Landslide Cases in the Great Lakes: Issues and Approaches

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A review of the experience gained over a decade of studying the mechanics of coastal bluff erosion and stability along the Great Lakes shorelines and the approaches to dealing with this problem are presented. Shore recession affects planning, design, and maintenance of transportation facilities in coastal areas in a significant way. The erosional processes resulting in significant mass wasting include wave erosion, solifluction, rain impact and rill-sheet erosion, wind erosion, sapping, and ice erosion. Another important process is mass sliding and slumping of bluff materials in response to and in conjunction with the erosional processes. The methods of approach to this problem are grouped as structural (stabilization) and nonstructural (planning and management) solutions. These approaches and the required information to implement either of them are discussed. Because of the length of the slopes and the variable nature of geology and soil properties, a probabilistic approach to stability analysis has been adopted for planning and managing the stabilization efforts. Two specific cases, one in an undeveloped segment and another in an urban segment of the shoreline, are presented to demonstrate the stabilization approach.

This paper presents a review of the experience the author has gained over a decade of studying the mechanics of coastal bluff erosion and stability along the Great Lakes shorelines. Nearly 65 percent (10 444 km) of the 16 047-km-long Great Lakes shoreline is designated as having significant erosion; about 5.4 percent (860 km) of it is critical. The total damage to the U.S. shoreline of the Great Lakes due to wave action during the high-lake-level period, May 1951 through April 1952, is placed at about $50,000,000 (1952 price level). Nearly 32 percent of the U.S. shoreline of the Great Lakes, not including the islands, consists of erodible bluffs. Extensiveness of the shoreline formed in erodible bluffs and dunes (an often complex response of this type of shoreline) to wave erosion makes slope processes an important part of the shore recession problem. The shore recession, in turn, affects the planning, design, and maintenance of transportation facilities in coastal areas in a significant way. The coastal bluff processes are briefly described, and the methods of approach to this problem are presented along with two specific cases.

SLOPE PROCESSES

The interaction of driving forces (gravity and climate) and soil shear resistance results in a number of processes that lead to debris production and removal. The commonly encountered processes in the Great Lakes coastal bluffs can be separated into two broad groups, mass and particle movements. In the mass group, debris begins to move as a coherent unit (rigid body movement or viscous flow). Movements in which particles move individually, with little or no relation to their neighbors, are particle movements. The erosional processes caused by waves, currents, rain, groundwater, and winds seem to be mainly particle movements. These concepts are presented in Figure 1.

Wave Erosion

Probably the most significant geomorphic process along the Great Lakes shoreline is the erosion and removal of shoreline materials by waves. Wave action is important, both in itself and in initiating and perpetuating other geomorphic processes in segments of the shoreline where bluffs are encountered. The most notable factor that affects the wave erosion in the Great Lakes is water-level fluctuation. For the long-term water level changes, the intervals vary from 10 to 30 years and the magnitudes are up to 2 m.

Sliding and Slumping

Slides (both rotational and translational) and flows (including solifluction) are the two types of movements most commonly encountered in the Great Lakes region. Coastal bluffs are in constant evolution because of the combined effects of toe erosion, slides, and face degradation.

Edil and Vallejo (*1*) described bluff stability at two sites on the shore of Lake Michigan. Where unexpected stability occurred, it could be explained in a rational manner by the process of delayed failure that results from the unloading of clays by erosion. Barring the presence of gross inhomogeneities, rotational slides involving approximately circular rupture surfaces have been observed and analyzed in the Great Lakes bluffs formed in cohesive soils (*1–3*). Deep-seated rotational slips occur in clay soils but are not observed in sands. One method of analysis of rotational slides that is accurate for most purposes is that advanced by Bishop (*4*). The failure arc predicted by the Bishop method has been found to compare very well with the actual failure surfaces in bluffs in the Great Lakes and other places. Using the effective stress approach and the Bishop method, Vallejo and Edil (*5*) developed stability charts for rapid evaluation of the state of stability of actively evolving Great Lakes coastal slopes. These charts indicate the stability status as well as the type of potential failure, whether deep or shallow, to which the bluffs may

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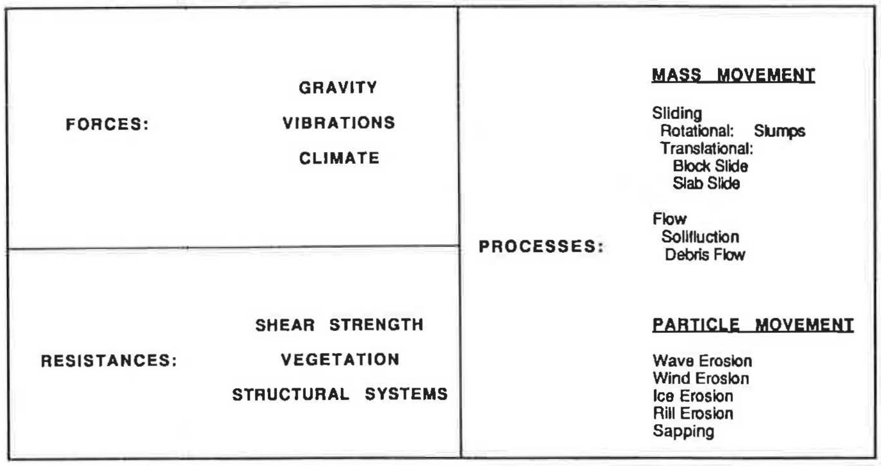

FIGURE 1 Forces, resistances, and slope processes in Great Lakes.

be subjected. The geometric changes can be discerned from the stability charts.

A translational slide in which the moving mass consists of a single unit that is not greatly deformed, or a few closely related units, may be called a block slide. An example of such a failure involving a block of fractured till in the upper part of a coastal bluff in Milwaukee County, Wisconsin, was reported by Sterrett and Edil (*6*). Translational slips can also occur in a homogeneous soil mass. In particular, granular materials such as sand and gravel fail in surface raveling and shallow slides with the failure surface parallel to the slope surface. Similar failures occur in a mantle of weathered or colluvial (granulated) material from clay slopes and are referred to as slab slides. An infinite slope analysis is often representative of such failures. Sterrett (*7*) reported slab slides with a depth of about 6.0 m from Milwaukee County. This depth coincided closely with the depth of desiccation cracking and soil structure change from fine to prismatic beds to massive intact blocks. Sterrett (*7*) also observed that frozen slabs of soil measuring 0.6 m × 10 m × 13 m failed in early spring, and attributed this failure to differential melting of the bluff face.

Flows and Solifluction

Flows commonly result from unusually heavy precipitation, thaw of snow, or frozen soil. The flows observed in the Great Lakes bluffs take place mostly in spring and result primarily from ground thawing and snow and ice melting. Therefore, they can be classified largely as solifluction. Solifluction is the downslope movement of water-saturated materials that follows thawing in previously frozen slopes. Vegetation appears to be the most restraining factor for solifluction. The size of the flows along the western Lake Michigan shoreline varies from 0.3 to 0.6 m wide up to 15 to 20 m wide and 21 m long. A number of approaches for the analysis of solifluction failures have been suggested. Vallejo (*8*) introduced a new approach to the analysis of solifluction that reflects the particulate structure of the flowing mass. Vallejo and Edil (*9*) applied this analysis, with successful field verification, to a coastal bluff in Kewaukee, Wisconsin. The critical depth of thaw normal to the slop face at which failure occurred was measured to be 0.25 m.

Rain Impact, Rill and Sheet Erosion, Sapping, Wind, and Ice Erosion

These processes are also important in general mass wasting that occurs in the exposed coastal slopes of the Great Lakes. In slopes formed in granular material, these processes are dominant. A description of these processes given by Sterrett (*7*) determined on the basis of field observations that most of the material removed from the slopes during summer is by way of sheet-wash and rill erosion. Sterrett (*7*) found that the universal soil-loss equation, in its modified form as suggested by Foster and Wischmeier (*10*), is useful in predicting soil loss from steep slopes.

STRATEGIES FOR DEALING WITH ACTIVELY EVOLVING SLOPES

The most significant characteristic of the coastal slopes in many areas of the Great Lakes shoreline is the fact that they are actively evolving natural slopes—the slope geometry continually changes. This characteristic sets these slopes apart from other natural slopes in terms of stabilization approaches. There are basically two approaches to the problem of actively evolving coastal slopes. The first approach involves structural or stabilization solutions on a site-specific basis. The structural approach, with some additional considerations, is similar to other natural slope stabilization efforts. A proper stabilization program should include (a) protection against wave action in all cases, (b) slope stabilization against deep slips if needed (important in the delayed instability often observed in bluffs formed in stiff clay soils), and (c) stabilization against face degradation and shallow slips. Shore protection is a major component and may be more costly than the slope stabilization. The problems associated with the execution of this category of solutions are of two types: (a) many attempts are not engineered and fail to cope with the problems, and (b) those engineered solutions often neglect to consider all aspects of the problem as described. For stabilization works, site-specific studies are undertaken at numerous locations. In these studies, an attempt is made to identify and understand slope-stability problems at a single site over a relatively short period of time.

The second approach, the nonstructural planning and management approach, is particularly suitable in undeveloped tablelands where hazard mitigation to transportation facilities can be planned and managed over an extensive part of the shoreline (the size of a county or at least several kilometers are usually considered). These studies are usually aimed at minimizing future losses. In this case the need for understanding bluff processes is critical because predictions of future recession over a long period of time with changing water-level and climate conditions are necessary. This approach necessitates models of bluff evolution (*1,11*). The main problem concerning prediction of slope evolution is understanding the response times to environmental changes and the time necessary for bluffs to pass through an evolutionary sequence. Evidence from other areas with evolving slopes such as riverbanks and marine coasts as well as from the Great Lakes suggests that there are possibly three time scales over which the natural cycles of evolution take place: 2 to 3 years, 50 to 100 years, and thousands of years.

The main tool used in the nonstructural or management approach is the establishment of a setback requirement for new transportation facility development. This requires a knowledge of coastal recession over a long time—say, 30 to 50 years—and the determination of stable slope angles. Typically, historical aerial photographs are used to establish the recession rates and geological and geotechnical analyses are used to determine the stable slope angles. Research conducted primarily during the last decade or two has identified the operating processes and their possible magnitudes (*12*). A nonstructural setback distance can be estimated as shown in Figure 2 (*13*). Setback distance consists of two components. Erosion risk distance is the distance from the existing bluff edge that could be affected by recession of the bluff over time

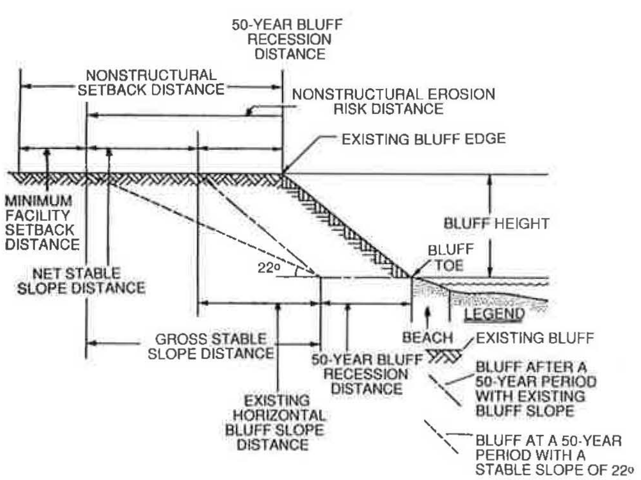

FIGURE 2 Procedure for estimating nonstructural setback distances (*13*).

and by the regrading of the bluff to a stable slope angle. Minimum facility setback distance is considered to be an additional safety zone intended to prevent facilities from being placed too close to the bluff edge. The solution strategies for active coastal slopes are given in Table 1.

TWO CASE HISTORIES

In this section slope conditions and stabilization strategies at two specific sites along the southwestern Lake Superior and the western Lake Michigan shorelines are presented, respectively.

Madigan Beach—Lake Superior

The bluffs of this site are reached by a 4-km secondary road that is directed northeastward of U.S. State Highway 2, approximately 25 km east of Ashland, Wisconsin. The site is located 610 m west of the mouth of Morrison Creek in the Bad River Indian Reservation. This site provides an example of a situation in which the main highway is quite far from the shore for immediate threat and the upland is undeveloped. The shoreline profile at the Madigan Beach site consists of bluffs rising 18 m above the beach and maintaining temporary steep inclinations in excess of 40 degrees with the horizontal. The processes of undercutting and slumping are evident and the bluff faces are mostly exposed because there are no vegetation and trees, which are dominant in the upland. Geologically, northwestern Wisconsin, where Madigan Beach slopes are, is a glaciated area.

The borings and observations of the materials exposed on the bluff face indicated the presence of a 4.5- to 6-m-thick reddish-brown stiff, silty, clay layer of low plasticity on the top, underlain by a thick (more than 12 m), very dense, brown sandy silt or silty sand. The geotechnical properties of the bluff materials are presented in Table 2. This highly erodible (cohesionless) sandy silt makes up most of the bluff material and is underlain by a reddish brown, rather stiff clay layer of high plasticity, mostly below the lake level. A mineralogical analysis of this lower clay layer revealed the presence of quartz, illite, kaolinite, and a small quantity of montmorillonite. The difference in the plasticities of the upper and lower clay layers appears to stem primarily from the difference in their clay fractions (26 and 63 percent, respectively) rather than from a mineralogical difference.

Madigan Beach appears to be subject to severe climatological forces and this, coupled with the erodible (cohesionless) materials forming the bulk of the bluffs, results in a highly active environment for slope evolution. Photoreconnaissance surveys conducted periodically since 1974 have revealed the action of bluff face degradational processes such as sheet wash, solifluction, seepage effects, and also the dominant action of waves. The sandy silt material of the bluffs, while highly erodible under surface processes, is strong below the surface when it is confined (the effective angle of internal friction is 37 degrees). This makes the bluffs highly stable against immediate rotational slips, and they sustain fairly steep inclinations, in excess of 40 degrees, in many parts of the shoreline. This situation is helped by the presence of clay layers some 6 m thick that cap the top of the bluffs.

TABLE 1 SUMMARY OF STRATEGIES FOR COASTAL BLUFF RECESSION

PROCESSES	SOLUTIONS	
	STRUCTURAL (STABILIZATION): DESIGN	NONSTRUCTURAL (MANAGEMENT): PREDICTION
TOE EROSION	SHORE PROTECTION (groins, seawalls, etc.)	SHORE RECESSION RATE (long-term, cyclic)
DEEP ROTATIONAL SLIPS	SLOPE STABILIZATION (regrading, dewatering, etc.)	STABLE SLOPE ANGLE AGAINST DEEP SLIPS
FACE DEGRADATION AND SHALLOW SLIPS	SURFACE PROTECTION (vegetation, drainage)	ULTIMATE ANGLE OF STABILITY

TABLE 2 GEOTECHNICAL PROPERTIES OF SOIL SAMPLES FROM MADIGAN BEACH BOREHOLES

Soil Description and Depth	Group Symbol*	Texture** %Sand	%Silt	%Clay	Liquid Limit %	Plasticity Index	Water Content %	Unit Weight (kN/m³)	Average+ N	Angle ++ of Internal friction ϕ	Cohesion++ Intercept C (kPa)
Reddish brown, stiff silty clay of low plasticity (Upper clay, 0-6m)	CL	10	64	26	31	14	18	21.4	18	19°	80
Brown, very dense sandy silt or silty sand (6-16m)	ML or SM	7-18	78-90	3-4	---	---	18	19.9	>100	37°	0
Reddish brown, rather stiff clay of high plasticity (Lower clay 16m)	CH	11	26	63	51	31	31	19.2	14	21°	0

* According to the Unified Soil Classification (ASTM Designation: D 2487-69 and D 2488-69).

**4.76 mm > Sand > 0.074 mm; 0.074 mm > Silt > 0.002 mm; 0.002 mm > Clay

+ Standard Penetration Resistance, N(ASTM Designation: D 1586-67).

++Drained Shear Strength Parameters.

To monitor the changes in slope morphology, a number of cross sections (perpendicular to the shoreline) have been periodically surveyed. The changes in most of the cross sections were indicative of face degradational processes without any deep rotational slips. The bluff top recession, measured using 1976 and 1978 aerial photographs, indicated a variation of recession on the order of 3 to 11 m during this period. In 1977, a shore protection demonstration project was initiated, which involved the placement of longard tubes filled with sand in different configurations (*14*) along the beach, as shown in Figure 3. Longard tubes are constructed of a geotextile, and they come in various sizes. Those used at Madigan Beach were 1.75 m in dia and weighed about 4500 kg/m once they were hydraulically filled with sand. Four of the tubes were placed at the toe of the bluff parallel to the shoreline acting as a seawall and six 33-m-long tubes were placed perpendicular to the shoreline to act as groins. The recession rates measured in the field, as well as discerned from aerial photographs, indicated a significant reduction in the recession from the 1976–1977 values. It should be noted, however, that one of the sections experienced a large recession of 2.9 m during this period because of a deep slip.

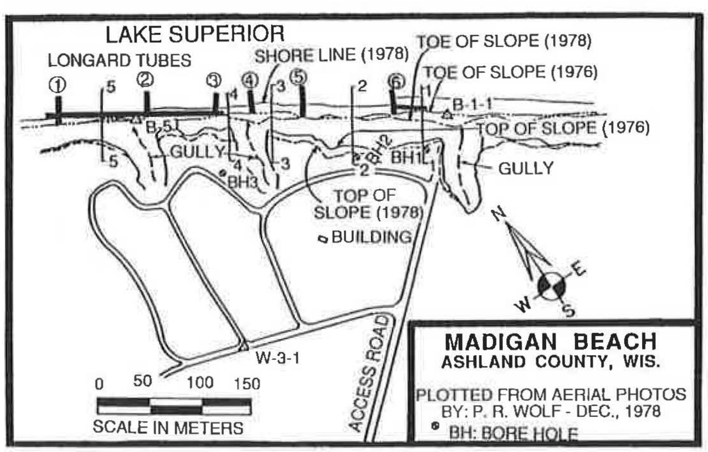

FIGURE 3 Longard tube layout and bluff conditions at Madigan Beach (number circled indicates tube location).

The stability of the bluffs was analyzed using the Bishop method in a conventional manner. The method used the effective stress analysis of slope stability, the drained strength parameters of the bluff materials, and the measured pore-water pressures as estimated from the piezometer readings. The stability analyses performed on the initially measured profiles of Cross-sections 1, 2, and 3 (Figure 3) resulted in safety factors of 1.36, 1.16, and 1.30, respectively, which indicated the general stability this segment of the site had against sliding. When surveyed in 1976, Cross-section 2 had already gone through a deep-seated major slide involving a 3.4-m drop of an 8.8m-wide section at the top. The initially surveyed profile of Cross-section 4 resulted in safety factors less than unity and were therefore unstable. A safety factor of less than unity implies potential instability (long-term) for a currently standing slope because the analysis performed is an effective stress (long-term stability) analysis, and it may also imply that some of the assumptions, such as zero pore pressure above the groundwater table, are too conservative. This bluff maintained its stability until the latter part of the summer of 1978, when slumping occurred, which resulted in the recession of the bluff top as marked by the approximate intersection of the predicted failure surface.

Another cross section, Cross-section 5, was also analyzed; it was found to have potential for slumping as shown by the failure surface in Figure 4. The bluff segment in this area was chosen for a slope stabilization demonstration. After considering several alternatives—including terracing, berms, and various combinations of these—it was decided to regrade the bluff to a uniform slope of about 25 degrees. On the basis of a similar analysis, this inclination was determined to be safe. The bluff stabilization demonstration included surface water diversion in the upland and seeding of the bluff face to prevent surface erosion. The project was completed in the fall of 1977. The toe of the bluff was protected with a longard tube that was placed parallel to the toe and another longard tube placed perpendicular to the shoreline (see Figure 3), which effectively built up the beach in front of the longard tube acting as a seawall. The six longard tubes that were placed as groins perpendicular to the shoreline were all destroyed by spring 1980. The other tubes that were put as seawalls, for the most part, lasted for a while and did provide some degree of protection. In 1984, these tubes were mostly deflated and buried in the beach sand, exposing the stabilized bluff section to renewed wave attack.

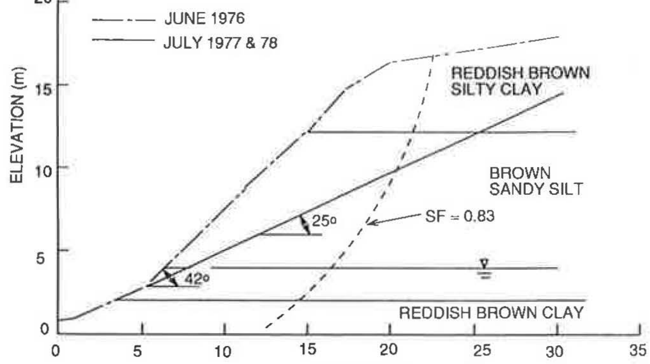

FIGURE 4 Bluff profiles in Cross-section 5.

Whitefish Bay—Lake Michigan

The topland in the case of these slopes is in a highly developed urban setting in a northern suburb of Milwaukee, Wisconsin. Within 50 m from the edge of the bluffs are expensive homes, behind which a major urban road passes. A 1983 landslide threatened six to eight homes when their front yards dropped nearly a meter. This coastal area is formed in glacial deposits ranging up to 40 m in thickness. The layer nearest to the surface is known as the Ozaukee Till, which consists of a silty clay with reddish color. The Oak Creek Till lies beneath the Ozaukee Till; it is composed of a pebbly silty clay. Lake sediments of various textural characteristics are often found between these two tills. Directly beneath the Oak Creek Till lies another very stiff till known as the New Berlin Till, which was deep below the lake level at this location. This till is coarse-grained, sandy in texture, and dominated by pebbles and cobbles. These formations are often exposed in the active slopes along the western shore of Lake Michigan. The stratigraphy was identified on the basis of field surveys, historic geological records of soil boring data, and new soil borings performed at selected sections of the shoreline.

The geotechnical properties of the glacial materials forming the bluffs were obtained from historic data and laboratory analysis of the samples collected from the exposed slope surface and soil borings. In terms of the known values and standard deviations based on the samples collected from different parts of the shoreline, the strength parameters and the unit weights of the bluff material are presented in Table 3.

Along the northern Milwaukee County shoreline, groundwater generally flows toward the lake and is discharged into the lake either at or below the base of the bluff. The lake sediments consisting of coarse-grained soils may act as water-bearing units. Additionally, a perched water table is usually found within the fractured Ozaukee Till at the top of the bluffs, which produces seeps on the bluff face. The groundwater elevations used in the slope stability analysis of each profile site were based on observed groundwater seepage, soil boring data, groundwater observation wells, and electrical resistivity analysis.

The area shoreline studied extending nearly 3.7 km, was considered to be a single section with similar physical and erosion-related characteristics. Field surveys were conducted to delineate the section boundaries and to identify the causes and types of beach erosion and slope failure occurring in the section. Unlike the earlier case, both conventional deterministic slope stability analyses and probabilistic soil stability analyses were conducted to establish the state of stability of this entire section.

Conventional Analyses

The worst-case groundwater conditions are assumed in the conventional deterministic analyses. Late winter and early

TABLE 3 GEOTECHNICAL PROPERTIES OF SOILS ENCOUNTERED IN SLOPES ALONG LAKE MICHIGAN'S MILWAUKEE COUNTY SHORELINE

Soil Unit	Unit Weight (kN/m^3)	Friction * Angle (Degrees)	Cohesion* Intercept (kPa)
Tills			
New Berlin	21.7	34 ± 3	0.5 ± 0.25
Oak Creek	21.2	30.5 ± 2	5.0 ± 3.75
Ozaukee	21.1	30 ± 3	7.5 ± 5.0
Fractured Ozaukee	21.1	30 ± 3	0 + 3
Lake Sediments			
Medium Fine Sand	18.9	33 ± 2	0 + 0.25
Sand and Gravel	18.9	33 ± 2	0 + 0.25
Silt	20.4	31 ± 2	200 ± 50
Silt and Fine Sand	17.3	31 ± 2	0.5 + 0.5
Clay and Silt	20.4	27 ± 2	22.5 ± 17.5
Fine Sand and Silt	19.7	33 ± 2	5.0 ± 3.75
Gen'l Lake Sediment	19.7	27 ± 3	5.0 ± 3.75
Fill			
Concrete Rubble/Soil	20.4	35	0

spring have been found to be the most critical periods for the stability of Lake Michigan coastal slopes for both deep-seated and shallow slides (*5*). The slope stability program named STABL (*15*) was used to calculate the safety factor for a slope profile measured in an adjacent segment of unfailed bluff within the study area. The lowest safety factor calculated was 0.95, and the corresponding failure surface was located in the middle portion of the bluff slope. However, a bluff segment in the study area had already failed in a rotational slide. The safety factor for this failed bluff was calculated to be 1.56 (*16*).

In the deterministic approach, mean values of soil properties based on the available test results in Table 2 were used in computing a safety factor against failure, along with the geometrical and geological measurements of the slope at the specific site. An assessment of the general stability of slopes of a shoreline section, based on the deterministic safety factors of a number of slope profiles located within the section, could conceivably be in error if the slope parameters chosen as input for the stability program are not reliable. For example, adequate information may be available at certain sections because of the excellent exposure of slope soil stratigraphy and the rather uniform characteristics of the slopes and soils within the section. At such locations, adequate information is available for what may be called a reliable analysis. However, for many other sections, between borehole locations and poorly exposed slope faces because of vegetative cover, soil properties, position of stratigraphic unit interfaces, and groundwater table in the slope have to be interpolated or estimated based on an examination of the general slope conditions. In view of the variability of soil properties and other slope parameters that have to be estimated, the computed safety factors would not be as reliable for these profiles as they would be for the profiles at the borehole locations and those with good exposure.

Probabilistic Analyses

The objective of the probabilistic analysis was to verify the results obtained from the deterministic slope stability analyses and to provide an assessment of overall slope stability within the entire section, rather than just specific profile sites. The probabilistic model used is based on the Monte Carlo method (*17–19*). As much as possible, the uncertainty and variability of the basic controlling geological, geotechnical, and environmental factors involved must be incorporated into the model. The significant variable parameters for this model included soil strength parameters, watertable elevation, and soil interface lines. Slope geometry for the slope profiles was measured in the field, representative of the general morphology of the slopes in the section; therefore, it was not considered to be a significant source of variability. The β-distribution was used to generate the variable parameter distributions. Mean values and standard deviations of the variable parameters and the limits of their variation were required to generate β-distributions of these parameters. The spread of these distributions was established on the basis of the measured or expected standard deviations of these randomly varying properties. By setting limits on the variation of the parameter, unreasonable possibilities such as negative cohesion are avoided.

The original STABL program was modified to permit the input of random variables (*19*). A random number generator randomly generates the values of the variables based on the given β-distribution of the variables. For each combination of these variables, a slope stability analysis is performed, based on the consideration of approximately 100 random failure surfaces. At least 20 such analyses are needed to provide an assessment of the risk of instability. Running more than 1,000 analyses does not seem to provide any additional information (*19*). The probabilistic stability analysis performed on the unfailed slope segment in the study area indicated that of the 200 analyses, 159, or 80 percent, gave minimum safety factors of less than 1.0. For the same slope profile, the deterministic safety factor was calculated to be 0.95, as indicated earlier.

Hazard Classification

A set of general guidelines was developed to classify the slope sections for rotational slides on the basis of their stability. The cumulative probability of safety factors being less than or equal to unity can be used as a way of assessing the overall landslide hazard potential. Southeastern Wisconsin Regional Planning Commission (*13*) adopted a hazard classification system in which a probability of more than 75 percent (75 percent of the safety factors less than 1) indicated unstable or high-hazard condition and less than 25 percent stable or low-hazard condition. On the basis of this classification, this section of Whitefish Bay was classified as unstable.

Stabilization Effort

A structural solution was pursued because of the immediate threat presented to the homes and the urban highway directly behind these slopes. This solution involved construction of a terraced berm using concrete demolishing debris. Use of a waste material reduced the cost significantly. The berm was constructed from the beach level up. Dumping materials from the slope top was avoided to prevent further movements of existing slump blocks and an access road was cut to the beach. The configuration of the berm is shown on Figure 5. The toe of the berm was protected against wave action by limestone riprap. A 0.3m-thick soil cover was placed on the berm surface to provide vegetation. A deterministic slope stability analysis was also performed using the profile after restoration as shown in Figure 5. This analysis indicates a stable bluff slope with respect to rotational sliding. The lowest safety factor calculated was 1.75, with a corresponding failure circle beneath the top portion of the fill layer.

SUMMARY

The processes that operate on coastal slopes often lead to significant mass wasting, instability, and recession of the slopes. Transportation facilities existing on the upland are threatened by the slope failures and shoreline recession. Stabilization strategies for long stretches of coastal slopes are described in terms of management or stabilization approaches. Because of the length of the slopes and the variable nature of geology and soil properties, a probabilistic approach to stability analysis has been adopted for planning and managing the stabilization efforts. Two case histories from the Great Lakes region were presented to demonstrate the low-cost stabilization approach. Once a hazard classification is established, a variety of methods can be used to stabilize unsafe or marginally unsafe coastal slopes. Potential slope stabilization measures commonly used include regrading the slope to a stable angle, installing groundwater drainage systems to lower the elevation of the groundwater and prevent groundwater seepage from the face of the bluff, constructing surface water control measures, and revegetating the slope.

The stable long-term inclination used in the cutbacks of the oversteepened bluffs of the western Lake Michigan shoreline is 22 degrees (1V:2.5H). The typical material used in slope stabilization and restoration fills is concrete rubble. Depending on the type of material used for filling, a steeper angle than the usual stable angle of 22 degrees (often approximating 35 degrees) may be used for portions of the filled bluff slopes. The restored slopes are normally terraced or contain compound slopes. Filling should begin at the slope bottom, and some bluffs may need to be filled only along the lower portions of the slope. Soil cover a minimum of 0.3 m thick is placed over the rubble fill, and seeding and mulching is required to develop a vegetable cover. Adequate toe protection is required for long-term stability. Usually a riprap revetment of rock or quarry stones is used for this purpose. Before the placement of the fill materials, trench drains are usually required to intercept and divert the groundwater of the water-bearing lacustrine deposits usually found at the mid-height of the bluffs.

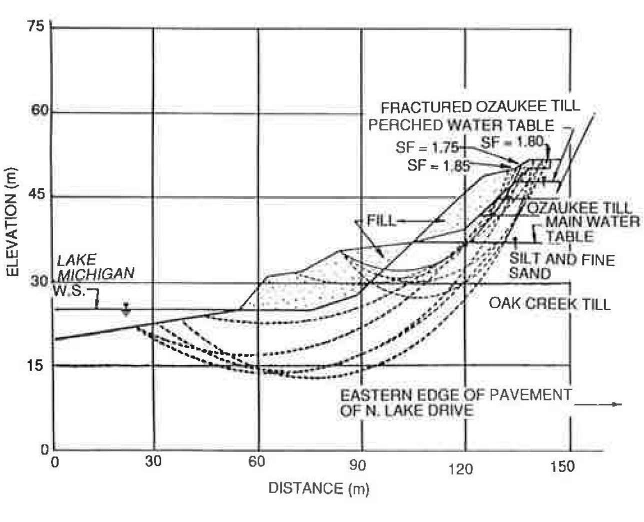

FIGURE 5 Bluff profile from Whitefish Bay (*13*).

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