THE UNIVERSITY OF MANITOBA

# AN ANALYSIS OF FIELD AND LABORATORY DATA FOR UNSTABLE RIVER BANKS IN THE METROPOLITAN WINNIPEG AREA

by

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A THESIS

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## LIST OF SYMBOLS

$\alpha$ — Angle which the base of a sliding block subtends with the horizontal, in degrees.

$\beta$ — Angle which the surface of a slope subtends with the horizontal, in degrees.

$\gamma$ — Total unit weight of soil, in pounds per cubic foot.

$\Phi$ — Total stress angle of internal friction, in degrees.

$\Phi'$ — Effective stress angle of internal friction, in degrees.

$\Phi'_A$ — Effective angle of internal friction required for static equilibrium when a slope is actually on the verge of sliding, in degrees.

$\Phi'_D$ — Effective angle of internal friction required for static equilibrium as determined by the sliding Block slope stability analysis, in degrees.

$\Phi'_P$ — Effective angle of internal frictional strength measured at peak failure stresses in the direction shear test, in degrees.

$\Phi'_R$ — Effective angle of residual internal friction measured at residual failure stresses in the direct shear test, in degrees.

$\sigma_n$ — Normal, or consolidation, stress used in direct shear testing of soils, in pounds per square inch.

$\tau$ — Shear stress induced at the failure plane during a direct shear test, in pounds per square inch.

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|---|---|
| C | Total stress cohesion, in pounds per square inch, pounds per square foot or kips per square foot. |
| $C_P'$ | Effective cohesive strength measured at peak failure stresses in the direct shear test, in pounds per square inch. |
| $C_R'$ | Effective residual cohesive strength measured at residual failure stresses in the direct shear test, in pounds per square inch. |
| $C_U$ | Undrained total stress cohesion, in pounds per square foot or kips per square foot. |
| F | Effective soil reaction at the base of a sliding block, in kips per running foot. |
| U | Hydrostatic, or pore water, force acting on a soil surface, in kips per running foot. |
| W | Total weight of a sliding block, in kips per running foot. |

## LIST OF ABBREVIATIONS

| | |
|---|---|
| Degs. | Degrees |
| ° | Degrees |
| Ft. | Feet |
| ' | Feet |
| Kips | 1000 pound units |
| K.S.F. | Kips per square foot |
| No. | Number |
| P.C.F. | Pounds per cubic foot |
| P.S.I. | Pounds per square inch |

SUMMARY

This thesis is a study of data obtained for two unstable river banks in the Metropolitan Winnipeg area. Records of slope movements and ground water pressures are presented, as obtained in the field by means of slope indicators and piezometers. These provide previously unavailable information regarding pore pressures and bank movements varying with time.

The shape of the slip line was estimated from slope indicator readings and observations of surface cracking. Residual shear strength parameters were found from direct shear tests performed on clay samples from each site.

A simplified method of slope stability analysis was developed to permit ready analysis of piezometer data obtained at frequent intervals of time. A computer program was prepared to process the large amount of data obtained. The analysis permits a correlation of shear strength parameters required for static equilibrium of the river banks at various times of the year. The strength parameters required for equilibrium in the field, for times of impending slope movements, were found to fall within the range of residual values as determined in the laboratory by direct shear tests.

# CHAPTER I

## INTRODUCTION

Over the years, there has been an increasing demand to develop river bank areas in the Metropolitan Winnipeg area. With increased population, greater use has been made of the banks for apartments, bridges, pipe line crossings and parks. Areas previously deemed unsuitable for construction are now receiving consideration for development.

A lack of adequate data, particularly with regard to pore pressures, is the cause of considerable uncertainty in analyzing the stability of the river banks. Uncertainty has also existed in knowing the actual configurations of existing failure surfaces, shear strength parameters, and when slope movements occur.

### I GENERAL DESCRIPTION OF THE STUDY

The locations of the two sites studied, namely the Tache Avenue and St. Vital sites, are given in Figure 1. Data on pore pressures and slope movements had been previously obtained on the river bank at the St. Vital site for a limited time only. It was decided to continue monitoring this site with additional instrumentation to provide more detail. To give a more general basis for analysis, a second site (Tache Avenue) was selected and instrumented. For both sites, pore pressure and slope indicator readings were obtained for a full cycle of winter, spring, summer and fall conditions.

A simple means of analysis of failed banks was developed to permit an appropriate correlation of piezometer readings, river levels, slope movements and shear strength parameters*.

It was realized that with only two banks investigated, the results of the study should not necessarily be considered typical for all river banks in the Winnipeg area. A limited number of laboratory tests were performed to determine residual shear strength parameters. The complete range for these parameters might not have been established. Although it is an important factor, the variation in river bank profile at the toe of each bank due to scour and silting during flood stage was not included in the present study. The numerous soundings required for such a record were considered to be beyond the scope of this thesis.

## II ORGANIZATION OF THE THESIS

Chapter II. A review is made of previous approaches to the problem and their limitations.

Chapter III. The data are described. Details of field instrumentation and laboratory testing are presented. Examples of slope indicator, piezometer, and direct shear test data are given. The complete records, because of their bulk, are included in the appendices in table form.

Chapter IV. The data obtained are analyzed. The derivation of the simple slope stability analysis technique using full residual shear

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* Some of the terms commonly used in the field of soil engineering are defined in Appendix E.

strength is presented, together with graphical illustrations of the results.

Chapter V. The results of the study are discussed. Conclusions are made through consideration of the findings.

# CHAPTER II

## THE HISTORY AND PRESENT STATUS OF SLOPE STABILITY ANALYSIS IN THE METROPOLITAN WINNIPEG AREA

### I DESCRIPTION OF SOIL CONDITIONS

The soil profile in the Metropolitan Winnipeg area has been previously described by Baracos (1)*. Topsoil is usually encountered to a depth of up to one foot. This is successively underlain by three distinct clay units, namely; brown clay, mottled brown and grey clay, and grey clay. Silt layers ranging from several inches to several feet in thickness are sometimes found within the clay or at surface. The clays usually have a laminated structure. To a depth of approximately six to eight feet the structure is nuggety, friable and sometimes slicken-sided in addition to being laminated. This structure is likely due to frost action and weathering.

The grey clay, which is found beneath the mottled brown and grey layer, usually contains small silt inclusions which become larger and more numerous with depth. Silty glacial till is normally found immediately below the grey clay. The till is often soft within the top two feet. Inclusions of grey clay are sometimes encountered in this upper till layer. The till becomes hard and cemented with depth.

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* Numbers in brackets refer to the corresponding reference listed in the bibliography.

Limestone bedrock generally underlies the till. Sometimes the till layer is absent and the grey clay rests directly on the bedrock.

The clays, glaciolacustrine in origin, were deposited by sedimentation in the former Lake Agassiz. The variation in color with depth is suspected to be due to oxidation of the clay minerals. The upper layers are characterized by the rich brown color of ferric oxides. The color becomes grey with depth, indicating the presence of ferrous oxides due to incomplete oxidation.

The most important soils from the viewpoint of slope stability are the clays, since this is the material within which slope failures usually occur. A high content of montmorillonite has been found in Winnipeg clays. Quigley (10) found that the clays contained about 80 percent montmorillonite-illite within the clay size fraction. Their plasticity is generally high, with a plasticity index of approximately 60 (1). The activity ratio, defined as the ratio of plasticity index to percentage of minus two micron sizes (2), is found to be approximately 0.75 to 1.0. This figure is arrived at by assuming the plasticity index to be 60 (1) and the percentage of minus two micron sizes to vary between 60 and 80 as found by Quigley (10).

## II PREVIOUS APPROACHES TO ANALYSIS

The analysis of slope stability involves statical indeterminacy (8). In order to perform such analyses, simplifying assumptions have been made to render them statically determinate. The approaches to slope stability vary in the nature of their simplifying assumptions.

The approaches to slope stability analysis discussed herein involve the assumption of a failure zone in the shape of an infinitely long cylinder of uniform cross section. In section, the failure zone is bounded by a slip line consisting of a circular segment along which the slide tends to move. The assumption of infinite length makes the problem a two dimensional one. A section of unit width, taken perpendicular to the bank, can be used for analysis. The forces acting on the cut faces of this section can be ignored, since this is a plane strain condition.

Taylor's friction circle method (16). The actuating force vectors are considered to be the total weight and the resultant boundary neutral forces. The resisting forces consist of the resultant cohesion and the resultant intergranular frictional force. The intergranular forces acting on segments of the slip surface are assumed to act at an obliquity of $\Phi'_D$*. Hence their lines of action are all tangent to a smaller circle having the same center as the slip surface. This smaller circle is called the friction circle. The approximation is made that the resultant intergranular force acting on the entire slip surface is also tangent to the friction circle. The existence of tension cracks at the top of the bank is usually ignored, or taken care of by using a simple reduction in average shear strength.

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* Symbols used in the text are defined in the List of Symbols.

As a result of the assumptions, the problem becomes statically determinate. The solution for the safety factor with respect to soil strength is accomplished by a consideration of moment equilibrium about the center of the slip circle.

Taylor (16) analyzed a large number of cases by means of the friction circle method and devised a chart of stability numbers from the results. This chart facilitates the solution of safety factors for slopes of a standard cross-sectional shape, such as the one shown in Figure 2. Bank height, slope angle and assumed soil strength parameters are required in using this method.

Methods of slices. The soil within the failure zone is divided into elements consisting of vertical slices. An attempt to solve the equilibrium of forces yields one statical indeterminacy per slice. There are four unknown forces acting on each element, while only three equations of static equilibrium are available for their solution.

The Fellenius solution (12) assumes that the lateral forces acting on each slice are equal and opposite, even though this would only be valid for an infinite bank of constant slope angle. The problem then becomes statically determinate and the solution for the soil mass as a whole can be obtained by considering moment equilibrium about the center of the slip surface.

Bishop's method of slices (3) is more rigorous than the above. It utilizes the concept that the sum of vertical components of lateral forces for all the slices is equal to zero. A trial and error procedure is necessary for evaluation of the safety factor. A simplified version of Bishop's method assumes that the sum of the lateral forces acting on each individual slice is equal to zero. Little and Price (7) have developed a computer program which solves for the safety factor by means of trial and error convergence using Bishop's simplified method.

Shear strength parameters. All the methods discussed so far can be used for effective stress analysis or total stress analysis. In the case of effective stress analysis it is necessary to either assume a pore pressure distribution or actually determine this distribution by means of field instrumentation. When total stresses are used, knowledge of pore pressure distribution is not required.

The peak shear strength parameters $c'_p$ and $\Phi'_p$ are normally used in the effective stress approach, for normally consolidated clays in which no previous failure has taken place. These parameters can be evaluated by means of tests, such as triaxial compression or direct shear, on soil samples. They are derived from the maximum shearing resistance of the soil.

In the total stress analysis of clay slopes, frictional strength is assumed to be zero and cohesion is considered to be the only strength component resisting slope movement. That is, the undrained strength is used. This approach is justified when the slope being analyzed is composed of relatively impermeable clay and the short term stability is being sought. In such a case the undrained cohesion, $c_u$, can be used. This parameter can be evaluated in the laboratory by means of rapid shear tests such as direct shear, unconfined compression, or undrained triaxial tests.

Baracos (1) and Sutherland (15) recommended the use of a reduced value of undrained strength for evaluation of long term stability by means of total stress analysis. The value of cohesion they recommended was 500 pounds per square foot, based on several bank sections that were analyzed.

## III LIMITATIONS OF PREVIOUS APPROACHES

The various classical methods of analysis that have been described are used successfully in many areas of the world. Results obtained from the analysis of river banks in the Metropolitan Winnipeg area, however, have sometimes been misleading. River bank failures have occurred in areas where analyses of slope stability indicated that these banks were stable.

The method of total stress analysis using a reduced undrained strength value, as described previously, has not overestimated the safety factors for the river banks as consistently or to the same degree as the conventional methods of analyses using peak shear strengths (1). Because it does not take pore pressures into account, there will always be some uncertainty involved in the results of this analysis. Baracos (1) suggested that implied or computed safety factors of even 1.5 could be considered unsafe.

<u>Configuration of the slip line</u>. Baracos (1) noted that the slip line is usually an approximately circular arc somewhat flattened at its

base, where it becomes tangent to the till or bedrock. This shape can vary considerably, as evidenced by the points of known slip determined for the St. Vital site in this study, shown in Figure 7. The circular arc assumption is not always appropriate.

Methods have been devised (5, 6, 13) whereby the classical approaches to stability analysis can be used with an assumed slip line having the shape of a logarithmic spiral. An attempt to fit such a curve to the St. Vital case was unsuccessful.

Morgenstern and Price (9) developed a stability analysis by which irregular shapes of slip lines could be incorporated. In this method the equations of static equilibrium are satisfied, various pore pressure distributions can be utilized, and any shape of slip line can be assumed. It is essentially a method of slices in which the vertical interslice forces are considered to vary as an arbitrarily assumed function of the horizontal interslice forces. This method has the drawback of being complicated and difficult to use. For the sake of expediency of calculation, the decision was made to exclude this method from the study and to utilize a simpler method which also takes into account measured pore pressures and uses a simple geometry to approximate the failure surface. This was done since a large number of slope stability analyses had to be made, as can be seen in Appendix A.

Tension cracks filled with water can have an important effect on the results of stability analyses (14). This is not always taken into account in the classical methods, but it is included in the method used in this study.

Pore pressure distribution. Pore water forces are a key factor in effective stress analyses. Accurate knowledge of these is required if results are to be realistic. There is a necessity for an extensive study of pore pressure distributions in the river banks of the Metropolitan Winnipeg area. Without accurate pore pressure data, slope stability calculations will continue to be unreliable.

Shear strength parameters. Total stress analysis using a reduced cohesion is not reliable because it is too empirical an approach. It could yield excessively high safety factors in some cases, or be excessively conservative in others because of the uncertainty involved in choosing a reduced cohesion value.

Effective stress analysis, using appropriate shear strength parameters would be more realistic. Skempton (11) found, from his studies of London clay, that the shear strength parameters decrease from the peak strength values after initial failure. The results of large strain direct shear tests indicated that eventually the cohesion approached a small value close to zero and the friction angle approached a constant lower value, which can be defined as the residual friction angle. He suggested that subsequent movements of clay banks after initial failure are controlled by residual strength parameters, rather than by peak values. This appears to be verified for banks in the Winnipeg area. The use of peak strength parameters has yielded unreasonably high safety factors for the assumed pore pressure distributions (15), suggesting that these parameters are too high.

Baracos (1) suggested that the Winnipeg river banks were originally extremely steep when the river channels were formed. Subsequent failures occurred, leaving old slip lines. Soil strengths along such slip lines are low. These slip lines are difficult to detect from test borings because the soil above and below the slip line was left intact. Old failures have been masked by surface weathering and vegetation.

In his letter to W. D. Hurst, City Engineer of Winnipeg, Casagrande (4) stated that all major recent failures occurred during subsidence of river levels after severe floods. The clay was left permanently weakened.

Consideration of the above points leads to the conclusion that effective stress analyses should generally make use of residual, rather than peak, shear strength parameters in areas where failures have occurred in the past. It is likely that many areas along these banks have old slip lines, as yet undetected, along which the strength has been reduced.

## IV METHOD OF ANALYSIS EMPLOYED

In the method employed, an attempt was made to approximate closely the shape of the existing slip lines, as interpolated from the slope indicators. Pore pressure distributions used were interpolated from pore pressure readings, as determined in the field. The stability of the sites studied was assumed to be controlled by residual shear strength parameters.

Configuration of the slip line. A probable curved slip line for each site was estimated by interpolation between points of known slip such as slope indicator locations and visible surface cracks. A reasonably close fit for each slip line was found to be a series of three straight lines, as illustrated in Figure 2. A tension crack was included for one of the sites, since this crack was observed in the field. The failure zone was divided into a system of three sliding blocks with vertical interfaces to facilitate the analysis. The geometry of the sliding blocks was taken to fit the assumed slip pattern, based on the slope indicator data and observed surface movement.

Pore pressure distribution. Field instrumentation to determine pore water forces was established at each site. The locations of piezometers at the St. Vital and Tache Avenue sites are shown in Figures 3 and 4, respectively. Readings of pore pressure were taken, often at weekly intervals, during 1969 and 1970. The intention was to analyze the stability of each site as it varied with time using pore water forces determined by interpretation of field data.

A graphical determination of pore water forces acting on the failure surface was obtained by drawing equipotential lines to fit the observed piezometer readings. Examples of equipotential lines are shown in Figures 5.1 to 5.4 and 6.1 to 6.4. Pore pressure heads were determined at points where the equipotential lines crossed the lower boundary of each sliding block, by subtracting elevation head from total head. The pore pressure heads at each of these points was represented graphically by a vector of suitable length acting normal to the surface. A smooth curve was drawn joining these vectors, thereby estimating the distribution of pore pressure heads acting on the lower surface of each sliding block. Resultant pore water forces were then determined by estimating the areas bounded by these pore pressure distributions.

Shear strength parameters. Considering that the banks selected for study already showed signs of instability, it was considered appropriate to approach the problem initially on the basis of using residual shear strength parameters (11). The following considerations also tended to justify this approach. The presence of old slide planes proposed by Baracos (1) and Casagrande (4), certainly must have reduced the strength to something below the peak value. Terzaghi and Peck (17) proposed that strength loss could even occur in banks that had not previously failed. Progressive failure could be caused by stress concentrations at discontinuities, such as tension cracks, thereby weakening the soil. The use of peak shear strength parameters appeared to be precluded.

The values of $c'_R$, as obtained from laboratory tests, were relatively small. As shown in Table 1, $c'_R$ was found to vary between 0.0 pounds per square inch, and 1.5 pounds per square inch, with an average of 0.7 pounds per square inch. The conservative assumption was made that the cohesive force was zero.

## CHAPTER III

## FIELD AND LABORATORY PROCEDURES

### I FIELD INSTRUMENTATION

The location of some river bank slide areas in Metropolitan Winnipeg, after Baracos (1), are shown in Figure 1. Locations of both sites studied by the writer are indicated on this figure.

The St. Vital site was studied because partial piezometer data dating back to 1962 were available for this site. These data are given in Table XI. Van Cauenberghe (18), had studied this bank previously, and two slope indicators and ten piezometers had been installed in conjunction with the study. Subsequently, three additional piezometers were installed and the taking of readings at all the piezometers was continued.

The Tache Avenue site was chosen because it was a failure of a bank much steeper than the St. Vital bank. The Tache Avenue site had an over-all slope angle of about twelve degrees, while the St. Vital site had an over-all slope angle of about six degrees. Thus it was felt that a comparison of the soil and water conditions at each site might yield some relevant information regarding slope stability of river banks in the Metropolitan Winnipeg area.

<u>Slope indicators</u>. The slope indicators, obtained from the Slope Indicator Company of Seattle, have an outside casing diameter of 3.18

inches. The casing has four equally spaced longitudinal grooves which serve as guides for the slope indicating probe that is lowered down the casing. The probe is approximately 1.5 feet long and 2 inches in diameter. It has two sets of wheels, spring loaded, which run in opposite longitudinal grooves of the casing.

An instrument consisting of a wheatstone bridge circuit is connected to the probe by a cable. Inside the probe, forming part of the circuit, is a pendulum which is electrically wired so that the wheatstone bridge must be re-balanced each time the probe is at a different inclination. The balancing procedure for the wheatstone bridge yields readings which are proportional to the slope of the probe, hence these readings also indicate the slope of the casing as the probe is lowered to various depths.

The changes in horizontal position of the casing at various times after installation are determined by numerically integrating the changes in slope along the casings. Proper seating of the bottom of the casing in an immovable layer is required to provide a reference point of zero movement. The calculation of horizontal movement at various depths in the casing makes use of the assumption that the bottom of the casing does not move.

The casings were installed in holes bored vertically into the banks at desired locations, to depths that would ensure proper seating in the stable glacial till. The holes were back-filled around the casings with bentonite grout to ensure a snug fit in the holes, so that