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Shoreline erosion on selected artificial lakes in Iowa

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Shoreline erosion on selected
artificial lakes in Iowa

by

Bryant Mervin Berg

A Thesis Submitted to the
Graduate Faculty in Partial Fulfillment of the
Requirements for the Degree of
MASTER OF SCIENCE

Department: Civil Engineering

Major: Geotechnical Engineering

Signatures have been redacted for privacy

Iowa State University
Ames, Iowa
1980

TABLE OF CONTENTS

	Page
OBJECTIVES	1
INTRODUCTION	2
TERMINOLOGY	3
REVIEW OF LITERATURE	6
Variables Influencing Shoreline Erosion	6
Shoreline Protection Methods	19
DESCRIPTION OF STUDY AREAS	29
Prairie Rose Lake	29
Big Creek Lake	31
FIELD STUDY	34
Soil Sampling	35
Shoreline Inventory	39
Profiling	41
DISCUSSION OF FIELD STUDY	71
The Shoreline Erosion Process	72
SHORELINE EROSION MODEL ASSUMPTIONS	79
Discussion of Model Assumptions	79
SHORELINE EROSION MODEL	84
Existing Methods for Predicting Shoreline Erosion	84
COMPARISON OF FIELD OBSERVATIONS WITH SHORELINE EROSION MODEL	96
SUMMARY AND CONCLUSIONS	105
RECOMMENDATIONS FOR FURTHER STUDY	107

	Page
BIBLIOGRAPHY	108
ACKNOWLEDGMENTS	111
APPENDIX A: SOIL DATA SUMMARY	112
APPENDIX B: STEEL REFERENCE PIN LOCATIONS	125
APPENDIX C: SEDIMENTATION CALCULATIONS	129
APPENDIX D: EFFECTIVE FETCH AND WAVE BASE DEPTH CALCULATIONS	132

OBJECTIVES

Because there are many new methods of shoreline protection that have not been utilized in the State of Iowa, the first objective of this research is to review state-of-the-art shoreline protection methods, especially those methods that look promising for use on artificial lakes in Iowa.

The second research objective is to gain a better understanding of the variables influencing the shoreline erosion process on artificial lakes and the third objective is to predict the location and severity of shoreline erosion at any position on an artificial lake.

INTRODUCTION

Shorelines which result from impounding water in artificial reservoirs are often subject to wave erosion which produces unsightly scars on the landscape, increases water turbidity, contributes to reservoir sedimentation, and endangers facilities or structures along the shoreline. The conventional remedy for wave erosion has been the use of loose rock riprap, but this is a solution with physical and economic limitations. In Iowa, where suitable riprap is expensive, there is a need for alternative, economical methods of stabilizing shorelines to prevent or retard wave erosion. In order to develop economical alternative methods of protecting shorelines, a thorough understanding of the shoreline erosion process is necessary.

The process of shoreline erosion on artificial lakes is a complex phenomenon involving many variables. Presently, the ability to predict the location and maximum extent of shoreline erosion on man-made lakes is limited by a lack of understanding of the many variables that influence the shoreline erosion process. Some of these variables include the surrounding topography, geologic setting, soil characteristics, wave action, and climate. An understanding of how and to what extent these variables influence the shoreline erosion process would greatly aid in the design of stable shorelines.

TERMINOLOGY

Several terms which may be unfamiliar are used in the text to describe features of the erosion profile. Therefore, Figure 1 is provided to illustrate the features of an idealized shoreline erosion profile, to define terms, and to aid in the discussion of the erosion profiles development.

Immediately after the impoundment of water at a man-made lake, the erosion profile begins to develop. As waves attack the original surface, material is eroded and an abrasion platform and wave cut cliff form. The formation and recession of the wave cut cliff is a cyclic process. Initially, the attacking waves erode a portion of the original surface producing a vertical wave cut cliff. As the waves continue to attack the base of the wave cut cliff, they notch out a portion of soil creating a slope which is steeper than the original. Subsequently, the slope may become so steep that it is unstable. This unstable slope fails in shear as the soil becomes saturated and wave attack continues. Initially, the clumps of soil which fall to the base of the wave cut cliff provide protection from further wave action, but as the waves continue to erode the soil a vertical wave cut cliff is again formed and the cycle repeats itself. As the cycle progresses and the shoreline recedes, an abrasion platform develops. The abrasion platform is a gently sloping surface which develops gradually as the shoreline recedes and wave action reshapes the original surface (Strahler, 1963). If

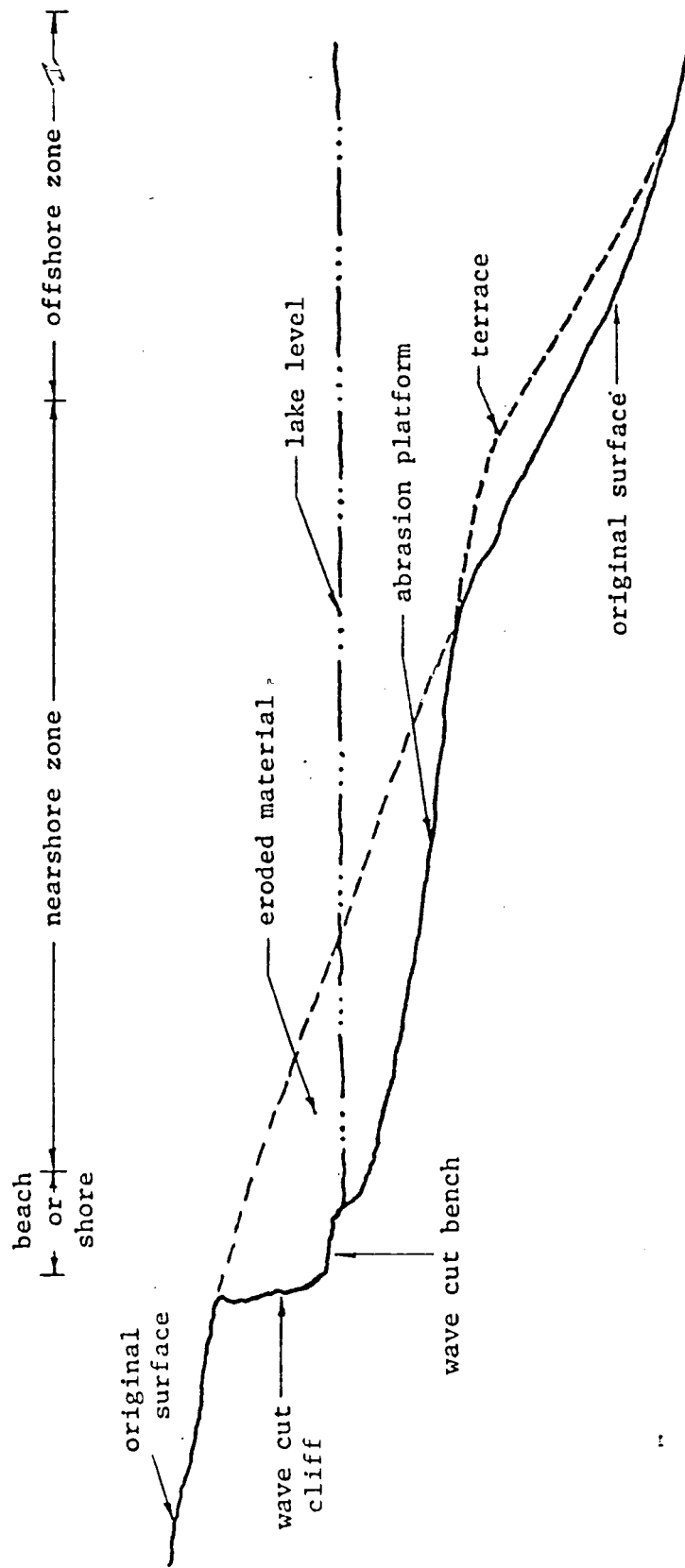


Figure 1. Erosion profile terminology

conditions permit, the material eroded from the original surface may be redeposited offshore from the abrasion platform forming a depositional feature called a terrace (Strahler, 1963). The formation of the terrace depends on the steepness of the original surface, any longshore currents which may move sediments laterally along the shore, and the physical properties of the eroded soil. As the abrasion platform and terrace develop, they will help dissipate the energy of breaking waves and reduce further shoreline recession. The abrasion platform and terrace form the nearshore zone of the erosion profile. Another feature of the erosion profile, the wave cut bench, may form in the eroded soil at the base of the wave cut cliff when the water level fluctuates or when resistant shoreline materials are encountered.

REVIEW OF LITERATURE

For convenience, the literature review is divided into two parts. The first part discusses the variables influencing shoreline erosion and the second part covers shoreline protection methods.

Variables Influencing Shoreline Erosion

The design of a proper shoreline protection system requires an understanding of the variables influencing the shoreline erosion process. The most important of these variables are: geologic information, geotechnical properties, shore and nearshore geometry, climate, and wave action.

Geologic information

The geology and stratigraphy at a proposed lake site are important variables to consider when designing shoreline protection. An understanding of the local stratigraphy enables the engineer to predict what materials may be encountered at points of interest along the shoreline and what materials may be exposed as the erosion process continues.

In Iowa, most all man-made lakes and reservoirs are constructed in areas where either glacial till or loess or both are the materials along the shoreline. These two materials exhibit varying degrees of erosion resistance, therefore, it is important to know the extent and location of these materials along the shoreline.

The use of soil survey reports, available from the Soil Conservation

Service of the United States Department of Agriculture, geological reports, and aerial photographs aid in determining which soils and parent materials may be encountered at the proposed lake site.

Soil properties

The discussion of soil properties is divided between cohesive and non-cohesive soils. These two soil categories vary in their resistance to shoreline erosion because of different factors influencing their strength.

Most information regarding the mechanics of soil erosion is the result of investigations of sediment yields from watersheds, tractive forces that a river exerts on its wetted perimeter and soil detachment due to raindrops; therefore, the erosion characteristics of cohesive and non-cohesive soils cannot be directly applied to soil erosion by wave action, however, some of the factors discussed provide an indication of a soil's resistance to wave action.

Buller et al., cited by Keown et al. (1977), reported that the erosion characteristics of non-cohesive soils which are controlled by gravitational forces and the basic parameters affecting the erosion of non-cohesive soils such as particle size, grain shape, gradation, moisture content, and relative density are fairly well-understood. Gibbs, as cited by Keown et al. (1977), has conducted studies on non-cohesive soils correlating the permissible unit tractive force to the particle diameter.

Although the parameters affecting the erosion of non-cohesive soils are fairly well-understood, there is a basic lack of understanding of

the erosion characteristics of cohesive soils, which seem to be influenced by many different factors (Keown et al., 1977). Gibbs also attempted to correlate the permissible unit tractive force with the void ratios of cohesive soils. However, this is limited by a lack of knowledge concerning the various parameters which control the erosion resistance of cohesive soils. Partheniades and Paaswell (1970) reported that efforts to correlate gross soil properties, such as Atterberg limits, plasticity index, bulk density and mechanical soil composition to the erosion resistance of cohesive soils have had little success. Also, soil shear strength cannot be used as a unique parameter to determine a soil's resistance to erosion. On the basis of limited research, Partheniades and Paaswell concluded that the physico-chemical properties of cohesive soils are the most important variables in determining the erosion resistance of cohesive soils. More recently, Sargunam et al. (1973) conducted research on the physico-chemical factors involved in the erosion of cohesive soils and concluded that the pore fluid composition of a cohesive soil, as it influences the swell potential of a soil, affects its erosion resistance. Arulanadan (1975) reported that the composition of the eroding fluid, the type and amount of clay minerals, and the cation exchange capacity can all affect the erosion rate of cohesive soils.

Although a better understanding of the factors influencing the erosion characteristics of cohesive soils is becoming a reality, this understanding is the result of tests where cohesive soils are subjected to shear stresses or tractive forces and not wave forces. Tests where

cohesive soils are subjected to wave action may reveal other important variables which determine a soil's erosion resistance.

Shore and nearshore geometry

The shore and nearshore geometry are very important variables influencing the shoreline erosion process. The slope of the shore or beach affects wave run-up and movement of material on the shore and the slope of the nearshore profile determines the breaking point of a wave and the rate at which energy is dissipated as a wave shoals. The slope of the shore also affects how material will move when subjected to wave action. In general, the steeper the slope the greater the potential for mass movement down the slope. Mass movement may be initiated by wave erosion at the toe of the slope. After the waves have eroded enough material to produce a critical slope, mass movement follows.

Beach Erosion Board (1962) reported on the importance of shore slope and roughness on wave run-up. The wave run-up on a shore of given roughness tends to increase as the slope steepens, until a critical slope is reached at which point run-up decreases, as the shore slope increases.

The slope of the nearshore profile is also important in influencing the shoreline erosion process. As waves, which are usually generated in deep water, move into shallower water of the nearshore zone, the decreasing depth influences the wave's characteristics. As a wave begins to shoal, it loses energy as the wave motion interacts with the bottom surface. This energy dissipation results in less energy available for

shoreline erosion.

The slope of the nearshore profile also influences the breaking point of a wave. The United States Army Coastal Engineering Research Center (1973) reported that as a wave approaches the shoreline it reaches a depth of water so shallow that the wave will collapse or break. This depth being equal to about 1.3 times the wave height. Other research by Ippen, Kulin, and Galvin, as cited by the United States Army Coastal Engineering Research Center (1973), shows that the nearshore slope has a significant effect on the breaking point of a wave. As the nearshore slope steepens, the waves break nearer to shore.

Climate

Climate plays an important role in the shoreline erosion process. Precipitation, temperature, and wind are all important variables to consider. The annual amount of precipitation influences water level fluctuations and groundwater levels in the surrounding topography. Where bluffs have formed near the shoreline, the groundwater movement becomes very important. Water seepage in the bluffs may cause sloughing as the soil becomes saturated and unstable.

Temperature variations become most important in areas where ice formation can occur. Miller (1971) noted the importance of ice formation and movement on lakes. In his study of the Iowa Great Lakes he recorded the movement of soil and rock material along the shoreline by ice masses.

Wind movement on a lake is the most important climatic variable influencing shoreline erosion. The duration, direction, and velocity of the wind are important variables influencing wave generation.

Wave action

Predicting wave action has consistently troubled engineers. It is essential that engineers be able to predict wave heights because many important elevations are established by wave heights. Examples of such elevations are freeboard allowances, railroads and other structures. Wave height information is also important in determining the quality and extent of slope protection and designing boat ramps, docks, and other recreational facilities to provide minimal disturbance from wave action.

Early engineers experienced much frustration in developing rational wave forecasting equations. This frustration is best summarized by Shield (1895) who reported, "In view of the numerous conditions which affect the height of waves, it seems doubtful if it is possible to construct any reliable formula by which it (wave height) can be predicted." Despite early harbor engineers' frustration and disappointment, there has been significant progress in the past 100 years.

Thomas Stevenson's work is probably the most valuable early contribution to the prediction of wave heights and the study of wave mechanics. Stevenson made numerous observations of waves in canals, fresh water lakes, and the open sea. The relationships he developed are purely empirical, but were considered fairly reliable because of his large number of observations (Shield, 1895; Gaillard, 1904;

Molitor, 1935; and American Society of Civil Engineers, 1948).

Stevenson proposed that the height of a wind generated wave is proportional to the square root of the fetch. He established relationships between the fetch and wave height based on a 78 mile per hour wind, and expressed the first of these relationships as follows:

for fetches greater than 30 miles

$$h = c\sqrt{f} \quad (1)$$

where

h = height of wave in feet

f = the fetch or distance to the windward shore in nautical miles

c = a coefficient which varies with the strength of the wind

For strong gales, where the water is of sufficient depth to allow the waves to fully form the formula may be changed to:

$$h = 1.5\sqrt{f} \quad (2)$$

Stevenson also proposed the following equation for shorter reaches.

for fetches less than 30 miles

$$h = 1.5\sqrt{f} + (2.5 - \sqrt[4]{f}) \quad (3)$$

Molitor (1935), using observational data reported by Gaillard in 1904, made the next significant contribution to the prediction of wave heights by modifying the work of Stevenson to include the effects of wind velocity. Molitor reported that for a given wind velocity, V , in miles per hour, and fetch, D , in statute miles, the wave height, h , may be estimated using the following equations:

for values of D greater than 20 miles

$$h = 0.17\sqrt{VD} \quad (4)$$

and, for values of D less than 20 miles

$$h = 0.17\sqrt{VD} + 2.5 - \sqrt[4]{D} \quad (5)$$

Molitor reported that the reason for the introduction of the wind velocity term into the Stevenson formulas was to render them applicable to varying wind conditions. The original Stevenson formulas were based on a 78 mile per hour wind, and naturally a 30 mile per hour wind will produce waves of a lesser height than a 78 mile per hour wind; likewise, there may be situations where the wave height produced by a 100 mile per hour wind is necessary for breakwater design. Molitor further reported that these formulas are approximate and that when actual observations are available they should be utilized.

The American Society of Civil Engineers (1948) provided a much needed summary of available wave forecasting techniques in their paper entitled a "Review of Slope Protection Methods." This paper provided a 'state of the art' summary of the available shoreline protection methods and some much needed criteria for the design of riprap slope protection. The authors noted that information available for the design of any form of slope protection is meager and hoped that their paper would stimulate interest in the area of slope protection.

It is important to note that after fifty years the main source of information on the prediction of wave heights is still based on Stevenson's work as modified by Molitor. The American Society of Civil Engineers (1948) reported that although the Stevenson-Molitor formulas are still in general use, experience has shown that they do not always

assure reliable predictions of the critical wave height and on several instances it was reported that the equations predicted wave heights that were greatly exceeded in actual situations. Therefore, they recommended that wave heights computed by the Stevenson-Molitor equations be assumed as approximations and an indication of the average conditions. The American Society of Civil Engineers also reported two more recent formulas proposed by Wolf and Creager. The equations are:

$$\text{Wolf: } h = (0.0335V - 0.28) \sqrt{D} \quad (6)$$

$$\text{Creager: } h = \frac{D^{0.37} V^{0.48}}{c} \quad (7)$$

where

h = height of wave in feet

D = fetch in statute miles

V = wind velocity in miles per hour

c = value of conservatism

If a value of $c = 3.41$ is used in the Creager formula it produces results that are almost identical to the Stevenson-Molitor formulas, except for short fetches. For fetches of length less than five miles, the Stevenson-Molitor equations predict greater wave heights than the Creager equation.

Prior to 1942, most attempts at predicting wave heights were based on empirical relationships and were not always reliable. The ability to obtain more reliable wave heights and periods became more important before and during World War II in order to plan large scale amphibious landings.

Early work in the area of developing semi-empirical relationships for the prediction of wave heights was completed by Sverdrup and Munk for the United States Navy Hydrographic Office (1951). Sverdrup and Munk combined the theoretical equations of hydrodynamics with observed data to develop semi-empirical relationships which govern deep-water wave generation and decay. Bretschneider (1953) modified Sverdrup and Munk's results to include the effect of energy added from wind stress.

The previously discussed work concerns the prediction of deep water waves which differ from shallow water waves. Shallow water waves occur in water where the depth is approximately equal to or less than one half the wave length. In some cases shallow water waves may result from waves which originated in relatively deep water and advance to water of decreasing depth. As waves approach the shoreline, the decreasing water depth has a pronounced effect on wave characteristics.

Bretschneider (1954) presented a numerical method to determine the generation of wind waves over a shallow bottom in which the effects of bottom friction and percolation in a permeable sea bottom are taken into account. This numerical method is based on successive approximations where wave energy is added due to wind stress and subtracted due to bottom friction and percolation. Bretschneider reported that the prediction of waves in shallow water is more difficult than the prediction of waves in relatively deep water, because in shallow water the depth and type of bottom will have a limiting effect on the rate of wave growth. The number of variables involved in wave generation over a shallow bottom makes a mathematical investigation of this phenomenon

very complicated.

Sibul (1955) performed a laboratory analysis of the generation of wind waves in shallow water and demonstrated that wave heights are affected by depth. More specifically, his experiments indicated that depth affects wave heights when the ratio of the depth to wave height is less than 5. Sibul plotted his results against the dimensionless parameters developed by Sverdrup and Munk to obtain the following relationship:

$$\frac{gH}{U^2} = 3.25 \times 10^{-3} (gF/U^2)^{0.435} \quad (8)$$

where

F = fetch length in feet

U = wind velocity in feet per second

H = wave height in feet

g = acceleration due to gravity

Rearranging terms this equation may be written as:

$$H = \frac{0.00325U^2}{g} \left(\frac{gF}{U^2} \right)^{0.435} \quad (9)$$

Saville (1962) proposed a formula for predicting wave heights on deep water inland reservoirs based on Sverdrup and Munk's work as modified by Bretschneider. Their formula is based on the study of two inland reservoirs and proper recognition must be given to the physical conditions at other reservoirs or inland lakes before the formula is applied. The equation proposed by Saville (1962) is:

$$H = \frac{0.0026 V^2}{g} \left(\frac{gF_e}{V^2} \right)^{0.47} \quad (10)$$

where

H = wave height in feet

g = acceleration due to gravity

F_e = effective fetch in feet

V = wind velocity in feet per second

The method recommended by the U.S. Army Coastal Engineering Research Center for the prediction of waves which are generated in shallow water and move into deeper water is based on Bretschneider's results as modified by Ijima and Tang (U.S. Army Coastal Engineering Research Center, 1973).

Bhowmik (1976), using relationships developed by Sibul (1955), developed a nomograph relating the wave height to the effective fetch and wind velocity. The nomograph is easy to use and gives results similar to those achieved when using the method recommended by the United States Army Coastal Engineering Research Center.

The difficulty engineers face when trying to develop rational formulas for predicting wave heights is accounting for the many variables which influence wave action. The most important variables are: duration, direction, velocity of the wind, effective fetch, and surrounding topography.

The Beach Erosion Board (1962) in its study of waves on two different inland reservoirs established relationships between the wind velocity

on land and the wind velocity on a body of water. Results showed that the ratio of the wind velocity over water to the wind velocity over land increases as the effective fetch increases until the ratio equals 1.31. The study indicated that as the wind velocity remains constant, wave heights get progressively larger until a limiting wave height is reached.

Prior to 1954, fetch was defined as the greatest straight line distance over which the wind blows from the windward shore to the shore where the waves impinge. The American Society of Civil Engineers (1948) reported that the fetch is usually defined as the normal distance from the windward shore to the structure being designed, and that because of the unusually large waves that are generated in certain situations the 'effective' fetch may be a curved path, such as wind sweeping down a curved valley. The American Society of Civil Engineers (1948) reported that the determination of the correct value for the fetch is difficult because the effects of topography on wind and waves have not yet been extensively studied, and that further research in this area would greatly aid the field of wave forecasting.

Saville (1954) presented a method for the determination of the effective fetch on inland reservoirs with irregular shorelines. Saville reported that the effect of fetch width in limiting wave growth has long been recognized, but has generally been neglected because; for the generation of waves in the ocean, most of the fetches will have widths of the same magnitude as their lengths. However, when determining fetches for artificial lakes and reservoirs, the width of the fetch is generally limited by land masses. This limiting of fetch width has a pronounced

effect on wave generation, and may significantly reduce wave heights.

It is known that waves are generated not only in the predominant wind direction, but also at various angles to the predominant wind direction. As a result of this phenomenon, the wave energy reaching a particular shoreline will be the sum of the wave energy produced by the wind in the predominant direction plus the wave energy generated by the wind at various angles either side of the prevailing wind direction. Saville (1954) made several different assumptions as to the directional variation of wind strength and found that results based on wind strength varying as the cosine of the angle up to 45 degrees either side of the predominant wind direction most nearly conform with existing wave forecasting methods. Examples of computing the effective fetch are found in Appendix D.

The topography surrounding a man-made lake or reservoir is very important in determining wave action. Beach Erosion Board (1962) reported that relatively high bluffs, hills, or trees bordering the site, may exert a significant effect on the air currents causing turbulence which may affect wave action on the lake.

The many factors influencing the generation of wind waves makes the prediction of wave characteristics very difficult. The method selected for predicting wave action will depend on the physical characteristics of the body of water being examined.

Shoreline Protection Methods

Interest in developing economical and reliable shoreline protection methods has increased significantly in recent years. The loss of

valuable shoreline property along the coasts and the Great Lakes has stimulated many new ideas for controlling shoreline erosion. The large amount of literature available on shoreline protection methods makes it impossible for a complete examination, therefore, the most important developments and some promising methods for usage in Iowa are discussed.

Development of protection methods

The use of materials for shoreline or bank protection did not become popular until the late 1800s and early 1900s. California, State of, Department of Public Works, Division of Highways (1960) reported that prior to 1920 there was very little need for bank protection methods because most highways and structures were located in areas where hazardous situations could be avoided. This was the case in California until the middle to late 1920s when large floods produced such extreme damage that an investigation into different bank protection devices was begun.

An important reference concerned with the slope protection of earth dams was published in 1948 by the American Society of Civil Engineers. This was one of the first publications to examine the problem of slope protection and assemble the available information. An objective evaluation of several types of shoreline protection methods was made. The methods reviewed were: dumped stone riprap, hand placed riprap, grouted riprap, concrete slabs and blocks, porous concrete paving, bituminous paving, vegetative cover, and miscellaneous protective measures. Riprap was extensively used for shoreline protection in the

early 1900s thus most early publications on slope protection concentrate their efforts on riprap and the different placement methods. The authors noted that detailed design information for the construction of riprap or any other means of slope protection is very limited and they hoped their paper would generate enough interest so that advances could be made.

Office, Chief of Engineers (1949) of the United States Army Corps of Engineers presented results of one of the first studies aimed at evaluating different methods of slope protection. This report is the result of an extensive survey, conducted in 1946, regarding the effectiveness of various slope protection practices found on selected dams throughout the United States. The purpose of the study was to determine which methods of slope protection were most economical and practical. Only the most common types of slope protection were analyzed (dumped stone riprap, hand placed riprap, and concrete revetment). The study concluded that dumped riprap with a suitable filter blanket is the most satisfactory type of slope protection. It also recommended that hand placed riprap not be used in areas where heavy ice conditions occur and a higher quality of stone is necessary for slope protection equal to that of dumped riprap. The other method examined was concrete revetments which performed satisfactorily under moderate wave action only. They recommended that monolithic construction be used for concrete revetments and the number of expansion joints be kept to a minimum.

Davis et al. (1973) of the Bureau of Reclamation, Denver, Colorado presented a paper examining riprap slope protection methods. In-depth studies were made of 50 case histories of the use of riprap for upstream protection on earth dams. They reported that the rational design formulas available to engineers for predicting wave heights gave highly variable results for the size of rock required. It was also found that no single, currently available laboratory test adequately evaluated the quality and durability of riprap, but several of the available physical properties tests provide an indication of durability. Conclusions were that where difficulties with slope protection on certain dams required maintenance, maintenance costs plus operating costs would never approach the initial cost of providing maintenance free slope protection. They also recommended that specifications be changed to include more rock of the larger sizes.

The use of riprap for protecting the shoreline of small artificial lakes and reservoirs is also very popular. Bhowmik (1976) conducted a study of riprap usage in the state of Illinois. Bhowmik developed a methodology for designing effective riprap protected shorelines by analyzing long-term wind data as they affect wave characteristics and the forces acting on individual riprap particles. Also examined were the physical quality of riprap materials and proper selections of filter materials.

Other methods of slope protection which have become popular in recent years include soil-cement and chemical soil stabilizers. Portland Cement Association (1965) discussed the proper method for

constructing soil-cement slope protection and reviewed the condition of one of their first soil-cement test projects, the Bonny Reservoir located near Hale in eastern Colorado. This site was chosen because of its exposure to freeze-thaw conditions, wave action and successive wetting and drying. The Bonny Reservoir was designed and constructed in 1951 by the Bureau of Reclamation. The construction consisted of placing 7 feet wide overlapping sections 350 feet long on top of each other in a stair-stepped fashion producing a 2:1 slope. This provided a minimum soil-cement thickness of 2.7 feet measured perpendicular to the slope. After ten years of exposure, core samples were taken from the site and evaluated for their compressive strengths. The average compressive strengths ranged from 2000 to 2160 psi. The results of this first reservoir site using soil-cement as the slope protection method proved that soil-cement can be as effective as riprap with a cost of 30 - 50%, depending on riprap availability. Portland Cement Association further recommends that only sands and very sandy soils be stabilized with Portland cement for water resources applications.

In addition to using Portland cement as a soil stabilizer for use in slope protection, other chemicals and even vegetation have shown promise in the prevention of slope erosion. Morrison and Simmons (1977) of the Bureau of Reclamation conducted screening tests on 30 different liquid soil stabilizing materials. Some of the liquid soil stabilizing materials used included liquid cutback asphalt, elastomeric emulsions, latex emulsions, polyvinyl acetate emulsions, urethane liquid and liquid resin solution. The soil used for the screening tests was a fine grained

sand. Once treated, the soil samples were subjected to water erosion in wave simulating devices, water jets, wind and outdoor weathering. Most of their work was aimed at providing soil stabilization for such problems as temporary dust control, erosion control at construction sites and stabilizing secondary roads. However, one of the liquid soil stabilizers, a urethane product, exhibited excellent erosion resistance to wave action. Another part of their study consisted of examining the possibility of binding gravel size particles for application as riprap material. One liquid soil stabilizer, an elastomeric emulsion, exhibited satisfactory compressive strength and adequate resistance to wave action, but upon exposure to outdoor conditions showed signs of weakening and deteriorating after a four year period. Conclusions were that several of the liquid soil stabilizers provide adequate protection for erosion control and that with refinement some of these stabilizing agents may be applicable for shoreline stabilization on small artificial lakes and reservoirs.

Another method of shoreline protection which may prove suitable on small artificial lakes and reservoirs is the use of cellular concrete blocks or 'monoslabs'.

Parsons and Apmann (1965) constructed an experimental revetment of cellular concrete blocks on the banks of an eroding river to determine the cellular block's effectiveness in comparison to riprap. After an eight year test period, only three of the original 600 cellular concrete blocks had been lost from flow conditions which included the impact of large ice flows and estimated shear stresses of 3.2 psf. Adjacent riprap

was unable to withstand the same conditions. When cellular concrete blocks are mass produced they cost approximately the same as riprap and provide comparable protection. If cellular concrete blocks were used for shoreline protection, they would be easier to transport to the site and would provide a more accessible beach.

Keown et al. (1977) of the United States Army Engineer Waterways Experiment Station in Vicksburg, Mississippi conducted an extensive literature survey on the 'state of the art' in streambank protection methods in which they examined most every feasible method of riverbank stabilization. Although this publication deals with methods and materials which are used for riverbank stabilization, many of these same methods and materials can be used equally as well for shoreline protection. To expedite their literature search they divided the different riverbank protection methods into categories. These categories are: single component revetments, bulkheads, soil stabilization and river training structures. For shoreline erosion the number of divisions can be changed to five.

These five divisions and examples of each are:

single component revetments

asphalt blocks

cellular blocks

ceramic blocks

concrete blocks

rubble

sack revetment

stone riprap

tetrapods

monoslabs

mattresses, matting, and revetment pavement

articulated concrete mattresses

asphalt pavement

bituminous mattresses

ceramic mattresses

concrete pavement

erosion-control matting

fascine mattresses

gabions

log and cable

rock and wire mattresses

synthetic mattresses, matting, and tubing

timber and brush mattresses

used tire matting

grouted riprap

bulkheads

concrete or stone

fiber

metal (steel facing)

timber

soil stabilization

asphalt emulsions

grout

organic mixtures and mulches

soil cement

temperature control

vegetation

offshore breakwaters

floating tires, log etc.

Most of these methods must be supplemented with a suitable filter blanket, either of sand and gravel or synthetic materials. The filter blanket prevents the fine silty, sandy, clayey material of the original bank from being washed through the outer protective cover thereby possibly causing subsidence and failure. The Bureau of Reclamation (1965) reported that the criteria used to design a proper sand and gravel filter blanket are credited to Terzaghi and Bertram. It was found that by using the correct filter ratio, which is defined as the ratio of the 15% particle size of the coarse layer to the 85% particle size of the finer layer, the washing of finer materials through the slope protection material can be prevented. In protecting the slopes of earth dams it has been found that a filter ratio of 5 or less between successive layers will provide adequate protection.

The selection of a shoreline protection method depends on many factors: wave action, foundation materials, availability, and economics. The large amount of eroded shoreline on many man-made lakes makes it economically impossible to provide complete shoreline protection, therefore, only selected areas of shoreline can be protected. These

areas of shoreline may be selected on the basis of recreational opportunities provided or severity of erosion which has taken place.

Presently, as a result of the Water Resources Development Act of 1974, there are several shoreline erosion demonstration projects in the United States on the Great Lakes and coastal areas. The purpose of this program is to determine economical and effective means of controlling shoreline erosion. A report will be prepared upon completion of the demonstration projects to assist private landowners and public agencies in selecting the proper methods and materials for controlling shoreline erosion.

DESCRIPTION OF STUDY AREAS

The study of shoreline erosion consisted of examining two man-made lakes, Big Creek Lake and Prairie Rose Lake. These lakes were selected for study in order to compare and contrast shoreline erosion on lakes in loess and in glacial till.

Prairie Rose Lake

Prairie Rose Lake and its park facilities were opened to the public in 1962. The lake is located approximately six miles east and three miles south of Harlan, the county seat of Shelby County. Prairie Rose Lake was designed and constructed under the twenty-five year conservation plan initiated by the Iowa Conservation Commission in 1933. Reconnaissance surveys for Prairie Rose Lake were begun in 1938 and in 1952 the proposed acquisition map for the present site was drafted. Prairie Rose Lake has 8.5 miles of shoreline surrounding a 218 acre body of water. The lake's watershed is approximately 4490 acres of which 443 acres surrounding the lake form a state park. The deepest point in the lake is approximately 26.5 feet, near the dam, with 5 - 8 feet being the average depth. Prairie Rose State Park has facilities for camping and boating with a limit of six horsepower engines (Iowa Conservation Commission, 1977).

Climate

The climate of Shelby County area is humid to subhumid. The average annual temperature for the summer months is 72.5°F and 22.8°F

for the winter months. The annual average precipitation is about 29 inches per year, most of which occurs during the growing season. The prevailing winds for this area are out of the southwest in the warm months and out of the northwest in the cool months.

Vegetation

The native vegetation of the upland areas in Shelby County was prairie grass, mainly big bluestem, most of which has disappeared as a result of farming and grazing practices. The timber areas in Shelby county are mostly limited to the steeper sloping areas and the flood plains of the streams and rivers (Soil Conservation Service, U.S.D.A., 1961).

Topography and geology

Prairie Rose Lake is located on the western portion of the Southern Iowa Drift Plain. The topography in this region consists of steeply rolling hills interspersed with uniformly level upland divides and level alluvial lowlands (Prior, 1976).

The hill summits near Prairie Rose Lake have approximately 20 - 30 feet of loess, the bulk of which was deposited during post-Tazwell time of the Wisconsin age glacial period. Beneath the Wisconsin age loess is the Yarmouth-Sangamon weathering surface developed on Kansan till, which outcrops on valley sideslopes where it has been exposed by slope beveling. Beneath the thick deposits of Kansan till is the Aftonian weathering surface which developed in Nebraskan till. Although the Kansan till and its paleosol outcrop on the

valley sideslopes, the streams have not eroded deep enough to expose the Nebraskan till and Aftonian weathering surface. Beneath the Nebraskan till is the preglacial bedrock topography.

Soils

In general, the soils of Shelby County have developed in relatively thick loess that covers glacial till deposited during the Nebraskan and Kansan glacial periods. Prairie Rose Lake is located within the Marshall soil association which covers approximately 4260 square miles or 7.6% of the state's total area (Fenton et al., 1967). Slope gradients in this area range from 1 - 30% with most of the gradients falling into the 2 - 14% slope category. Marshall soils occupy about 45% of the area. Marshall soils are well drained soils which developed from loess under native prairie vegetation and occupy most of the area around Prairie Rose Lake.

Big Creek Lake

Big Creek Lake is located in Polk County approximately 1½ miles northwest of Polk City and was created as part of the Saylorville Reservoir project to protect Polk City from Big Creek Floods. Big Creek Lake, which opened in November of 1972, has approximately 21.5 miles of shoreline surrounding the 861 acre lake. The lake's watershed is approximately 51,000 acres of which 2025 acres surrounding the lake are state park grounds. The deepest point in the lake is approximately 52.5 feet (United States Environmental Protection Agency, 1976). Big Creek Lake provides excellent sailing and fishing.

Climate

Polk County's climate is subhumid to humid making it ideally suited for agriculture. The average annual temperature for the summer months is 74.4°F and for the winter months is 25.5°F. The average annual rainfall is approximately 31 inches, 70% of which occurs during the growing season. The prevailing winds are out of the south in the warm months and out of the northwest in the cool months (Soil Conservation Service, U.S.D.A., 1960).

Vegetation

The native vegetation of this area was prairie grasses and hardwood trees. The hardwood forests usually grew along the major streams with prairie grasses covering the areas which are now extensively used for farming (Soil Conservation Service, U.S.D.A., 1960).

Topography and geology

Big Creek Lake is located on the southern portion of the Des Moines Lobe near the Bemis Moraine. The topography may be described as variable, with areas of flat to slightly irregular land intermixed with bands of rough, knobby terrain. Numerous ponds and marshes dot the landscape along with some glacial lakes (Prior, 1976).

The geologic setting of the Big Creek Lake area is mostly the result of glacial activity. The most recent glacial activity was the Cary substage of the Wisconsin age glacial period which occurred approximately 13,000 years ago. Beneath the Wisconsin age Cary till is loess, most of which was deposited during post-Tazwell time of the Wisconsin

age glacial period. Beneath the loess are the Kansan and Nebraskan tills with their respective Yarmouthian-Sangamonian and Aftonian weathering surfaces. On the surface of the Cary till are areas of local loess and wind deposited fine sands which have been blown from the Des Moines river bottom to upland areas. These wind deposited materials form a thin mantle, 2 to 3 feet in thickness (Soil Conservation Service, U.S.D.A., 1960).

Soils

In general, the soils surrounding Big Creek Lake in Polk County developed from Wisconsin age glacial till and glacial till derived sediments and are part of the Clarion-Nicollet-Webster soil association of central Iowa. Approximately 75% of this area has level to gently sloping topography, but Big Creek Lake is located near the southern boundary of this soil association area which is more hilly as a result of stream dissection (Fenton et al., 1967).

The soils on the gently sloping to steeply sloping valley walls surrounding Big Creek Lake are chiefly the Hayden and Lester soils which have developed in glacial till. The soils which are found in the nearby level areas of the valley bottoms are usually the Colo, Waukegan, Dickinson, and Dorchester soils formed from outwash or alluvium. The soils on the level to gently sloping upland drainage divides are the Clarion, Nicollet and Webster soils weathered from glacial till (Fenton et al., 1967).

FIELD STUDY

The field study of shoreline erosion on Prairie Rose Lake and Big Creek Lake consisted of two phases: first, soil sampling and shoreline inventory and second, profiling selected transects perpendicular to the shoreline. The first phase of the field work provided the opportunity to become familiar with the topography, stratigraphy, and areas of shoreline with and without problem erosion. Soil data collected in the field were in situ shear strengths and unit weights. Soil samples were returned to the laboratory where particle size analyses and Atterberg limits tests were performed. The shoreline inventory consisted of mapping the extent and severity of erosion and areas which are protected by riprap or other means.

The second phase of the field work, profiling selected transects perpendicular to the shoreline, was completed after a reconnaissance of the erosion problems at both lakes. The selection of transects to be profiled was based on the following criteria: area which, in the judgement of the park rangers, exhibited severe erosion problems, shoreline accessibility for field work, and position on the lakes relative to the fetch.

The profiling of selected sites was executed as follows: Initially, 2' x $\frac{1}{2}$ " \varnothing steel rods were placed at a measured distance and direction inland from the edge of the wave cut cliff. Attempts were made in October of 1979 to profile selected transects from a small boat, but difficulty in obtaining accurate depth and horizontal measurements from

the boat forced abandonment of this method. Therefore, profiling was completed after an ice cover had formed on the lake with depth measurements made through holes bored with an ice auger. Vertical and horizontal control was obtained with an automatic level and steel tape. Measurements made on the ice provided more accurate results and established an accurate reference point for future profile measurements.

The existing profiles were then drawn. Aerial photographs, ground photographs, and original topographic maps were used to draw the profile of the original surface on the same paper. By comparing the original surface and existing profile the degree of erosion which had taken place at each site could be evaluated. Values measured at each profiling site include: original surface slope, existing nearshore profile slope, wave cut cliff height, horizontal shoreline recession, and profile direction.

Soil Sampling

Soil sampling at Big Creek Lake and Prairie Rose Lake started in the summer of 1979 and was completed in the fall with laboratory analyses performed during the winter months.

Soil samples were collected at seven locations along the shoreline at Big Creek Lake and two locations along the shoreline at Prairie Rose Lake. In addition, one upland sample was collected at Prairie Rose Lake. The extensive shoreline erosion at Big Creek Lake provided exposures from which to sample the soils at various vertical positions on the wave cut cliff whereas soil samples taken at Prairie Rose Lake

had to be taken from pits dug near the shoreline.

Big Creek Lake

Figure 2 shows the locations of soil sample sites at Big Creek Lake. A summary of soil properties is provided in Appendix A.

In general, the soils surrounding Big Creek Lake are glacial till which has a loamy texture. Most soils contain approximately 40 - 45% sand. Clay contents are generally between 15 and 19%, except for soils at sites 2 and 15 which have clay contents in excess of 20%. Gravel content is generally less than 6%, except for sites 3 and 17 which have sand and gravel layers present. Sample 1 at site 7 exhibited an extremely high silt content of 74%.

Shear strengths of soil surrounding Big Creek Lake are variable depending on a soil's location and moisture content. Shear strengths generally decreased from top to bottom of the wave cut cliff as the moisture content increased. The range of soil shear strengths is 3380 psf (162 kN/m²) to 56 psf (3 kN/m²). Very low shear strengths were exhibited by the sand and gravel layers at sites 3 and 17.

Dry unit weights generally varied from 105 pcf (16.5 kN/m³) to 115 pcf (18.0 kN/m³), except for sample 1 at site 7 which had a dry unit weight of approximately 95 pcf (14.9 kN/m³).

Prairie Rose Lake

Figure 3 shows the locations of soil sample sites at Prairie Rose Lake. Soil properties are summarized in Appendix A.

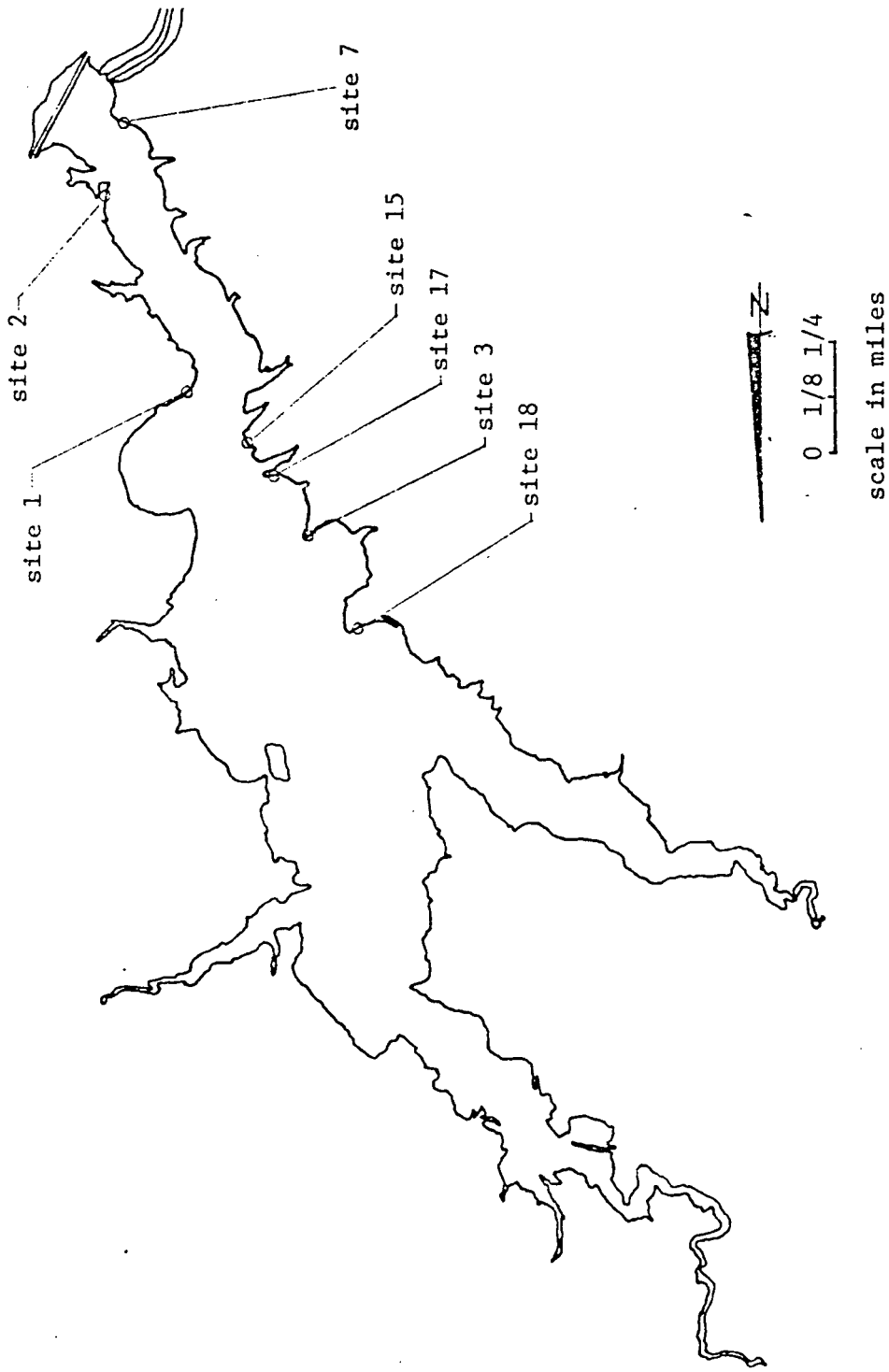


Figure 2 . Big Creek Lake soil sampling locations

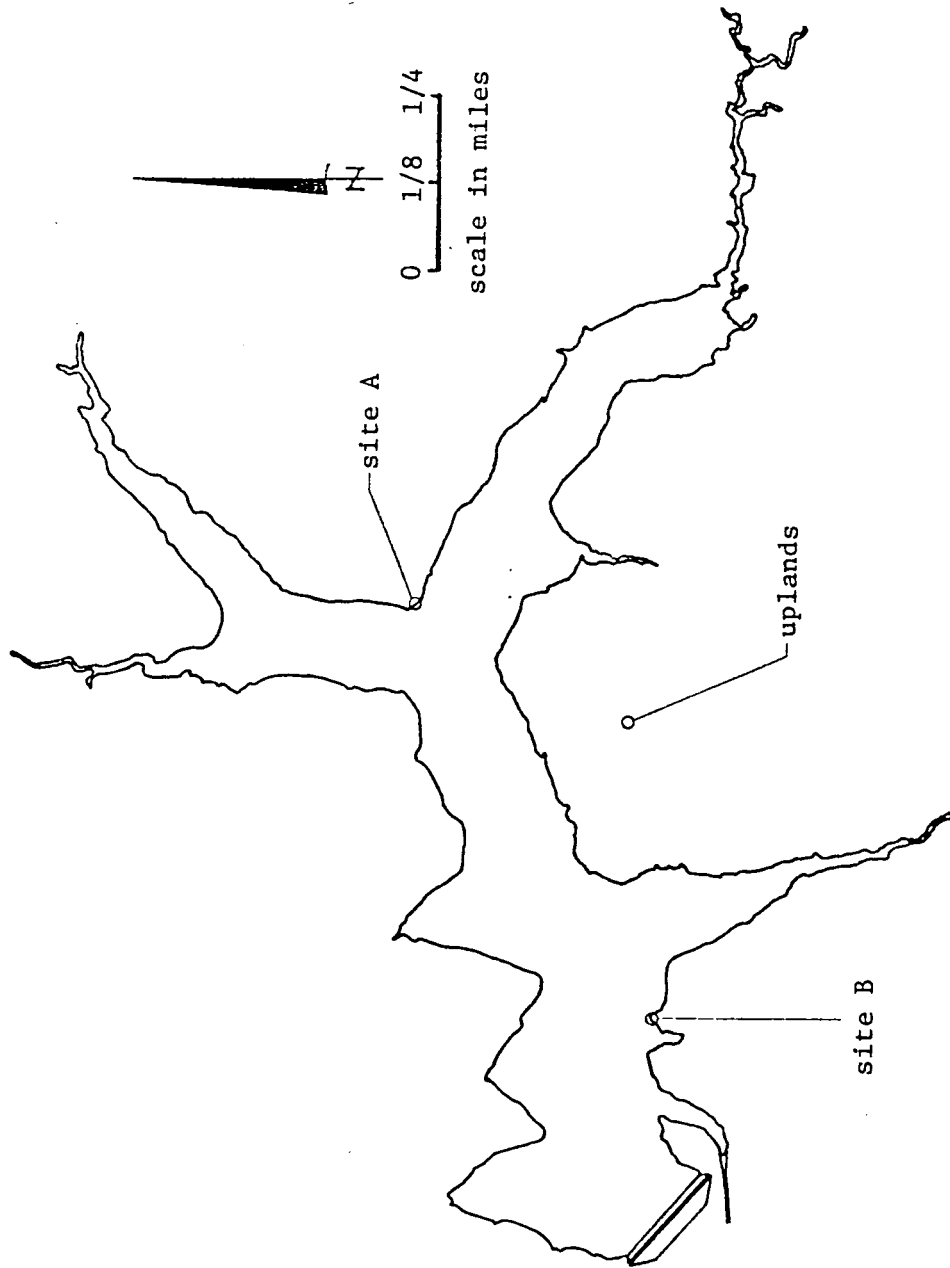


Figure 3. Prairie Rose Lake soil sampling locations

All of the soils sampled at Prairie Rose Lake are loess which has a silty clay loam texture with the silt content varying from 55 - 70% and clay content varying from 28 to 36%.

Dry unit weights were consistently at 92 pcf (14.4 kN/m³) and shear strengths were approximately 850 psf (40.6 kN/m²) at 30% moisture content.

Shoreline Inventory

The shoreline inventory at Big Creek Lake and Prairie Rose Lake consisted of mapping areas with problem erosion and areas with shoreline protection.

Big Creek Lake

Figure 4 shows the shoreline inventory taken at Big Creek Lake. Areas with erosion are classified according to the height of the wave cut cliff. Three classifications of erosion are defined, 1' to 2' wave cut cliffs (slight erosion), 2' to 5' wave cut cliffs (moderate erosion), and 5' and higher wave cut cliffs (severe erosion). Also inventoried at Big Creek Lake are areas with shoreline protection.

Big Creek Lake has approximately 21.5 miles of shoreline of which 18.8% show erosion. Slight erosion amounts to about 8.5%, moderate erosion approximately 7.8%, and severe erosion approximately 2.5%.

Riprap, the only form of shoreline protection, at Big Creek Lake is used on approximately 2.5% of the total 21.5 miles of shoreline.

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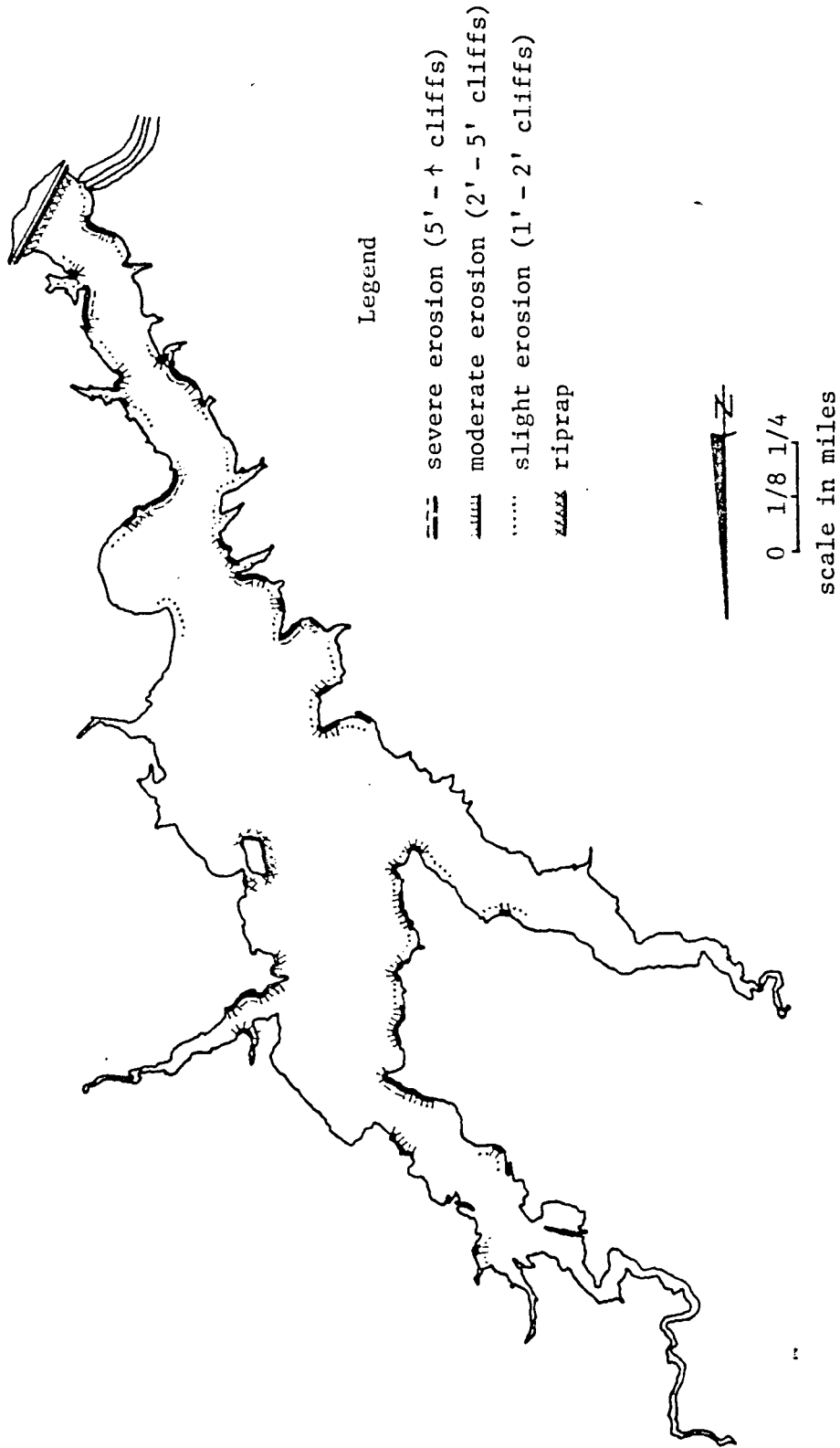


Figure 4 . Big Creek Lake shoreline inventory and protection inventory

Prairie Rose Lake

Figure 5 shows the shoreline inventory taken at Prairie Rose Lake. No inventory of wave cut cliffs could be made because riprap has been placed at several positions along the shoreline; however, an inventory of riprap placement was made.

Prairie Rose Lake has approximately 8.5 miles of shoreline, of which 2.5 miles or approximately 30% have been riprapped. Of these 2.5 miles, approximately 60% is very sparse and provides little protection from wave action.

Profiling

Profiles, measured perpendicular to the shoreline, at Big Creek Lake and Prairie Rose Lake, were obtained for the purpose of estimating the quantity of erosion and to characterize the geometry of the wave cut surface.

Big Creek Lake

Profiles were measured at three selected sites at Big Creek Lake. Two of the sites, 3 and 17, are located on the west side of the lake, approximately mid-way along the northwest-southeast axis. The other site, 2, is located at the southeast end of the lake, near the dam. Figure 6 shows the locations of these profiling sites.

Site 2

Site 2 is located near the dam at the southeast end of Big Creek Lake. One profile was measured at this site in the direction N45°W.

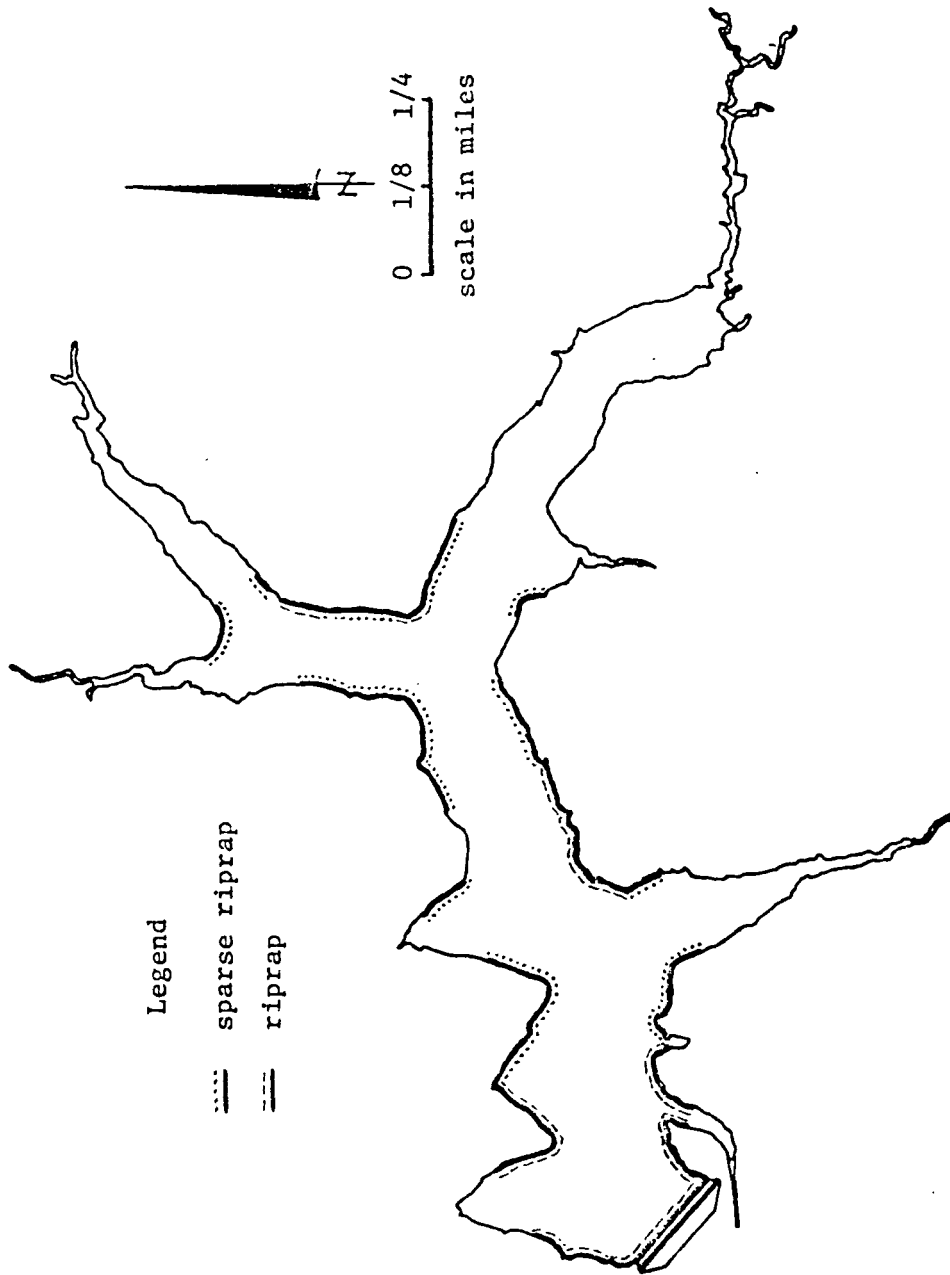


Figure 5. Prairie Rose Lake shoreline protection inventory

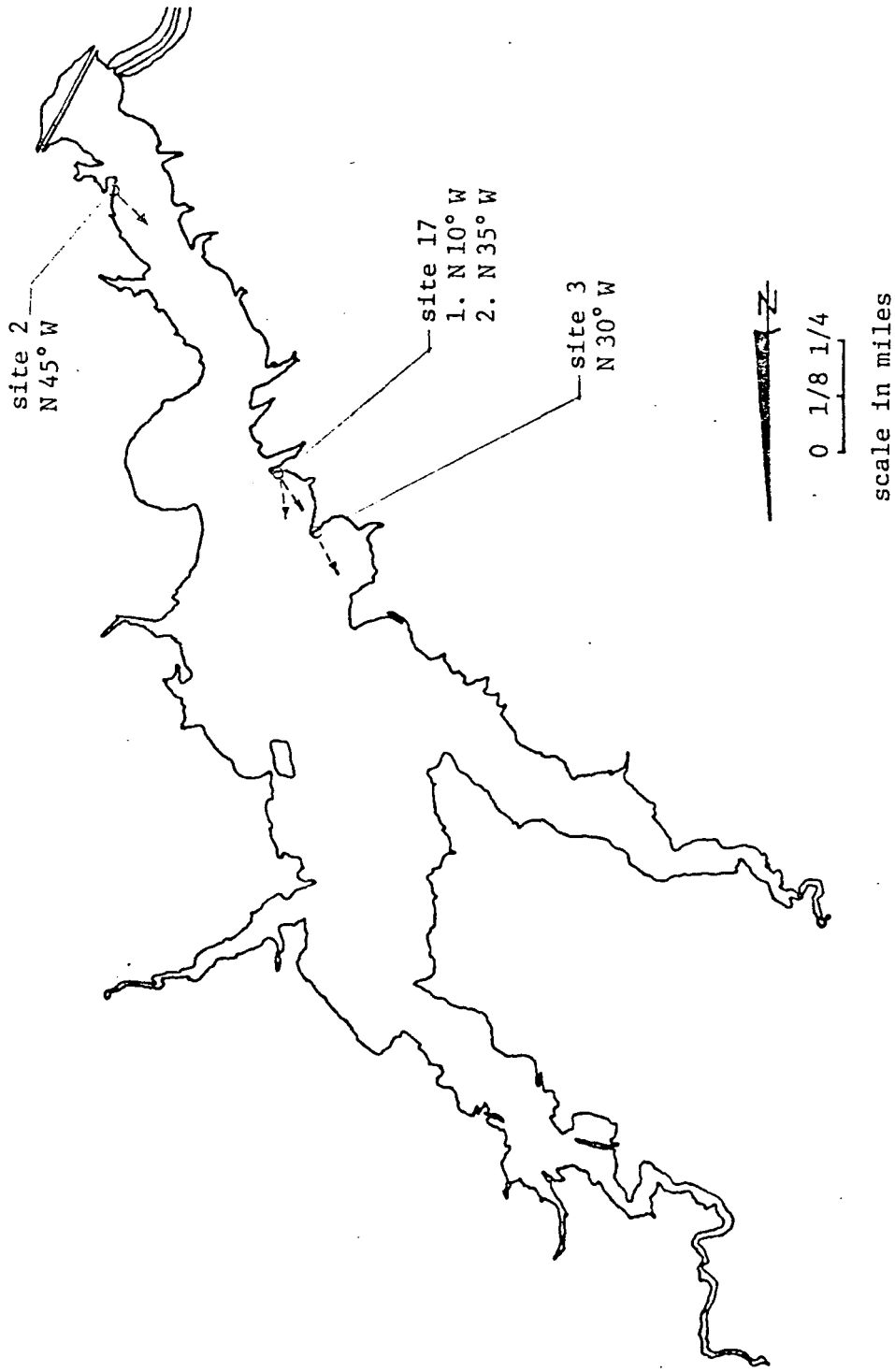


Figure 6. Big Creek Lake profile locations and directions

Plots of the existing profile and a plot estimating the original surface relative to the existing profile are shown in Figure 7. A plan view of the original topography is also provided along with a photograph of site 2 in Figures 8 and 9, respectively.

The height of the wave cut cliff at site 2 is 17.5 feet on approximately a 175% slope. Active erosion is taking place at site 2 evidenced by soil clumps with vegetation still on them along the shoreline.

The quantity of erosion taken place at site 2 was determined by planimetering the shaded area between the existing profile and the estimated original surface. This provides a volume of erosion per foot of shoreline at right angles to the measured profiles. At site 2 there is approximately 235 cubic feet of erosion per foot of shoreline.

Another variable measured at site 2 was the magnitude of horizontal shoreline recession, which is the distance from the intersection of the original surface with the lake level to the intersection of the existing profile with the lake level. At site 2 the horizontal shoreline recession is approximately 20 feet. Big Creek Lake was full in November of 1972, therefore this horizontal shoreline recession has occurred over a period of approximately 8 years. This is an average annual rate of horizontal shoreline recession of about 2.5 feet/year. Three sets of measurements made on the dates of December 11, 1979, March 3, 1980, and July 18, 1980 from the steel reference pin to the edge of the wave cut cliff indicated that no significant recession of the wave cut cliff edge or shoreline had occurred.

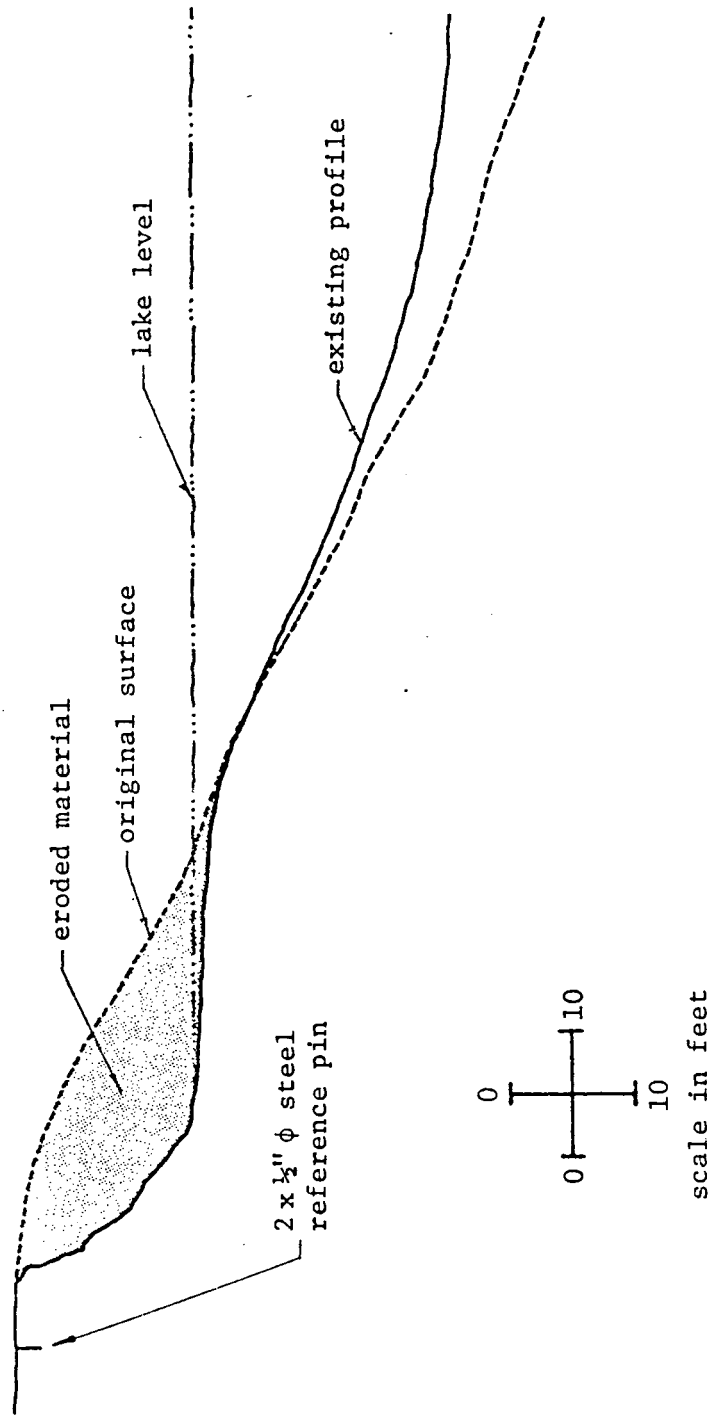


Figure 7. Site 2 Big Creek Lake: measured profiles in the direction N45° W

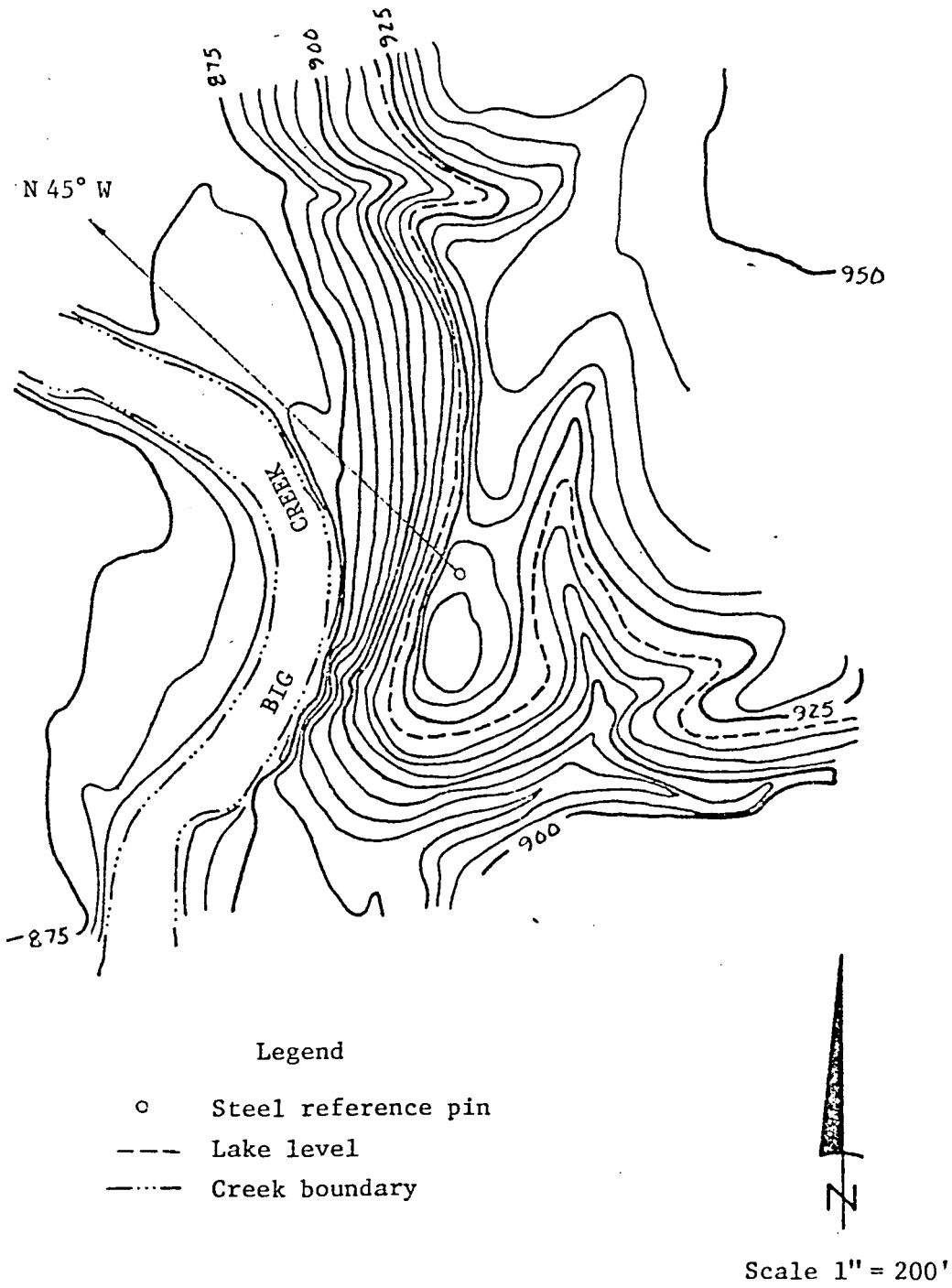


Figure 8. Site 2 Big Creek Lake plan view

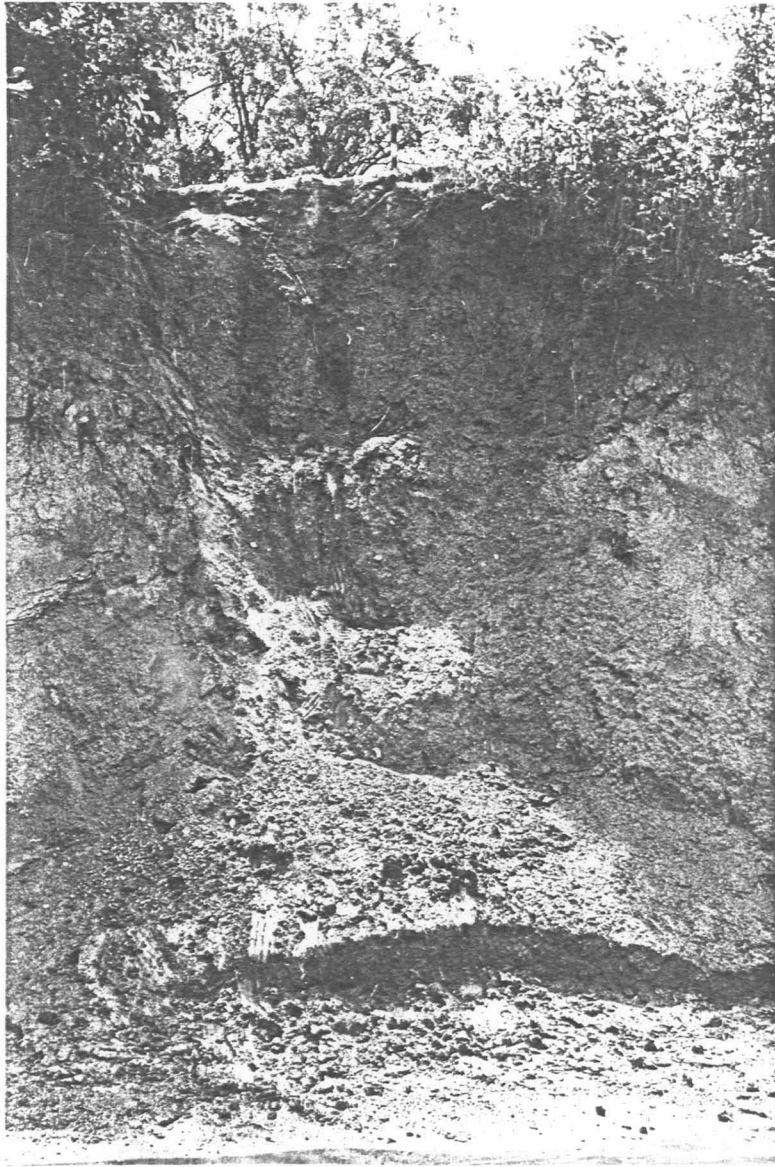


Figure 9. Site 2 Big Creek Lake

On July 18, 1980, an examination of the abrasion platform was made. Probing and sampling revealed that a layer of sediment (2 feet \pm) covered the abrasion platform. This observation revealed that a considerable amount of the sediment on the abrasion platform had been eroded and redeposited offshore since the previous observation on March 3, 1980, presumably resulting from the frequently high wind velocities and wave action of the spring months. At site 2 the slope of the existing near-shore profile is approximately 7% and the original surface had a slope of 25 to 50%.

Prior to the impoundment of the lake, site 2 was located adjacent to the channel of Big Creek (Figure 8), which had been eroding the toe of the valley side slope. The stream erosion created a steep bluff for wave action to attack when the reservoir was filled.

Site 3

Site 3 is located on the west side of Big Creek Lake approximately mid-way along the northwest-southeast axis of the lake. One profile was measured at site 3 with an azimuth of N 30°W. Plots of the measured existing profile and an estimated plot of the original surface relative to the existing profile are found in Figure 10. A plan view of the existing shoreline measured from the steel reference pin, a plan view of the original topography, and a photograph of the wave cut cliff are shown in Figures 11, 12, and 13, respectively.

The height of the wave cut cliff at site 3 is approximately 6.0 feet, measured vertically. The quantity of erosion which has taken

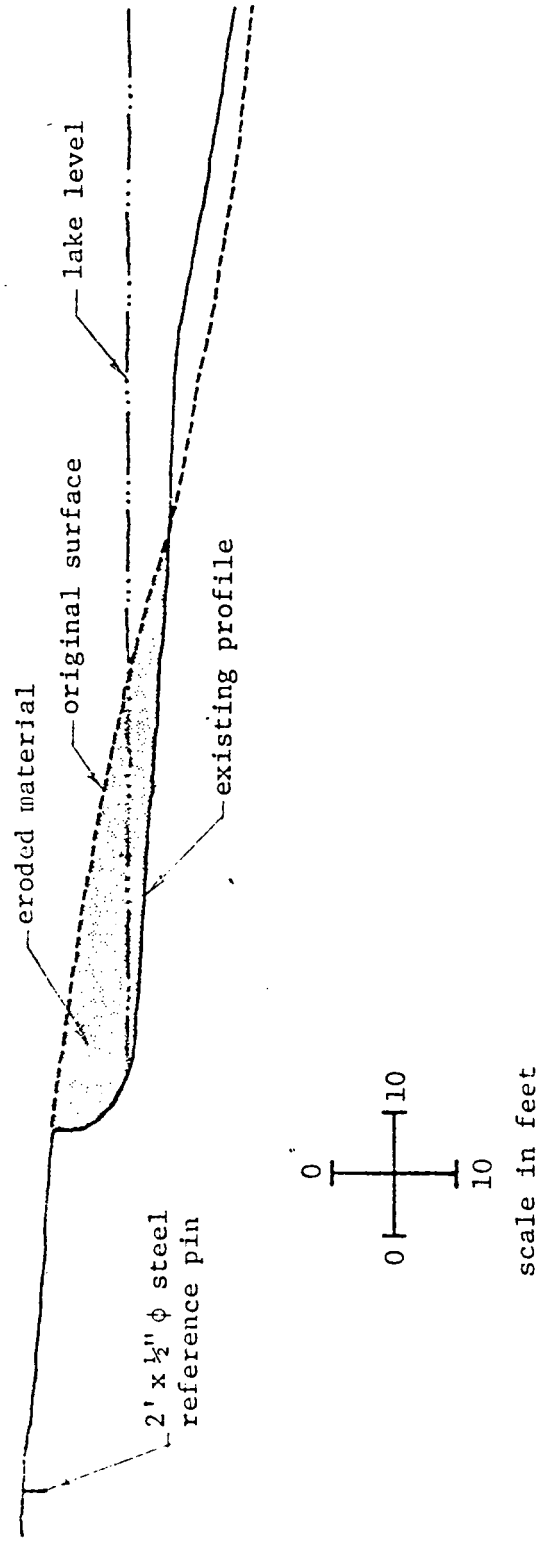


Figure 10. Site 3 Big Creek Lake: measured profiles in the direction N 30° W

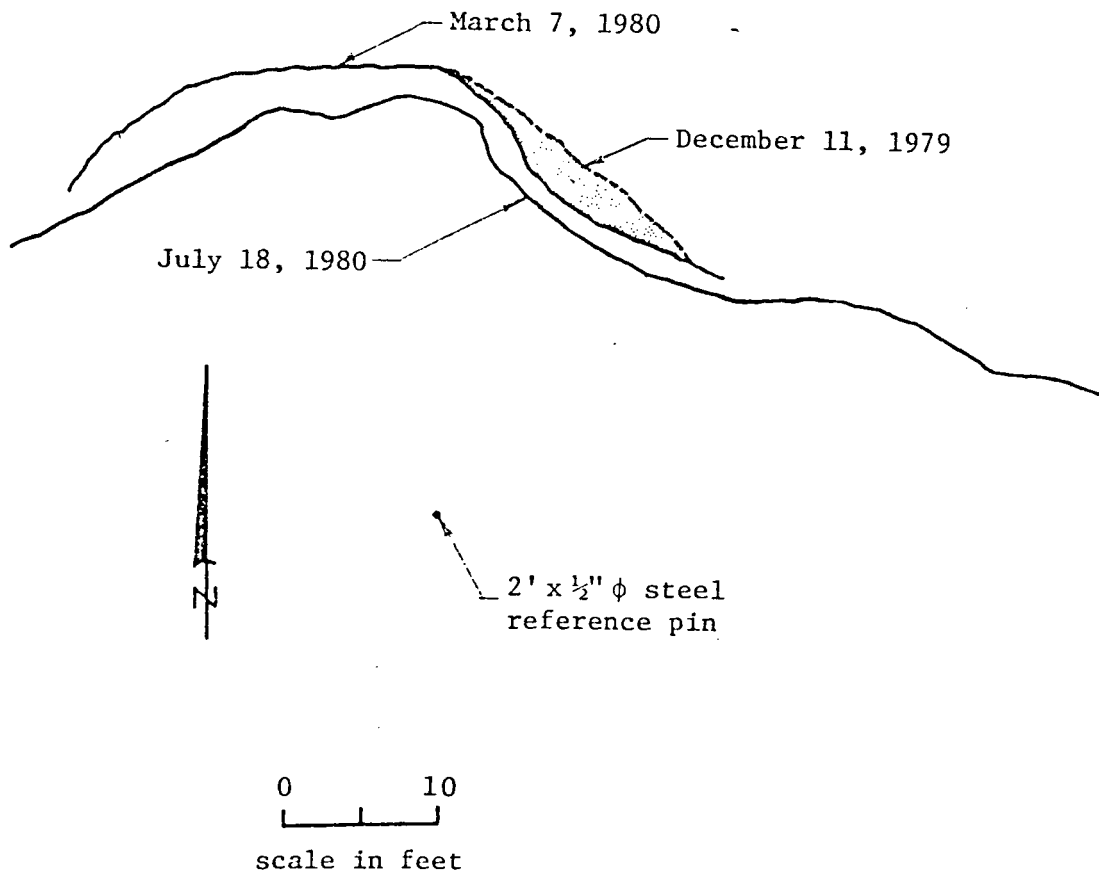


Figure 11. Site 3 Big Creek Lake: recession of wave cut cliff edge

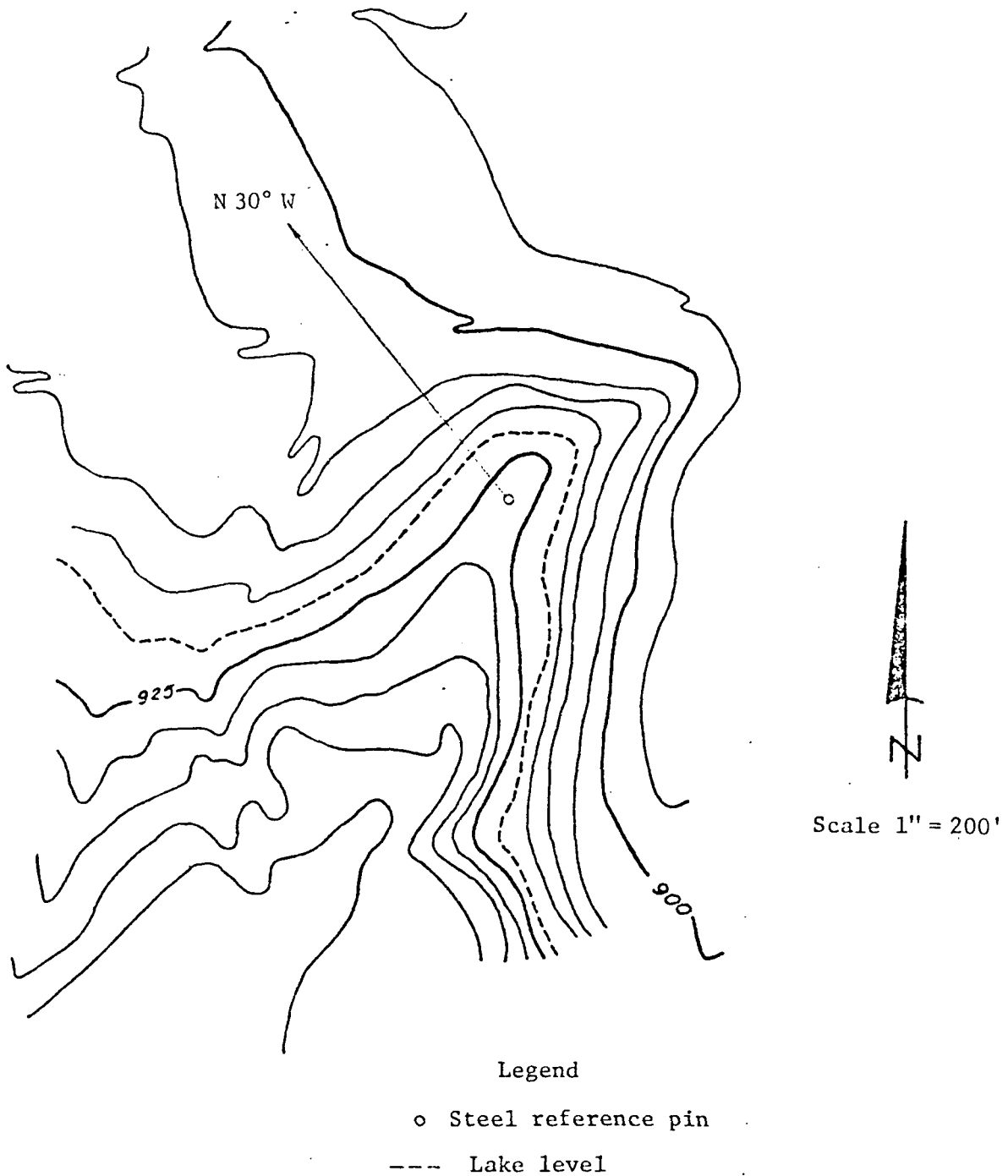


Figure 12. Site 3 Big Creek Lake plan view

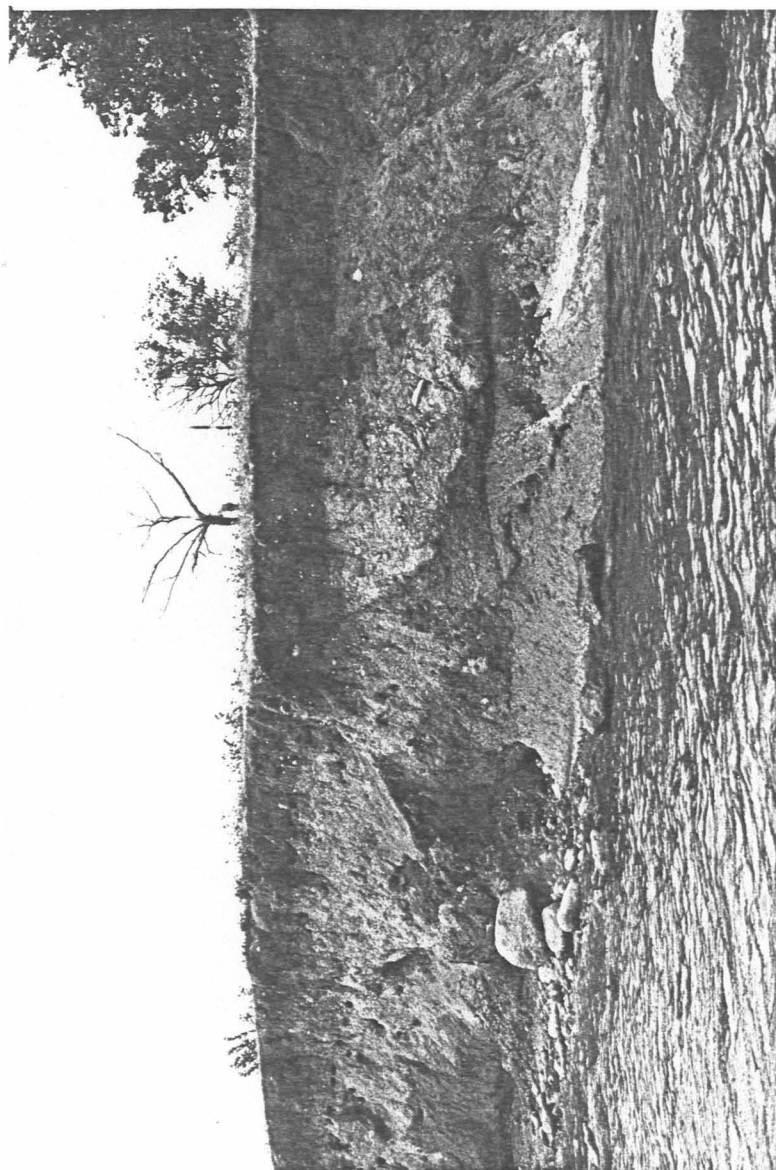


Figure 13. Site 3 Big Creek Lake

place at site 3, determined by the same method used at site 2, is approximately 180 cubic feet of erosion per foot of shoreline. Also measured at site 3 was the erosion which had taken place northeast of the steel reference pin (Figure 11). An inspection of the steel reference pin on December 11, 1979 and March 7, 1980 revealed that approximately 200 cubic feet of soil had eroded at site 3 between those dates. Another inspection of the steel reference pin on July 18, 1980 revealed that considerably more erosion had taken place due to the frequently high wind velocities and wave action of the spring months. Examination of Figure 11 reveals that the wave cut cliff edge receded as much as 4 feet in some areas. In the direction N 30°W the wave cut cliff edge had receded 3.5 feet since March 7, 1980.

Other variables measured at site 3 included the rate and magnitude of horizontal shoreline recession and the slopes of the original surface and existing nearshore profile. The horizontal shoreline recession at site 3 is approximately 32 feet, which is an average annual rate of horizontal shoreline recession of about 4.0 feet/year. The slopes of the existing nearshore profile and original surface are 6% and 15 - 20%, respectively.

Site 3 has several features which influence the erosion process. At several locations along the shoreline there are weak sand and gravel lenses near the base of the wave cut cliff which have been eroded away causing the soil to slump. These weak sand and gravel lenses, visible in the photographs of Figures 14 and 15, have very low shear strengths, around 60 psf (2.9 kN/m^2) at the shoreline. Also, many large



Figure 14. Site 3 Big Creek Lake - sand and gravel layers

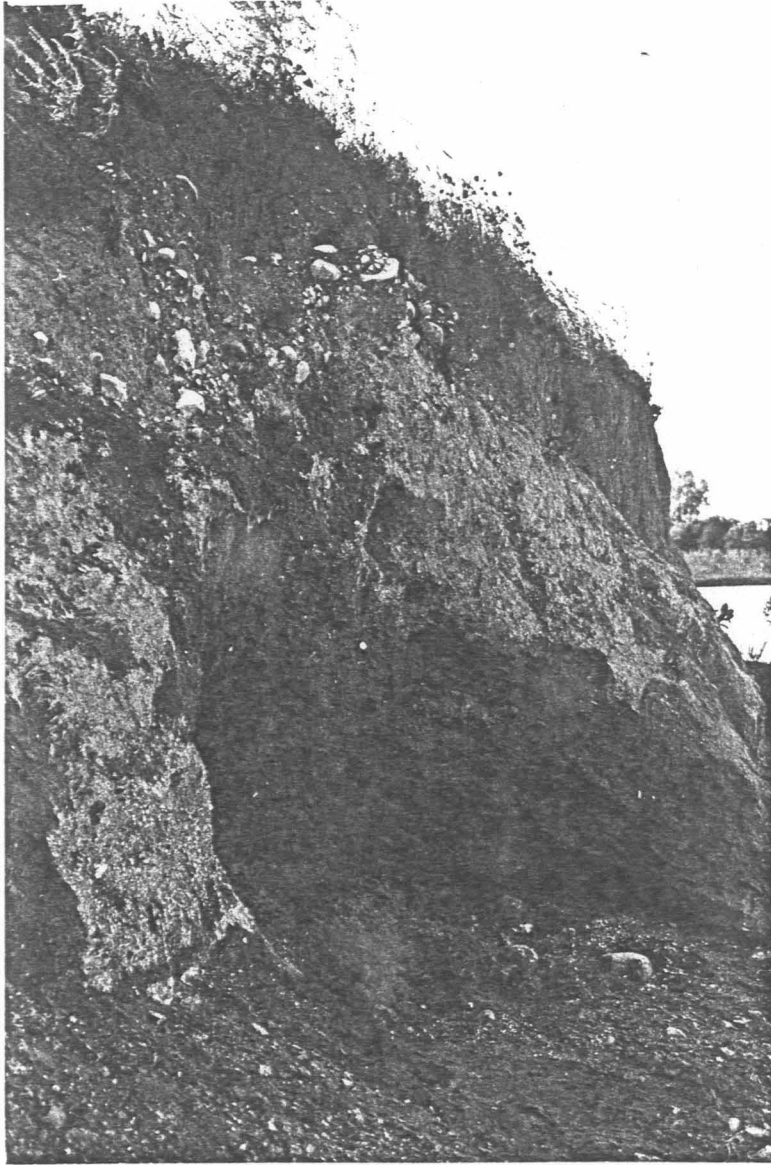


Figure 15. Site 3 Big Creek Lake - wave erosion of sand and gravel layers

boulders have been redeposited on the abrasion platform as the surrounding finer grained soil has eroded and the shoreline receded. These boulders have started to form a natural defense against wave action.

The weak sand and gravel lenses, the long effective fetch, and the orientation of the site with respect to the prevailing northwesterly winds, make site 3 one of the most actively eroding headlands on Big Creek Lake.

Site 17

Site 17 is located on the west side of Big Creek Lake directly southeast of site 3. Two profiles were measured at site 17, one in the direction of N 10°W and the other in the direction N 35°W. Plots of the existing profiles and estimated plots of the original surface relative to the existing profiles are found in Figures 16 and 17. A plan view of the original topography at site 17 and a photograph of the wave cut cliff are provided in Figures 18 and 19, respectively.

The height of the wave cut cliff at site 17 is approximately 6.5 feet measured vertically. The quantity of erosion which has taken place at site 17, determined through comparisons of the original surface and existing profile, is approximately 170 cubic feet of erosion per foot of shoreline in the direction of N 10°W, whereas 150 cubic feet of erosion per foot of shoreline has occurred in the direction N 35°W. The profile parallel to N 35°W is perpendicular with the existing shoreline.

The horizontal shoreline recession at site 17 in the direction N 10°W is approximately 32 feet, whereas parallel to N 35°W it is nearly 28

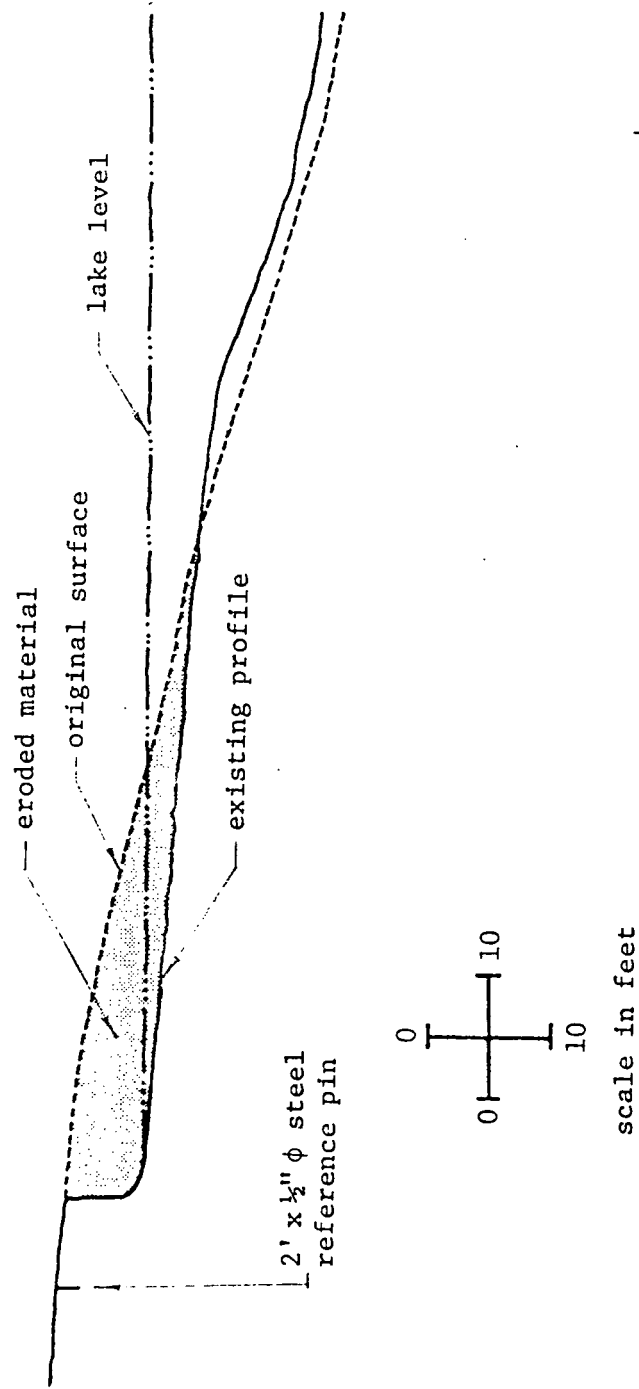


Figure 16. Site 17 Big Creek Lake: measured profiles in the direction N10° W

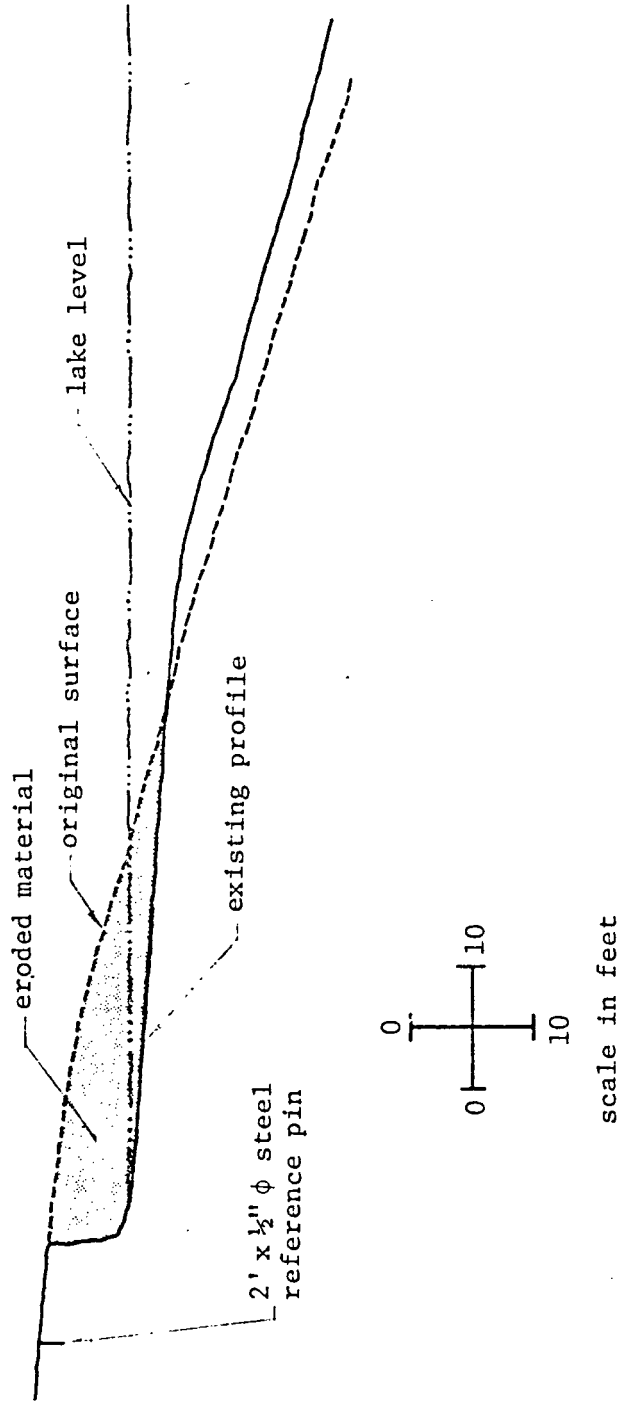
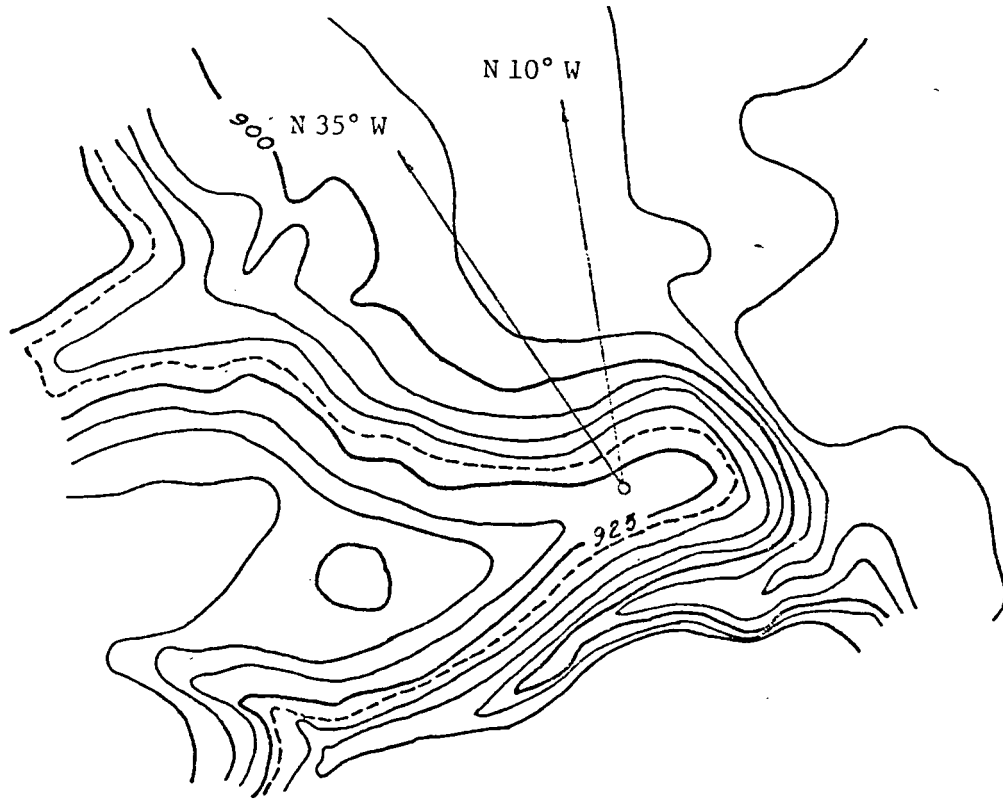


Figure 17. Site 17 Big Creek Lake: measured profiles in the direction N 35° W



Legend

- Steel reference pin
- Lake level



Scale 1" = 200'

Figure 18. Site 17 Big Creek Lake plan view



Figure 19. Site 17 Big Creek Lake

feet. This is an average annual rate of shoreline recession of 3.5 feet/year in the direction N 35 W. Measurements at site 17 indicate that there has been no significant recession of the top of the wave cut cliff between the dates of December 11, 1979 and July 18, 1980; however, measurements made on July 18, 1980 indicated that the base of the wave cut cliff had receded 3.0 feet. Examination of the abrasion platform at site 17 revealed that a thin veneer of sediments covers the platform. These sediments range from coarse sands and gravels near the shoreline to finer silts and sands offshore.

Measurements of the existing nearshore profile slopes indicate that parallel to both directions, N 10°W and N 35°W, the slopes are approximately 7.5%. Measurements of the original surface slopes are 25% in both directions, N 10°W and N 35°W.

Similar in configuration and orientation to site 3, site 17 contains some weak layers of sand and gravel, with shear strengths less than 200 psf (9.6 kN/m²), which have eroded at the waterline causing the soil mass to slump into the lake.

Table 1 presents a summary of the Big Creek field study.

Prairie Rose Lake

Shoreline profiles were measured at two selected sites at Prairie Rose Lake. One of the sites, site A, is located at the northeast end of the lake and the other site, site B, is located at the southwest end of the lake as shown in Figure 20.

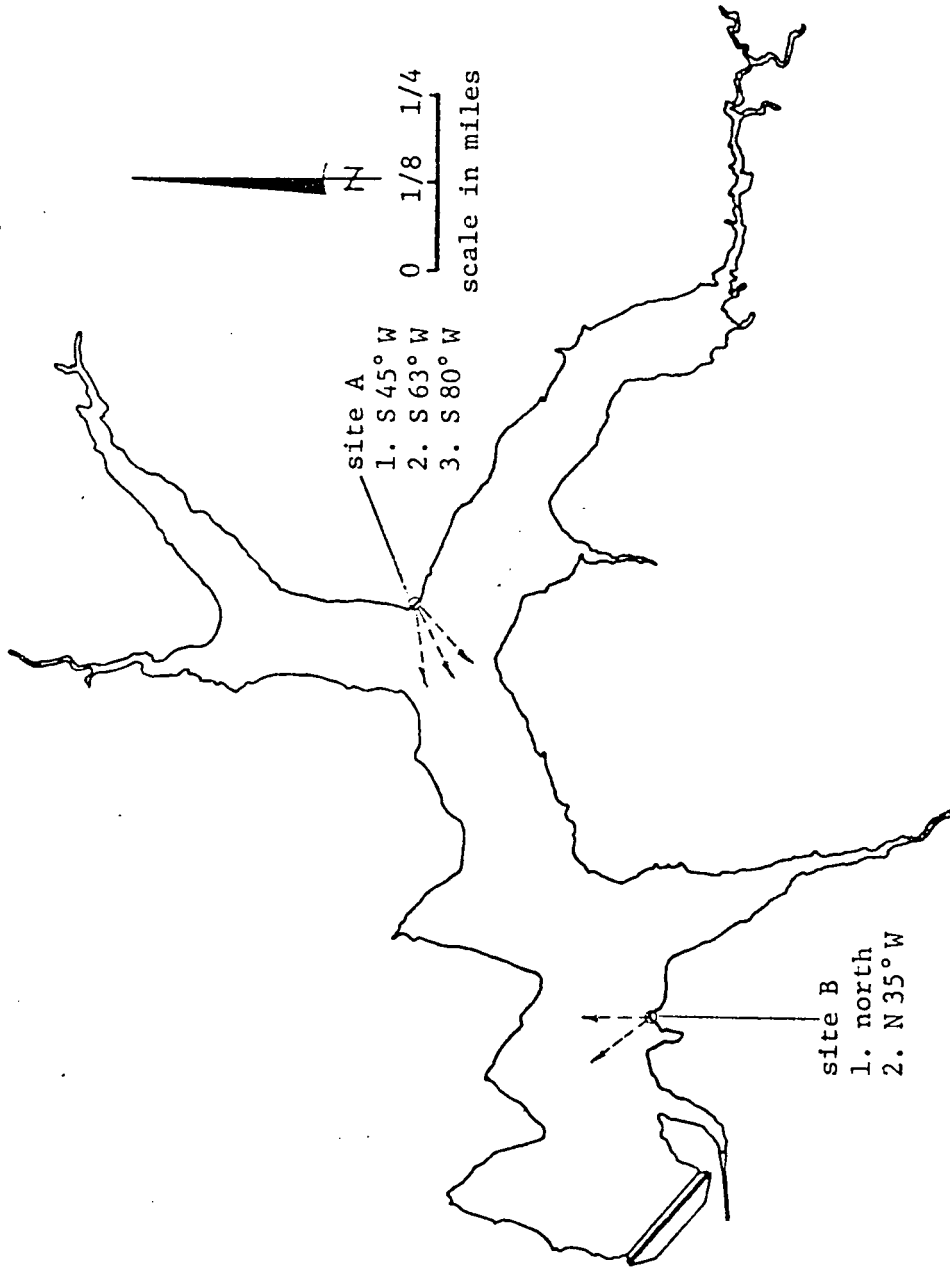


Figure 20. Prairie Rose Lake profile locations and directions

Table 1. Big Creek field study summary

Location	Wave cut cliff height (feet)	Nearshore profile slope (%)	Original surface slope (%)	Volume of eroded material (ft ³ /ft of shoreline)	Annual rate of erosion (ft/yr)	Effective fetch (ft)
Site 2	17.5	7	25 - 50	235	2.5	1640
Site 3	6.0	6	15 - 20	180	4.0	2640
Site 17	6.5	7.5	25	150	3.5	2750

Site A

At site A profiles were measured in three directions from the steel reference pin: S45°W, S 63°W, S 80°W. All three profiles are nearly the same, therefore, only the profile measured in the direction S 63°W is discussed.

A plot of the existing profile and an estimated plot of the original surface are shown in Figure 21. A plan view of the original topography and a photograph of site A are shown in Figures 22 and 23, respectively. It was difficult to plot the original surface on the existing profile of site A because of no distinct slope breaks or landforms. Also, it appears that there may have been some reshaping and grading of the headland at this site prior to the placement of riprap; therefore, no attempt was made to estimate the quantity of erosion which had taken

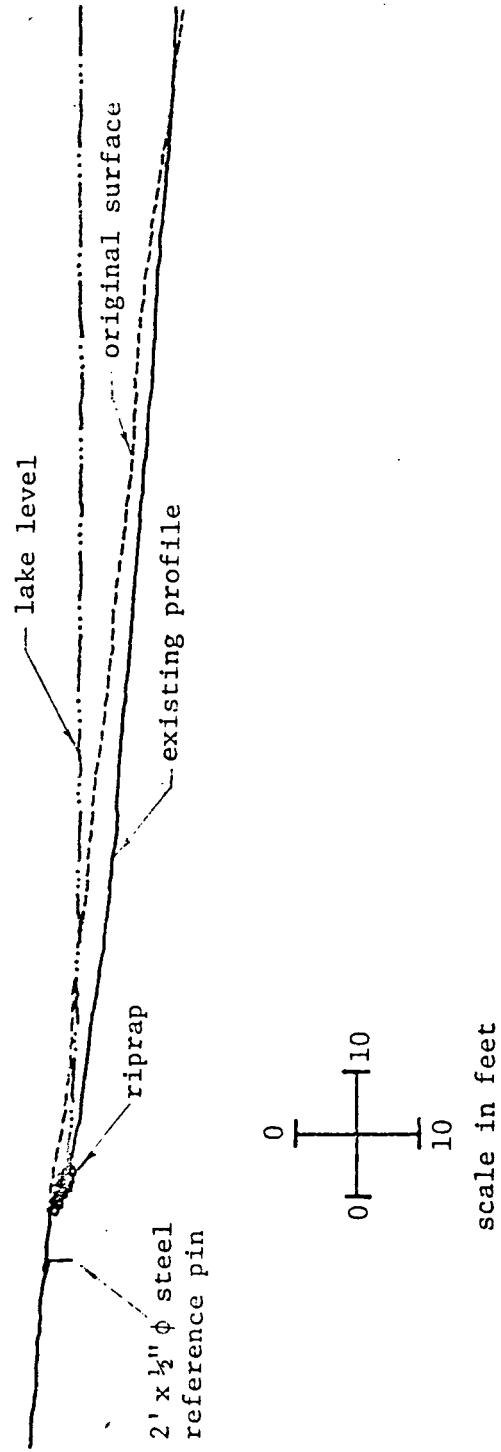


Figure 21. Site A Prairie Rose Lake: measured profiles in the direction S 63° W

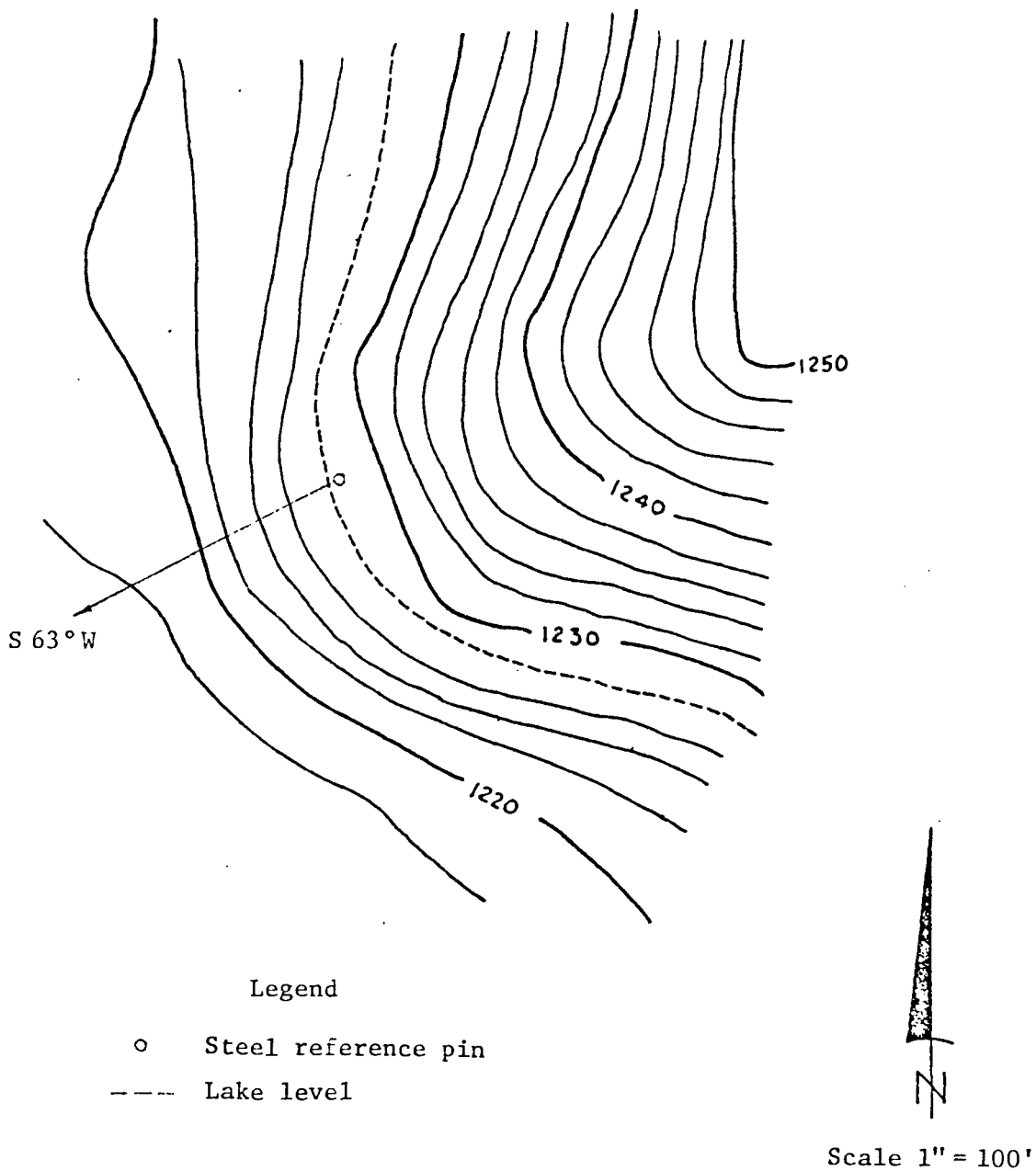


Figure 22. Site A Prairie Rose Lake plan view

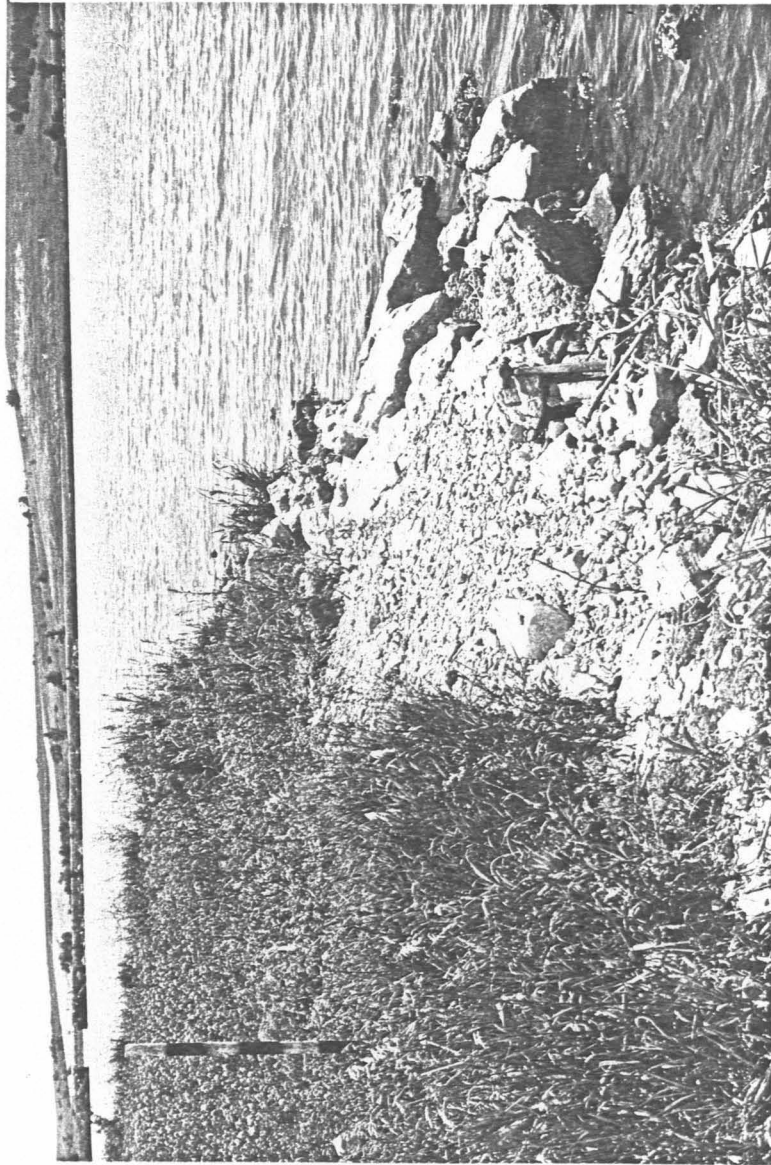


Figure 23. Site A Prairie Rose Lake

place. Measurements of the slopes of the existing nearshore profile and original surface were made and found to be 7% and 10%, respectively.

Site B

At site B, profiles were measured in two directions from the steel reference pin: North and N 35°W. The profile measured in the north direction is discussed relative to erosion quantities and shoreline recession because this profile is most nearly perpendicular to the existing shoreline. In Figure 24 a plot of the measured existing profile and an estimated plot of the original surface is shown. A plan view of the original topography and a photograph of the site are found in Figures 25 and 26, respectively.

The height of the wave cut cliff is approximately 12.5 feet on a 70% slope. The quantity of erosion which had taken place at site B, determined through a comparison of the existing profile and original surface, is approximately 50 cubic feet of erosion per foot of shoreline. The horizontal shoreline recession at site B is approximately 10 feet, which is an average annual rate of horizontal shoreline recession of about 0.5 feet/year. On July 15, 1980 the wave cut cliff at site B was remeasured and it was found that a small amount of erosion had occurred upslope from the shoreline but no significant recession had taken place.

Measurement of the original surface slope reveals a slope of 25 - 40%. No measurement could be made of the existing nearshore profile slope because it has only started to develop.

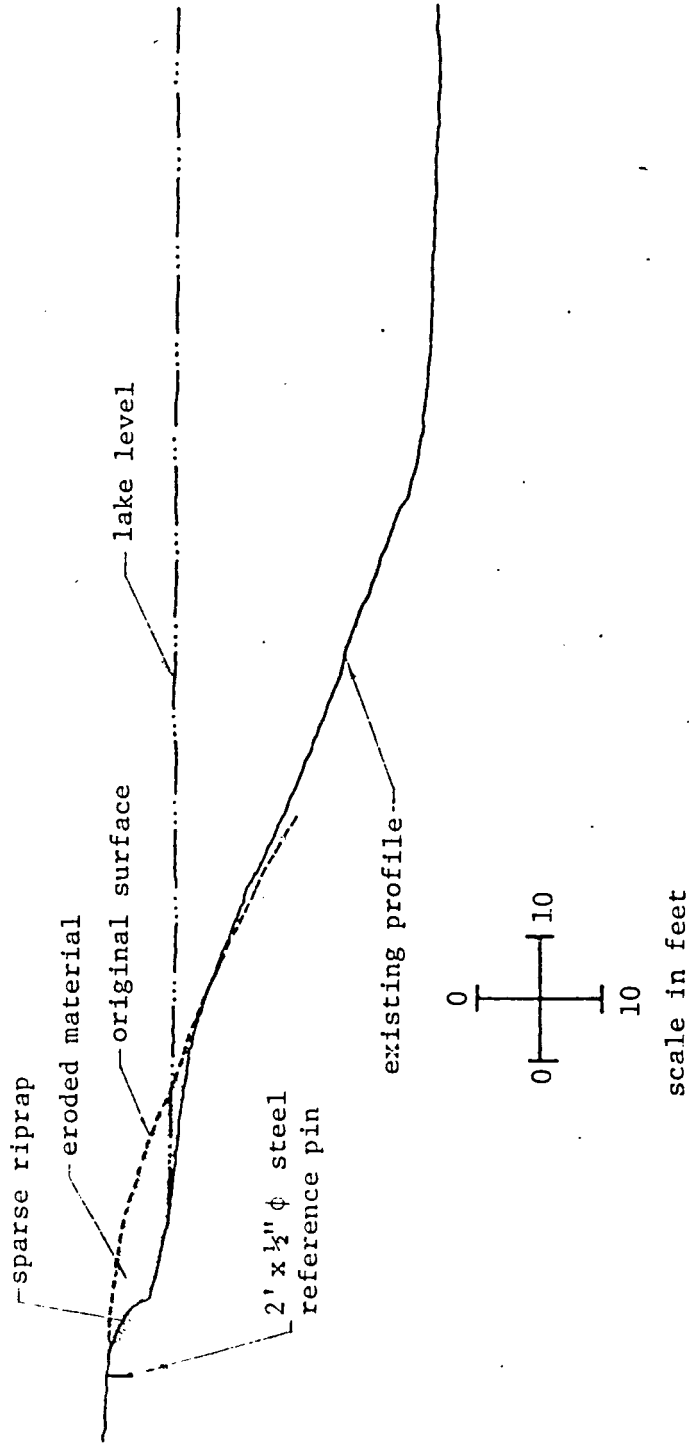


Figure 24. Site B Prairie Rose Lake: measured profiles in the direction north

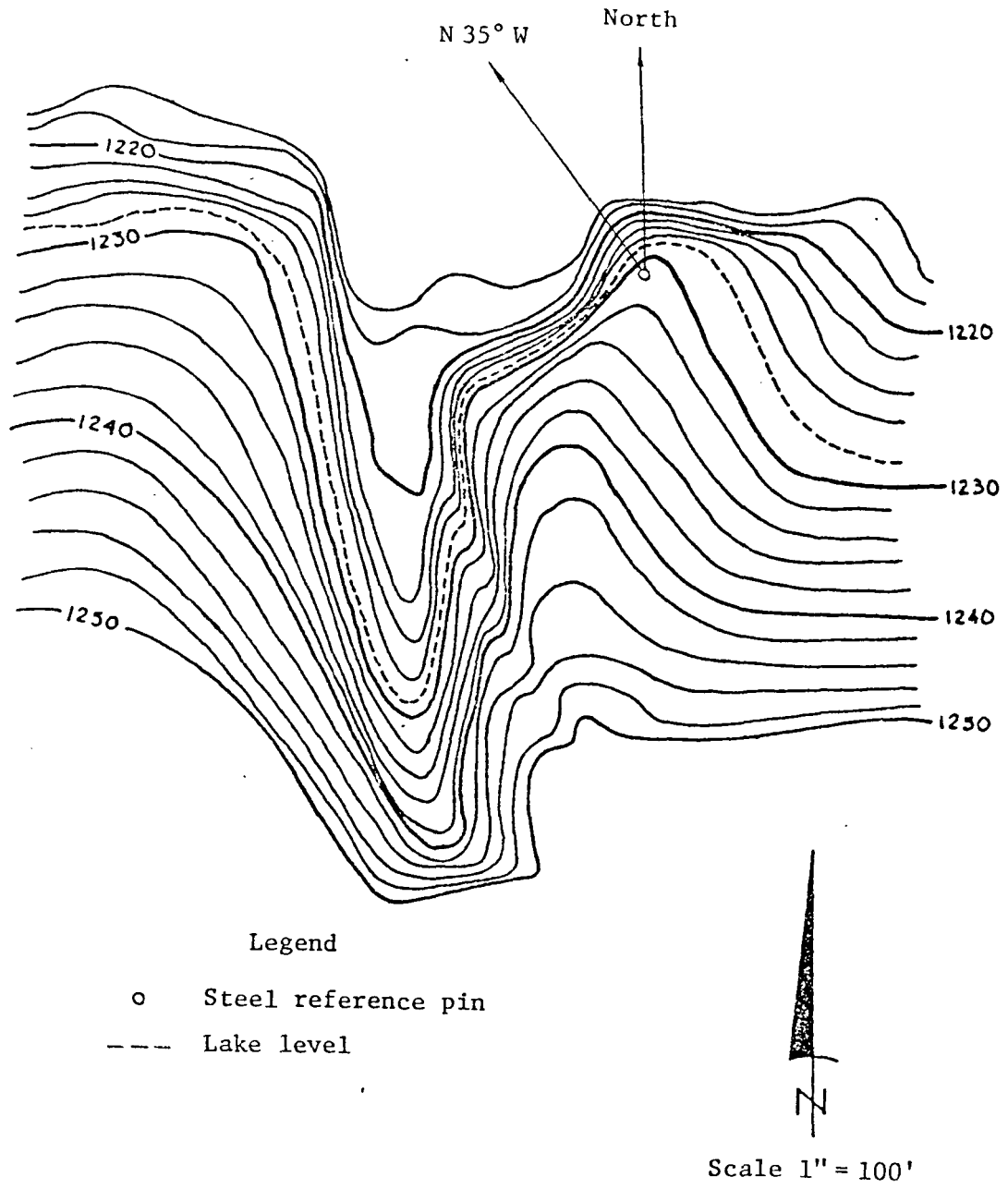


Figure 25. Site B Prairie Rose Lake plan view



Figure 26. Site B Prairie Rose Lake

DISCUSSION OF FIELD STUDY

The objective of this aspect of the study is to gain a better understanding of the shoreline erosion process on man-made lakes. This knowledge would give an engineer the ability to predict which areas would be most susceptible to erosion and to estimate the maximum quantity and horizontal extent of erosion which might occur before an equilibrium profile is reached.

The benefits of predicting the location and estimating the quantity of shoreline erosion are many. One of the more obvious is controlling a sediment source. Many artificial lakes are receiving large amounts of sediment from shoreline erosion. Estimates for Big Creek Lake indicate that 55% of the sedimentation is due to shoreline erosion and 45% is due to tributary streams' contributions. The details of the calculations are in Appendix C and the following paragraphs summarize the procedure.

The Big Creek Lake basin consists of two types of drainage, bluffs and uplands. The sediment contribution from bluff drainage is approximately one order of magnitude greater than the sediment contribution from upland drainage (Upper Mississippi River Basin Coordinating Committee, 1970). Sediment yields for bluff drainage are approximately 500 tons/mi²/year and for upland drainage approximately 50 tons/mi²/year, depending on the size of the drainage area. Using these sediment yield values and areas of bluff and upland drainage obtained from U.S.G.S. topographic maps, the sediment contribution from tributary streams was determined to be 45,850 tons for the period since the lake was opened.

The sediment contribution from shoreline erosion was determined by selecting an idealized erosion profile for each of the three erosion classifications, determining the quantity of erosion per foot of shoreline and multiplying that quantity times the length of shoreline having that erosion classification. This yielded a shoreline erosion sediment contribution of 55,070 tons. The idealized erosion profiles were selected on the basis of field observations and are conservative in the volume of erosion estimated. For example, the idealized erosion profile selected for the severe erosion classification yielded an average erosion volume of 130 cubic feet per foot of shoreline, whereas calculations based on field observations at specific sites indicated values ranging from 150 to 235 cubic feet per foot of shoreline. These calculations, even though they appear unusually high and will require modification with the collection of more field data, indicate the importance of controlling shoreline erosion as a potential sediment source.

The prevention of inaccessible beach areas and unsightly scars along the shoreline is another benefit of controlling shoreline erosion. Finally, a good understanding of shoreline erosion processes may lead to the design of shorelines which are less likely to erode from wave action.

The Shoreline Erosion Process

Shoreline erosion is a dynamic process of continual wave attack and sediment movement until an equilibrium profile develops. The equilibrium profile is attained when the horizontal recession of the

shoreline ceases. There may be movement of sediment within the nearshore zone, but no net gain or loss. The knowledge of how equilibrium profiles on artificial lakes develop would be very useful in predicting the maximum extent of shoreline erosion.

Equilibrium profile

The relationships between the equilibrium profile and the many variables which influence its formation are very complicated, making a field study of this process very difficult. Rector (1954), realizing this difficulty, conducted a laboratory study of the formation of equilibrium profiles in which some of the variables could be controlled. Rector's work consisted of subjecting surfaces of known material characteristics and slopes to varying wave conditions and examining the formation of the equilibrium profiles. Rector reported that the most important variables influencing the formation of equilibrium profiles are: wave characteristics, material properties, original surface slope, and fluctuating lake levels.

Wave characteristics The length, height, period, and energy of a wave are all important in the formation of an equilibrium profile (Figure 27). As a wave approaches the shore, its total energy consists of two parts, kinetic energy and potential energy. The kinetic energy of the wave is due to the water particle velocities and the potential energy is due to the fluid mass above the wave trough (United States Army Coastal Engineering Research Center, 1973). Airy, as cited by the United States Army Coastal Engineering Research Center (1973), reported

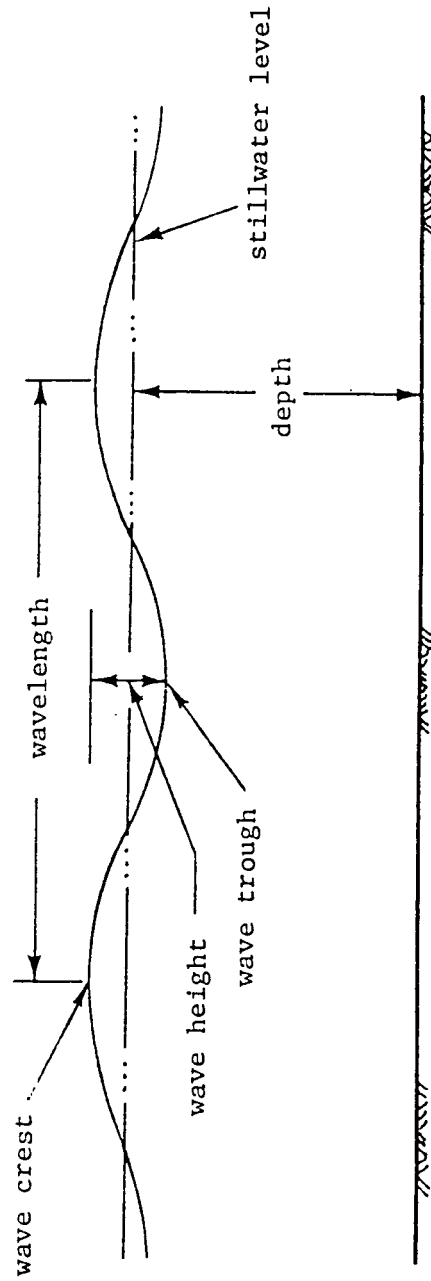


Figure 27. Wave characteristics

that if the potential energy is measured relative to the mean water level and all waves are propagated in the same direction, the potential and kinetic energy of a wave will be equal. Therefore, the total energy in one wave length per unit crest width may be given by the following relationship:

$$E = E_{\text{kinetic}} + E_{\text{potential}} = \frac{\rho g H^2 L}{16} + \frac{\rho g H^2 L}{16} = \frac{\rho g H^2 L}{8} \quad (11)$$

where

E = total wave energy

ρ = mass density of water

g = acceleration due to gravity

H = wave height

L = wave length

A significant amount of this total energy is dissipated in the nearshore and shore regions.

Material properties The properties of the materials comprising the original surface also influence the formation of the equilibrium profile. Characteristics of the material such as particle size, shape strength and density influence how wave action shapes the profile.

Original surface The slope of the original surface determines how much material is available for the formation of the nearshore profile and also the horizontal location of where wave action causes movement of materials (Rector, 1954).

Fluctuating lake levels The formation of the equilibrium profile is greatly influenced by fluctuations in the lake level. If an equilibrium profile has formed at a certain lake level and for some reason the lake level would raise or lower, the reshaping process would be renewed.

Field Observations

With these variables in mind, the measured profiles at Big Creek Lake were analyzed. Big Creek Lake was selected for examination because of the significant amount of erosion which has taken place; and unlike Prairie Rose Lake which has approximately 30% of its shoreline riprapped, Big Creek Lake has few shoreline erosion countermeasures.

The slopes of the nearshore profiles of all three profiles show striking similarity. The nearshore profile slope of site 2 is approximately 7.0%, site 3 is approximately 6.0%, and site 17 is approximately 7.5%. The similarity of these nearshore profile slopes is the basis for the following shoreline erosion model. Although the nearshore slopes are very similar, there are differences in other aspects of the shores. Variables which may have influenced these differences are discussed below.

Site 2 This site is quite different from the other two measured sites at Big Creek Lake. The most obvious difference is the height of the wave cut cliff, which is 17.5 feet measured on approximately a 175% slope. This, in part, is due to the very steep original surface slope

at site 2. The original surface slope was so steep, 25 to 50%, that any horizontal recession of the shoreline results in a large quantity of eroded material and produces a relatively high cliff. The steep slope of the original surface also appears to have had a significant influence on the nearshore profile development. Unlike sites 3 and 17 which have developed terraces as part of their nearshore profiles, site 2 does not have this depositional feature. It appears that the steepness of the original surface caused the eroded material to be deposited at some position offshore instead of forming a part of the nearshore profile. It is speculated that if the angle of repose of the terrace sands is less than the original surface slope, the sand will slide offshore. Longshore currents may also account for the absence of a terrace here.

Sites 3 and 17 The nearshore profile developed at site 3 is similar to the nearshore profile developed at site 17. The height of the wave cut cliff at site 3 is 6.0 feet measured vertically and the slope of the original surface between 15 and 20%. The gradual slope of the original surface at site 3 may have resulted in the terrace becoming an integral part of the nearshore profile. The coarseness of the eroded material at site 3 may also have contributed to the formation of this depositional feature (Appendix A).

Both sites are subjected to approximately the same wave conditions and have developed similar nearshore profiles. The height of the wave cut cliff at site 17 is approximately 6.5 feet measured vertically and the slope of the original surface is approximately 25%. As with site

3, the relatively gradual slope of the original surface has resulted in the terrace forming an integral part of the nearshore profile.

SHORELINE EROSION MODEL ASSUMPTIONS

Based on observations at the three measured profiles of Big Creek Lake, a conceptual model can be developed for the formation of the erosion profile. The small number of field observations limit the model, but it provides a basis for future observations and perhaps for predicting the maximum extent of shoreline erosion at specific sites.

Discussion of Model Assumptions

In order to present the shoreline erosion model certain assumptions are made, which include wave theory and classification, water particle motion, wave generation, water level fluctuations, and nearshore profile slope.

Wave theory and classification

The three-dimensional nature of waves, the irregularity of their shape, and the variability of their occurrence make their mathematical description very difficult. Numerous attempts have been made at developing theoretical relationships which describe wave motion, however, the problem lies in obtaining agreement between theory and field observations (United States Coastal Engineering Research Center, 1973).

For this study the classical small-amplitude or linear wave theory proposed by Airy, as cited by the United States Army Coastal Engineering Research Center (1973), is utilized. The small-amplitude wave theory was selected for its simplicity and ease of application. It should be noted that for special circumstances such as shallow water and very

shallow water near the breaker zone other wave theories will more accurately predict wave motion.

The classical theory of small amplitude waves, as discussed by the United States Army Coastal Engineering Research Center (1973), is used to describe simple oscillatory waves. Simple waves can be described in elementary mathematical terms. Examples of simple waves are sinusoidal or simple harmonic waves since the profiles of these waves can be described by either a sine or cosine function. Waves are considered oscillatory if the water particle motion can be described by orbits (Figure 28). Once a wave form has developed, it will either move relative to the fluid, move with the fluid, or stand still. The waves discussed here are considered progressive, that is, the wave form moves relative to the fluid.

Simple sinusoidal oscillatory waves are generally described by their length, height, period, and depth of water in which they occur. The depth of water in which a wave progresses has a significant effect on the wave's characteristics, therefore waves are classified according to the depth of water in which they occur. Waves are classified as either deep water waves, transitional water waves, or shallow water waves based on a criterion known as relative depth, which is the ratio of the water depth to the wave length. Deep water waves occur when the ratio is greater than one half, transitional water waves occur as the ratio varies from one twenty-fifth to one half, and shallow water waves occur when the ratio is less than one twenty-fifth. For this model, it is assumed that deep water wave characteristics prevail, that is, the

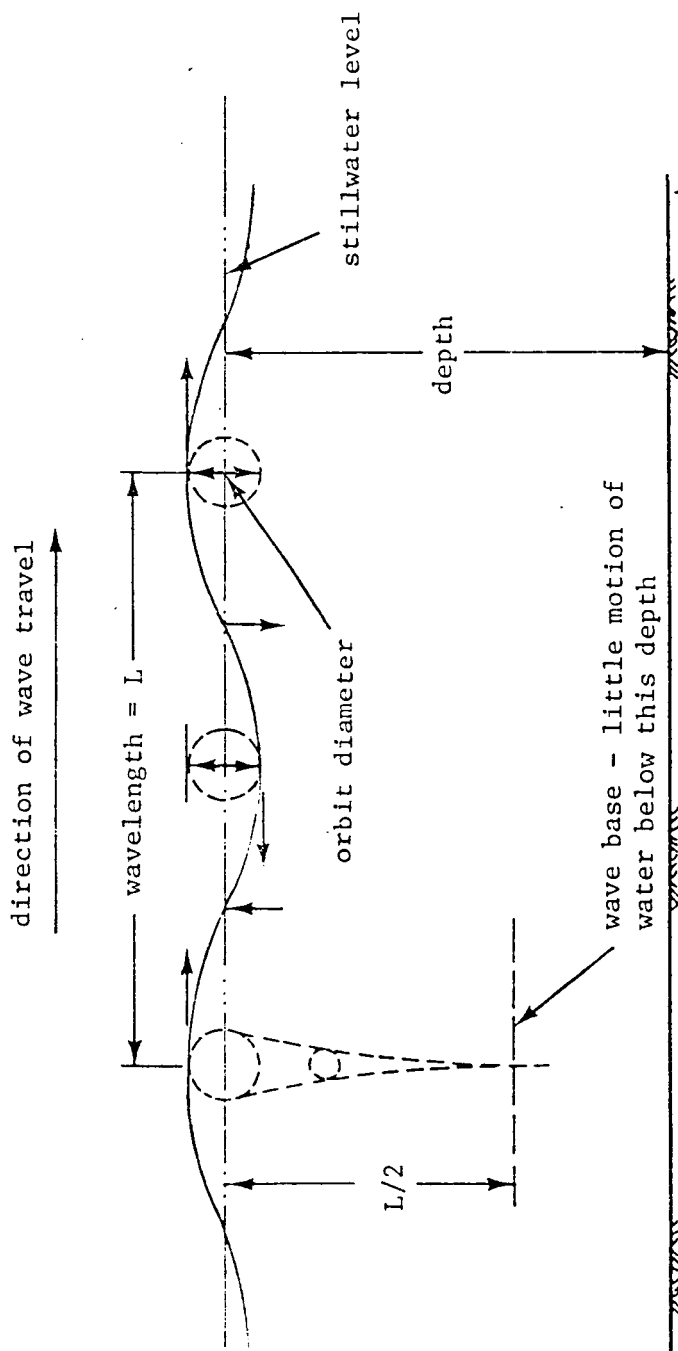


Figure 28. Orbital movement of water particles in a deep water wave

wave's characteristics are independent of depth. This assumption is reasonable for the height of the waves being considered and the average depth over which they occur. On most artificial lakes, waves are usually generated in areas of relatively shallow water and move into deeper water where their characteristics are independent of depth, for most wave heights. The study of shallow water waves is quite complex because of the elliptical orbits of the water particles and the interaction of this water particle movement with the lake bottom.

Water particle movement

The movement of water particles within a wave form is another variable to consider. As discussed previously, the movement of water particles in shallow water waves is an elliptical path. However, the water particle movement in deep water waves is circular. The circular paths followed by the water particles of deep water waves decreases in diameter exponentially to a depth equal to one half the wave length, where below there is little or no water particle displacement (United States Army Coastal Engineering Research Center, 1973). This depth, equal to one half the wave length, is termed the wave base (Figure 28).

Wave generation

The three measured profiles at Big Creek Lake are subjected to varying wave conditions as the wind changes its velocity and direction, therefore, the profiles are being shaped by varying wave conditions. For this model it is assumed that the wave characteristics resulting from the most frequently occurring wind velocity in a northwesterly

direction control the equilibrium profiles. For Big Creek Lake the northwesterly winds have the highest average wind speed, approximately 13.5 miles per hour, and occur most frequently (National Oceanic and Atmospheric Administration, 1978).

Water level fluctuations

Although water level fluctuations will have a significant effect on equilibrium profile formation, this variable will be omitted for simplicity.

Slope of nearshore profile

It is assumed that the nearshore profile slope is 7%. This is consistent with the field observations at Big Creek and Prairie Rose Lakes.

SHORELINE EROSION MODEL

There are three purposes for developing a shoreline erosion model. First, it allows for the prediction of the maximum horizontal shoreline recession and wave cut cliff height; second, it provides a basis for future field observations; third, it includes the design of stable shorelines as a part of reservoir design.

Existing Methods for Predicting Shoreline Erosion

Presently, there are no accurate methods for determining the magnitude of horizontal shoreline recession and wave cut cliff heights, however, the Missouri River Division of the Corps of Engineers in Omaha, Nebraska has developed a general approach to predicting the ultimate extent of shoreline erosion.¹ Field observation at the Ft. Randall and Garrison Lake projects indicated that stable nearshore profiles develop on a 1 on 14 slope (depending on the material), wave cut cliffs develop on a 4 on 3 slope, and the slope beyond the abrasion platform develops on a 1 on 3 slope. Using these slope dimensions to form a template, the ultimate extent of shoreline erosion is estimated at the point where the area of the eroded material equals the area of the material deposited offshore. Proper adjustments must be made in areas adjacent to old river channels and areas with longshore drift. Using a concept similar to the one developed by the U.S. Army Corps of Engineers and observational data

¹Personal communication with Ross Black, Iowa Geological Survey, Iowa City, Iowa.

collected at Big Creek Lake, a shoreline model is proposed. This model enables the prediction of the potential maximum horizontal shoreline recession and the maximum wave cut cliff height of the equilibrium profile, knowing the slope angle of the original surface and the wave base depth.

The discussion of the shoreline erosion model consists of examining two hypothetical profiles normal to the shoreline, a gradually sloping profile and a steeply sloping profile (Figures 29 and 30).

The variables include wave base, potential maximum length of the original surface influenced by wave action, potential maximum horizontal shoreline recession, wave cut cliff height, and formation of a terrace.

Symbols for the variables are:

WBD - wave base depth

X_m - potential maximum horizontal length of original surface influenced by wave action without terrace development

X_r - potential maximum horizontal recession of original shoreline without terrace development

X_a - horizontal length of equilibrium nearshore profile

T_x - horizontal length of terrace development

H - wave cut cliff height

i_o - original surface slope in percent

i_e - equilibrium nearshore profile slope in percent

ℓ - length of abrasion platform

A - angle between abrasion platform and horizontal

B - angle between abrasion platform and original surface

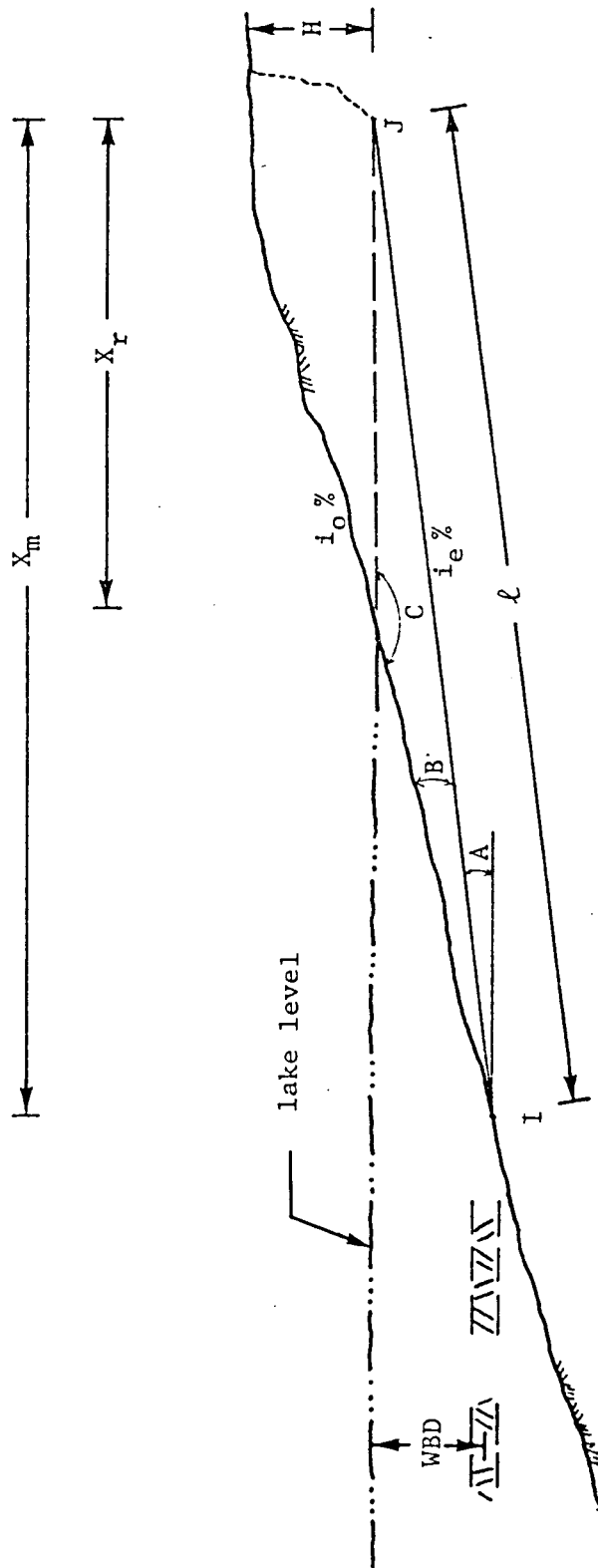


Figure 29. Gradually sloping original surface

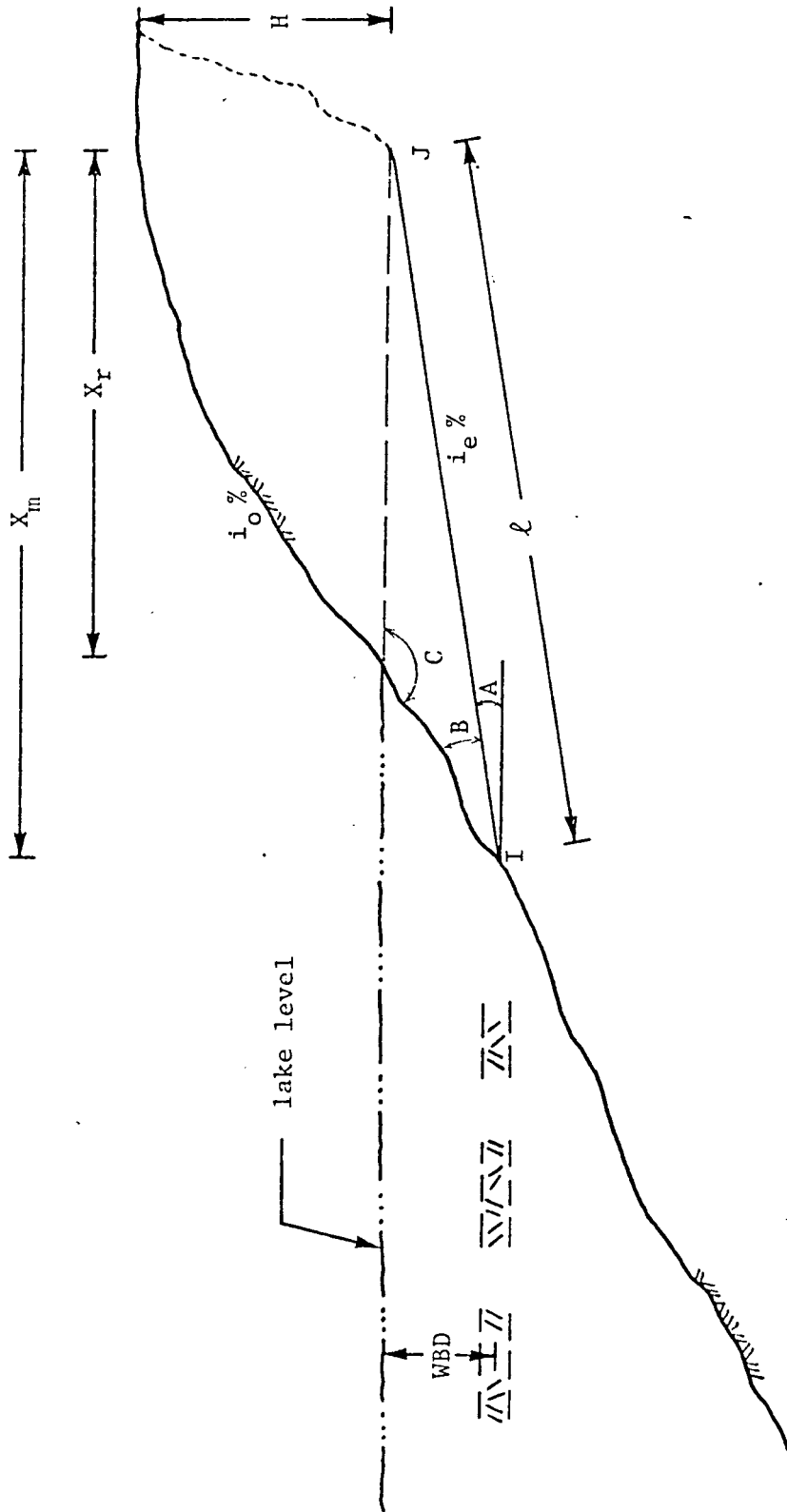


Figure 30. Steeply sloping original surface profile

Wave base depth

The depth to which significant water particle motion is influenced by wave action is known as wave base. Wave base is an average position and cannot be designated as a discrete depth. For a given set of wind conditions, effective fetch, and average water depth, wave height and length can be calculated according to deep water small-amplitude wave theory. Once the wave length is calculated, the wave base depth can be determined as equal to one half the wave length.

The estimated intersection of the wave base with the original surface, point I in Figures 29 and 30, is significant in equilibrium profile development. It is assumed that because the wave base is approximately the deepest point of water particle motion, erosion of the original surface will occur above this depth. As a wave approaches the shoreline, its energy will be dissipated as the friction between the water particle motion and the original surface causes sediment movement. This point of intersection is the approximate starting point for the development of the equilibrium nearshore profile. The extension of this equilibrium nearshore profile, of slope i_e , to a position where it intersects the existing lake level, point J, should then be the potential maximum horizontal distance of the original surface influenced by wave action, X_m . Based upon field observations, the slope of this nearshore profile is assumed to be a constant.

The potential maximum horizontal distance of the original surface subjected to wave action, X_m , is a function of wave base depth and slope of the equilibrium nearshore profile. The deeper the wave base and the

more gradual the equilibrium nearshore profile slope the greater X_m will be.

Examining the profiles in Figures 29 and 30 reveals that X_m can be geometrically related to wave base depth, abrasion platform length, and the angle the abrasion platform makes with the horizontal, A , by the following relationships:

$$\frac{WBD}{X_m} = \tan A$$

rearranging terms and using the identity $\tan A = \frac{1}{\cot A}$

$$X_m = WBD \cot A \quad (12)$$

X_m may also be related to the length of the abrasion platform as follows:

$$X_m = \ell \cos A \quad \text{or} \quad \ell = X_m \sec A$$

by substitution

$$WBD \cot A = \ell \cos A$$

and

$$\ell = WBD \csc A \quad (13)$$

The potential maximum horizontal shoreline recession, X_r , is the horizontal distance from the intersection of the original surface with the lake level to the intersection of the lake level with the equilibrium profile. X_r is a function of wave base depth, original surface slope, and equilibrium nearshore profile slope. As the wave base depth increases and the equilibrium nearshore profile slope decreases, X_r will approach X_m .

As with X_m , X_r can be geometrically related to the abrasion platform length, the angle between the original surface and the abrasion

platform, B, and angle C by the following relationships:

$$\frac{X_r}{\sin B} = \frac{\ell}{\sin C} \quad \text{where } C = 180 - (A + B)$$

by rearranging terms and substitution

$$X_r = \frac{\ell \sin B}{\sin [180 - (A + B)]} \quad (14)$$

substituting $X_m \sec A = \ell$

$$X_r = \frac{X_m \sec A \sin B}{\sin [180 - (A + B)]} \quad (15)$$

finally, substituting $WBD \cot A = X_m$

$$X_r = \frac{WBD \cot A \sec A \sin B}{\sin [180 - (A + B)]} \quad (16)$$

Therefore, if angle A is assumed to be constant of 7% a graph can be plotted showing how X_r , the potential maximum horizontal shoreline recession, varies as a function of the original surface slope and wave base depth (Figure 31). Examining Figure 31 reveals that as the wave base depth increases, the amount of horizontal shoreline recession becomes very sensitive to small increases in the original surface slope. When the original surface slope becomes greater than 25%, the increase in X_r becomes smaller, for the wave base depths plotted. The curves for each of the wave base depths in Figure 31 intersect the original surface slope axis at 7%. This is consistent with field observations which showed that for original surface slopes equal to or less than 7% there was little or no shoreline erosion.

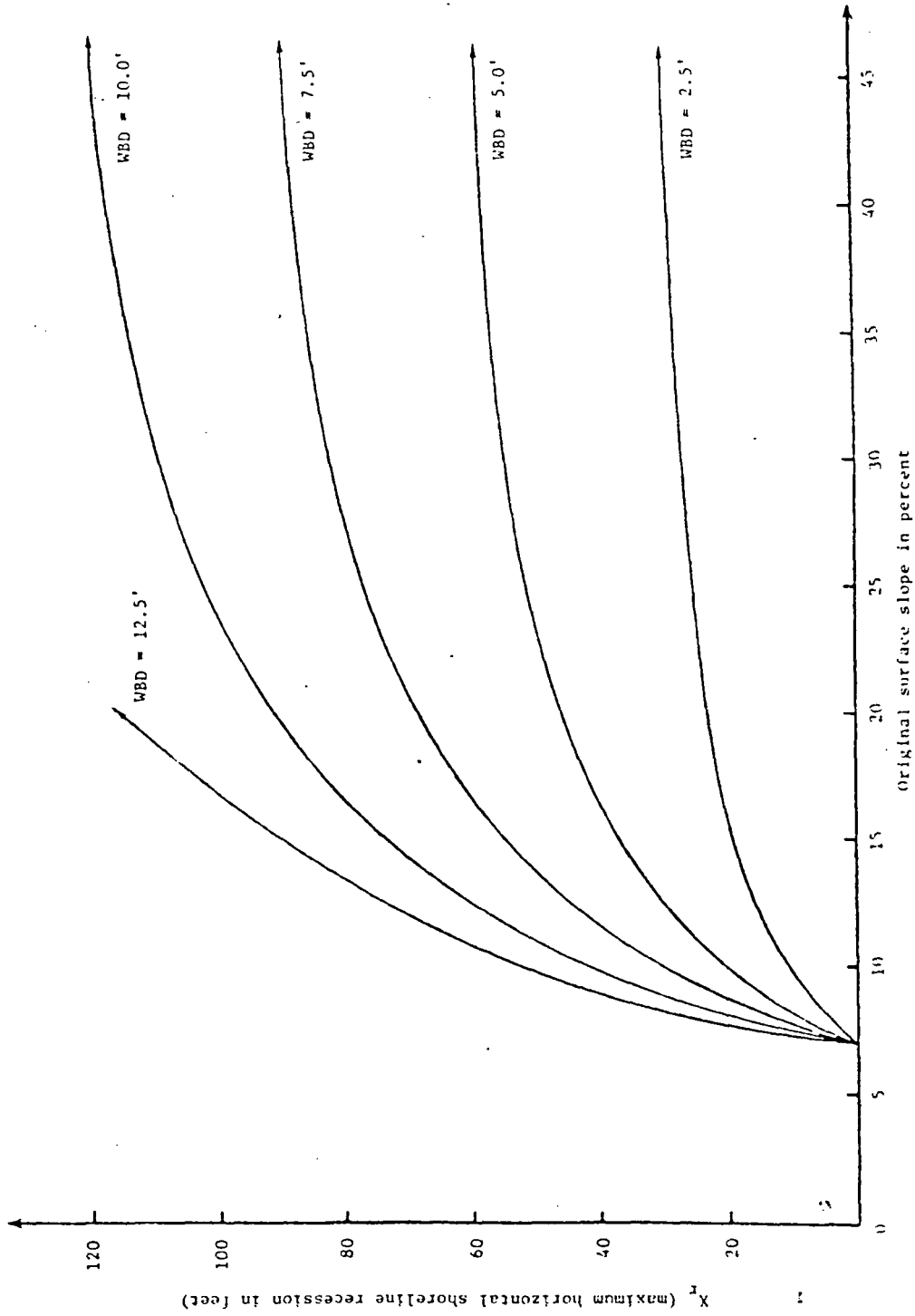


Figure 31. X_r versus original surface slope as a function of wave base depth (WBD)

The wave cut cliff height, H , can be geometrically related to the horizontal shoreline recession and the original surface slope as follows:

$$H = X_r \tan (A+B) \quad (17)$$

where

$A+B$ equals the angle the original surface makes with the horizontal. By substituting the value for X_r into the immediately preceding equation.

$$H = \left(\frac{WBD \cot A \sec A \sin B}{\sin [180 - (A+B)]} \right) \tan (A+B) \quad (18)$$

Therefore, knowing the wave base depth and the slope angle of the original surface, the height of the wave cut cliff can be determined (Figure 32). Figure 32 shows that as the wave base depth increases, the height of the wave cut cliff becomes more sensitive to increases in the original surface slope.

Terrace development

Observations at Big Creek Lake indicated that some of the eroded material of the original surface has been redeposited at an offshore position. This depositional feature is called a terrace. The horizontal length of terrace development is given the symbol T_x . The measured profiles at Big Creek Lake indicate that terrace development depends on original surface slope, wave action, longshore sediment movement, and soil properties. It appears that the steeper the original surface slope the less the tendency for terrace development. If the original surface slope is greater than the angle of repose of the sediment, an offshore terrace may not develop because the eroded material will slide down

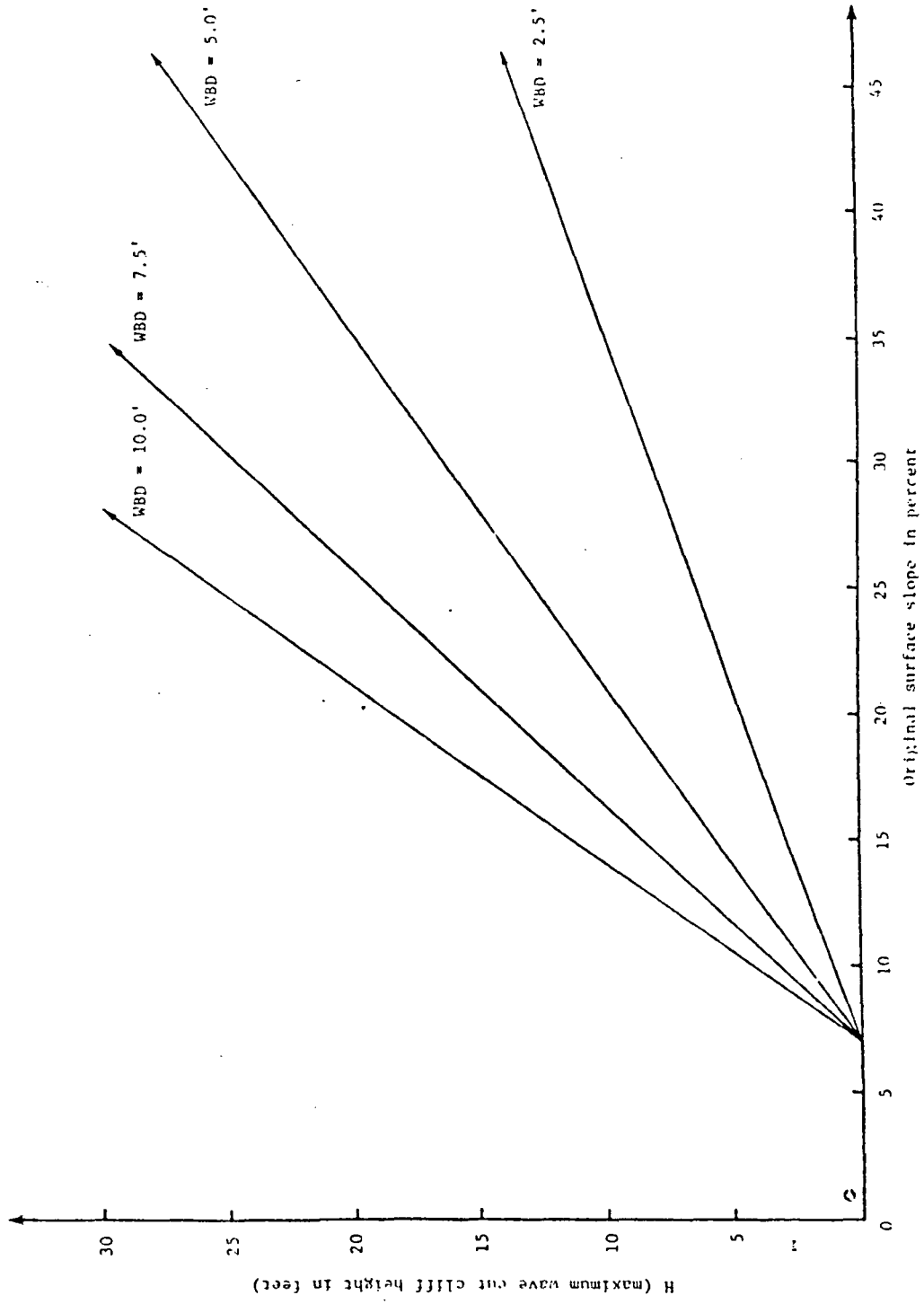


Figure 32. H versus original surface slope as a function of wave base depth (WBD)

the slope into deeper water offshore. Longshore currents, resulting from wave fronts hitting the shoreline at an angle, may also affect terrace development by transporting eroded material laterally along the shore.

Understanding how the terrace feature develops is very important, because its development may influence how much of the original surface erodes before equilibrium is reached. It was previously stated that a certain horizontal distance, X_m , of the original surface is subjected to wave action; however, as a terrace develops and becomes an integral feature of the equilibrium nearshore profile, the distance X_m may be reduced by the distance T_x . This reduction in the distance X_m may be explained by considering the dissipation of a wave's energy as it approaches the shore. As a wave enters the nearshore zone, its energy will begin to be dissipated as the interaction of the water particle motion with the abrasion platform generates friction. The deposition of the terrace extends the length of the abrasion platform lakeward by a distance T_x and results in energy dissipation beginning at a position further offshore. Thus, less of the original surface will be eroded if a terrace develops. In the case where a terrace develops, a variable X'_m defines the distance $X_m - T_x$, which is the potential maximum horizontal length of the original surface influenced by wave action when a terrace develops. In Figure 33 J' is the point of maximum erosion with terrace development and the symbols X'_r , X'_m , T_x , and X_a relate to that situation. If no terrace develops, J is the point of maximum erosion and the terms X_m and X_r are relevant.

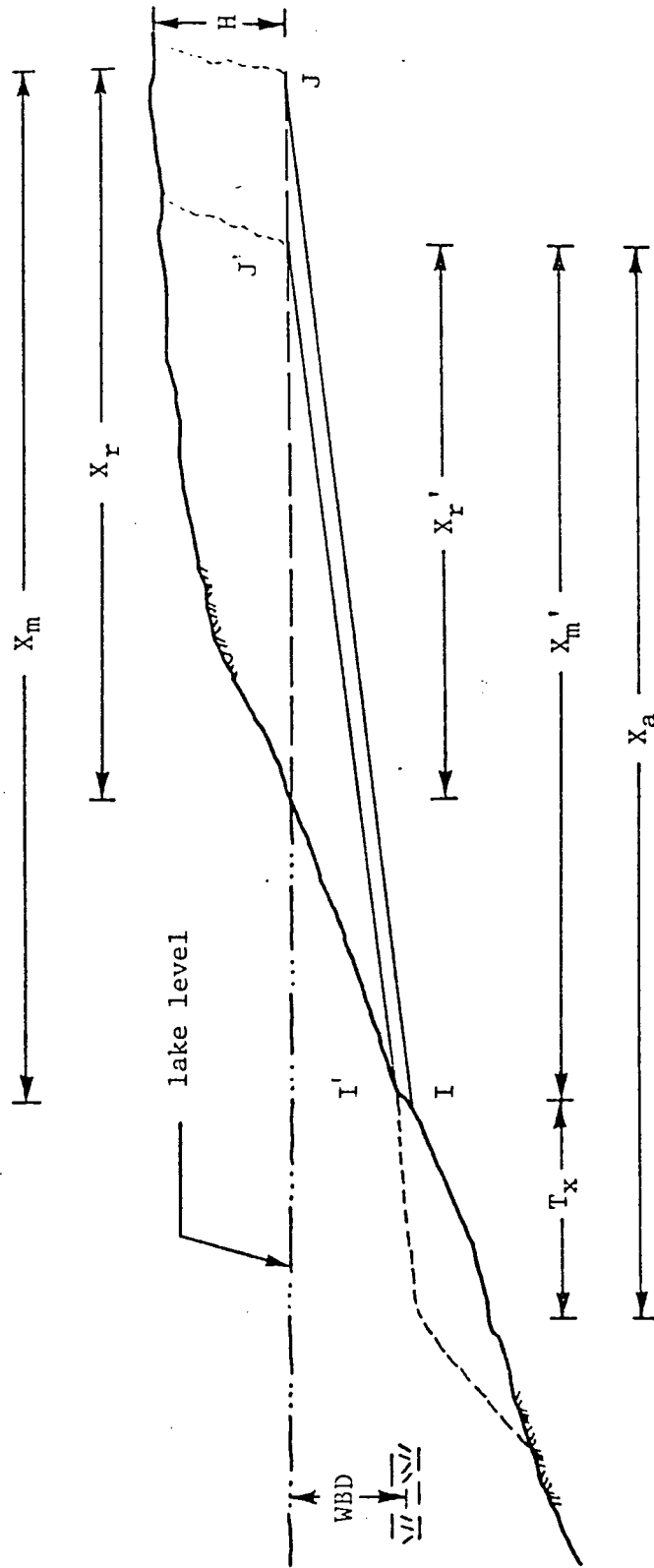


Figure 33. Gradually sloping original surface profile with terrace development

COMPARISON OF FIELD OBSERVATIONS WITH
SHORELINE EROSION MODEL

In order to test the adequacy of the shoreline erosion model, field observations of wave cut cliff height and horizontal shoreline recession at the three selected sites on Big Creek Lake are compared to theoretical model predications. All calculations to determine the theoretical model predications are found in Appendix D, with a summary of the calculation procedures following.

To predict the wave cut cliff height and horizontal shoreline recession, the wave base depth must be calculated and the original surface slope measured. To calculate the wave base depth, the wave period, which is the time required for two wave crests to pass a fixed point, must be determined for a selected wind velocity and effective fetch. The wind velocity used for determining the wave period is the most frequently occurring out of the direction perpendicular with the existing shoreline and parallel with the measured erosion profile, which is northwest for the sites at Big Creek Lake. For the Big Creek Lake area, the National Oceanic and Atmospheric Administration (1978) reported that the most frequently occurring wind velocity in a northwesterly direction averages 15 mph and occurs 3.4% of the time. Overall, the wind occurring out of the NNW-NW-WNW are the most frequent, occurring 22.3% of the time. The calculations for effective fetch are completed according to the method proposed by Saville (1954) as previously discussed in the review of literature.

Upon selection of the wind velocity and determination of the effective fetch, the wave period may be determined either by field observations or estimated from nomographs found in Volume 1 and Errata of the United States Army Coastal Engineering Research Center's Shore Protection Manual, chapter 3, section 3.6. A method is provided for determining the period of waves which are generated in shallow water and progress to deeper water. For the wind velocity considered, the resultant waves are of such magnitude that the relative depth (d/L) is greater than one half; therefore, the waves' characteristics will not be affected by depth. The United States Army Coastal Engineering Research Center (1973) reported that the wave period for oscillatory waves is independent of depth.

After determining the wave period, the wave length can be calculated using the following equation, reported by the United States Army Coastal Engineering Research Center (1973) for deep water waves:

$$L = 5.12T^2 \quad (19)$$

where

L = wave length in feet

T = wave period in seconds

Once the wave length has been calculated, the wave base depth can be determined as equal to one half the wave length.

Knowing the wave base depth and the angle that the original surface makes with the horizontal, the theoretical wave cut cliff height and potential maximum horizontal shoreline recession can be determined using the following equations:

$$H = \left(\frac{WBD \cot A \sec A \sin B}{\sin [180 - (A + B)]} \right) \tan (A + B) \quad (20)$$

and

$$X_r = \frac{WBD \cot A \sec A \sin B}{\sin [180 - (A + B)]} \quad (21)$$

A summary of the predicted and actual wave cut cliff heights and horizontal shoreline recession for the three sites at Big Creek Lake is found in Table 2. The predicted and actual data points have also been replotted in Figures 34 and 35 to aid in the discussion.

Site 2

Of all three sites on Big Creek Lake, site 2 appears to be approaching equilibrium at the slowest rate. It has the least amount of horizontal shoreline recession, 20 feet, and the lowest measured annual average rate of horizontal shoreline recession, 2.5 feet/yr. The ratio of the actual horizontal shoreline recession to the theoretical horizontal shoreline recession is also the lowest, 0.38, which indicates that site 2 may be far from reaching equilibrium. The rate of horizontal shoreline recession at site 2 may be reduced when slope failure occurs, because the large height of the wave cut cliff results in a tremendous amount of soil slumping to the shore. This soil mass then provides protection for the base of the wave cut cliff until there has been sufficient time for wave action to erode the soil and transport it offshore. Field observations on March 3, 1980 and July 18, 1980 indicated that a considerable amount of sediment had been removed from the abrasion platform and redeposited offshore. This removal of sediment, which was deposited

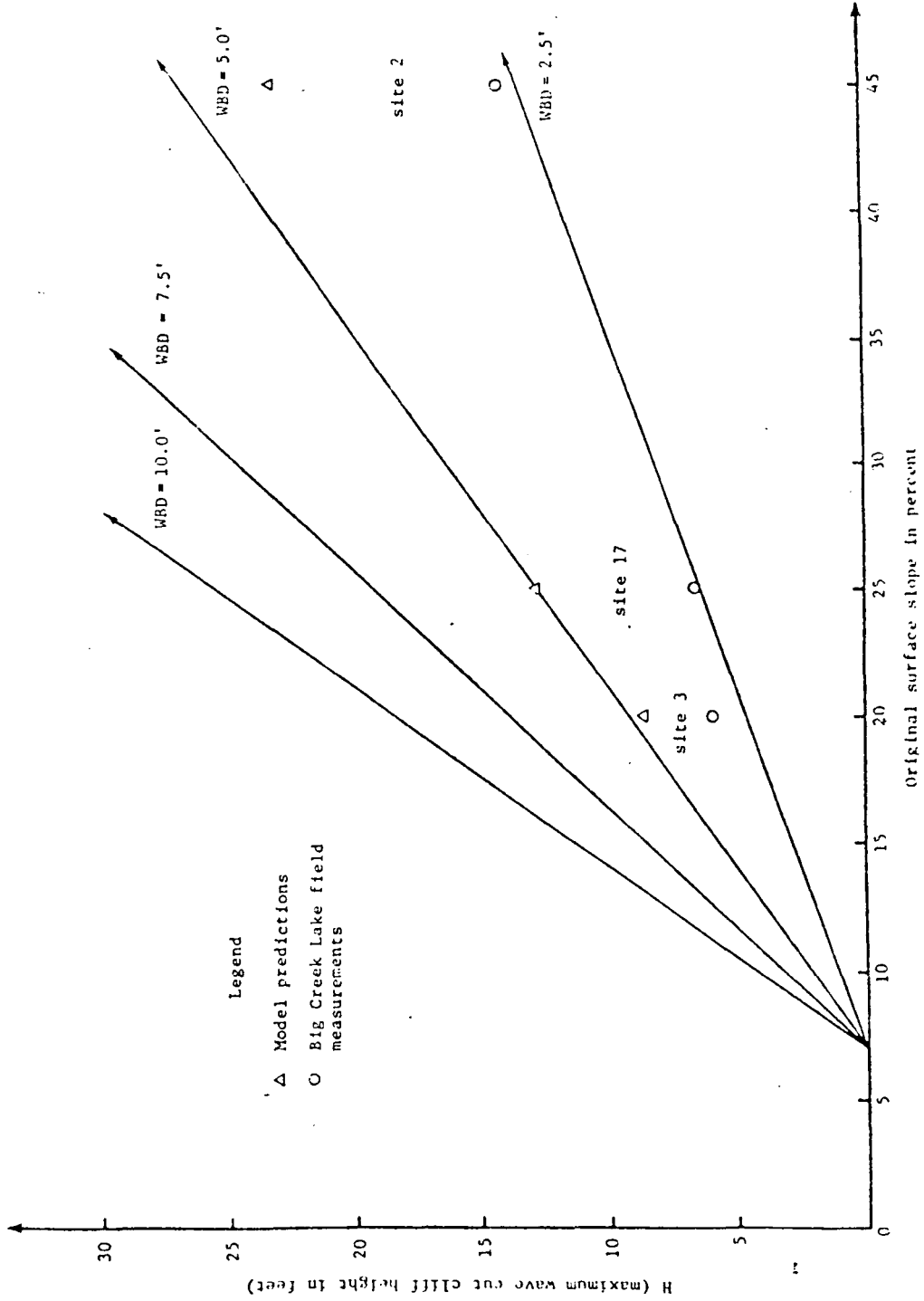


Figure 34. H versus original surface slope as a function of wave base depth (WBD)

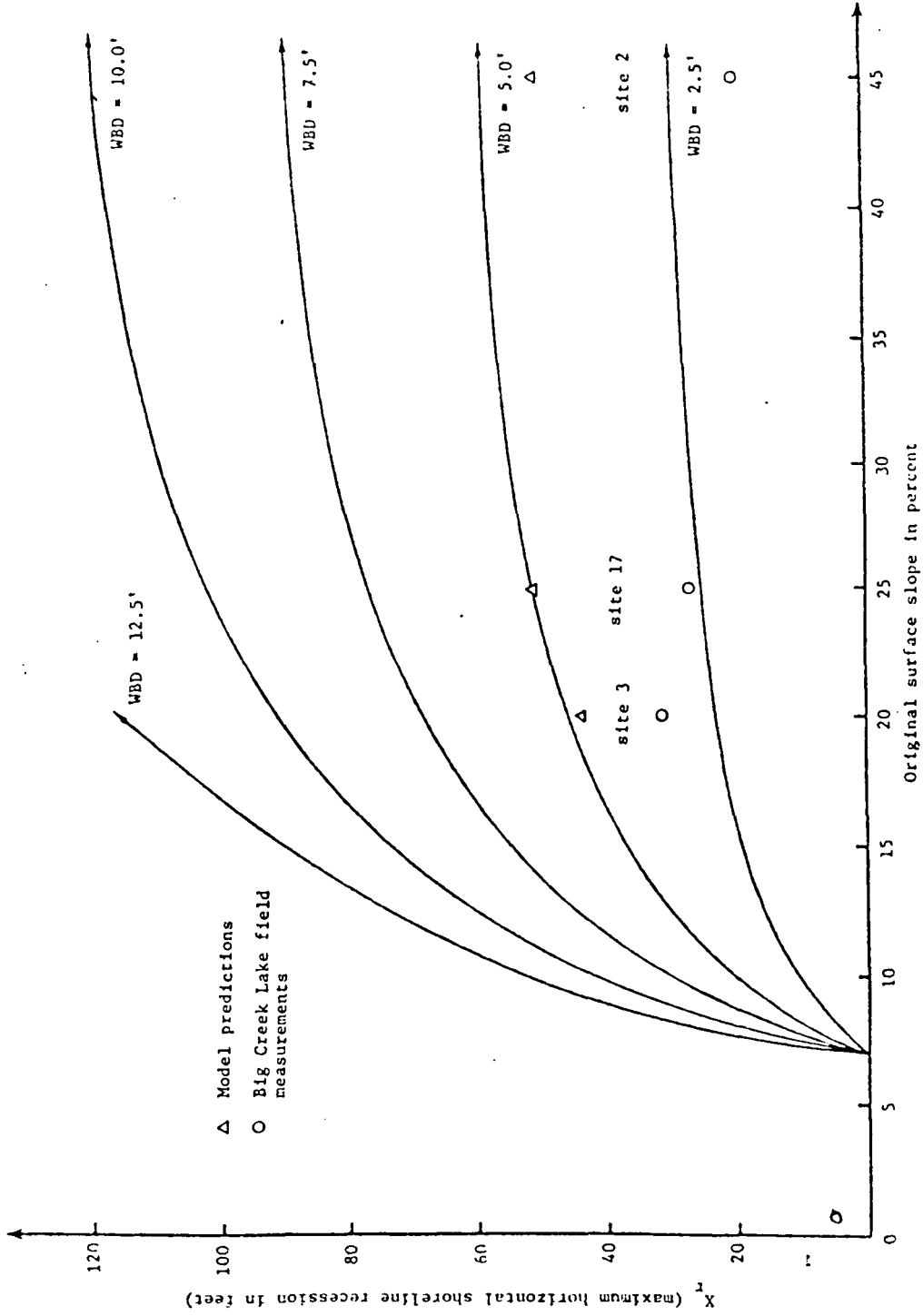


Figure 35. X_r versus original surface slope as a function of wave base depth (WBD)

Table 2. Big Creek Lake wave cut cliff height and shoreline recession summary

Site	Effective fetch (feet)	Wave period (sec)	Wave length (feet)	Wave base depth (feet)	H(a) ^a	H(t) ^b	H(a)/H(t)	X _r (a) ^c	X _r (t) ^d	Original surface slope (%)	
										X _r (a)	X _r (t)
2	1640	1.30	8.6	4.3	14.5	23.5	0.62	20	52	0.38	45
3	2640	1.37	9.6	4.8	6.0	8.8	0.68	32	45	0.71	20
17	2750	1.40	10.0	5.0	6.5	12.8	0.51	28	51	0.55	25

^aActual vertical wave cut cliff height in feet.

^bTheoretical vertical wave cut cliff height in feet.

^cActual horizontal shoreline recession in feet.

^dTheoretical potential maximum horizontal shoreline recession in feet.

on the abrasion platform from a previous slope failure, indicates that active erosion is taking place at site 2.

The slow rate of horizontal shoreline recession at site 2 may also be influenced by the soil's shear strength, which determines the stability of the wave cut cliff. Shear strengths measured at the base of the wave cut cliff at site 2 are approximately 750 psf (35 kN/m²), which is considerably higher than the shear strengths of the sand and gravel layers at sites 3 and 17 which are approximately 60 psf (3 kN/m²).

The ratio of the actual vertical wave cut cliff height to the theoretical vertical wave cut cliff height, 0.62, indicates that site 2 may be closer to equilibrium than site 17 where this ratio is 0.51. When examining the theoretical vertical wave cut cliff height it should be recognized that the model assumes a uniform and continuous original surface slope, while at site 2 it is not possible to attain a vertical wave cut cliff height of 23.5 feet because the original surface becomes level near the steel reference pin.

Site 3

Site 3 is the most erosion active site measured on Big Creek Lake. Measurements made on March 7, 1980 and July 18, 1980 indicated that 3.5 feet of horizontal shoreline recession had occurred between those dates. The total horizontal shoreline recession at site 3 is 32 feet and the annual average rate of horizontal shoreline recession is 4.0 feet/yr. The ratio of $X_r(a)$ to $X_r(t)$ at site 3 is 0.71, indicating that site 3 may be nearing equilibrium. The ratio of $H(a)$ to $H(t)$, 0.68, at site 3 is also the highest of the three measured sites.

Site 3 appears to be approaching equilibrium at a faster rate than the other sites on Big Creek Lake because of the low shear strengths of the sand and gravel layers at the base of the wave cut cliff and the larger waves resulting from an effective fetch which is 1000 feet greater than at site 2. The shear strengths of the cohesionless sand and gravel layers at the base of the wave cut cliff are approximately 60 psf (3 kN/m²), producing very unstable slopes which are easily eroded.

Site 17

The shoreline erosion rate at site 17 is between that of site 3 and site 2. Total horizontal shoreline recession at site 17, in the direction N 35°W, is 28 feet, which is an average annual rate of horizontal shoreline recession of 3.5 feet/yr. The ratio of $X_r(a)$ to $X_r(t)$ is 0.55 and the ratio of $H(a)$ to $H(t)$ is 0.51, indicating that site 17 is approximately one half of the way to reaching equilibrium.

Although site 17 is not eroding at as fast a rate as site 3, field measurements made on March 3, 1980 and April 18, 1980 showed that erosion at the base of the wave cut cliff had resulted in approximately 3.0 feet of shoreline recession while the top of the wave cut cliff had not receded. These measurements and observations indicate that active erosion is taking place at site 17 and that equilibrium has not been reached.

There is ample evidence that all three sites are still actively eroding and thus should plot below the equilibrium curve. Site 3 and 17 may be nearer to equilibrium than indicated by the actual to

theoretical shoreline recession and wave cut cliff height because of the terraces present.

SUMMARY AND CONCLUSIONS

The study of shoreline erosion on selected artificial lakes in Iowa had two objectives. These objectives were to review currently available techniques for shoreline protection and to gain a better understanding of the shoreline erosion process on man-made lakes.

The first objective, a review of shoreline protection methods, was completed as part of the review of literature. Selected techniques which looked promising for use on man-made lakes are: soil-cement stabilization and interlocking concrete blocks or 'monoslabs'.

The second objective, to gain a better understanding of the shoreline erosion process on man-made lakes, was approached by studies at Big Creek Lake in Polk County, and Prairie Rose Lake in Shelby County. These two lakes were selected for study because of the differences in their geologic and topographic settings. Big Creek Lake was used more extensively in the study because of its severe shoreline erosion problems and its close proximity to Ames. Discussion and analyses of the data collected at Big Creek Lake led to the development of a shoreline erosion model. The model contains equations which can be used to estimate X_r , the potential maximum horizontal shoreline recession and H, the maximum wave cut cliff height of the equilibrium profile. These equations are:

$$X_r = \frac{WBD \cot A \sec A \sin B}{\sin [180 - (A+B)]}$$

and

$$H = \left(\frac{WBD \cot A \sec A \sin B}{\sin [180 - (A+B)]} \right) \tan (A+B)$$

Comparisons of field observations at Big Creek Lake with predictions made using the shoreline erosion model suggest that the model produces a reasonable estimate of the maximum horizontal shoreline recession and the maximum wave cut cliff height. Although future field observations may require modification of the model, present observations indicate that active erosion is occurring at all three sites and that these sites have not reached equilibrium.

RECOMMENDATIONS FOR FURTHER STUDY

Future work should include the continued monitoring of shoreline erosion at the three sites on Big Creek Lake and the two sites on Prairie Rose Lake. Additional data on wave cut cliff heights and original surface slopes should be collected at Big Creek Lake and compared with the model and several more sites should be profiled at both lakes. The continued monitoring of existing sites at Big Creek Lake and Prairie Rose Lake and the collection of additional data may require modification of the model as new relationships are discovered. To test the model's application to varying geologic and topographic conditions, shoreline erosion should be examined at several more man-made lakes and compared with model predications.

Finally, a potentially erosive shoreline at a proposed man-made lake site should be designed as suggested by the model specifications before the reservoir is filled and the site monitored after filling.

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APPENDIX A:
SOIL DATA SUMMARY

Soil data collected at Big Creek Lake and Prairie Rose Lake are summarized in Appendix A. Soil data collected in the field included in situ unit weights and shear strengths. Laboratory analyses of collected soil samples included Atterberg limits, particle size analysis, and textural classification.

Unit weights: In situ unit weight measurements were made using the Eley Volumeter manufactured by Soiltest, Incorporated. Two to three samples were taken, moisture contents measured and wet and dry unit weights calculated (Tables 3 and 4).

Shear strength: In situ shear strength measurements were made using the Torvane shear device manufactured by Soiltest, Incorporated. At each position four readings were taken along with moisture samples and then average shear strength values calculated. The sensitive vane adapter was utilized for the low strength sand and gravel layers and the high-capacity adapter was used for stiff glacial till (Tables 5 and 6).

Atterberg limits: Liquid limits were determined according to the procedure outlined in AASHTO (American Association of State Highway Officials) designation: T89-60, pages 202 - 209 and the plastic limit and plasticity index according to AASHTO designation: T90-61, pages 210 - 211 of the Asphalt Institute (Tables 7 and 8).

Particle size analysis: The mechanical analysis of soil samples was completed according to AASHTO designation: T88-57, pages 191 - 201 of the Asphalt Institute (1969). Samples were dispersed using an air-jet apparatus and a 152 H standard hydrometer was used for the hydrometer analysis. Hydrometer readings were temperature adjusted and a specific

gravity of 2.65 was assumed for particle size calculations (Tables 9 and 10).

Textural classification: The textural classification of soil samples was determined by using Figure 7-3, page 148 of Spangler and Handy (1973). The following particle size classifications were used:

gravel size particles	> 2.0 mm
sand size particles	2.0 - 0.074 mm
silt size particles	0.074 - 0.002 mm
clay size particles	< 0.002 mm

The uniformity coefficient, the ratio of D_{60} to D_{10} , was also calculated when possible. D_{60} equals the maximum diameter of the smallest 60 % by weight and D_{10} equals the maximum diameter of the smallest 10 % by weight (Tables 9 and 10).

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Table 3. Big Creek Lake unit weight measurements

Site	Position (ft)	Wet unit weight		Dry unit weight		Moisture content (wt. %)
		(pcf)	(kN/m ³)	(pcf)	(kN/m ³)	
1	5.0	136.7	21.4	118.6	18.6	15.3
		135.4	21.2	117.3	18.4	15.2
		136.7	21.4	118.6	18.6	15.1
2	3.0	129.2	20.2	110.4	17.3	17.3
		130.4	20.4	112.3	17.6	16.2
		132.3	20.7	111.7	17.5	18.2
	8.0	132.3	20.7	115.4	18.1	14.9
		130.4	20.4	113.6	17.8	14.9
		131.0	20.5	114.2	17.9	14.9
	15.0	128.5	20.1	108.0	16.9	19.1
		127.9	20.0	106.1	16.6	20.6
		126.7	19.9	105.5	16.5	20.2
7	3.0	118.6	18.6	96.7	15.2	22.6
		115.4	18.1	93.0	14.6	24.1
	7.0	124.8	19.6	104.8	16.4	19.3
		129.8	20.3	109.8	17.2	18.3
	9.5	137.3	21.5	120.3	18.9	14.1
		137.3	21.5	120.5	18.9	13.9
15	3.0	124.8	19.6	107.3	16.8	16.2
		127.9	20.0	109.2	17.1	16.8
17	3.0	126.7	19.9	111.1	17.4	14.4
		129.2	20.2	112.9	17.7	14.7

Table 3. (cont.)

Site	Position (ft)	Wet unit weight		Dry unit weight		Moisture content (wt. %)
		(pcf)	(kN/m ³)	(pcf)	(kN/m ³)	
18	2.5	115.4	18.1	101.7	15.9	13.4
		115.4	18.1	102.3	16.0	12.9
	Wave cut bench	136.0	21.3	116.7	18.3	16.3
		133.5	20.9	114.2	17.9	17.0

Table 4. Prairie Rose Lake unit weight measurements

Site	Position (ft)	Wet unit weight		Dry unit weight		Moisture content (wt. %)
		(pcf)	(kN/m ³)	(pcf)	(kN/m ³)	
A	2.0	120.4	18.9	91.7	14.4	31.1
		120.4	18.9	91.7	14.4	31.4
B	2.2	117.9	18.5	91.7	14.4	28.9
		118.6	18.6	93.0	14.6	27.9

Table 5. Big Creek Lake shear strength measurements

Site	Position (feet)	Shear strength (psf) (kN/m ²)	Moisture content (wt. %)	Wave cut cliff height (feet)	Approx. slope (percent)	
1	2.5	3380	162	9.8	8.0	vertical
	5.0	1620	77	16.0		
	8.0	1400	67	17.4		
2	3.0	1400	67	17.8	17.5	175
	8.0	1320	63	15.0		
	15.0	740	35	20.2		
3	2.5	720	34	--	6.0	vertical
	shoreline	880	42	14.2		
25' North of site 3	glacial till	1060	51	17.0		
	sand layer	56	3	moist sand		
	sand layer	440	21	moist sand		
7	3.0	620	30	23.3	14.0	225
	7.0	800	38	21.0		
	9.5	1740	83	14.5		
15	3.0	1480	71	18.0	5.0	vertical
17	3.0	1840	88	15.1	6.5	vertical
18	2.5	1520	73	13.3	5.0	vertical
	wave cut bench	1560	75	16.9		

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Table 6. Prairie Rose Lake shear strength measurements

Site	Position (feet)	Shear strength (psf) (kN/m ²)	Moisture content (wt. %)	Wave cut cliff height (feet)	Approx. slope (percent)
A	2.0	800 38	31.3	5.0	70
B	2.2	880 42	27.7	12.5	70

Table 7. Big Creek Lake Atterberg limits

Site	Position (feet)	Liquid limit	Plastic limit	Plasticity index
1	2.5	28	16	12
	5.0	26	15	11
2	3.0	33	16	17
	8.0	24	15	9
	15.0	28	15	13
3	2.5	25	16	9
7	3.0	30	22	8
	7.0	28	14	14
	9.5	25	14	9
15	3.0	32	16	16
17	3.0	27	16	11
18	2.5	26	16	10
	wave cut bench	27	15	12

Table 8. Prairie Rose Lake Atterberg limits

Site	Position (feet)	Liquid limit	Plastic limit	Plasticity index
A	2.0	47	22	25
B	2.2	45	24	21
Uplands	1.5	41	26	15

Table 9. Big Creek Lake particle size distributions

Site	Position (feet)	Textural Classifi- cation	Sand				D ₁₀	D ₆₀	Uniformity coefficient	
			Gravel (%)	Coarse (%)	Fine (%)	Silt (%)				Clay (%)
1	2.5	loam	5.9	9.5	28.8	38.3	17.5	0.0018	0.1	556
	5.0	loam	5.7	9.0	28.5	39.3	17.5	<0.001	0.1	-
	shoreline	--	3.2	48.1	48.2	0.5	0.0	0.25	0.45	1.8
2	3.0	clay loam	2.6	8.5	36.9	26.0	26.0	<0.001	0.13	-
	8.0	loam	5.3	10.4	30.3	36.5	17.5	<0.001	0.11	-
	15.0	sandy clay loam	3.2	11.7	39.1	25.0	21.0	<0.001	0.15	-
	shoreline	--	3.4	24.8	70.8	1.0	0.0	0.18	0.34	1.9
3	2.5	loam	5.5	9.8	34.7	32.5	17.5	<0.001	0.13	-
7	3.0	silty loam	0.0	0.3	7.6	74.1	18.0	<0.001	0.034	-
	7.0	loam	1.8	8.2	28.5	45.5	16.0	<0.001	0.07	-
	9.5	loam	2.6	8.7	38.7	38.5	11.5	0.001	0.11	110
	shoreline	--	19.1	49.5	31.1	0.3	0.0	0.24	0.90	3.7
15	3.0	clay loam	3.0	10.6	32.9	28.5	25.0	<0.001	0.11	-

Table 9. (cont.)

Site	Position (feet)	Textural Classifi- cation	Gravel				Sand			D ₁₀	D ₆₀	Uniformity coefficient
			(%)	Coarse (%)	Fine (%)	Silt (%)	Clay (%)					
17	3.0	loam	9.1	10.0	24.9	40.0	16.0	<0.001	0.09	-		
	sand and gravel layers		40.6	39.6	17.5	2.3	0.0	0.26	2.02	7.8		
	shoreline		8.4	73.2	18.1	0.3	0.0	0.32	0.75	2.3		
18	2.5	loam	5.5	11.1	29.4	35.0	19.0	<0.001	0.11	-		
	wave cut bench	loam	4.5	10.7	27.3	41.5	16.0	<0.001	0.08	-		

Table 10. Prairie Rose Lake particle size distributions

Site Position (feet)	Textural Classifi- cation	Gravel			Sand			D ₁₀	D ₆₀	Uniformity coefficient
		(%)	Coarse (%)	Fine (%)	Silt (%)	Clay (%)				
A	2.0 silty clay loam	0.0	0.9	7.7	55.4	36.0	<0.001	0.022	-	
B	2.2 silty clay loam	0.0	0.1	0.3	65.6	34.0	<0.001	0.018	-	
Uplands	1.5 silty clay loam	0.0	0.0	0.2	72.0	27.8	<0.001	0.018	-	

APPENDIX B:
STEEL REFERENCE PIN LOCATIONS

Steel reference pins, $2' \times \frac{1}{2}''\phi$, were positioned at three locations at Big Creek Lake and two locations at Prairie Rose Lake using a Brunton pocket transit and steel tape. The purpose of the steel pins is to provide an accurate reference point from where future profiles and horizontal shoreline recession can be measured. The following figures show the locations of the steel reference pins as of July 1980.

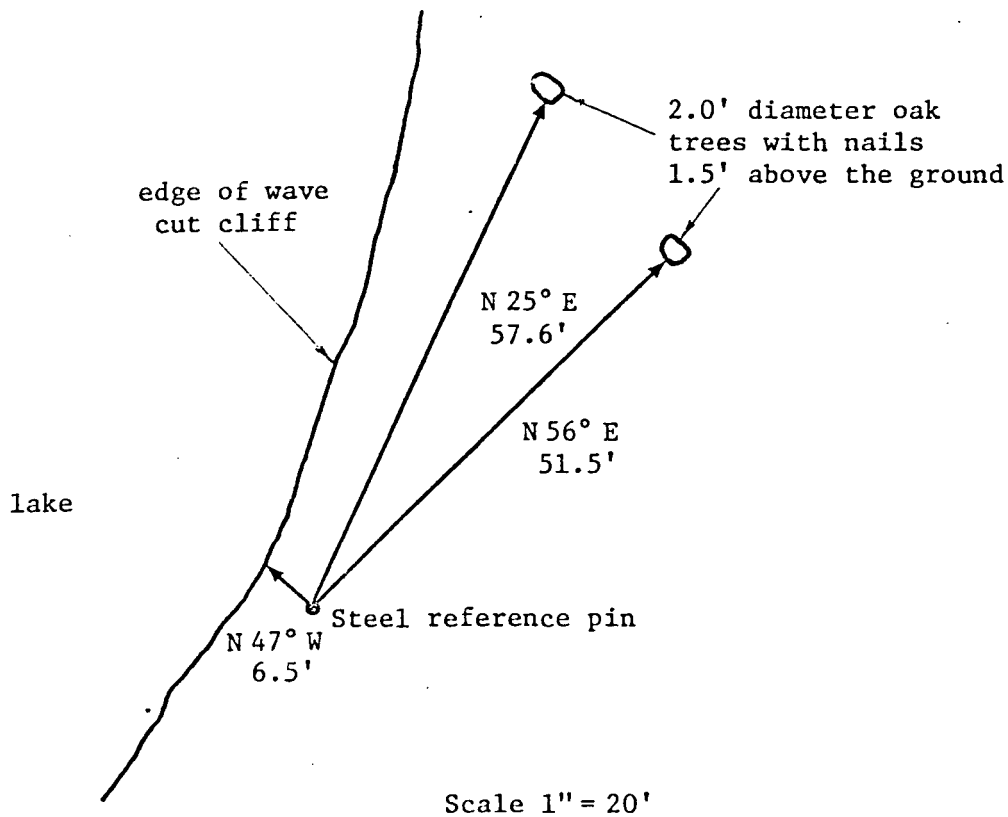


Figure 36. Site 2 Big Creek Lake

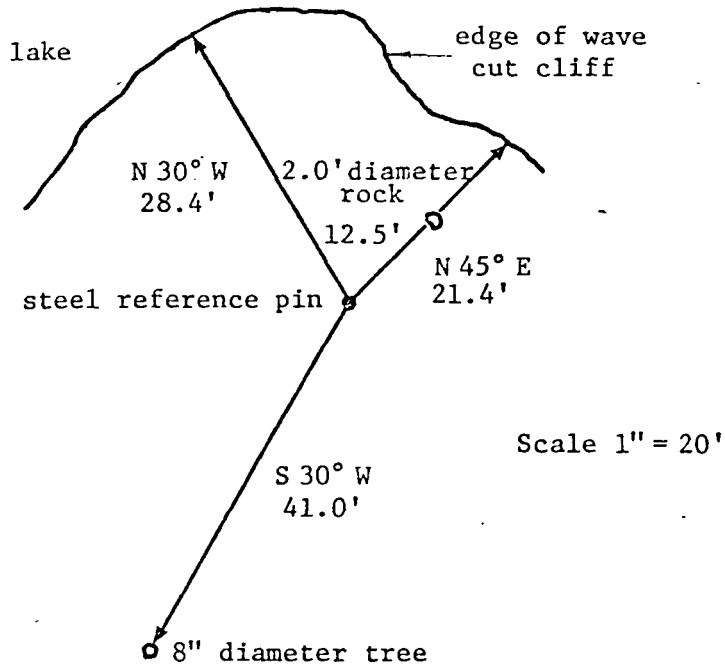


Figure 37. Site 3 Big Creek Lake

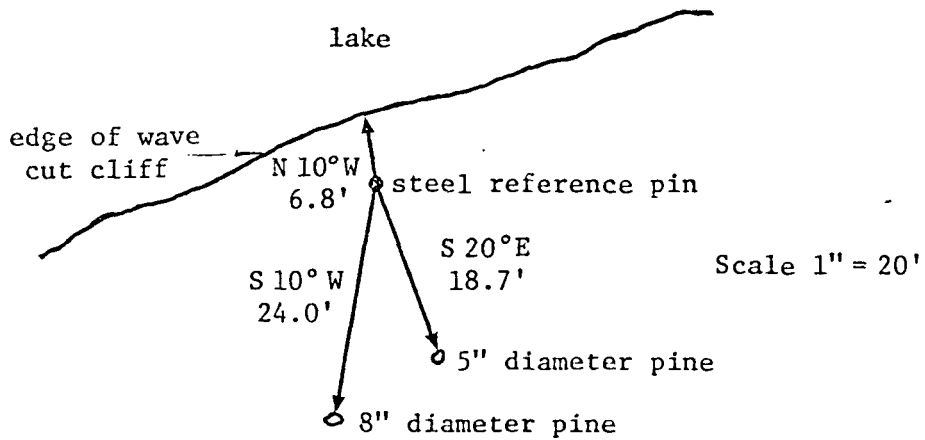


Figure 38. Site 17 Big Creek Lake

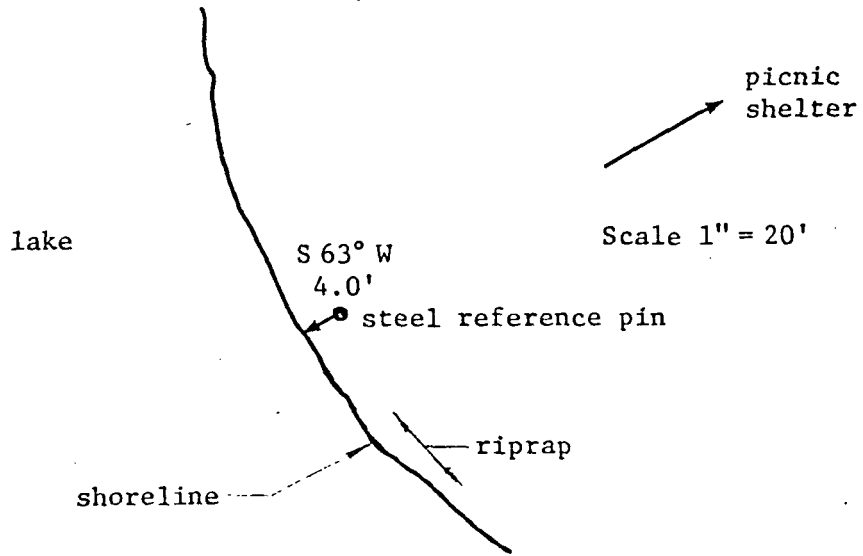


Figure 39. Site A Prairie Rose Lake

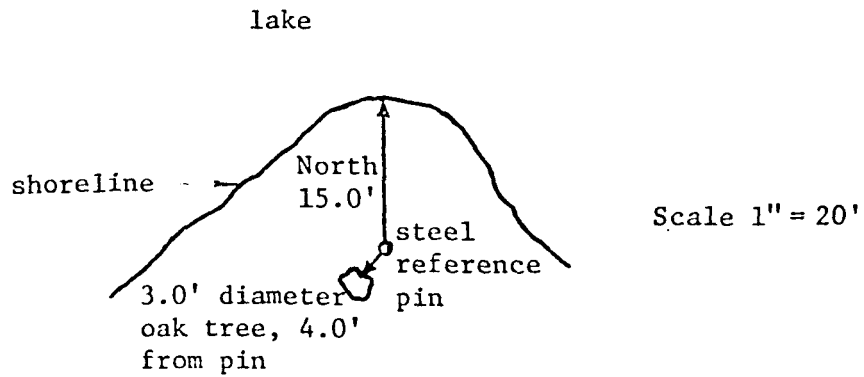


Figure 40. Site B Prairie Rose Lake

APPENDIX C:
SEDIMENTATION CALCULATIONS

Shoreline Erosion Sedimentation Calculations

Slight erosion (1' - 2' wave cut cliffs)

Assumptions: 1' vertical wave cut cliff

7% nearshore profile slope

10% original surface slope

Eroded volume: 15 ft³/ft of shoreline

$$15 \text{ ft}^3/\text{ft of shoreline} \times 9600 \text{ ft of shoreline} = 144,000 \text{ ft}^3$$

Moderate erosion (2' - 5' wave cut cliffs)

Assumptions: 3' vertical wave cut cliff

7% nearshore profile slope

15% original surface slope

Eroded volume: 55 ft³/ft of shoreline

$$55 \text{ ft}^3/\text{ft of shoreline} \times 8850 \text{ ft of shoreline} = 486,750 \text{ ft}^3$$

Severe erosion (5' to ↑ wave cut cliffs)

Assumptions: 10' wave cut cliff on 175% slope

7% nearshore profile slope

30% original surface slope

Eroded volume: 130 ft³/ft of shoreline

$$130 \text{ ft}^3/\text{ft of shoreline} \times 2850 \text{ ft of shoreline} = 370,500 \text{ ft}^3$$

$$\text{Total volume of shoreline erosion} = 1,001,250 \text{ ft}^3$$

$$1,001,250 \text{ ft}^3 \times \frac{110 \text{ lbs}}{\text{ft}^3} \times \frac{1 \text{ ton}}{2000 \text{ lbs}} = 55,070 \text{ tons}$$

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Tributary Sedimentation Calculations

Big Creek Lake

Opened November 1972

Been in operation 7.5 yr

Drainage area = 77.3 mi²

Located in Land Resource Area 103 (Upper Mississippi River Basin Coordinating Committee, 1970, page 6-79).

Approximately 5 mi² of bluff drainage (adjacent to creek) with very small drainage area, sediment yield approximately 500 tons/mi²/yr.

$$5 \text{ mi}^2 \times 500 \text{ ton / mi}^2/\text{yr} \times 7.5 \text{ yr} = 18,750 \text{ tons}$$

72.3 mi² of drainage with a sediment yield of approximately 50 tons/mi²/yr.

$$72.3 \text{ mi}^2 \times 50 \text{ ton/mi}^2/\text{yr} \times 7.5 \text{ yr} = 27,100 \text{ tons}$$

Total = 45,850 tons

Summary

Tributary sedimentation \approx 45,850 tons

Shoreline erosion sedimentation \approx 55,070 tons

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APPENDIX D:

EFFECTIVE FETCH AND WAVE BASE DEPTH CALCULATIONS

Site 2Table 11. Effective fetch calculations -
profile direction N 45° W

α	$\cos \alpha$	X_i	$\bar{X}_i \cos \alpha$
42	0.743	650	480
36	0.809	730	590
30	0.866	1070	930
24	0.914	990	900
18	0.951	1070	1020
12	0.978	1270	1240
6	0.995	2280	2270
0	1.000	3300	3300
6	0.995	3670	3650
12	0.978	3710	3630
18	0.951	2490	2370
24	0.914	930	850
30	0.866	690	600
36	0.809	290	230
42	0.743	200	150
Total	13.512		22,210

$$\text{Effective fetch} = \frac{22,210}{13.512} = 1640 \text{ ft}$$

where

α = angle the radials make with the wind direction

X_i = component of length of each radial in a direction parallel
with the wind direction

See figure 41.

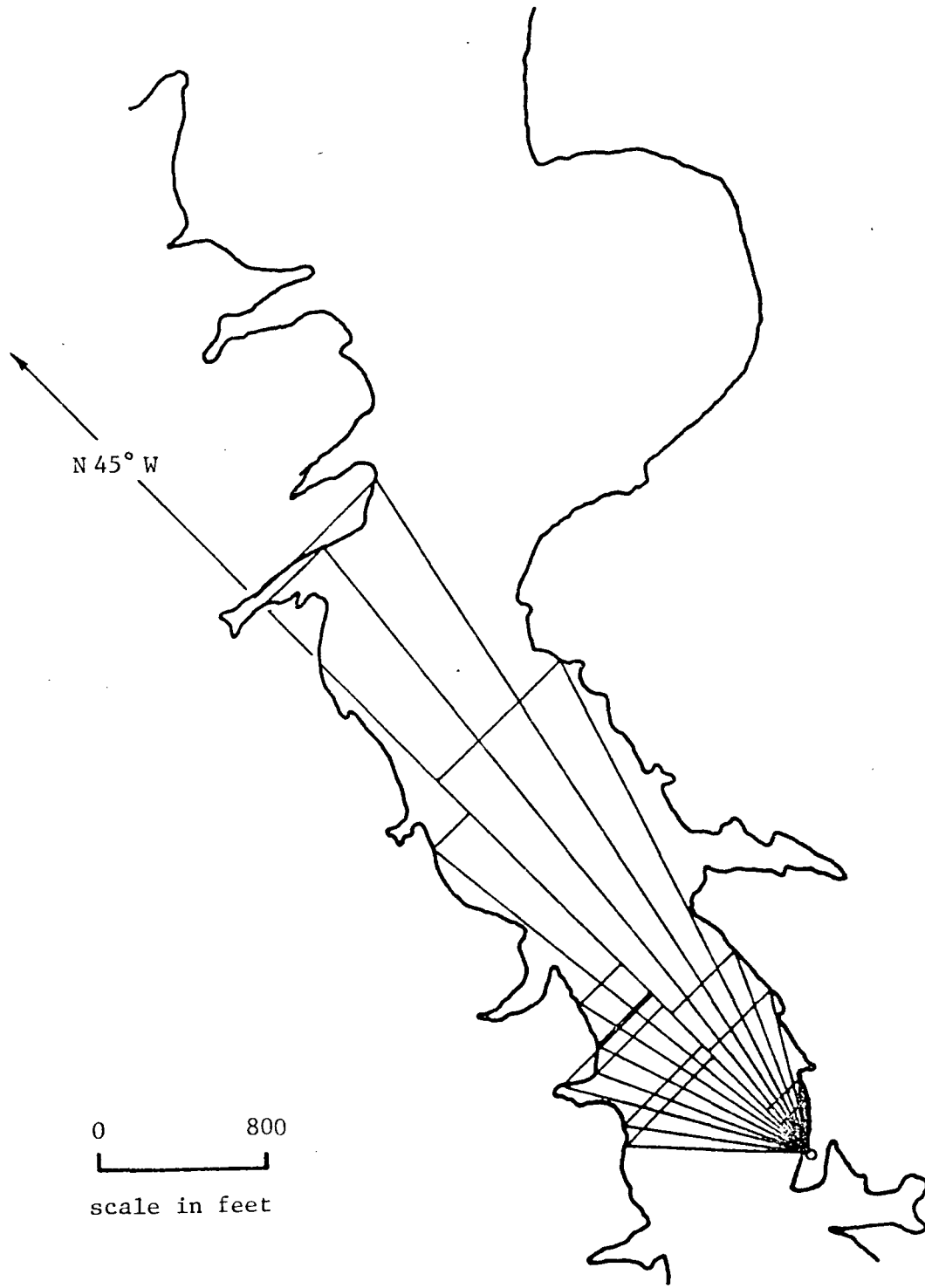


Figure 41. Site 2 Big Creek Lake effective fetch computations

Wave base depth calculations

Effective fetch = 1640 ft

Average depth in northwesterly direction = 40 ft

Wind velocity = 15 mph

Figure 3-28, page 3-50, Errata, United States Army Coastal Engineering Research Center (1973) Shore Protection Manual.

Wave period (T) = 1.30 seconds

Wave length (L) = $5.12 T^2 = 5.12(1.30)^2 = 8.65$ ft

Wave base depth = wave length/2 = $8.65/2 = 4.3$ ft

Site 3

Table 12. Effective fetch calculations - profile direction N 30° W

α	$\cos \alpha$	X_i	$X_i \cos \alpha$
42	0.743	650	480
36	0.809	740	600
30	0.866	970	840
24	0.914	1030	940
18	0.951	1100	1050
12	0.978	1100	1080
6	0.995	1110	1100
0	1.000	1110	1110
6	0.995	1310	1300
12	0.978	5720	5600
18	0.951	7420	7060
24	0.914	7000	6400
30	0.866	4470	3870

Table 12 (cont.)

α	$\cos \alpha$	X_i	$X_i \cos \alpha$
36	0.809	2650	2140
42	0.743	2750	2040
Total	13.512		35.610

$$\text{Effective fetch} = \frac{35,610}{13.512} = 2640 \text{ ft}$$

See Figure 42.

Wave base depth calculations

Effective fetch = 2640 ft

Average depth in northwesterly direction = 20 ft

Wind velocity = 15 mph

Figure 3-24, page 3-48, Errata, United States Army Coastal Engineering Research Center (1973) Shore Protection Manual.

Wave period (T) = 1.37 seconds

Wave length (L) = $5.12 T^2 = 5.12(1.37)^2 = 9.61 \text{ ft}$

Wave base depth = wave length/2 = $9.61/2 = 4.8 \text{ ft}$

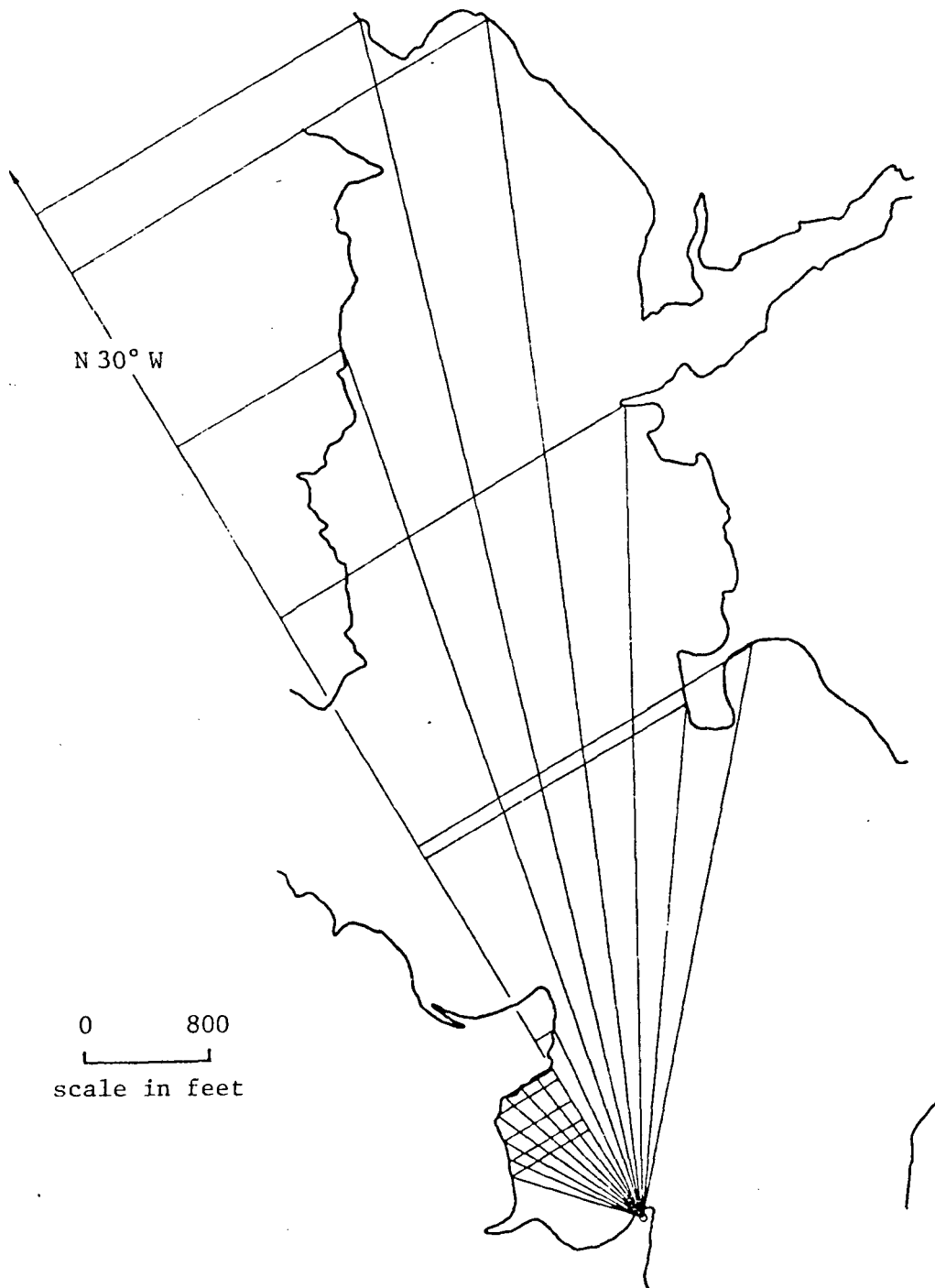


Figure 42. Site 3 Big Creek Lake effective fetch; computations

Site 17Table 13. Effective fetch calculations -
profile direction N 35° W

α	$\cos \alpha$	X_i	$X_i \cos \alpha$
42	0.743	240	180
36	0.809	450	360
30	0.866	490	420
24	0.914	570	520
18	0.951	690	660
12	0.978	780	760
6	0.995	840	840
0	1.000	960	960
6	0.995	2300	2290
12	0.978	6160	6020
18	0.951	7420	7060
24	0.914	7900	7220
30	0.866	5240	4540
36	0.809	3580	2900
42	0.743	3300	2450
Total	13.512		37,180

$$\text{Effective fetch} = \frac{37,180}{13.512} = 2750 \text{ ft}$$

See Figure 43.

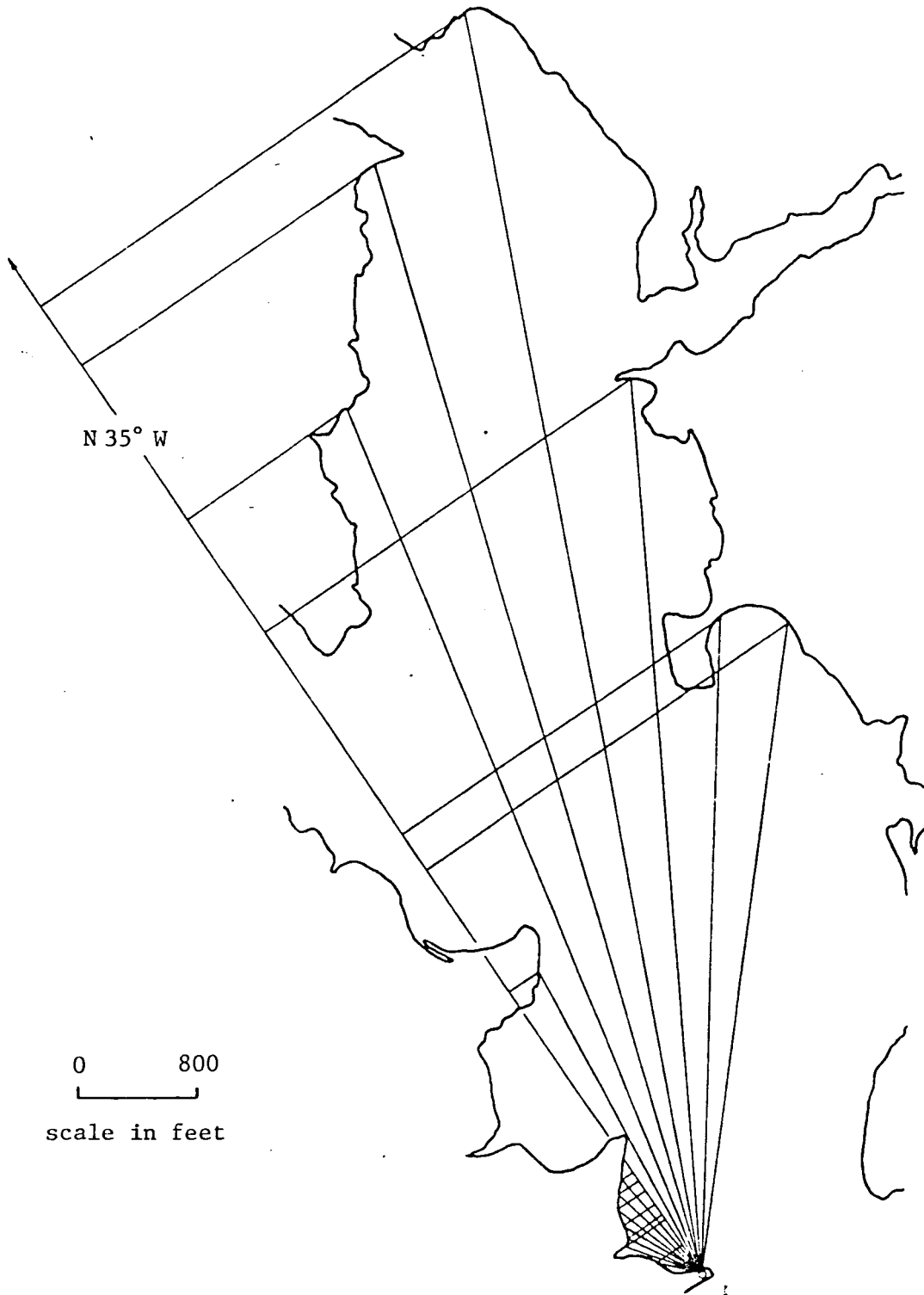


Figure 43. Site 17 Big Creek Lake effective fetch computations

Wave base depth calculations

Effective fetch = 2750 ft

Average depth in northwesterly direction = 20 ft

Wind velocity - 15 mph

Figure 3-24, page 3-48, Errata, United States Army Coastal Engineering Research Center (1973) Shore Protection Manual.

Wave period (T) = 1.40 seconds

Wave length (L) = $5.12 T^2 = 5.12(1.40)^2 = 10.0$ ft

Wave base depth = wave length/2 = $10.0/2 = 5.0$ ft