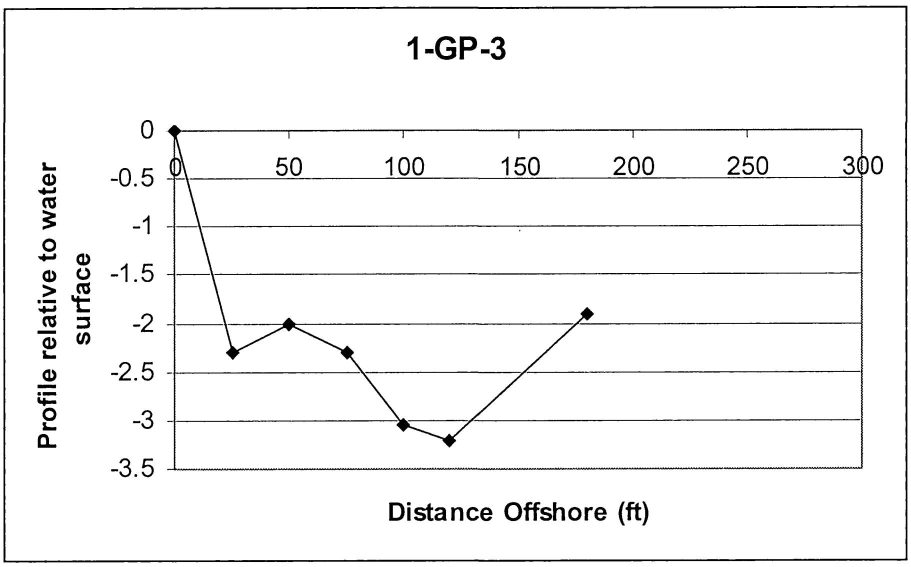


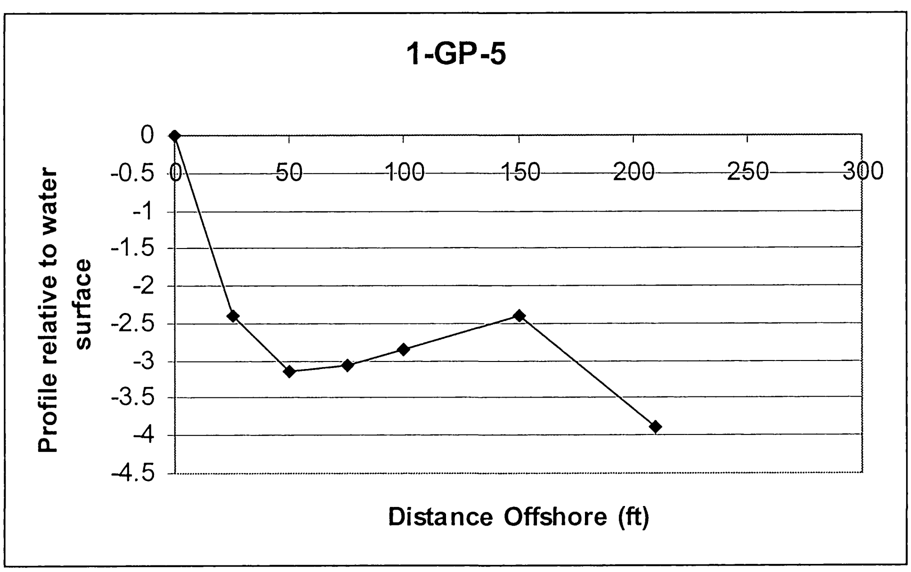
Figure 8. Cross-section of stone groin to extend from high water level to approximately 50 ft offshore of low water datum. The purpose is to intercept CKD materials transported along the shoreline.

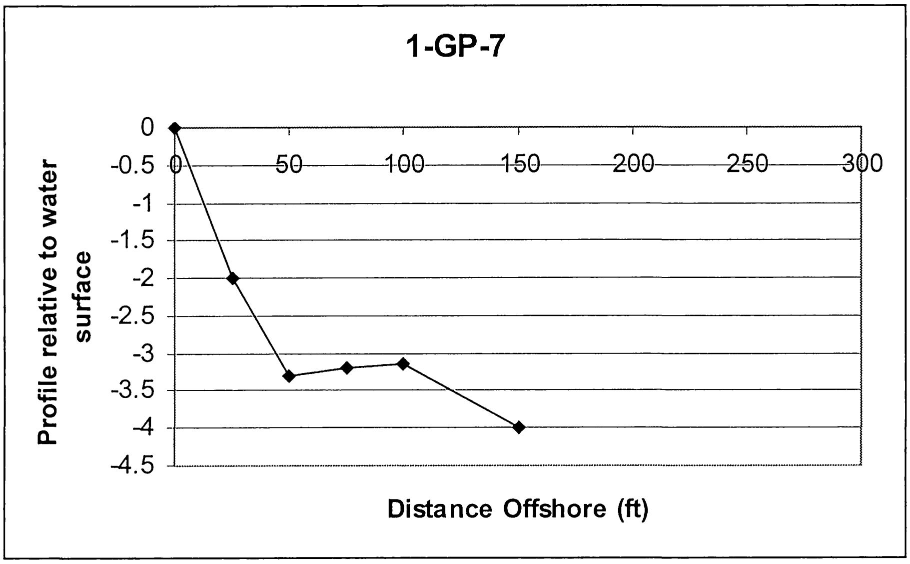
APPENDIX - OFFSHORE PROFILES IN AREAS 1-3

These profiles were measured August 7, 2001. The mean water level for Lake Huron was reported to be 577.6 ft on that date. The wave climate was minimal and water depths were estimated with a survey rod. Distances offshore were measured with a tape measure except for distances greater than 200 ft that were estimated. Distances are relative to the survey stakes used to identify the profile. In some instances a berm of CKD protruded above the free surface.

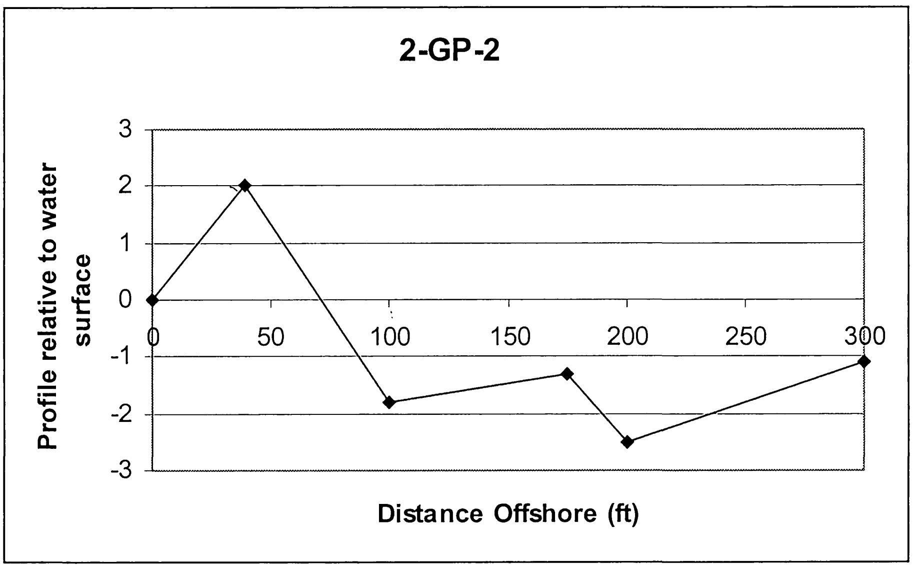

1-GP-1
Profile relative to water surface
Distance Offshore (ft)
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
0
50
100
150
200
250
300


1-GP-3
Profile relative to water surface
Distance Offshore (ft)
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
0
50
100
150
200
250
300

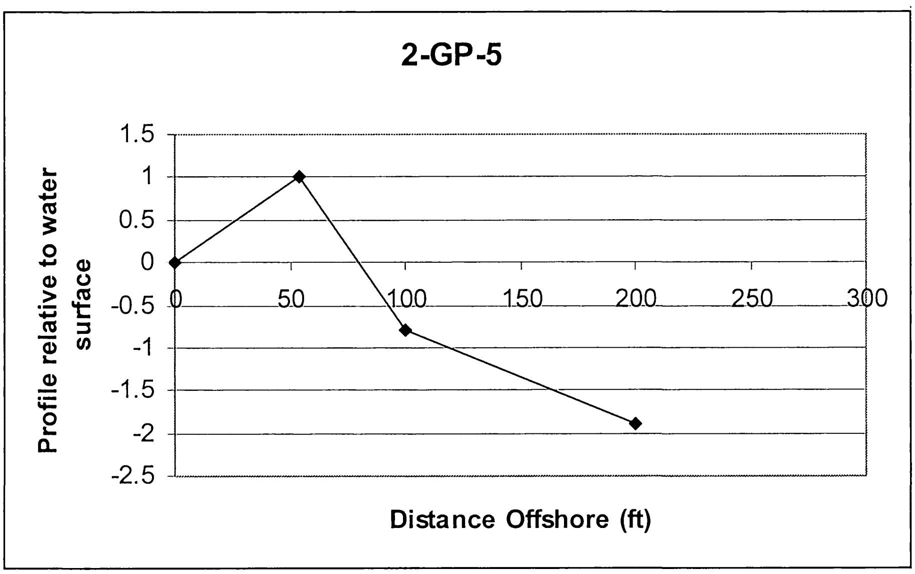

1-GP-5
Profile relative to water surface
Distance Offshore (ft)
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
0
50
100
150
200
250
300



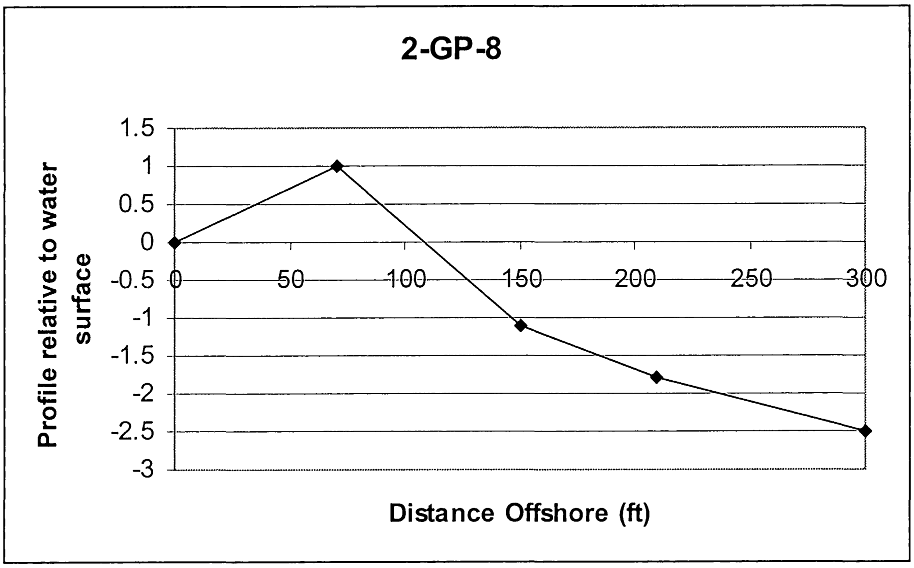
1-GP-7
Profile relative to water surface
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
0
50
100
150
200
250
300
Distance Offshore (ft)

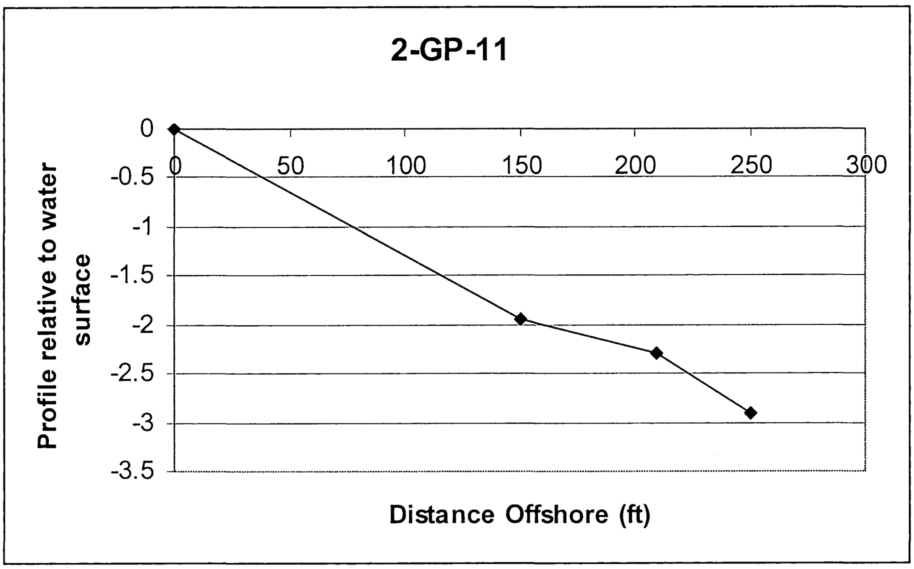


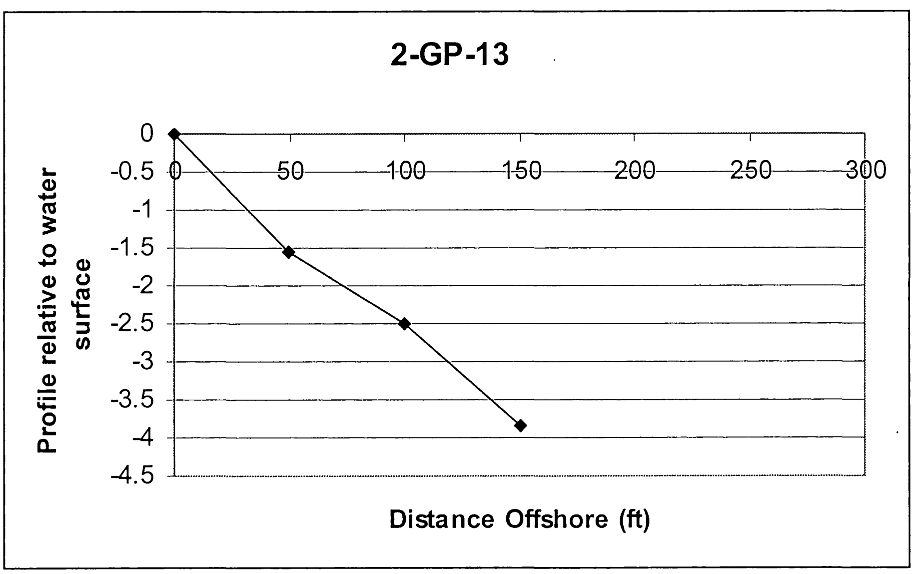
2-GP-2
Profile relative to water surface
3
2
1
0
-1
-2
-3
0
50
100
150
200
250
300
Distance Offshore (ft)

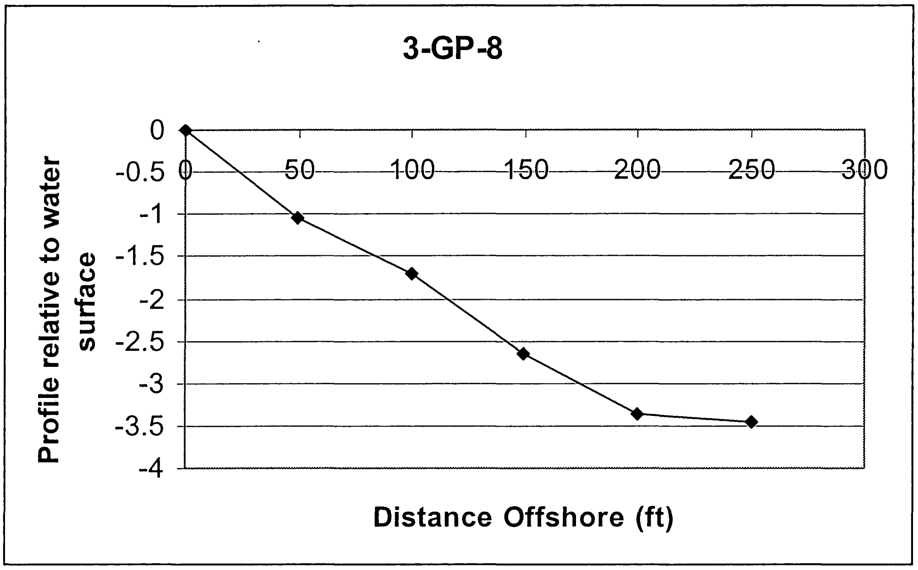


2-GP-5
Profile relative to water surface
1.5
1
0.5
0
-0.5
-1
-1.5
-2
-2.5
0
50
100
150
200
250
300
Distance Offshore (ft)


2-GP-8
Profile relative to water surface
1.5
1
0.5
0
-0.5
-1
-1.5
-2
-2.5
-3
0
50
100
150
200
250
300
Distance Offshore (ft)


2-GP-11
Profile relative to water surface
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
0
50
100
150
200
250
300
Distance Offshore (ft)


2-GP-13
Profile relative to water surface
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5
0
50
100
150
200
250
300
Distance Offshore (ft)


3-GP-8
Profile relative to water surface
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
0
50
100
150
200
250
300
Distance Offshore (ft)

AIIM SCANNER TEST CHART # 2

Spectra

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Times Roman

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Century Schoolbook Bold

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

News Gothic Bold Reversed

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Bodoni Italic

4 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
6 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
8 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789
10 PT ABCDEFGHIJKLMNOPQRSTUVWXYZabcdefghijklmnopqrstuvwxyz;:",./?$0123456789

Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋡⋠⋗⋖≡
6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋡⋠⋗⋖≡
8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋡⋠⋗⋖≡
10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⋡⋠⋗⋖≡

White

Black

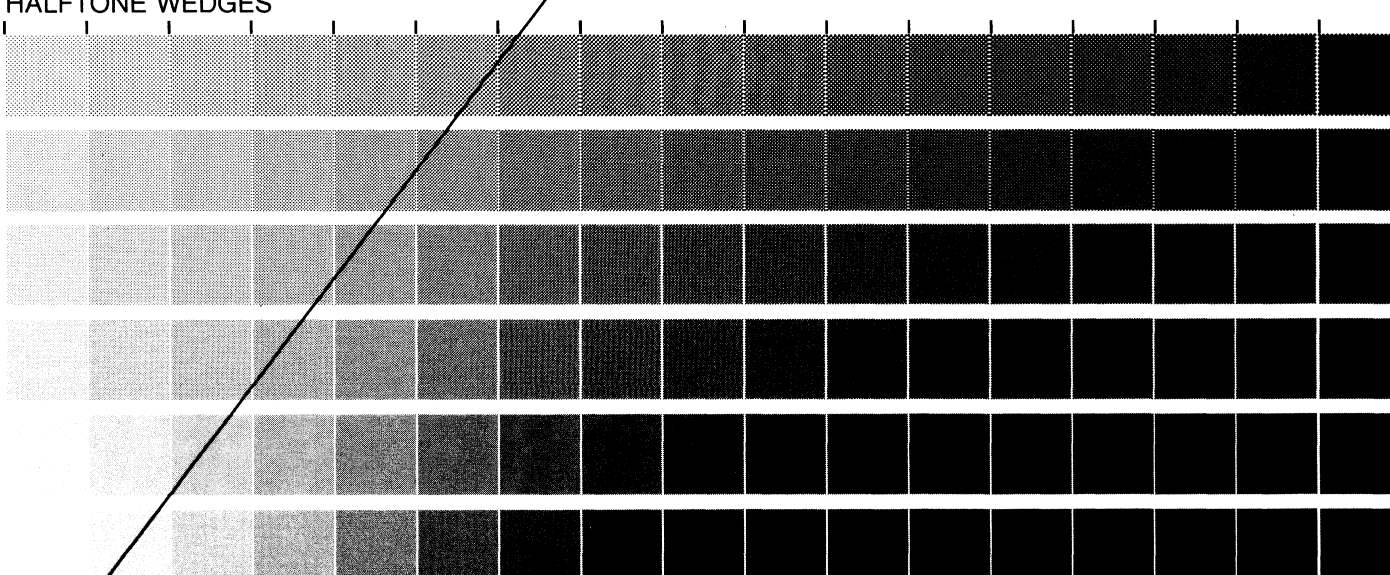
Isolated Characters

e	m	1	2	3	a
4	5	6	7	o	•
8	9	0	h	l	B

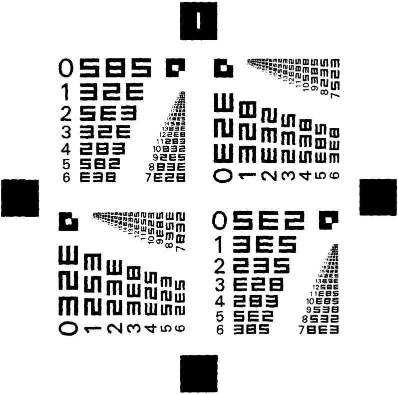
0123456
A4 Page 6543210

MESH HALFTONE WEDGES

65
85
100
110
133
150



RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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