ENGINEERING ANALYSIS OF LAKE ERIE BLUFF

EROSION PHENOMENA

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Presented in Partial Fulfillment of the Requirements
for the Degree Master of Science

By

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Robert Chieruzzi

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December, 1957
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I. INTRODUCTION

Although soils have been one of the most vital of our natural resources, man's knowledge of them for engineering purposes is still too inadequate to accurately predict their behavior under certain conditions of stress.

The fact that geomorphic processes, those physical and chemical actions responsible for the formation of soils, are themselves complex interactions is reason enough not to expect soil behavior to be otherwise. Continual modification of the earth's surface by these processes produces not only the ultimate degradation of the rock materials into individual soil grains but the landform resulting from their deposition as well. Eventually, the landform itself becomes a victim and the soil grains are again free to be transported by the same or different geologic agents and deposited in a manner in which totally different stress conditions are imposed. With these actions, variations in soil profile and structure are undoubtedly introduced, the extent depending on the number and kind of geologic agents present, as well as the magnitudes of their corresponding forces. It is certainly apparent, therefore, that a comprehensive evaluation of soil behavior involves knowing something about the effect of a number of complex actions, much as in the case of the rather unpredictable human behavior.
A good example of these complex interactions is bluff erosion, the subject of this thesis. Where once the combined aggrading manifestations of the glaciers and lakes resulted in the present soil formation, the combined degrading actions of the lake, groundwater, runoff and other erosional processes have now taken over, producing the disintegration and eventual disappearance of the bluff formation.

To some, this phenomena is still considered as an uncontrollable act of God. However, with the advent of a more quantitative approach in dealing with the behavior of soils, engineers know what measures are necessary to effectively combat this mass wasting of a most valuable natural resource. Unfortunately, the cost of such corrective actions are generally prohibitive, partly because the lack of a more comprehensive and precise understanding of soil behavior has necessitated the use of design criteria not necessarily congruent with engineering economy. With the increasing demand of lakefront property, engineering economic analyses, of course, will eventually appear to be more favorable. This will not, however, necessarily increase and stimulate more knowledge of these basic activities nor will it represent, any more so than now, the more economical approach for such corrective measures.
Many property owners, desperate in their efforts to maintain their land, have literally dumped millions of dollars into the lake without any measurable degree of success. Many forms of corrective measures have been attempted, even groins where sufficient beach forming material is lacking. The fact that most of the methods employed require only a nominal initial investment is inducement enough for the property owners to try them. In many cases, perhaps, the total spent through the years for such piece meal measures might far exceed that which would have been required initially for a much more permanent solution. Some of those who have unsuccessfully fought this battle for years are now convinced that more should be learned of the basic interactions before wildly embarking upon any more extensive remedial work.

Obviously, there is a logical need for a systematic study of the bluff erosion phenomena. Although a number of such studies (1)(2)* have been made by various governmental agencies, their individual scopes have been generally limited to the hydraulic aspects of bluff erosion. The underlying philosophy upon which such studies have been based is that an adequate beach in front of a bluff serves as the stabilizing factor necessary to maintaining equilibrium of the bluff materials, implying that the wave

*Numbers in parentheses refer to the list of References.
energy is expended upon the beach rather than at the bluff toe. Consequently, the majority of the work has been concerned with designing such structures as are necessary to develop adequate beaches or, where beach building is difficult, structures to withstand the direct wave attack.

Much emphasis has also been placed in the obtaining of geological data such as origin of the bluff materials, their mode of deposition and their movements. Little or no mention, however, is found in the literature concerning the more quantitative analyses of the strength properties of these materials and their capabilities to withstand the various erosional forces operating in the area. The fact that so little has been done along this line has furnished sufficient impetus for such a study.

The possibility of any two natural earth slopes or bluffs existing under identical stress-resistance conditions is considered quite remote. Not only are the number of variables subject to analysis quite numerous, but their corresponding actions become too complex to evaluate with sufficient accuracy to make valid comparisons. In spite of their apparent individualistic character, all slopes, however, have one most important engineering property in common, namely, shearing resistance. The very fact that a slope exists at all is an indication that some such resistance to the ever acting force of gravity must
by necessity be present. In fact, without it, all terrain would be quite featureless, quite similar to that which occurs for all bodies of water.

The magnitude of such shearing resistance is, of course, quite variable and is reflected to some degree in the steepness of the slopes. At any rate, it is a fundamental criteria that for a slope to remain stable, it must have sufficient shearing resistance to counteract the stresses imposed upon it. Whenever this state of equilibrium ceases to exist, it is logical to infer that some change is introduced into the stress-resistance relationship, decreasing the ratio of the resisting to the motivating forces to some value less than unity. Whatever the causes responsible for the change may be, the result produced is the same. In all cases, some deformation or movement of the soil mass occurs.

There is no obvious reason to suspect that the phenomena of bluff erosion operating today are materially different that they were at any time during their existence. This assumption is very much in accordance with the concept of uniformitarianism, a fundamental tenet of geomorphology expressed by Hutton in 1785. It says that the same physical and chemical processes and laws operating today have operated throughout geologic time, although not necessarily with the same intensity. This implies that bluff erosion
is a continual process, but also one of varying degree, the magnitude of which depends on the unbalanced forces. Applied to the universe as a whole, this concept is probably quite applicable, but for specific localized areas such as the Lake Erie shoreline, where all forces present during the glacial age are not now present, it is not totally true. Within a shorter time period, however, one comparable to that usually estimated as the life of engineered structures or slightly longer, the concept is more valid. For instance, the bluff erosion problem along Lake Erie was present as early as 1906 at least, if not longer. A survey (3) made in that year mentioned the fact that the reason for the survey was to establish the parts of a highway destroyed and injured by the washing and sliding of the land within said highway occasioned by natural drainage. Because no mention was made of any violent cataclysm producing the erosion of the highway, inference can be made that erosion then is similar to that present today, where a gradual but unceasing loss of soil is the case.

Bluff erosion is designated here as the disintegration of any portion of the bluff proper and its subsequent movement downwards and lakewards. Basically, it is the result of unbalanced force conditions about either an individual grain or an intact soil mass.
Responsible for these conditions are a number of mechanisms, both physical and chemical, whose modes of action tend to produce either a decrease in shearing resistance or an increase in the shearing stresses, or both. Included among those processes associated with a decrease in shearing resistance are the following: leaching, increased moisture content, frost action, rain impact, wetting and drying, expansion and contraction, oxidation, etc.

Increased motivating forces, producing increased shear stresses, are a result of the following more important mechanisms: seepage forces, removal at toe and undercutting by wave action, rain impact, concentrated and sheet runoff, frost action, and activities of man such as removal of material at the toe, steepening of the slope, or loading at the top. The force of gravity, of course, is always present and is assumed to be of a constant magnitude.

Since most of these mechanisms are subject to considerable variation because of environmental conditions, the corresponding effects on stability will vary accordingly. To pin the full responsibility for failure on a specific mechanism is often the tendency but it usually is only the triggering action. Actually, all other mechanisms are proportionately responsible as well,
because they contributed to a degree for sometime but the combined effect never being sufficient to initiate movement.
II. PURPOSE

The inability to exercise full control over a sufficient number of variables at any given time has prevented the development of a more complete understanding of the bluff erosion phenomena along Lake Erie. To conduct a comprehensive study into the many mechanisms operating continuously and periodically is obviously a monumental task. In view of this, the objective of this study will be an attempt to investigate the behavior of several of the more important mechanisms and to determine what specific information is lacking and, hence, required to obtain a more precise, quantitative evaluation of their respective roles in this phenomena.

Results so obtained will facilitate the formulation of long range planning for a continuing study which will endeavor to develop a higher correlation between the established fundamental theories (geology, mechanics, soil mechanics, and hydraulics) and actual field observations.

Not until this is accomplished will an engineering economic solution for this problem be possible.
III. PLAN

The investigation will commence with a discussion of the generally accepted theoretical concepts. Included will be an explanation of the nature of the more important variables and the existing methods employed for their evaluation. The combined actions of the operating mechanisms will be resolved into terms of either shearing stress or resistance, and then totaled in accordance with established techniques for stability analyses. This, then, will permit a theoretical determination of the relative importance of the roles played by those mechanisms subject to quantitative evaluation in terms of shearing characteristics.

An analysis of a specific bluff area, Perry Township Park, incorporating as many of the theories as are found applicable, will afford an opportunity to determine the adequacy of these theories to accurately predict the data obtained by field observations. Comparisons of theoretical results with field observations will provide the basis upon which recommendations for modifications, applicable to both theory and data collection, can be formulated.
IV. SHEARING RESISTANCE

A. Introduction

The role of shearing resistance as a most important and useful parameter in bluff erosion analysis is apparent. Although considerable progress has been made towards developing standardized tests to obtain values for shearing resistance, most of the ones presently used are valid only under specially controlled conditions. The variations in the direct shear test simulating actual pore water conditions is a prime example. Much still remains a matter of personal judgment and experience. Nevertheless, even if such results, at best, are found to occur within a range, they provide the engineer a more firm basis upon which he can exercise his judgment. There is no doubt that with increased usage, improvements in techniques and procedures will narrow the range of results.

Unfortunately, many physical and chemical processes have been determined as contributing to the alteration of the soil structure, but little or nothing has been done to relate this in terms of shearing resistance. This situation exists probably because much is yet to be learned of the manner in which many of these processes effect shearing resistance,
B. Nature

The nature of shearing resistance varies with the soil type. In granular soils it consists of a combination of rolling and sliding frictional resistance between the grains. For granular soils in a dense state, there is a certain amount of interlocking between the grains which provides additional resistance, the sharper and more angular the grains, the better. The resultant value of the above factors is designated as the angle of internal friction, $\phi$.

For cohesive soils, the angle of internal friction approaches zero, and the amount of colloidal size grains increase. Associated with these soils are two different forms of shearing resistance, cohesion and apparent cohesion. Apparent cohesion, attributed to the presence of capillary pressures, will disappear when the soil becomes submerged because the surface menisci are destroyed and the tension in the water is dissipated. This kind of resistance is considered a result of pressures exerted on the material at some time in the past, the effects of which have been retained. True cohesion is sometimes thought of as frictional resistance at zero normal pressure, the result of intermolecular bond of the adsorbed layers that separate the grains at points of contact. It, too, is influenced by water content which tends to cause
greater separation of particles, reducing the effect of molecular attraction and corresponding cohesion values.

Descriptions of the generally accepted apparatus and procedures for shear testing are found in all textbooks on Soil Mechanics and will not be dealt with here.

C. Effect of Soil Structure and Environmental Conditions

Initial soil structures and stress conditions, past and present, imposed upon it by environmental conditions exert considerable influence upon shearing resistance. Much has been written on sedimentation by geologists and others about various modes of deposition and the intrinsic characteristics associated with each, but quantitative relationships remain to be developed. Density, stratification, non-homogeneity, orientation of mineral particles, and presence of binders are among these characteristics. Appropriate correlations between these and shearing resistance values will, undoubtedly, increase the knowledge of shear behavior of soils.

Heavy pre-consolidation loads on clay deposits have been determined to be responsible for high shearing strengths as well as high sensitivity values. Marine clays, because of high salt and carbonate contents, sometimes have high shearing strengths. The presence of iron oxides in some soils produces similar results. Fissures and other
discontinuities, attributed to unequal pressure distribution of loads such as glaciers, may invalidate shearing test results if not given due consideration.

Moisture and temperature fluctuations influence the rate of the weathering processes which continuously act upon the soil structure.

Most of the above factors serve as aids in the interpretation of test results, but the pore water conditions, in addition, help determine the type of test to be run.
V. MECHANISMS OF BLUFF EROSION

An attempt is made to delve into several of the more important mechanisms associated with bluff erosion that are susceptible, at least in part, to quantitative evaluation. Included are rain impact, concentrated runoff, frost action, subsurface moisture, and wave action.

A. Rain Impact

When rain hits the ground, soil splashes to varying heights can be seen going in all directions. According to Ellison (4), as much as 100 tons of soil per acre may be splashed by the most beating types of rain on bare soil which is highly detachable. Variables affecting the impact of the raindrop are its velocity, size, intensity and inclination to the ground. Soil resistance to it is directly proportional to grain size and specific gravity, cohesive and frictional resistance, cementation, stone percentage, compaction, and vegetation cover.

\[
\text{Amount of soil detached} = f (v, d, i, I, R) \quad \text{where,}
\]

\[
v = \text{Velocity, ft. per sec.} \\
d = \text{Raindrop size, mm.} \\
i = \text{Intensity, in. per hr.} \\
I = \text{Inclination.} \\
R = \text{Soil impact resistance factor.}
\]

Ellison found by controlled lab tests on a Muskingum silt loam in a flat terrain that the amount of soil splashed occurred as a function of the following expression:

\[
(v^{4.33} d^{1.07} i^{0.65})
\]
According to this, velocity appears to exert the most influence and the size of the raindrops the least.

The energy of the splash process tends to be uniform on each area of the bluff so subjected. This uniformity tends to produce maximum erosion at the crest of the slope where the least amount of energy is required to transport the soil. Further downslope, however, more energy is devoted to resplashing soil already splashed from above. Consequently, it appears that at some critical distance downslope no new soil will be detached from its original position. Variation in this distance most likely occurs, depending on the characteristics of the rainfall.

B. Concentrated Runoff

The portion of the soil structure altered by this process is mainly confined to the exposed surface. Energy is primarily spent on individual grains by actions including rolling, dragging, lifting and abrasion. Because irregularities in the soil surface are inherently present, these activities are ultimately confined to channeled areas which progress from minor irregularities to major ditches or ravines.

The degree of activity = \( f (E, m, R) \), where

\[
E = \text{Energy of flow, ft. lbs.} \\
m = \text{Quantity of abrasives.} \\
R = \text{Surface flow resistance factor.}
\]
The energy of flow is usually expressed in terms of specific energy where:

\[ E = d + \frac{v^2}{2g} \]

and

\[ E \text{ is Specific Energy, } \text{ft.} \]

\[ d \text{ is Depth of flow, } \text{ft.} \]

\[ \frac{v^2}{2g} \text{ is Velocity head, } \text{ft.} \]

The surface flow resistance factor for the soil is a function of many factors among which are grain size, density, compaction, cohesive and frictional resistance, and solubility of any binders present.

Lab tests (5) have indicated that for a given set of conditions, surface flow in which abrasive sediments were injected, appreciable scouring resulted, but for clear water, none was evident.

The relative importance of each of the many variables has not yet been determined, but Hjulstrom has developed in Fig. 1 a relationship between velocity and particle size. Although specific conditions for which these results are valid are not known, certain inferences can be made. The relationships appear to be quite logical. Clayey and coarse-grained sandy soils are less erodible than silty soils under identical flow conditions, each for a different reason. For clays, cohesion plays an important part and in coarse sands, the size of particle becomes significant.
Figure 1

After F. Hjulstrom
Uppsala Univ. Geol. Inst.
Bull. XXV, 1935
Judging from the lack of other relationships, velocity and grain size appear to be the most susceptible to evaluation. From field measurements, such as slope, dimensions of channels, and depth of flow observations, velocity of flow can be calculated by Manning's formula,

\[ V = \frac{1.5}{n} R^{0.7} S^{0.5}, \]

where

- \( V \) = Velocity, ft. per sec.
- \( R \) = Hydraulic Radius, ft.
- \( S \) = Slope, ft. per ft.
- \( n \) = Roughness factor.

Velocities obtained in this manner and correlated with soil type, shearing resistance, and amount of soil removed offer the best practical approach to evaluating the importance of concentrated runoff to the overall bluff erosion phenomena.

C. Frost Action

Associated with this phenomena are several detrimental actions upon the soil structure, the primary result of which is decreased shearing resistance. For practically all areas in the United States the thawing index exceeds the freezing index; thus, a complete cycle of frost action occurs annually. This means that the heat stored in the ground during the summer gradually becomes dissipated with the onset of winter until frost begins to appear and gradually penetrates to some maximum depth. Upon reaching this point, the soil begins a thawing process, the completion of which occurs when
sufficient heat again becomes available. In areas farther north, however, where the freezing index is greater than the thawing index, only portions of the soil become thawed each summer and refrozen the following cold period.

Frost action, beginning with the freezing of the porewater, is subsequently followed by formation of ice crystals which continues as long as moisture is available. Associated with this, of course, is expansion of the frozen water which produces heaving in the soil mass. Ultimately, completion of the cycle with thawing, frost action has succeeded in disrupting the soil structure rather extensively at least to the depth of frost penetration.

Because temperatures fluctuate above and below the freezing point quite often and erratically in the Lake Erie area, a number of short cycles of thawing and freezing occur. The depths effected each time depend on the duration and magnitude of the temperature fluctuations. With each individual freeze-thaw cycle, distinct movements of soil on a slope occur. Upon heaving, the movement is directed outwards and normal to the slope. In thawing, the soil does not return to its original position and much of the shearing strength appears to have been lost in the process. The increase in volume accompanied by increased moisture content produces a saturated soil, a condition most conducive to reduction of cohesive forces as well as internal
frictional forces. The blocked drainage of the meltwater by the still frozen layer below also acts detrimentally by building up of a hydrostatic condition. Unable to withstand these actions combined with the force of gravity, the soil moves downslope, the rate and magnitude of which depend mainly on the moisture content and degree of slope. Precipitation occurring at this time serves to accelerate the movement considerably. This combination of circumstances represents a situation as being most susceptible to severe erosion.

As thawing nears completion much the same situation as above exists. No frozen layer is present below, however, to impede drainage, but the total depth of altered soil is now completely thawed and in a saturated condition with great tendencies to move downslope. Spring rains about this time represent, for many areas, the major portion of the total annual precipitation. Again, under optimum conditions severe erosion could result.

The effect of frost action is noted by Hursh (6) who reported freezing and thawing to cause "...erosion of one foot of soil in a single winter on a 1-1 slope clay bank...". Baver (7) found alternate freezing and thawing a factor in developing soil structure which produces a granulating action on soil clods that is usually more
effective than wetting and drying. Shelburne (8) found that distress varied with the amount of moisture available, the cycles of freezing and thawing, and depth of frost penetration. Similar observations are found in other literature sources.

According to Winn (9), the factors necessary and favorable for ice segregation, frost heave, and frost penetration are:

1. Capillary saturation of the soil at beginning and during freezing process.
2. A free supply of water from within or without.
3. A minimum percentage of three to ten percent of grains smaller than 0.02 mm.
4. Gradual decrease in temperature of air above the soil to below freezing temperature.

These factors are then influenced by the following: radiation, convection, conduction, evaporation, vegetation, snow cover, soil type, accumulation of water and others.

In view of the many variables requiring evaluation, the Corps of Engineers adopted a wise approach which consisted of making field observations and measurements of certain variables and investigating their correlations with values obtained by theoretical analyses. Certain variables found to be most important were then further investigated by controlled laboratory tests. Results to date have indicated that fairly reliable predictions can be made of depths of freeze and thaw by using the standard equation
for heat transfer with certain modifications. Although these equations were developed for permafrost regions, it is believed they are applicable for the area under study as well.

Below is a brief summary of relationships developed by the Corps of Engineers (10). Complete derivations and assumptions will be found in the Appendix.

Standard equation for heat transfer is:

\[ Q = \frac{K (V_1 - V_2)}{X} At = \frac{T}{R} At \]

where

- \( A \) = Area, sq. ft.
- \( Q \) = Total amount of heat transferred in Btu's.
- \( V_1, V_2 \) = Temperatures of surfaces, deg. Faranheit.
- \( K \) = Thermal conductivity, Btu per hr., per deg. F., per ft.\(^2\), per ft. thickness.
- \( t \) = Time in hours.
- \( X \) = Thickness of layer, ft.
- \( T \) = Temperature difference between surfaces, deg. F.
- \( R = \frac{X}{K} \) = Thermal resistance in deg. F. per Btu.

Depth of thaw \( X_t = \left(\frac{48KI_a}{1.434wd}\right)^{\frac{1}{2}} \), and

Depth of frost penetration \( X_f = \left(\frac{48K}{1.434 wd}\right)^{\frac{1}{2}} \).

\( I_a = T't' \) = Maximum number of degree-days of thaw based on air temperature.

\( I_a = T_t = Maximum number of degree-days of freeze based on air temperature. \)

\( C \) = Correction factor for particular surface to change air temperature into ground temperature.

\( w \) = Moisture content, %.

\( d \) = Dry unit weight, lbs. per cu. ft.
The extent to which frost action contributes to the overall bluff erosion phenomena can best be determined by a combination of field observations and theoretical results.

D. Subsurface Moisture

Below the ground surface three general moisture regions exist, each with unique characteristics. They are ground water, capillary fringe, and the vapor area.

1. Ground Water

Most important of the regions is ground water which fluctuates with variations in precipitation. Unless it is artesian, its uppermost surface, the water table, will roughly parallel the profile of the terrain surface above. It is generally moving constantly under a slight gradient of about one per cent for flat terrain and under a considerably higher gradient when flowing beneath steep terrain toward a much lower elevation.

The presence of ground water may affect the stability of a bluff in several ways. All soil beneath the water table will be fully saturated and its grain to grain contact pressure is reduced by the buoyant effect of the water. The frictional force due to the internal frictional properties of the soil is thereby reduced. With the lowering of the water table, however, the inter-granular pressures increase. Since cohesion is also a
function of moisture content, its value fluctuates with fluctuation of the water table also.

The larger the gradient under which the ground water flows, the more significant the effect of seepage will be. As the energy due to hydraulic head is gradually dissipated, seepage as a force or drag is exerted on the soil grains in the direction of flow. Where the seepage force exceeds the intergranular force in granular soils, a quick condition will exist. Because clays possess cohesion, such a condition does not generally occur, but seepage pressures still exist. Of considerable importance to seepage pressure analysis are factors such as stratification, anisotropy and boundary conditions. Flow nets, utilizing Darcy's Law, are probably the best method to estimate location of possible critical gradients and seepage pressures for relatively homogeneous soil, but are not as well suited for highly stratified deposits such as varves.

The effects of seepage may be observed in the slope face. Theoretically, for a homogenous, isotropic soil the seepage or phreatic line tends to dip downwards with the slope and intersects it shortly above an impervious stratum where small semi-circular erosion pits may develop.
Evidence of a washboard pattern may strongly indicate the presence of highly stratified soils where the sands and silts have been retrogressively removed by seepage pressures. Quantitative analysis for this condition will be most complex, if possible at all. An estimate of the amount removed may be obtained by actual measurements of the "troughs".

Leaching of soluble cementing materials such as salts, calcium carbonates and iron oxides is another detrimental effect of ground water. Depending on the amounts present, the shear strength of certain soils may be appreciably decreased because of it. For a number of slope shear failures it has been traced as being the triggering action. At present, not much has been done to correlate the loss in shear strength with the removal of cementing agents except as indicated by sensitivity values. By appropriate chemical tests, the presence and amount of most cementing agents can be determined.

2. Capillary Fringe

Located immediately above the ground water table and extending to varying heights and depending on the capillary properties of the soil, is the capillary fringe. The smaller the pores in the soil, the greater the height will the capillary rise be. The water acting as in tension exerts an intergranular pressure which may
develop to significantly high values for clay size soils. In most stability analyses this increased effect on the intergranular pressure is ignored, mainly because the ascertaining even of rough estimates necessitates considerable amount of theorizing. Besides, excluding it in such analyses is on the conservative side.

3. Vapor Area

Little is known of how vapor moisture affects the soil structure, but its contribution to bluff erosion is considered to be of little significance.

With respect to the bluff erosion phenomena, the actions of the subsurface moisture contributing significantly are the buoyant effect, decrease in cohesion, leaching, and seepage. Of these, only the buoyant effect can be determined readily, once the elevation of the water table is known. Theory and supplemental field observations of ground water conditions over a period time are required for seepage calculations. Controlled laboratory testing is required for evaluation of the other two actions.

E. Wave Action

The importance of wave action with respect to bluff erosion cannot be over emphasized. Although the depositional aspects of wave action are beneficial, the erosional aspects are most damaging. Because of the latter, the stability of the bluff proper is continuously in
jeopardy. Equilibrium under these conditions is impossible to achieve.

In general, most of the damage is caused by incoming waves and littoral currents. Since most offshore beaches found in lakes slope rather gently, waves break at some distance from the shore, producing waves of the translatory type therein. Upon striking the shore or structure, a dynamic impact is delivered, the magnitude of which is somewhat reduced by refraction caused by the inclination of the incoming waves with respect to the shore.

With increased inclination, greater littoral currents are produced. These are very effective in the abrasion and movement of material at the toe of the bluff.

For storm periods, beach conditions may change considerably, depending on the severity of the storms. Waves become higher and breaking then usually occurs at the shore or structure, producing hydrostatic as well as impact forces.

Wetting, either directly by the waves or spray, results in a decrease of cohesion. When wetting is alternated with drying, shrinkage cracks develop in clay bluffs, facilitating the entry of water therein. Upon freezing of such water in fissures, cracks, etc., the gaps of course, become widened.
The following is quoted directly from *Wave and Lake Level Statistics*, U. S. Corps of Engineers, Technical Memorandum No. 37, 1953.

"Although for structural design purposes the important factor is the size of the maximum probable wave (within a certain time period), for computations involving sand movement and littoral drift, a more desirable parameter would be some averaged factor including within it the effect of both height and period, the variation of these parameters, and the duration that waves of each particular category exist. Present day knowledge indicates that sand movement by wave action is best correlated with the amount of energy transmitted forward (and eventually on to the beach) by the waves. The total energy per unit width in each wave is, in deep water:

\[ E_0 = \frac{WHL^2}{8} \left[ 1 - 4.93 \frac{H^2}{L} \right] = \frac{Wg}{16} \frac{H^2 T^2}{L^2} \left[ 1 - 4.93 \frac{H^2}{L^2} \right] \]

- **E₀** = Energy, ft. lbs. per ft. of wave crest.
- **W** = Unit weight of water = 62.4 lbs. per cu. ft.
- **g** = Acceleration due to gravity = 32.2 ft./sec/ sec
- **H** = Wave height, ft.
- **T** = Wave period, sec.
- **L** = Wave length, ft.

Only one-half of the energy is transmitted forward from deep water toward the shore, and it is this amount of energy that eventually reaches the shore line. The total energy transmitted forward in any given period
of time is then $E_0/2$ times the number of waves occurring in that period of time, and

$$E_t = \frac{E_0}{2} x \left(\frac{3600t}{T}\right) = 7.155 \times 10^4 H^2 \frac{H}{T} \left[1-4.93 \frac{H^2}{L}\right]$$

$E_t$ is the energy in ft. lbs. per ft. of wave crest and $t$ is the duration of the waves in years.

The possible utilization of the preceding equation depends upon the availability and adequacy of the data pertaining to the wave characteristics. Such data, mainly recorded by governmental agencies, must be statistically analyzed to be of practical use because of the wide range of wave heights, frequencies, and periods.

Assuming adequate data are available, the impact energy striking the bluff toe can be calculated. Only in recent years has any attention been given to the behavior of soils under repetitive loading. Although this has been in connection with highway foundations, the situation here is fundamentally the same. The bluff toe may undergo a fatigue phenomena which greatly weakens the structure, such as is the case of metals.

A comprehensive discussion of the tractive and eroding forces generated by waves and littoral currents is beyond the scope of the writer's understanding. Admitted by many experts in the field is the fact that
theory alone is not sufficient as yet to adequately describe the phenomena. In view of this situation, field observations must be heavily relied upon to obtain the desired relationships.
VI. STABILITY RELATIONSHIPS

A. Introduction

The obvious purpose for developing stability relationships is to provide a means by which the individual effects of the operating mechanisms can be appropriately totaled to indicate the degree of stability corresponding to a given set of bluff conditions. Another purpose, not seen anywhere, is to provide a means by which the relative importance of each operating mechanism can be determined with respect to the overall bluff erosion phenomena. Going one step further, it permits the determination of the effect on the stability by a change in climatological conditions as well as the differential degree of stability corresponding to each operating mechanism. To actually do this, the effect of climatological changes upon each of the operating mechanisms must be known. As found to be true in the preceding discussions, this is only possible to a very limited degree. By necessity, therefore, the stability analyses discussed below will include only those variables readily susceptible to quantitative evaluation.

Unfortunately, all bluff erosion is not as equally susceptible to the strict analytical approach. Most of bluff erosion occurs in two forms, either as a slide or a flow. A slide type of movement is generally
characterized by the displacement of an intact soil mass along some sheared surface. For a flow, however, no such shear surface exists and the movement resembles that of a viscous liquid. Although the basic reason for both types of movements is the same (unbalanced shearing stress-resistance relationship), the determination of shear properties is considerably more difficult to ascertain reliably for flow movements.

B. Slide Movements

All stability analyses for slide movements are based upon the concept that failure will occur unless the resultant shearing strength exceeds the shearing stress along the most critical surface in the mass. Commonly believed to be the case by many is that failure begins in an overstressed zone and progressively spreads to adjacent areas when they, in turn, are unable to withstand the additional stress burden imposed upon them. In relatively non-sensitive, cohesive soils, overstressed zones may exist without failure necessarily occurring.

A number of methods of stability analyses have been developed through the years, but the most practical and useful one is the "Slice" or Swedish method. It consists of dividing the profile of a displaced soil mass (bounded by a free bluff slope, ground surface and a curved shear surface) into vertical slices for which
vectors representing forces acting on each one, are drawn. These generally include the normal and tangential force components of the weight of the slice, capillary forces, buoyant uplift, and seepage force. The summation of these and the resisting forces along the surface of rupture are combined in the general equation of stability,

\[ G^s = \frac{\text{Shearing forces}}{\text{Resisting forces}} = cL + \sum (N_w + N_c - u) \tan \phi \]

where

- \( G^s \) = Overall factor of safety.
- \( c \) = Cohesion, lbs. per sq. ft.
- \( L \) = Length of slip surface, ft.
- \( N_w \) = Normal component of the weight of each slice, lbs.
- \( T_w \) = Tangential component of the weight of each action, lbs.
- \( N_c \) = Normal component of capillary force on each slice, lbs.
- \( u \) = Buoyant force, lbs.
- \( T_c \) = Tangential component of capillary force on each slice, lbs.
- \( \phi \) = Angle of internal friction.
- \( (F_s) \) = Seepage force, lbs.

The usefulness of this method is apparent; it is applicable for any shape of rupture surface, composite layers of soil, and partial submergence. In addition, a flow net can be superimposed on the cross action to include the seepage action.

The relative importance of each of the above factors and changes in that value, corresponding to fluctuations in climatological conditions, are determined in terms of the corresponding effects on the factor of safety for an assumed basic condition.
An example demonstrating this is developed below for the seepage effect. First, a basic condition is assumed such that no buoyant, capillary, or seepage effect is considered. The resulting factor of safety is equal to:

$$G_s = \frac{cL + \frac{N_w}{T_w} \tan \phi}{\frac{N_c - u}{T_c}}$$

Then, considering all variables except seepage, the factor of safety becomes:

$$G_s' = \frac{cL + \frac{N_w + N_c - u}{T_w + T_c} \tan \phi}{\frac{N_c - u}{T_c}}$$

Including seepage, the factor of safety is equal to that shown for the general condition, $G_s''$. The percentage of the decrease in stability corresponding to the seepage effect then equals to:

$$\frac{G' - G_s''}{G_s} \times 100$$

Assuming that a change in the climatological conditions has produced a change in the magnitude of the seepage force, new factors of safety as well as a new percentage responsibility assigned to seepage can be similarly obtained. A correlation can then be obtained between the magnitude of the climatological change and the effect on the factor of safety and seepage action, merely by determining the respective difference between the two sets of values.

Similar results can be obtained for the other variables by the same procedure.
C. Flow Movements

The phenomena associated with flow type movements in soils is similar to that of viscous liquids in that only slight shearing stresses if any, can be transmitted by either. Deformation under constant stress without appreciable volume change is also characteristic of flows. According to soil mechanics theory, the transformation of soil from the solid state to a plastic and finally to a fluid state consists of merely the addition of appropriate amounts of water. The rate of movement under a constant stress appears to be solely a function of the water content.

Under changing moisture conditions in an area, it is possible, therefore, to have corresponding degrees of rate of movement in the soil mass, ranging from plastic creep to laminar flow to tubulent flow. With increasing water content, stability analyses become more of a hydraulics problem than one of soil mechanics. It still remains, however, a case of an unbalanced shear stress-resistance relationship.

No specific quantitative analysis is available for all flow movements. Each mechanism tending to promote a flow condition should be analyzed individually with the aid of theory and supplemented by field observations.

For several specific situations the stability analysis that follows appears to be applicable. One such
situation is that which is produced by frost action. Assuming that frost penetration as well as thawing occurs uniformly within a bluff face, the lower surfaces of the corresponding effected areas are then parallel to the ground surface. Within these slab like areas, the soil, due to frost action, undergoes volume change and becomes more susceptible to surface infiltration. This, in addition to the already rather high moisture contents as a result of thawing, represents favorable development of hydrodynamic conditions where seepage flow is approximately parallel to the ground surface. For a unit area of the surface over which movement ensues, the general stability equation is:

\[ G_s'' = \frac{\bar{\sigma} \tan \phi + c}{f} = \frac{[(f_t - f_w) z \cos^2 i]}{f_b z \cos i \sin i + f_w z \cos i \sin i} (1) \]

\[ \bar{\sigma} = \text{Intergranular forces, lbs.} \]
\[ f = \text{Shearing forces, lbs.} \]
\[ f_t = \text{Combined unit weight of soil, lbs. per cu. ft.} \]
\[ f_w = \text{Unit weight of water, 62.5 lbs. per cu. ft.} \]
\[ z = \text{Vertical thickness of layer, ft.} \]
\[ i = \text{Inclination of slope.} \]
\[ \phi = \text{Angle of internal friction.} \]
\[ c = \text{Cohesion, lbs. per sq. ft.} \]
\[ f_b = \text{Buoyant unit weight of soil, lbs. per cu. ft.} \]

The normal component of the intergranular pressure acting on the surface is denoted by the term, \( (f_t - f_w) z \cos^2 i \). The shearing stresses in the denominator consist of two components, the tangential and the seepage force. The tangential force is expressed
as \( (f_b z \cos i \sin i) \) and seepage by \( (f_w z \cos i \sin i) \).

Deviations and assumptions are found in the Appendix.

Correlations similar to those determined as possible for slide movements are also obtained in the same manner for flows.
VII. AN EXAMPLE: BLUFF EROSION AT PERRY TOWNSHIP PARK

A. Introduction

The following analysis of bluff erosion for Perry Township Park provides an opportunity to investigate the adequacy and shortcomings of those existing theories and principles applicable to this area. It serves to demonstrate the kind of data normally obtainable, the various methods employed in their acquisition, their utilization in stability relationships, and the sufficiency of the conclusions arrived therefrom. Presumably, deficiencies in specific areas of the analysis will become quite apparent, and will form the basis upon which corresponding recommendations for improvements can be formulated.

B. Location and Description

Perry Township Park is located in Perry Township, Lake County, Ohio. As indicated in Fig. 2, it extends eastward from Perry Park Road along the Lake Erie Shoreline for approximately 862 feet. Although the area was once an orchard, it is now a publicly owned recreational park. Several times through the years portions of the public road through the area was either relocated or abandoned because of damage by bluff erosion. Usable area has been diminishing continuously; soon a major structure, a dance hall, will be in a serious position unless erosion is checked.
C. Geology

The Great Lakes Region in which Lake Erie is located is an ancient warped Paleozoic coastal plain (11). An uplift produced several major basins and domes, one of which was the Lake Erie Basin. It serves as the major path for at least two major continental ice sheets from the north-northeast direction. These two ice sheets, the older Illinoian and the younger Wisconsin, had a number of retreats and withdrawals, causing great fluctuations of lake levels and consequent changes in drainage patterns. One such important change resulted in the formation of the present Lake Erie and also the Niagra River, in which the Niagra Falls is located. This occurred because as the Wisconsin Glacier retreated, an outlet with a lower elevation, the Niagra River, was uncovered for the then existing Lake Maumee. Previously, it was at a level of about 185 feet above the present Lake Erie and drained westward and then southward to the Mississippi River.

Beach ridges, indicating shorelines of former lakes, are obvious landforms. On one such ridge is located U. S. Route 20. According to geologists, it represents the shoreline of Lake Warren.

A considerable portion of the area directly south of Lake Erie is part of a relatively smooth plain, sloping at the rate of 40 to 50 feet per mile towards the
Lake (12). According to Fenneman (13), it lies within the boundaries of the Eastern Lake Section of the Central Lowland Province of the Interior Plains Major Division and is characterized by the lacustrine deposits of glacial Lake Maumee.

The glacial deposits in Lake and Ashtabula counties are underlain by shales which belong in the Devonian System of rocks. The upper Devonian "Black shale" in Ohio forms the bedrock in a narrow band extending roughly in the north-south direction from Adams County on the Ohio River, northward to Erie County. The outcrop swings northeastward from Erie County and parallels Lake Erie from just east of Sandusky, Ohio to the Pennsylvania border. The "Cleveland" shale, comprising the top layer of 100 feet is typically black or brownish black and a fissile shale with a high content of carbonaceous matter in a fine state of direction. Pyrite either in fine crystals or in nodules or flakes is also a common impurity. Actually no outcrop of bedrock exists in this area and only one rock cliff emerges above lake level in Lake and Ashtabula Counties.

Overlying the bedrock is the glacial till or boulder clay plain, composed of a mixture of local material ground up beneath the ice and mixed with material from the Canadian regions to the east and northeast.
D. Lake Levels (4)

The level of Lake Erie fluctuates from month to month, depending on the precipitation in its drainage area. Records kept by Corps of Engineers since 1860 indicate that the maximum level is attained in June, only about 1.3 ft. higher than the lowest, occurring in February. During the period of record (1860-1952), the average elevation was 572.31 ft. The highest one-month average of 574.60 occurred in April, 1952 and the lowest of 569.43 occurred in February, 1936.

Special fluctuations due to barometric pressures and strong winds amounting to two and one-half ft. can be expected to occur about once every thirty years.

E. Submergence

Investigators (15) have found that the entire region of the Great Lakes has been slowly tilting upwards to the north and east during the past 50 years at the rate of from one-half to one foot per 100 miles per century. In so far as engineering structures are concerned, this effect has no practical significance.
F. Wind Data

The maximum wind movement, a function of the combined effect of velocity and duration, comes from the southwest. The wind rose for Fairport Harbor, shown in Fig. 4, indicates that in the higher velocities (25 mph and over), winds of the greatest duration come from the northwest to west directions. In the lower velocities, winds of the greatest duration originate from the southwest.

Since the prevailing winds from the southwest are nearly parallel to the shoreline, the corresponding waves that are generated create littoral currents along the shore in an easterly direction. Because of this, accretion usually occurs on the westerly side of the beach structures.

G. Climatological Data

The nearest U. S. Weather Bureau Office to the study area is located in Cleveland. Data from 1871 to June 1941 were recorded at the old Cleveland City Office, only a few blocks off Lake Erie. Those since that date have been recorded at Cleveland Hopkins Airport, located several miles inland from the Lake.

Shown in Fig. 5 are the mean monthly precipitations, based on 50 years of records for Cleveland. Also included are the monthly precipitation amounts for
Wind data based on records of the U.S. Coast Guard at Fairport, Ohio.

Notes: Figures at end of bars indicate average yearly percentage of occurrence of wind in the direction and intensity shown for the period of Feb. 1, 1932 to Jan. 31, 1942.

WIND ROSE FOR FAIRPORT, OHIO

Figure 4

Based on 50 years of records from U.S. Weather Bureau at Cleveland

Figure 5
Painesville in 1957. The mean monthly temperatures, recorded at Cleveland since 1871, are plotted in Fig. 6. Daily maximum and minimum temperatures and precipitation from Painesville records are shown in Figures 7 and 8.

From these data, the following observations are made:

1. For only three months (December, January and February) the mean monthly temperatures are below freezing.

2. Complete freeze-thaw cycles occur at least annually. This is evidenced by the fact that the thawing index (degree-days above freezing) is greater than the freezing index (degree-days below freezing). The corresponding index values obtained from mean monthly temperatures for 76 years are as follows:

   Thawing Index = 6,660 degree-days
   Freezing Index = 300 degree-days

3. Fluctuation of temperatures above and below freezing appears to be the case for the winter months rather than sustained freezing. Such a fluctuation occurred during 62% of the days during the interval between November 15 and April 1 of 1947. For 27% of these days, both the maximum and minimum temperatures were above freezing while the temperatures below freezing in existence throughout the day occurred during 11% of the days only.
Mean Annual Temp. 49.6°F.

Freeze Line

Mean Freezing Index (300 degree-days)

Mean Thawing Index (6600 degree-days)

From U.S. Weather Bureau Records, 1871-1947

Figure 8

Mean Monthly Temperatures for Cleveland, Ohio
4. In the months of July and September, the heaviest precipitation occurs. For any one year, however, this situation may not be true. This is shown in Fig. 5 where the monthly precipitations for 1957 vary considerably from the means.

5. The four heaviest rains during the winter months occurred during days when the temperature was always above freezing. Preceding two of these rains was a sustained period of below freezing temperatures.

H. Soils

The soil profile, in general, consists of lacustrine deposits overlying the glacial till with shale bedrock below. The upper portion of the lacustrine deposits appear to be more coarse and not as stratified as those below. In view of this variation, the bluff will be divided into these three distinct regions.

Region I will include the upper portion of the bluff down to an average elevation of about 606 ft. Predominantly found here are multi-colored clayey sands and silts with numerous erratically deposited sand pockets, containing fine to a course sand and pea gravel in places. Examples of these erratic deposits are shown in Photo 6.

Region II extends from about elevation 606 to 585 ft, averaging about 21 ft. Lacustrine deposits
are also found here, but more of a greyish-blue color and varved in nature. Numerous alternate layers of clayey silts and fine sands make up these varves. Their presence is especially noted in the western half of the area by their washboard appearance, presumably an indication of differential erosion by seepage pressures. An example is illustrated in Photo 5. Counted within a four foot depth, were some twenty-two sand layers alternated by clayey silts. Their thickness ranged from one-half to three quarters of an inch, while the clayey silts averaged about two inches.

Region III, consisting of the greyish-blue glacial till, forms the base of the bluff and extends below water to bedrock at an elevation of about 565 ft. Constituents include mostly clayey silts, shale fragments, and numerous erratic boulders, such as can be seen in Photo 7. Fissures, mostly vertical and containing oxidized material are extensively found.

Results of soil tests are tabulated in Tables I and II.

I. Rate of Erosion

Although land surveys for this area dating as far back as the late nineties are available, relatively accurate determinations of net bluff loss between surveys are difficult to obtain. Generally, the northern property line is stated as that which is delineated by the low water mark.
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<th>Sand %</th>
<th>Silt %</th>
<th>Clay %</th>
<th>L.L. %</th>
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<td>-</td>
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<tr>
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### TABLE II

**RESULTS OF STRENGTH TESTS**

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<th>w %</th>
<th>d /cu.ft.</th>
<th>$\phi$ Deg.</th>
<th>C #/sq.ft.</th>
<th>Tests</th>
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<tr>
<td>(Remold.)</td>
<td>580</td>
<td>-</td>
<td>-</td>
<td>30</td>
<td>480</td>
<td>&quot;</td>
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54
In 1946, however, a rather detailed topographic survey made by the Ohio Department of Public Works, serves as a good reference basis upon which all present and future rates of erosion can be determined. For two aerial coverages made in May and August 1957, respectively, topographic maps were compiled by the Institute of Geodesy, Photogrammetry and Cartography at Ohio State University. The map compiled from the aerial coverage in August 1957 is shown in Figures 9, 10 and 11. From these maps, net loss determinations between the dates mentioned above are possible. The results are shown for various vertical cross sections in Fig. 12. Fig. 13 shows the same data, but plotted with respect to longitudinal cross sections at five foot increments of elevation.

The following facts result:

1. Variations in rates appear to exist both vertically and longitudinally.

2. There is a general trend for greater erosion at the upper portion of the bluff and progressively less at the toe.

3. The least magnitude of erosion occurs consistently at the same general area for each longitudinal section, between stations 4 + 00 and 5 + 00.

4. The average rate of erosion varies up to five ft. per yr.

5. Although some erosion has taken place between May and August, 1957, no definite trend is visible. Even though considerable material at the toe appears to have been removed during this period, it does not necessarily signify that only one cycle of beach deposition and removal has taken place. Such cycles can occur during a single storm.
Perry Township Park

Plotting from Vertical Aerial Photography

8.13.57

Institute of Geodesy, Photogrammetry and Cartography
Ohio State University

Figure 11
Figure 12

Erosion between March 1946 and May 1957
Erosion between March 1946 and August 1957

EROSION PER CROSS SECTION
EROSION PER FIVE FOOT INCREMENTS OF ELEVATION

Figure 13
6. The growth of the bluff lakeward as indicated in stations 4 + 50 to 6 + 00 inclusive is the result of end dumping of debris.

J. Groundwater

The elevation of the groundwater table fluctuates considerably with the seasons of the year. Fig. 14 was compiled from groundwater observations in borings and represents the high and low positions for the data obtained. Differences in elevation for the two dates, representing spring and fall conditions, range from about six feet away from the lake to as much as nine feet near the edge of the bluff. No effort was made to obtain localized variations due to well pumping in the area.

The tendency for the contours at the right to curve can be explained by the presence of a gully whose bottom elevation is slightly higher than that of the beach. The curving of the dotted contours corresponding to the lower position may be due to relatively more coarse sand and pea gravel deposits found in that region.

From the above data and corroborated by field observations, the exit of seepage flow appears to be confined within the area, Region II, and distributed over the entire face through the varved layers rather than concentrated.

Seepage analysis becomes quite difficult under these circumstances. The sand layers, having the higher
permeability values, will, of course, act as passageways and seepage pressures can be assumed to act upon the sand grains almost exclusively. Evidence in the field shows this may be true. The trough like areas shown in Photo 5 all occur in the sand layers, indicating material removed, probably by seepage pressures.

In assuming a steady seepage condition and that all flow is through the sand layers, an estimate of the average seepage pressure in each layer can be obtained by treating each layer individually. Only the top layer may be under atmospheric pressure, while artesian conditions may likely be present in the other layers. Whatever the case may be, the contours in Fig. 14 represent the existing pressure conditions. One of the most critical seepage areas appears to be at the left where the contours are close together. Therefore, analysis of a section through borings, 15 and 5, follows for conditions on November 11, 1957.

An estimate of the seepage pressure is possible without a flow net here. The head lost between the two holes (the difference in water elevations, 605.7 and 597.5 ft.) is 8.2 ft., and can be assumed to be equally applicable for all layers. The average hydraulic gradient in each layer then equals to the head loss divided by the length of the layer or flow path. With depth, the average gradient decreases.
Fig. 14. Groundwater Contours

Lake Erie

Observation Wells
April 29, 1957

November 11, 1957

Scale: 1" = 100'

Groundwater Contours
For the top layer, the average hydraulic gradient is

$$\text{1 av. } = \frac{H}{L} = \frac{8.2}{120} = 0.070$$

The seepage pressure per volume then is

$$P_s = \gamma_w \times 0.070 \times 62.5 = 4.4 \text{ } \#/\text{cu.ft.} = 0.07 \text{ gm/cu.cm}.$$  

According to theory, seepage pressures act in the direction of flow. For quick conditions, seepage pressures generally act upward and the restraining action is merely a function of the specific gravity of the grains. However, the situation under discussion differs to a certain degree. The flow is practically horizontal and the resulting seepage pressure produces a corresponding horizontal thrust on the grains.

Frictional resistance between the grains appear to be the only force resisting this thrust. Additional motivating forces acting on the outside grains consist of hydrostatic as well buoyant forces.

Assuming the most critical condition for any such grain, having a diameter, d, being that no other grain is above it, the factor of safety against movement is expressed as

$$G_s = \frac{\gamma \tan \phi}{H + P_s} = \frac{\gamma_w V (G-1) \tan \phi}{0.5 \gamma_w d^2 + P_s V}$$

$$= \frac{0.7854 \gamma_w d^2 (G-1) \tan \phi}{0.5 \gamma_w d^2 + 0.7854 d^2 P_s}$$

$$= 0.5553$$
Substituting a range of values for $G$ and $\phi$ found by lab tests to be representative, the corresponding factors of safety result:

<table>
<thead>
<tr>
<th>$G$</th>
<th>$\phi$</th>
<th>$G_a$</th>
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<tbody>
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<td>2.70</td>
<td>30°</td>
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</tr>
<tr>
<td>2.70</td>
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<td>0.82</td>
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According to these results, it is apparent that seepage pressures are sufficient to produce movement of the soil particles in the uppermost sand layer. Assuming that the hydraulic gradients in other sand layers are approximately of the same magnitude as for the top one, similar conclusions can be made.

K. Frost Action

Although conditions favorable for frost action exist throughout the bluff area, they are especially present in Region II.

Here, the conditions are summarized as follows:

1. The percentage of soil grains smaller than 0.02 mm. average over 50 per cent.

2. The presence of ample groundwater serves as a source from which moisture can be drawn to the frozen area.

3. Because of the tempering effect of the lake, temperature drops to below freezing are generally gradual.

4. Duration of below freezing temperature may occur for at least a week.
A series of freeze tests, consisting of merely subjecting undisturbed samples to freezing temperatures, indicated an average of about 6% heave. Upon saturation of the same samples, and with the gradual lowering of the temperature, greater heaves, probably more representative of actual values, will undoubtedly result.

The approximate depth of frost penetration will be calculated using

\[ X_f = \frac{48 \cdot K \cdot I_a \cdot C}{1.434 \cdot w \cdot d} \]

following values will be used:

\[ I_a = 300 \text{ degree-days} \]
\[ d = 103 \text{ lbs. per cu. ft. (average of 16 unit weight determinations)} \]
\[ w = 22\% \text{ (average of 31 determinations made at various times)} \]
\[ K = 0.8 \text{ Btu per sq. ft. per deg. F. per hour per ft.} \]
\[ C = 1 \text{ (assumed that air and surface temperatures are same because of lack of any appropriate data.)} \]

The depth of frost penetration is

\[ X_f = \left( \frac{48 \cdot 0.8 \cdot 300 \cdot 12}{1.434 \cdot 22 \cdot 103} \right)^{\frac{1}{2}} = 23 \text{ inches} \]

This value compares very favorably with the 21 inches found by the U. S. Weather Bureau to be generally applicable for this area.

Depth of frost penetration can be calculated for any specific period of below freezing temperatures. For example: Between January 10th and 19th inclusive, both the maximum and minimum temperatures were below
freezing. The cumulative degree-days below freezing (freezing index) amounted to 165 degree-days. The corresponding depth of frost is

\[ X_f = \left( \frac{48 \times 0.8 \times 165 \times 12}{1.434 \times 22 \times 103} \right)^{\frac{1}{2}} = 4.8 \text{ inches} \]

Immediately following this period of freezing came temperatures sufficiently high to produce some thawing. The corresponding depth of thaw is calculated by

\[ X_t = \left( \frac{48 \times K \times I_\delta \times C}{1.434 \times \omega d} \right)^{\frac{1}{2}} \]

For frozen conditions, the K value changes to a higher value, 1.1 Btu's per hr. per sq. ft. per deg. f. per ft. The cumulative degree-days of above freezing, \( I_\delta \), amounts to 39.7 degree-days. It was obtained by averaging the maximum and minimum temperatures for that period of three days. A more proper technique would involve hourly temperatures rather than assuming equal durations for both maximum and minimum temperatures during each day.

The resulting depth of thaw equals to

\[ X_t = \left( \frac{48 \times 1.1 \times 39.7 \times 12}{1.434 \times 22 \times 103} \right)^{\frac{1}{2}} = 2.8 \text{ inches} \]

To facilitate other calculations for Region II, the frost and thaw depth equations are reduced to the
following respective forms:

\[ X_f = 0.38 \left( I_a \right)^{\frac{1}{3}} \]
\[ X_t = 0.44 \left( I'_a \right)^{\frac{1}{3}} \]

Obtained in similar fashion are the corresponding values for the other two regions:

**Region I**
\[ X_f = 0.47 \left( I_a \right)^{\frac{1}{3}} \]
\[ X_t = 0.59 \left( I'_a \right)^{\frac{1}{3}} \]

**Region II**
\[ X_f = 0.46 \left( I_a \right)^{\frac{1}{3}} \]
\[ X_t = 0.48 \left( I'_a \right)^{\frac{1}{3}} \]

It is noted that \( X_t \) values for all cases are higher than \( X_f \) values. This is directly related to the higher thermal conductivity values for frozen than unfrozen soil.

Also of interest is the fact that in Region I both \( X_f \) and \( X_t \) are greater than in the other two regions. The higher thermal conductivity of sands is largely responsible.

Although the above results are rough estimates, to say the least, they can be used as a basis with which field observations can be correlated.
I. Concentrated Runoff

The effects of concentrated runoff is evidenced by the numerous rills and gullies present, especially in Regions I and III. Typical examples are shown in Photos 10 and 11. In Photos 1 and 2, flows are seen encroaching on the beach. Near the eastern edge of the area a full grown ravine, Photo 9, has developed, presumably from a small channel some years ago. Soon, base level will be attained over a considerable stretch of the ravine, permitting the lake to encroach inland along with the corresponding wave action. A similar situation prevails on the adjacent property, approximately 400 feet west of this area.

Average slopes are tabulated in Table III for each of the three regions at the designated stations. Velocities for several slopes (20°, 30°, 45°) have been determined and tabulated in Table IV. To obtain these, several assumptions were made in the equation

\[ V = \frac{1.5}{n} R^{0.7} S^{0.5} \]

1. For \( n \), the value was taken as 0.040 (applicable for ditches and rivers with rough bottoms and much vegetation)

2. Channels were assumed to be semi-circular and flowing full.
**TABLE III**

**AVERAGE SLOPE IN DEGREES**

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<th>Region II</th>
<th>Region III</th>
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<td>39</td>
</tr>
<tr>
<td>0 ‡ 50</td>
<td>32</td>
<td>32</td>
<td>40</td>
</tr>
<tr>
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<td>Vert.</td>
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<td>30</td>
</tr>
<tr>
<td>1 ‡ 50</td>
<td>Vert.</td>
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<td>38</td>
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<td>30</td>
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<td>2 ‡ 50</td>
<td>33</td>
<td>16</td>
<td>30</td>
</tr>
<tr>
<td>3 ‡ 00</td>
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<td>37</td>
</tr>
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<td>28</td>
<td>37</td>
</tr>
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<td>6 ‡ 50</td>
<td>62</td>
<td>22</td>
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<td>45</td>
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<td>Slope Deg.</td>
<td>Diameter of Channel ft.</td>
<td>Velocity cm./sec.</td>
<td>Particle Size Erodible mm.</td>
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</table>
Also included in Table IV are the range in sizes of particles susceptible to movement by the corresponding velocities which were found. These values were obtained from Fig. 1. Assuming that the soil conditions in the area are similar to those for which relationships in Fig. 1 are applicable, the results indicate conclusively that concentrated runoff is capable of developing velocities of sufficient magnitude to produce erosion of grain sizes larger than those found to be representative of the area. Especially susceptible are Regions I and III, but in Region I, the channels only in the lower reaches may accumulate sufficient flow to be critical.

M. Wave Action

The area nearest to Perry Township Park for which wave height data are available is Cleveland. The wave rose in Fig. 15 indicates the per cent of time waves of various heights occur from each direction. Significant storm waves are seen to come from almost any direction between west and northeast, going in a clockwise manner. The greater wave heights appear to be from the west-northwest direction.

Shown in Table V are the durations in hours for various wave heights for each month. These represent the corresponding totals for three years, 1948, 1949 and 1950, and should be divided by three.
LAKE ERIE
STATION B (Cleveland)

WAVE ROSE SHOWING PERCENT OF TIME WAVES OF DIFFERENT HEIGHT OCCUR FROM EACH DIRECTION

FROM U.S. CORPS OF ENGINEERS TECHNICAL MEMORANDUM No. 37, 1953

Figure 15
### TABLE V

**DURATIONS OF WAVE HEIGHTS IN HOURS**

<table>
<thead>
<tr>
<th>Wave Height</th>
<th>0.5-1</th>
<th>1-2</th>
<th>2-3</th>
<th>3-4</th>
<th>4-5</th>
<th>5-6</th>
<th>6-7</th>
<th>7-8</th>
<th>9-10</th>
<th>10-11</th>
<th>11-12</th>
<th>16-17</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>60</td>
<td>336</td>
<td>240</td>
<td>114</td>
<td>78</td>
<td>30</td>
<td>12</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>876</td>
</tr>
<tr>
<td>February</td>
<td>108</td>
<td>330</td>
<td>132</td>
<td>108</td>
<td>54</td>
<td>48</td>
<td>12</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>804</td>
</tr>
<tr>
<td>March</td>
<td>12</td>
<td>180</td>
<td>294</td>
<td>174</td>
<td>96</td>
<td>54</td>
<td>36</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>-</td>
<td>876</td>
<td></td>
</tr>
<tr>
<td>April</td>
<td>24</td>
<td>270</td>
<td>228</td>
<td>162</td>
<td>102</td>
<td>48</td>
<td>12</td>
<td>12</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>864</td>
<td></td>
</tr>
<tr>
<td>May</td>
<td>18</td>
<td>372</td>
<td>270</td>
<td>150</td>
<td>42</td>
<td>30</td>
<td>12</td>
<td>6</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>906</td>
<td></td>
</tr>
<tr>
<td>June</td>
<td>84</td>
<td>234</td>
<td>186</td>
<td>54</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>558</td>
<td></td>
</tr>
<tr>
<td>July</td>
<td>36</td>
<td>198</td>
<td>174</td>
<td>48</td>
<td>48</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>504</td>
<td></td>
</tr>
<tr>
<td>August</td>
<td>162</td>
<td>390</td>
<td>144</td>
<td>18</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>714</td>
<td></td>
</tr>
<tr>
<td>September</td>
<td>126</td>
<td>360</td>
<td>168</td>
<td>84</td>
<td>54</td>
<td>30</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>834</td>
<td></td>
</tr>
<tr>
<td>October</td>
<td>36</td>
<td>222</td>
<td>156</td>
<td>78</td>
<td>24</td>
<td>6</td>
<td>6</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>540</td>
<td></td>
</tr>
<tr>
<td>November</td>
<td>18</td>
<td>84</td>
<td>150</td>
<td>108</td>
<td>78</td>
<td>18</td>
<td>24</td>
<td>12</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>516</td>
<td></td>
</tr>
<tr>
<td>December</td>
<td>42</td>
<td>282</td>
<td>204</td>
<td>108</td>
<td>42</td>
<td>24</td>
<td>54</td>
<td>6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>762</td>
<td></td>
</tr>
</tbody>
</table>
The month of May is indicated as having the most wave action while July and November have the least. However, the highest waves occurred during November. To evaluate the potential moving power of the waves themselves and littoral currents, refraction diagrams are an absolute necessity. This development is beyond the scope of this study.

Certain qualitative observations can be made, however. Existing concrete groins, rings, blocks, and debris affect the movement of waves and currents by dissipating some of the energy, especially during lower lake levels when more of these structures protrude out of the water. An indication of how effective these structures is denoted by the fact that the least amount of erosion in the whole bluff area happens to be between Station 4 4 00 and 5 4 00 directly behind the largest mass of concrete in the area.

Undercutting is more severe at the eastern portion of the bluff where no such structures exist. It is also explainable by the fact that this area is on the leeward side of existing structures with respect to the prevailing southwest littoral currents. The greatest extent of undercutting was observed during November as is indicated in Fig. 3. Also noteworthy is that according to the data concerning wave characteristics, the waves
up to 16 to 17 feet high occur during the month of November, the highest of any during the year.
Photo 1. Flow type movements  
(April'57)

Photo 2. Flow type movements  
(April'57)
Photo 3. Eastern portion of bluff area
(April '57)

Photo 4. Same bluff area as above
(Nov '57)
Photo 5. Lacustrine Deposits
(Nov '57)
Photo 6. Erratic Sand Deposits  
(Nov'57)

Photo 7. Block type movement of toe  
(Nov'57)
Photo 8. Wet slope from seepage and accumulation (April '57)

Photo 9. Ravine near easterly property line (April '57)
N. Summary

Based on field observations and engineering analyses of bluff erosion at Perry Township Park, the following statements are presented.

1. The rate of erosion for the past eleven years has averaged up to five feet per year. The greater rates have occurred at the top portions of the bluff. A reasonable explanation for this may be that the lower portions of the bluff are often covered with the flow material from up above and consequently, are exposed less to the erosion mechanisms than are the areas above.

2. Erosion is predominantly of the flow type in the upper two regions, whereas, the phenomena acting at the base portion of the bluff consists mainly of gulleying, abrasion, and block type movement. Apparently, the high shearing strengths, reflected by the steep slopes in the lower and upper regions and indicated by test results, have been sufficiently adequate to withstand the higher stresses associated with steep slopes. The lack of significant slide failures is further evidence of this fact.

3. Field observations, corroborated by theoretical analyses, indicate that mechanisms including seepage, concentrated runoff, frost action and wave action are potentially the main contributors to the bluff erosion phenomena here.
4. Groundwater tables fluctuate with the seasons; lowering as much as nine feet has been observed for a period from April to November. Majority of seepage activity is confined to the middle portion of the bluff (Region II). Because of pervious sand layers, the exit of seepage has been observed to be distributed throughout this area. Even though no rain fell for two weeks during one period in May, soft mushy areas up to a foot in depth were observed. Indentations or "troughs", present in these sand layers, strongly appear as being the result of seepage. Contours of groundwater elevations indicate the presence of larger gradients during the lower elevations in the fall.

5. Because of the rather steep slopes, high velocities of runoff occur, and the highly erodible sands and silts in the upper and lower regions are, therefore, susceptible to severe gulley formation and ultimately, deep channel development.

6. The combined presence of a very high percentage of the silt fraction, ample source of moisture (groundwater), and considerable fluctuation of temperatures above and below freezing indicates those conditions most favorable for frost action. Although all of the bluff is highly susceptible to frost action, Region II is especially so, mainly because of the presence of groundwater there.
According to U. S. Weather Bureau data and a theoretical analysis, the maximum depth of soil effected by frost action is normally between 21 and 23 inches. Highly possible is the deformation and subsequent erosion of soil to this depth, provided heavy rains occur at the optimum times, such as during thawing.

7. It can be inferred that the large mass of concrete structures located opposite section 4 + 00 has been sufficiently effective in dissipating wave energy. The smallest rate of erosion in the area was found to occur at this section.

8. Other concrete structures, rings, blocks, etc., appear to be effective to some degree also. The widest beaches in spring existed at the extreme ends of Perry Township Park and extended into the adjacent properties. Oddly enough, no concrete structures are located here and the bluff in both areas are steep. Also significant is the lack of flow material at the toe such as is found in the area proper. A simple explanation is that the waves were very effective in removing the flow material because no structures were there to dissipate some of the energy.
VIII. CONCLUSIONS

Following are the conclusions found in regards to the adequacy of existing methods for bluff erosion analyses, based upon the analysis performed for Perry Township Park.

In addition, recommendations for further studies and modifications in techniques and procedures are also included.

A. Although engineering analyses represents noble attempts to explain the phenomena of bluff erosion in terms of the shearing stress-resistance relationships, existing theories and procedures are not sufficiently progressed to predict soil behavior satisfactorily for a given set of conditions. An example is the unsuccessful attempt to adequately explain flow movements in this manner.

B. Existing methods for determination of shear strength are deficient in many ways. One is in the interpretation of the results. A small sample in the laboratory for instance, may have a very high shearing strength but because of discontinuities, fissures, etc. in the soil mass, some value considerably less is more appropriate.

C. Theory must be supplemented with systematic field observations. Only in this way can theory be modified and improved upon. Observations must, by necessity, be over a rather long period of time in order to obtain
A true picture of the fluctuations, normally associated with data of this sort.

D. Knowledge of the fundamental behavior associated with each of the mechanisms is not adequately known. More important, however, is the corresponding effect on the shearing resistance. Until the use of a better parameter is found, shearing resistance must continue to be the common basis upon which the effect of all the mechanisms can be combined by stability relationships.

E. Although there is close agreement between the maximum depth of frost penetration as found by a combination of a theoretical and empirical method and that furnished by the U. S. Weather Bureau, certain assumptions were made with the aid of very meager data. The fact that much is still to be learned of frost action is not surprising because of the many interactions yet to be investigated.

Field observations should include the following:

1. Ground and air temperature records to determine the correction factor used to convert the more generally available air temperatures recorded by U. S. Weather Bureau to ground temperatures.

2. Depth of maximum frost penetration

3. Depths of freeze and thaw corresponding to temperature change and duration.

4. Development of ice lenses and heaving.
Laboratory studies should include the evaluation of the thermal conductivities of the soils for the range of environmental conditions present.

F. Although various studies of the behavior of soils by rain impact have been made in the laboratory under simulated field conditions, results still have to be verified by observations of actual field conditions. Recommended, therefore, are field studies to obtain correlations of the amount of soil detached or splashed into collecting troughs with the impact energy as measured, possibly, by a sensitive scale consisting of a flat surface upon which raindrops strike.

G. Adequate theories in hydraulics are available to determine velocities of flow for assumed conditions of slope, roughness, channel configuration, and depth of flow. Proper evaluation of these, of course, would provide a better basis for the assumptions.

The grain size, density, cohesion, internal friction, and other characteristics of the soil should be correlated with the above data.

H. A clearer picture of the subsurface water conditions and its detrimental effect of which seepage is one of the more important can be obtained by the controlled
use of appropriate dyes which denote exit of seepage at bluff face.

Automatic recording piezometers are needed to obtain correlations between groundwater fluctuation and precipitation.

I. To correlate amount of erosion with change in environmental conditions, certain field studies could be done. They include:

1. Installations of flexible tubing at appropriate places to measure rate of creep flow in terms of slope change. Anchorage into the more firm material below is, of course, necessary. Flat plates placed on the surface may also include movement.

2. Indication of net loss for a given period by periodic measurements of differences between reference hubs and surface adjacent to hub. (Maps compiled from air photos are not sufficiently accurate to indicate net soil loss for short periods of time.)


J. Any fundamental study of wave action should include provisions for:

1. Measurements of impact energy against the toe of the bluff under various conditions of lake levels, wave heights, wind direction, etc.
2. Determining the effect of littoral currents in moving the material as well as undercutting. Velocity measurements are entailed.

3. Investigating the effect on the soil structure at the toe by repetitional impact loading by the waves.

4. Investigating the effect of the offshore profile in the breaking of the waves before striking the bluff. Needed will be underwater surveys.

K. Another lab study with potential significance is the development of a new test to replace the existing liquid limit test. Presumably, when a soil contains the percentage of moisture equal to the liquid limit, it has little or no shearing resistance. For a slope situation where soil is thawing and the water content increases, movement of that soil supposedly will commence when the moisture content becomes equal to the liquid limit. An analysis of the forces present, indicates that only the component of gravity tangent to the slope acts to movement. However, inherent in the technique used to determine the liquid limit, an impact force is delivered with every revolution of the crank. The two kinds of forces are not compatible. Therefore, a new device to determine the moisture content at which shearing resistance is negligible appears to have considerable merit.
APPENDIX

A. Depth of Frost and Thaw

Following is a derivation of the equations used to calculate the depths of frost penetration and thaw taken from the article by Henry Carlson, "Calculations of Depth of Thaw in Frozen Ground". Symposium on Frost Action in Soils, Highway Research Board, Special Report No. 2, 1952. The standard equation for heat transfer is:

\[ Q = \frac{K(V_1 - V_2)}{X} \text{At} = \frac{(V_1 - V_2)}{X/K} \text{At} = \frac{TA_t}{R} \]  

(1)

If \( t \) is in days and \( A = 1 \) sq. ft., then

\[ Q = \frac{24 Tt}{R} = \frac{24 I}{R} \]  

(2)

\( I \) = Freezing or thawing index in degree-days based on ground temperatures.

Considering only the latent heat of fusion, the number of Btu's per sq. ft. of area required to thaw the soil to a depth \( X \) is:

\[ Q = XL \]  

(3)

\( L \) = Latent heat of fusion of water per cu. ft. of soil.

\( L = 1.434 \) wd.

In the case of one thawing layer the average resistance during the period of thaw is taking place may be written \( \frac{R}{2} \) or \( \frac{X}{2K} \). The equation for the depth of thaw in one homogeneous layer is derived by equating (2) and (3) and using the average resistance:
\[ Q = xL = \frac{24I}{R/a} = \frac{24I}{x/2K} = \frac{48KI}{X} = \frac{48K'I_aC}{X} \]

\[ X = \left( \frac{48KI_aC}{L} \right) \frac{1}{2} \tag{4} \]

Equation (3) is applicable for determination of both the depth of frost penetration and thaw if the appropriate values for \( K \) and \( I \) are used.

The depth of frost penetration is

\[ X_f = \left( \frac{48KI_aC}{1.434 \text{ wd}} \right) \frac{1}{2} \]

The depth of thaw is

\[ X_t = \left( \frac{48K'I_aC}{1.434 \text{ wd}} \right) \frac{1}{2} \]

Assumptions used in the foregoing analysis are:

1. Thickness of the soil layer and the temperature difference between that surface are constants

2. Heat necessary to raise the ground temperature above the freezing point was neglected
B. General Stability Equation for a Thawed Soil Layer

Assumptions

1. Maximum permeability of flow in direction parallel to the slope

2. Perched water table, surface at ground surface

The intergranular vertical pressure on planes parallel to the slope is $p_{b} z \cos i$. The corresponding normal and shearing components are $p_{b} z \cos^2 i$ and $p_{b} z \cos i \sin i$, respectively.

The seepage force per unit volume is designated as $i_h p_w$, where $i_h$, the hydraulic gradient, equals one, then the volume is $z \cos i$ and the total seepage force is then $p_w z \sin i \cos i$. 
Then the general stability is expressed as follows:

\[ G_s = \frac{F_b z \cos^2 \phi}{F_b z \cos \phi \sin \phi + F_w z \sin \phi \cos \phi} \tan \phi + C \]

Reference

REFERENCES


REFERENCES (continued)

12. Shaffer, P. R., "Geology of the Lake Erie Shore in Ohio from Marblehead to the Pennsylvania Boundary", Second Annual Report, Beach Erosion Survey of Lake Erie Shores in Ohio, State Department of Public Works in Ohio, 1946.

