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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LIST OF SYMBOLS

α	Angle which the base of a sliding block subtends with the horizontal, in degrees.
β	Angle which the surface of a slope subtends with the horizontal, in degrees.
γ	Total unit weight of soil, in pounds per cubic foot.
Φ	Total stress angle of internal friction, in degrees.
Φ'	Effective stress angle of internal friction, in degrees.
Φ'_A	Effective angle of internal friction required for static equilibrium when a slope is actually on the verge of sliding, in degrees.
Φ'_D	Effective angle of internal friction required for static equilibrium as determined by the sliding Block slope stability analysis, in degrees.
Φ'_P	Effective angle of internal frictional strength measured at peak failure stresses in the direction shear test, in degrees.
Φ'_R	Effective angle of residual internal friction measured at residual failure stresses in the direct shear test, in degrees.
σ_n	Normal, or consolidation, stress used in direct shear testing of soils, in pounds per square inch.
τ	Shear stress induced at the failure plane during a direct shear test, in pounds per square inch.

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 slickensided 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 Φ'_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.

<u>Shear strength parameters</u>. 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 Φ'_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.

Configuration of the slip line. 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.

<u>Shear strength parameters</u>. 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.

Slope indicators. 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 subsequent casing movements would represent actual soil movements as closely as possible.

Piezometers. Two types of piezometers were used, namely the Casagrande stand-pipe and the pneumatic Thorpiezo piezometer.

The Casagrande stand-pipe piezometer consists of nylon tubing installed to a desired depth. A porous tip at the bottom facilitates the flow of ground water into or out of the tubing so that the water level can indicate ground water pressure at the porous tip. In reading the piezometer, a two wire cable is lowered down the tubing. The cable is connected to a galvanometer and a battery at ground level. When the bottom end of the cable makes contact with the water surface in the tubing, the galvanometer indicates electric current. The position of this water level can then be recorded according to the length of cable lowered down the tubing.

The tip of the pneumatic Thorpiezo piezometer consists of a well-point containing a closed diaphragm and valve system. A cable containing three strands of nylon tubing leads from the tip to the ground surface. A special instrument is connected to two of the strands of tubing at ground level. Air is forced down one of the lines to the tip. One pressure gauge indicates the air pressure being fed into the tip, while a second gauge registers air pressure coming back up through the second line.

When the air pressure at the diaphragm in the tip is equal to the ground water pressure, a valve in the tip closes. At this stage the air pressure indicated in the outflow line is equal to the ground water pressure and can be recorded as such.

The use of Thorpiezo piezometers has several advantages over the use of stand-pipes. Because a Thorpiezo is a closed system, very little flow of water is required to register pore pressure changes and thus the piezometer responds quickly to pore pressure changes, even in relatively impermeable clay. Thorpiezos do not freeze in the winter, since water in the system is confined to the well-point tip. Inundation of a site by high water levels does not prevent reading of the piezometers, since the tubing from all the piezometers can be led back to a central location at the top of the bank.

The St. Vital site. A cross-section of the bank showing locations of slope indicators and piezometers is presented in Figure 3. The logs of two test holes are also shown. The slope indicators were seated in hard till to provide reference points of zero movement at the bottom of each casing. Piezometers were placed at locations which would facilitate the determination of equipotential lines throughout the bank. Stand-pipe and pneumatic piezometers were both used. The bank at St. Vital is shown in Photograph 1. The piezometer data are shown in Table XI. Slope indicator data are presented in Tables XV, XVI and XVII.

The Tache Avenue site. The cross-section showing piezometer and slope indicator locations is presented in Figure 4. The logs of two test holes are also shown in this figure. The tips of slope indicators

No. 1 and No. 2 were seated in firm till and hard till, respectively. As evidenced by subsequent slope indicator readings, seating for both was adequate. Because of the advantages previously cited, pneumatic piezometers were used exclusively at this site. The piezometers were located so that the configuration of the equipotential lines could be estimated within the failure zone and especially in the vicinity of the slip line. Photograph 2 shows the bank at Tache Avenue. Piezometer data are presented in Table XII and slope indicator data are presented in Tables XVIII and XIX.

II DIRECT SHEAR TESTS

Typical samples of undisturbed clay from the banks were tested using shear boxes similar to those used by Skempton (9). The test set up is shown in Photograph 3.

In most cases each "undisturbed" thinwall tube sample selected for testing was divided into three test specimens which were tested separately. Each of the three specimens was consolidated at a different normal stress, ranging from 10 to 30 p.s.i., prior to testing in direct shear. The rate of shear was kept constant at approximately .005 inches per hour. This slow strain rate was used so that pore pressure build-up during testing would be negligible and effective stress conditions would be preserved. The shearing force induced by the movement was usually recorded at one hour intervals. After the shearing displacement had reached 0.1 inches, a reversed strain was applied to return the sample to its original configuration. Forward shear was then repeated for a displacement of 0.1 inches. Samples were subjected to ten such cycles of shear, to give a cumulative shearing movement of 1.0 inches.

Irregular shear planes were often found to have developed, causing excessive variation in shear force with displacement. That is, the shearing resistance of specimens after large strains had taken place fluctuated instead of approaching a constant residual value. The fluctuation was suspected to be due to the rough surface of failure, varying about 1/8 inch in relief between peaks and hollows. Such a rough shear plane is shown in Photograph 4. The irregular variation in shearing resistance with displacement after large total strains had taken place caused difficulty in estimating residual shearing resistance. This problem was substantially reduced in subsequent tests by cutting smooth shear planes with a wire saw after peak strength had been passed. Shearing was then continued along the pre-cut plane to determine residual shearing strength. Pre-cutting of failure planes seemed to reduce the variation in test results. That is, the range in Φ'_R for pre-cut samples is 8.3° to 9.5°, while for samples with irregular shear planes the range is 8.0° to 13.0°. A similar difference in ranges is observed for c'_R. The over-all range in Φ'_R was found to be 8.0° to 13.0° and that for c'_R was 0.0 p.s.i.* to 1.5 p.s.i. These results are shown in Table I.

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

TABLE I

DIRECT SHEAR TEST RESULTS

SAMPLE DESCRIPTION	SITE	DEPTH (FT.)	CHAINAGE (FT.)	SAMPLE NO.	PEAK ϕ'_P DEGS.	PEAK c'_P PSI	RESIDUAL ϕ'_R DEGS	RESIDUAL c'_R PSI	COMMENTS
HIGHLY PLASTIC MOTTLED BROWN AND GREY CLAY	ST. VITAL	11.0	1+30	SH-1			9.0	0.8	Sample pre-cut for failure & sheared during consolidation
	ST. VITAL	11.0	1+47	SH-203	18.5	1.5	13.0	0.5	Irregular shear planes observed
	TACHE AVE.	16.0	0+68	T-3	18.2	1.8	8.3	0.6	Samples cut for final failure
	TACHE AVE.	21.0	0+68	T-4	22.0	0.4	9.5	0.0	Samples cut for final failure
HIGHLY PLASTIC GREY CLAY WITH SILT LUMPS	ST. VITAL	20.0	1+30	SH-207	16.5	2.5	11.5	1.5	Irregular shear planes observed
	ST. VITAL	21.0	1+47	SH-205	13.5	1.5	8.0	0.5	Irregular shear planes observed
	ST. VITAL	21.0	1+47	SH-205	16.8	2.0	9.0	0.8	Samples cut for final failure
SILTY GLACIAL TILL	ST. VITAL	31.0	1+47	SH-218	33.0	0.0	31.5	0.0	

RESIDUAL AVERAGES

DESCRIPTION	ϕ'_R DEGS.	c'_R PSI
All clay samples	9.6	0.7
Mottled brown and grey clay	9.9	0.5
Grey clay	9.5	0.9
Pre-cut samples	9.0	0.5
Samples with irregular shear planes	10.8	0.8

CHAPTER IV

DEVELOPMENT AND USE OF A SIMPLE SLOPE STABILITY ANALYSIS

To permit ready analysis of the large amount of data obtained, simplifying assumptions were made and a convenient slope stability analysis technique was developed. The analysis was computerized to facilitate the solution for the shearing resistance required for statical equilibrium at each site. Values of shearing resistance required for statical equilibrium, as obtained for each day that piezometer readings were taken, were correlated with observed slope movements.

I DEVELOPMENT OF THE SIMPLIFIED ANALYSIS

Points of known shear movement were determined for both river banks as follows. Slope indicators provided two points of known slip at each site. Figures 7 and 8 illustrate the horizontal displacements of the slope indicator casings, due to slope movements. The point where the slip line intersected with each casing was interpreted from these displacements. An additional slip point was determined for each site by observations of surface cracking, at the tops of the banks, due to slope movements. The location of the St. Vital slip line, where it emerges at the toe, was estimated to coincide with the discontinuity in the river bank profile at station 2+60, as shown in Figure 7.

The control points which have been described were used for the graphical interpolation and extrapolation of the curved slip lines. These slip lines, illustrated in Figures 7 and 8, are somewhat flattened where they become tangent to the till layer and where there is a large component of horizontal movement. They are similar in shape to those described by Baracos (1). For the purpose of analysis, the simplest configuration of the failure surface was found to consist of three sraight lines, which fitted the curved slip lines. The interpolation and fitting, as described above, are shown in Figures 7 and 8. Two trial slip lines, shown in Figure 8, were used for the Tache Avenue site since the overtrust point at the toe could not be discovered in the river bank profile.

Assumptions. The soil mass undergoing movement was assumed to consist of a simplified system of three sliding blocks. Such a system is statically indeterminate. Statical determinacy was achieved through the following assumptions. The vertical component of base reaction for each block was assumed to be in vertical equilibrium with the weight of the block and the boundary forces. The second assumption was that the effective frictional reactions at the bases of all three blocks acted at the same obliquity (ϕ_D') to these surfaces.

The first assumption in the preceding paragraph is equivalent to stating that vertical forces are not transmitted through the vertical interfaces between blocks. The presence of such forces would change the distribution of the total weight of the blocks transmitted to the bases of the individual blocks. The total weight of the blocks is a constant defined by the geometry of the block system. The validity of the first assumption was checked by means of the simplified analysis which was developed. This will be further discussed subsequent to the derivation.

The second assumption, namely that Φ_D' was the same at the base of each block, was made because the same residual shear strength parameters were to be used for each block. These parameters also fall within a fairly narrow range, as given in Table I.

Residual cohesion was initially assumed to be zero, for the reasons previously described in Chapter II, Section II. This helped to simplify the derivation.

Classical methods of stability analysis which make use of straight slip lines are the Rankine, Coulomb and Culmann methods. The method of sliding blocks that was used in this study may be considered to be a modification of the Coulomb sliding wedge analysis in that a single sliding plane is not utilized necessitating additional assumptions regarding the distribution of forces at the base.

Derivation. A general example of a bank, showing the assumed slip lines and defining the force vectors used in the derivation, is presented in Figure 2. The block weights, and hydrostatic and pore water forces were assumed to be known. All symbols used in the derivation are defined in Figure 2.

The required friction angle for static force equilibrium was solved for as follows:

Because of the assumption that the vertical components of reaction at the base of each block are simply related to block weights, these forces become statically determinate. Hence the following three equations arise out of consideration of vertical force equilibrium for each block:

$$F_{Y1} = W_1 - U_{B1}\cos\alpha_1, \qquad 1.$$

$$F_{Y2} = W_2 - U_{B2}\cos\alpha_2, \qquad 2.$$

$$F_{Y3} = W_3 - U_{B3}\cos\alpha_3 + U_T\cos\beta. \qquad 3.$$

The three-block system may be considered to be a single free-body for consideration of horizontal force equilibrium. Accordingly, consideration of horizontal force equilibrium for the system as one unit yields the following equation, which interrelates the unknown horizontal components of frictional base reaction in terms of known quantities:

$$R_{L1} + U_{L1} + U_{B1}\sin\alpha_1 + U_{B2}\sin\alpha_2 + U_{B3}\sin\alpha_3 - U_T\sin\beta - F_{X1} - F_{X2} - F_{X3} = 0. \qquad 4.$$

Since the purpose of this derivation is to solve for Φ_D', equations are required by which Φ_D' can be related to the unknown horizontal and vertical components of frictional resistance. Consideration of the geometry of the problem, as shown in Figure 2, yields the following three equations:

$$F_{X1} = F_{Y1}\tan(\Phi_D' - \alpha_1), \qquad 5.$$

$$F_{X2} = F_{Y2}\tan(\Phi_D' - \alpha_2), \qquad 6.$$

$$F_{X3} = F_{Y3}\tan(\Phi_D' - \alpha_3). \qquad 7.$$

Equations 1, 2 and 3 may be substituted into equations 5, 6 and 7 in order to eliminate the unknown F_Y forces. This is done to obtain equations 5a, 6a and 7a:

$$F_{X1} = (W_1 - U_{B1}\cos\alpha_1)\tan(\Phi_D' - \alpha_1), \qquad 5a.$$
$$F_{X2} = (W_2 - U_{B2}\cos\alpha_2)\tan(\Phi_D' - \alpha_2), \qquad 6a.$$
$$F_{X3} = (W_3 - U_{B3}\cos\alpha_3 + U_T\cos\beta)\tan(\Phi_D' - \alpha_3). \qquad 7a.$$

These three equations may be substituted into equation 4 in order to eliminate the F_X terms, as follows:

$$\begin{aligned} &R_{L1} + U_{L1} + U_{B1}\sin\alpha_1 + U_{B2}\sin\alpha_2 + U_{B3}\sin\alpha_3 - U_T\sin\beta \\ &- (W_1 - U_{B1}\cos\alpha_1)\tan(\Phi_D' - \alpha_1) \\ &- (W_2 - U_{B2}\cos\alpha_2)\tan(\Phi_D' - \alpha_2) \\ &- (W_3 - U_{B3}\cos\alpha_3 + U_T\cos\beta)\tan(\Phi_D' - \alpha_3) = 0. \end{aligned} \qquad 4a.$$

Equation 4a may be simplified by collecting terms involving known values. Hence the following expressions are defined:

$$A = R_{L1} + U_{L1} + U_{B1}\sin\alpha_1 + U_{B2}\sin\alpha_2 + U_{B3}\sin\alpha_3 - U_T\sin\beta,$$
$$B = W_1 - U_{B1}\cos\alpha_1,$$
$$C = W_2 - U_{B2}\cos\alpha_2,$$
$$D = W_3 - U_{B3}\cos\alpha_3 + U_T\cos\beta.$$

Substitution of these expressions into Equation 4a yields:

$$A - B\tan(\Phi_D' - \alpha_1) - C\tan(\Phi_D' - \alpha_2) - D\tan(\Phi_D' - \alpha_3) = 0. \qquad 4b.$$

Equation 4b may be further modified to separate Φ_D' from the α's by means of the trigonometric identity

$$\tan(\Phi_D' - \alpha) = \frac{\tan\Phi_D' - \tan\alpha}{1+\tan\Phi_D'\tan\alpha}.$$

This is done to obtain the following equation:

$$A - \frac{B(\tan\Phi_D' - \tan\alpha_1)}{(1+\tan\Phi_D'\tan\alpha_1)} - \frac{C(\tan\Phi_D' - \tan\alpha_2)}{(1+\tan\Phi_D'\tan\alpha_2)} - \frac{D(\tan\Phi_D' - \tan\alpha_3)}{(1+\tan\Phi_D'\tan\alpha_3)} = 0. \qquad 4c.$$

By suitable manipulation, equation 4c can be transformed into a cubic equation of the form

$$\tan^3\Phi_D' + P\tan^2\Phi_D' + Q\tan\Phi_D' + R = 0, \qquad 4d.$$

where P, Q and R are functions of the base angles, block weights, pore water forces and hydrostatic forces. Once $\tan\Phi_D'$ is solved from equation 4d, the safety factor, in terms of the required friction angle for static equilibrium, can be solved for as follows:

$$\text{Safety Factor} = \frac{\tan(\Phi_R')}{\tan(\Phi_D')}, \qquad 4e.$$

where Φ_R' is the residual strength angle available.

To check the validity of the assumption that the vertical component of base reaction on each block is in equilibrium with the block weight and the boundary neutral forces, the simplified analysis was performed using various distributions of the total weights of the blocks. A vertical shear force was assumed to act at each interface, thereby redistributing the weights of the blocks. A vertical force equal in magnitude to ten percent of the weight of the top block was assumed to act at the vertical interface between the top block and the center block. Similarly, a vertical force equal in magnitude to ten percent of the bottom block weight was assumed to act at the interface between the center and bottom blocks. This led to four possible combinations of block weights, since the shear forces could be assumed to act in either the upward, or the downward direction. The adjusted block weights were used in the computer program which was developed to solve the equations that have been derived. The values of Φ'_D so determined deviate from those found without redistributing the block weights. The maximum deviations in these trial calculations, as shown in Table XX, were found to be 1.9° for the Tache Avenue site and 1.4° for the St. Vital site. These deviations were felt to be acceptable.

The use of the computer program. A computer program which was developed to solve equations 4d and 4e is given in Table II. The program was written in such a way that arbitrary values of assumed developed cohesion could be included in the analysis, even though cohesion was generally assumed to be zero. Provision was also made for analysis of the three-block system by the total stress method, in which the soil is assumed to be frictionless. The above modifications were made for the purpose of comparing the results of these analyses.

The weights and water forces were determined from the field data. Determination of weights was by graphically measuring the volumes of the blocks shown in Figures 7 and 8 and multiplying by the value of unit weight. Pore water forces were determined by graphical construction using interpolated equipotential lines, as described in Chapter II, Section IV. Examples of equipotential lines obtained from piezometer readings are presented in Figures 5.1 to 5.4 for the St. Vital site and Figures 6.1 to 6.4 for the Tache Avenue site.

River elevations at the Redwood Bridge, as recorded by the Federal Department of Energy, Mines and Resources, were obtained from the Province of Manitoba Water Control and Conservation Branch. The river elevations at the Tache Avenue and St. Vital sites were found to be higher than those at the Redwood Bridge by 1.0 feet and 2.3 feet, respectively. This was determined by level surveys. River levels at each site for any date desired were accordingly determined by adjustment of the levels at the Redwood Bridge. The hydrostatic force on the toe at each site was evaluated, based on the adjusted river levels.

The above data were entered into the computer program using the input data cards shown in Tables III, IV and V.

II RESULTS OF THE SLOPE STABILITY ANALYSIS

Values of Φ_D' were determined from the analysis. A plot of Φ_D' and Red River elevation against time is presented in Figure 9 . This plot indicates that , in general, Φ_D' decreases when the river level increases. This is to be expected, since the hydrostatic force acting on the toe of the slope tends to restrain slope movement. Other fluctuations in Φ'_D are due to the variations in pore pressures in the bank, which are influenced somewhat by river levels, but may be primarily influenced near the top of the bank by ground water seepage due to precipitation. Increases in pore pressures tend to promote movement.

Total slope movement, as determined from slope indicator readings, and values of Φ'_D were plotted against time in Figure 10. A useful relationship between total slope movement and Φ'_D was observed in this figure. It appeared that for each site there was a critical value of friction angle, Φ'_A, in that the bank did not move when Φ'_D was below this value, and moved when Φ'_A was exceeded. The value Φ'_A was found to be 8.1° for the St. Vital site. Two values of Φ'_A were determined for the Tache Avenue site, namely 10.8° and 12.7° for trial slip lines assuming base angles at the toe block of -9.7° and -13.5°, respectively. Calculations using the computer program were then made for safety factor, for each set of piezometer readings, using the value of Φ'_A as the value of Φ'_R. This gave a safety factor of unity when failure was impending. It will be noted that the values of Φ'_A so used fall within the range of Φ'_R values given in Table I. The computer outputs showing Φ'_D values and safety factors thus obtained are presented in Tables VI and VII.

A trial calculation was made which included a residual cohesion of 0.6 p.s.i. in the analysis of the Tache Avenue site. This resulted in values of Φ'_D, shown in Table VIII, which were lower than the computed residual strength angles during periods of movement.

A total stress analysis, using a reduced cohesion of 500 pounds per square foot, was also made by means of the computer program. Safety factors which indicated stable conditions at all times were calculated, as presented in Table IX. The lowest safety factors obtained were 1.97 for the St. Vital site and 2.08 for the Tache Avenue site. To give a safety factor of unity during periods of impending slope movement, values of reduced cohesion of 190 p.s.f. for the Tache Avenue site and 248 p.s.f. for the St. Vital site would have been required.

Total stress analyses by means of Taylor's stability charts (16) using a reduced cohesion of 500 p.s.f. were also made. This analysis is one of the methods commonly used in the Winnipeg area. It is the method recommended by Sutherland (15), which has been described in Chapter II, Section II. Six trials were performed for each site. In these trials, simple slope configurations were used to approximate actual bank configurations, as shown in the diagram at the head of Table X. The trial calculations which yielded the minimum safety factor for each site are presented in this table. Resulting safety factors were: 1.68 for the St. Vital site and 1.37 for the Tache Avenue site. The analysis would not have predicted failure unless values of reduced cohesion of 297 p.s.f. for the St. Vital site and 365 p.s.f. for the Tache Avenue site had been used.

CHAPTER V

DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

I DISCUSSION

There are several aspects of the study meriting discussion such as considerations of the locations of the sites and the results obtained through instrumentation and analyses.

Locations The Tache Avenue and St. Vital sites can be seen in Figure I to be locations on the concave sides of river bank curves. Many other unstable river banks have this feature in common. Considering that the soils forming the banks have been shown to be similar for a good part of the Winnipeg area (I), it would indicate that an investigation of these other sites might lead to similar findings. The results of the present investigation could therefore be applied in a preliminary consideration to other sites.

Profiles of both river banks, showing the logs of bore holes in addition to the locations of slope indicators and piezometers, are presented in Figures 3 and 4. The lack of information of soil conditions at the toe of the banks is obvious in these figures. To obtain information in these lower regions would entail considerable cost because of the difficulty of operating soil sampling equipment in these relatively inaccessible areas. Additional information in this regard would certainly lead to a refinement in the analysis, particularly if the soils at the toe are fluvial, rather than glaciolacustrine, in origin.

Figure 3 illustrates the extremely gentle slopes (6^{o} from the horizontal) that an unstable river bank can have in the Winnipeg area.

Pore Pressures The previously unavailable information on piezometric levels in the river banks in the Winnipeg area is summarized in Figures 5.1 to 5.4 and 6.1 to 6.4. Many sets of data had been examined to select these illustrative examples.

The information can be presented in a number of forms. Contours of piezometric pressure could have been plotted. Instead, contours of piezometric level, which combine piezometric head and elevation head, were drawn. The contours are based on limited information and involved extrapolation as well as interpolation, thus each represents one possible distribution.

Since the velocity of flow of water through soils is extremely small and leads to negligible velocity head, the contours of piezometric level can be considered to be in terms of total head where total head is the sum of pressure, velocity and elevation heads. If the simplifying assumption is made that the soils in the banks have homogeneous and isotropic permeabilities, the direction of seepage is theoretically perpendicular to contours of total head and in the direction of decreasing head.

This leads to some rather interesting observations. For example, Figure 5.1 indicates that the slip line feeds water into the bank. In this figure it will be noted that the equipotential lines of 55 (755 ft), 50 (750 ft) and 45 (745 ft) form a nest of approximate parabolas about the slip line. This is shown to a lesser extent in

Figures 5.2 , 5.3 and 5.4 . The conclusion is that the total head in this part of the slip line is being raised by the entry of water through the crack at the surface of the bank. Part of this water could be surface runoff. As was observed on this site, there is a pipe, leading from the Windsor Theatre (shown in Figure11), which intermittently discharges water into the crack at the top of the bank.

A similar nest of parabolic contours, shown between stations 1+00 and 1+15 at about elevation 710 in Figures 5.1 and 5.4 , may indicate the presence of an old slip plane.

In general, the direction of seepage in the St. Vital bank was downward, except where it is believed to have been affected by the presence of the slip line. This was true for the entire year, as shown by the data for the four dates shown.

The total head lines for the Tache Avenue site, shown in Figures 6.1 to 6.4 , also indicate generally downward flow into the bank. Figure 6.1 shows that the piezometric levels in the slide zone in winter are higher than winter river levels. This condition indicates that pore pressures in the bank are not readily relieved when the river is lowered, and that conditions favor instability during winter periods. The lowering of these piezometric levels to coincide with winter river level would have a very marked effect on the stability of the banks.

The effect of the spring high river level in raising the piezometric level within the bank is illustrated in Figure 6.2 . The inundation produces flow of water into the bank. The subsequent drawdown of the river to its summer level (Figure 6.3) was noted to leave a zone of high pressure near the toe of the bank, as indicated by a closed contour of total head equal to 45 (745 ft). This can be interpreted to indicate upward seepage near the toe under these conditions, which would certainly contribute to local instability. This may be an explanation for small slides sometimes observed at the toe.

It should also be noted that contours of equal head in the vicinity of the toe required considerable extrapolation. Inundated surfaces are theoretically equipotential lines. This theoretical consideration was used to assist in drawing the contours in the lower reaches of the bank for each site. The installation of piezometers at the toe would have contributed greatly to the accurate determination of these contours. The high cost of equipment that could operate at the toe of the bank prevented the installation of such piezometers.

Slope Indicators and Slip Planes Figures 11and12, in conjunction with Figures 7 and 8, show the locations of the slope indicators at the two sites. It is obvious that two slope indicators provide only limited information on the location of the slip line at a site. Additional slope indicators, particularly at the toe of the banks, would help to alleviate the uncertainty of the location of slip lines. There is, of course, a practical difficulty, in that slope indicators installed in the river itself would be affected by inundation and could be damaged by ice movements.

In this present study, the problem of uncertainty of the slip line locations at the toe was considered by taking, in the case of the Tache Avenue site, two possible positions of the slip line where it emerges at the toe. The significance of this is discussed in the section on "Correlation".

Figures 7 and 8 show that the slope indicators clearly establish the depths at which large components of lateral movement have taken place. They also illustrate that considerable interpolation and extrapolation was necessary to estimate the configuration of the slip lines.

Shear Strength Parameters It will be noted, in Table I, that the residual shear strength parameters fell within a relatively narrow range. Values for Φ'_R for clay ranged between 8.0° and 13.0°, and for C'_R, between 0.0 p.s.i. and 1.5 p.s.i. Much greater variation was observed for the peak shear strength parameters. For example Φ'_P ranged between 13.5° and 22.0°, and C'_P between 0.4 p.s.i. and 2.5 p.s.i. A large variation in peak strength is to be expected. Skempton (11) has shown that peak strength characteristics of clays depend on the consolidation history, while residual strengths do not depend on this.

The low values of C'_R shown in Table I do lead to a considerable simplification of the analysis, in that C'_R can be taken as equal to zero.

Figure 13 illustrates the difficulty in determining the residual parameters in the laboratory. To obtain shear stresses that were independent of the strain, the sample had to be strained a cumulative total of 1.0 inches, taken in 0.1 inch cycles. Even with this many cycles, it had been found necessary to use a cutting procedure along the shear plane to obtain this independence of residual shear strength with respect to strains. The rates of shear have to be slow enough so that excess pore pressures do not develop during shear. From a practical point of view, this means that for a single direct shear test at one normal pressure, about three weeks are required. This is an inconvenient requirement if several tests are necessary for any investigation.

Analysis The geometry of the simplified three-block system used for the analysis is shown in Figure 2. One of the problems in such an analysis is not knowing how much vertical force is transmitted through the vertical interfaces between blocks. The method of analysis derived in this study does permit consideration of how these interface vertical forces can influence the results. Such forces would redistribute the total weight of the three-block system so that different proportions of the weight would be carried by the individual blocks. (Such a redistribution was made, as shown in Chapter IV). It is obvious that such a redistribution of the weights does affect, to some extent, the value of Φ'_D obtained in this analysis. For example, the various combinations of redistributing ten percent of the weights of the top and bottom blocks shown in Table XX were used in the computer program. The resulting values of Φ'_D deviated from those obtained using the original block weights. These deviations, as shown in Table XX, were a maximum of 1.4° for the St. Vital site and 1.9° for the Tache Avenue site. The maximum percentage deviation, based on tan Φ'_A, was 21% for each site. The assumption of 10% redistribution of total weight may be quite severe. Such a redistribution theoretically should be applied only to intergranular stresses, which can have a vertical component on the block interfaces. Pore pressures, being hydrostatic, would only have a normal component.

Table X summarizes the results of total stress analysis using Taylor's Charts and the simplified slope configuration for both sites. The significance is that for both banks the cohesion required is substantially less than the 500 p.s.f. which Sutherland (15) has recommended for this type of analysis. A required cohesion for impending movement of 297 p.s.f. was obtained for the St. Vital site and that for the Tache Avenue site was 365 p.s.f.

<u>Correlation</u> The instability during winter conditions is reflected in Figure 10. Movement is shown to commence when the river level is lowered in the fall, and continues for the entire winter at a gradually decreasing rate. When the spring thaw begins in mid-March, increasing movements are again observed. In conjunction with Figure 9 , showing river levels, it will be noted that this increased movement preceeds by four or five weeks the rise of river level in the spring. Similar movements of lesser magnitude are also shown for the St. Vital site.

Figures 9 and 10 show that during the summer months there were periods in which the banks were in equilibrium. Very slight movements may be attributed to precipitation, which is an aspect of stability meriting further consideration.

The periods of equilibrium were correlated to the values of

Φ'_D to obtain a field evaluation of the residual angle of internal friction. On the Tache Avenue site, critical values of Φ'_A of 10.8° and 12.7° were obtained, considering two possible configurations of the slip line. For the St. Vital site a Φ'_A value of 8.1° was obtained. The values of Φ'_A so obtained depend somewhat on the distribution of weights of the blocks.

II CONCLUSIONS

The following conclusions result from the discussion of the results.

1. Many of the unstable river banks in the Winnipeg area are located on the concave sides of river bank curves.
2. Unstable river banks in the Winnipeg area can have slope angles as low as 6° from the horizontal.
3. Seepage of surface water into cracks at the top of a bank can have the effect of raising the pore pressures within a failure zone, thereby promoting slope movements.
4. The seepage in the banks studied was generally in the downward direction.
5. Pore pressures in the banks in winter are not readily relieved as the river is lowered, thereby favoring instability.
6. Zones of high pore pressure sometimes remain near the toe of a bank as spring drawdown occurs.
7. Considerable interpolation and extrapolation is necessary to estimate the configuration of the slip lines when only two

slope indicators are used.

8. The residual strength parameters for Winnipeg clay appear to fall within a relatively narrow range.

9. Residual cohesion can be taken as zero.

10. The simplified analysis developed in this study allows a correlation of the piezometer and river level data with observed slope movements.

11. Critical values of frictional strength, as calculated for periods of impending motion using the simplified analysis, fall within the range of residual strength perameters determined in the laboratory.

12. The two banks observed were in equilibrium during the summer months.

13. Slope movements are largest immediately after the fall drawdown takes place.

14. An appreciable increase in slope movement occurs when the spring thaw begins in mid-March.

III RECOMMENDATIONS FOR FURTHER STUDY

Tables XI to XIX have been included to present all the data obtained for the slope indicators and piezometers and river level observations. The slope indicator and piezometer readings are considered valuable data for those who may wish to make any other or similar analyses.

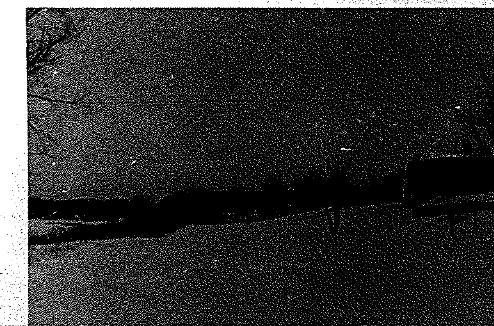
An attempt should be made to evaluate various remedial measures on both sites, by means of the computer program, using the data tabulated in this thesis.

Remedial adjustments in the data could take the form of increasing W_3 when loading of the toe is being considered or decreasing W_1 and W_2 when trimming of the slope to a smaller angle is being considered. The effect of reducing pore water forces through drainage could be simulated by reducing these values in the input accordingly. This approach would facilitate the evaluation of various remedial works regarding costs. The most economically sound method or combination of methods could then be selected and implemented. The effectiveness of these remedial works could be checked by suitable instrumentation.

More direct shear tests should be performed on Winnipeg clays. The values of the residual shear strength parameters could then be checked to find out whether the range is as limited as the present investigation seems to show. If the variation is not large then a reasonably conservative value of Φ'_R could be chosen which could be used for all clay river banks in the Metropolitan Winnipeg area.

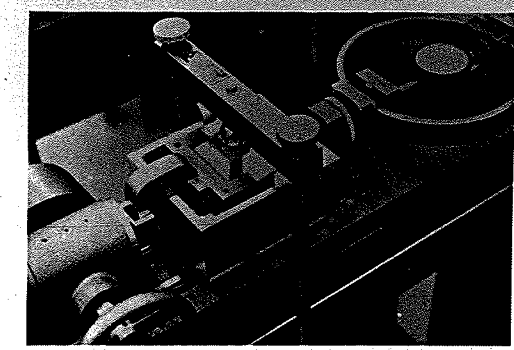
The computer program presented in this thesis should be modified to a higher degree of sophistication so that a trial and error approach could be used for the analysis of intact slopes. By means of iterative convergence, the most critical sliding block system could then be determined, and the slope stability of intact slopes could be analyzed.

Another area of study meriting further work is the correlation of slope movements to precipitation. An explanation may be found for the variation in pore pressures during the summer months, and slope movements resulting from them, in such a study.

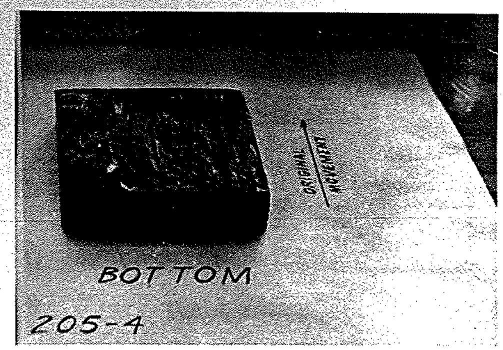


PHOTOGRAPH 1
RIVER BANK AT ST. VITAL SITE

PHOTOGRAPH 2
RIVER BANK AT TACHE AVENUE SITE



PHOTOGRAPH 3
DIRECT SHEAR TEST APPARATUS



PHOTOGRAPH 4
CLAY SPECIMEN WITH IRREGULAR SHEAR PLANE

LOCATION OF INSTRUMENTATION SITES AND SOME RIVER BANK SLIDE AREAS IN METROPOLITAN WINNIPEG

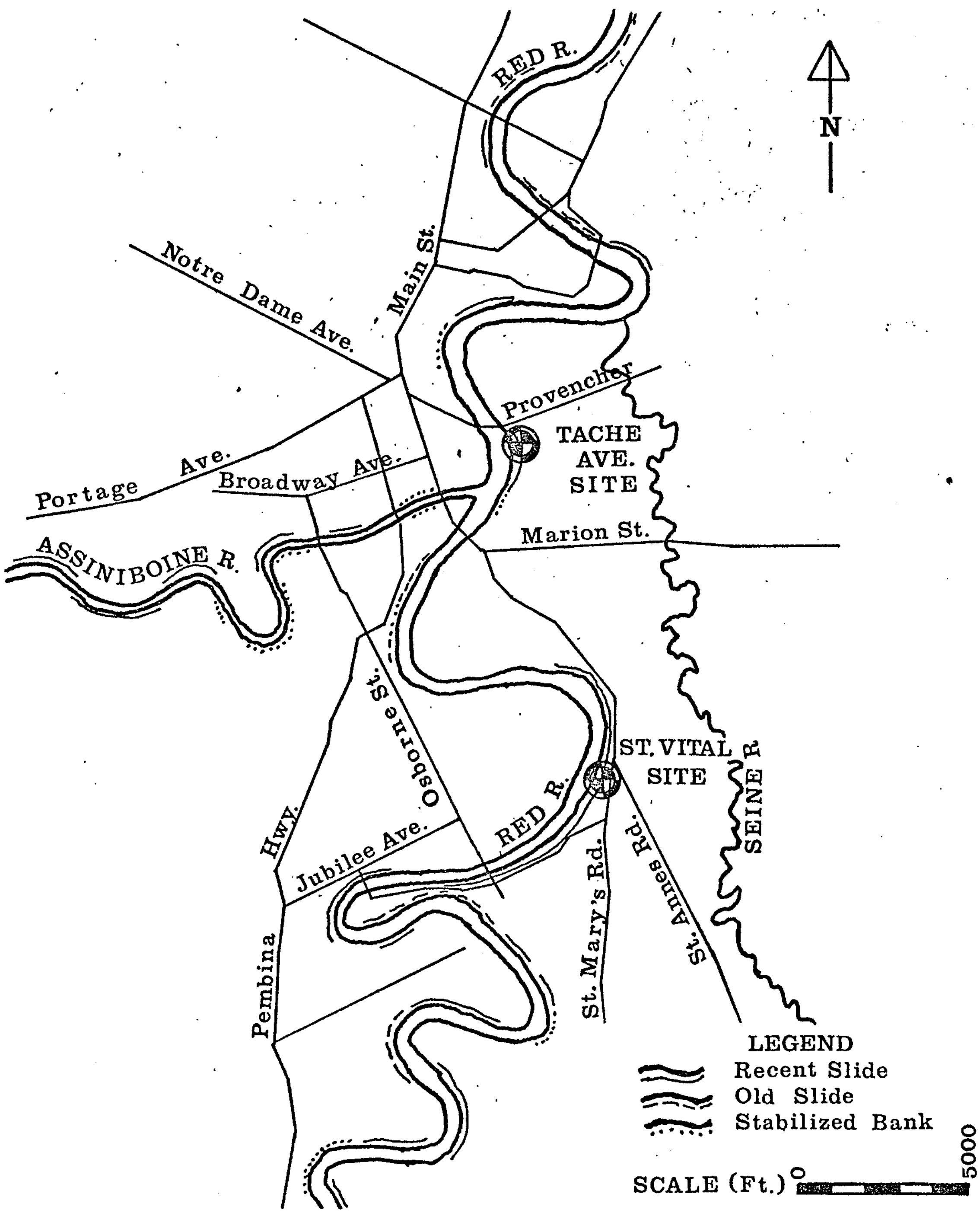
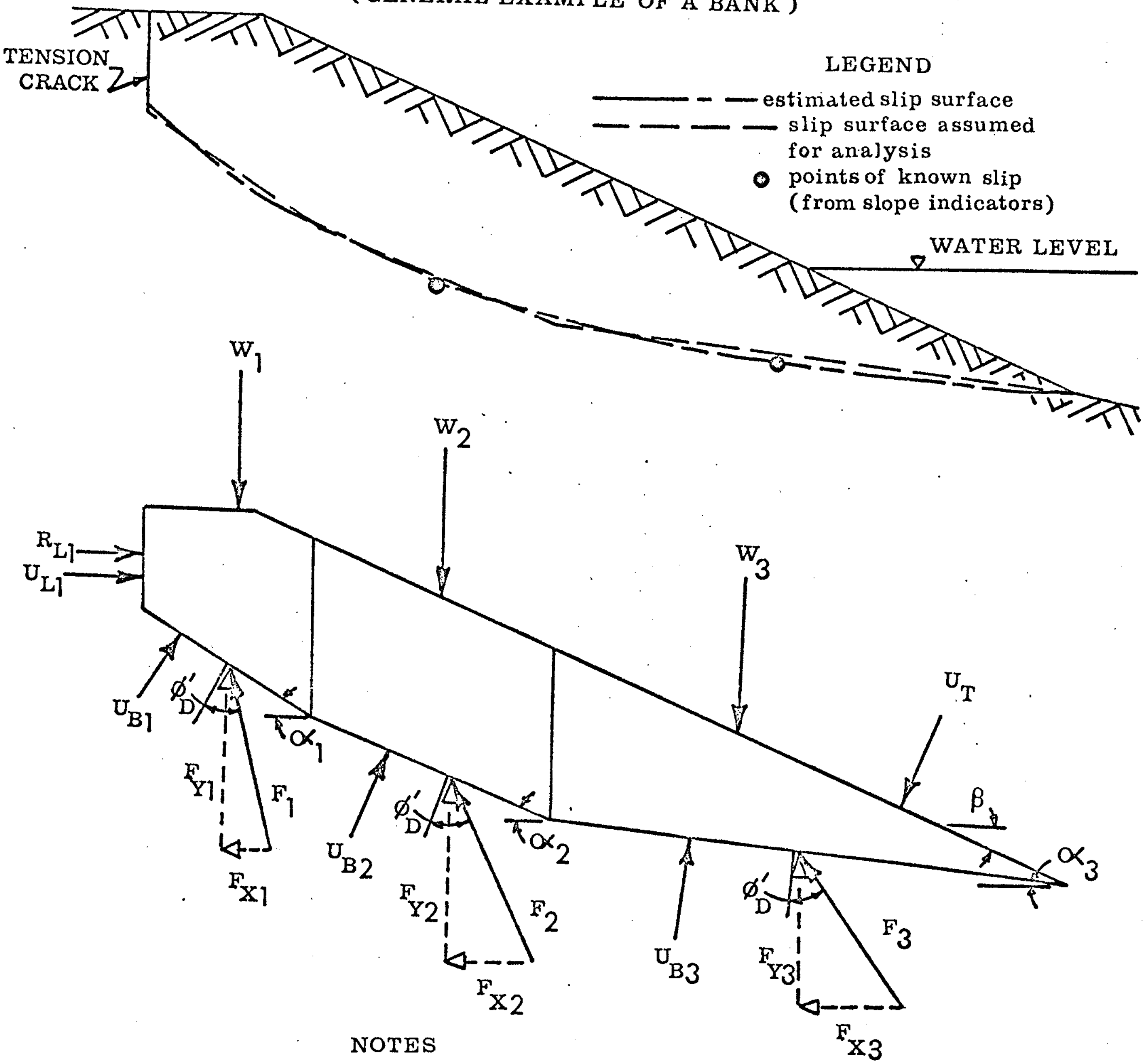


FIGURE 1

SLIP SURFACE ASSUMED AND FORCE VECTORS USED

IN THE SLIDING BLOCK SLOPE STABILITY ANALYSIS

(GENERAL EXAMPLE OF A BANK)



NOTES

1. R_{L1} = Effective Horizontal Soil Reaction at the Tension Crack
2. U = Hydrostatic or Porewater Force on Each Indicated Surface
3. W = Total Weight of Each Block
4. F = Effective Soil Reaction at the Base of Each Block
5. ϕ'_D = Effective Soil Friction Angle Required for Static Equilibrium

FIGURE 2

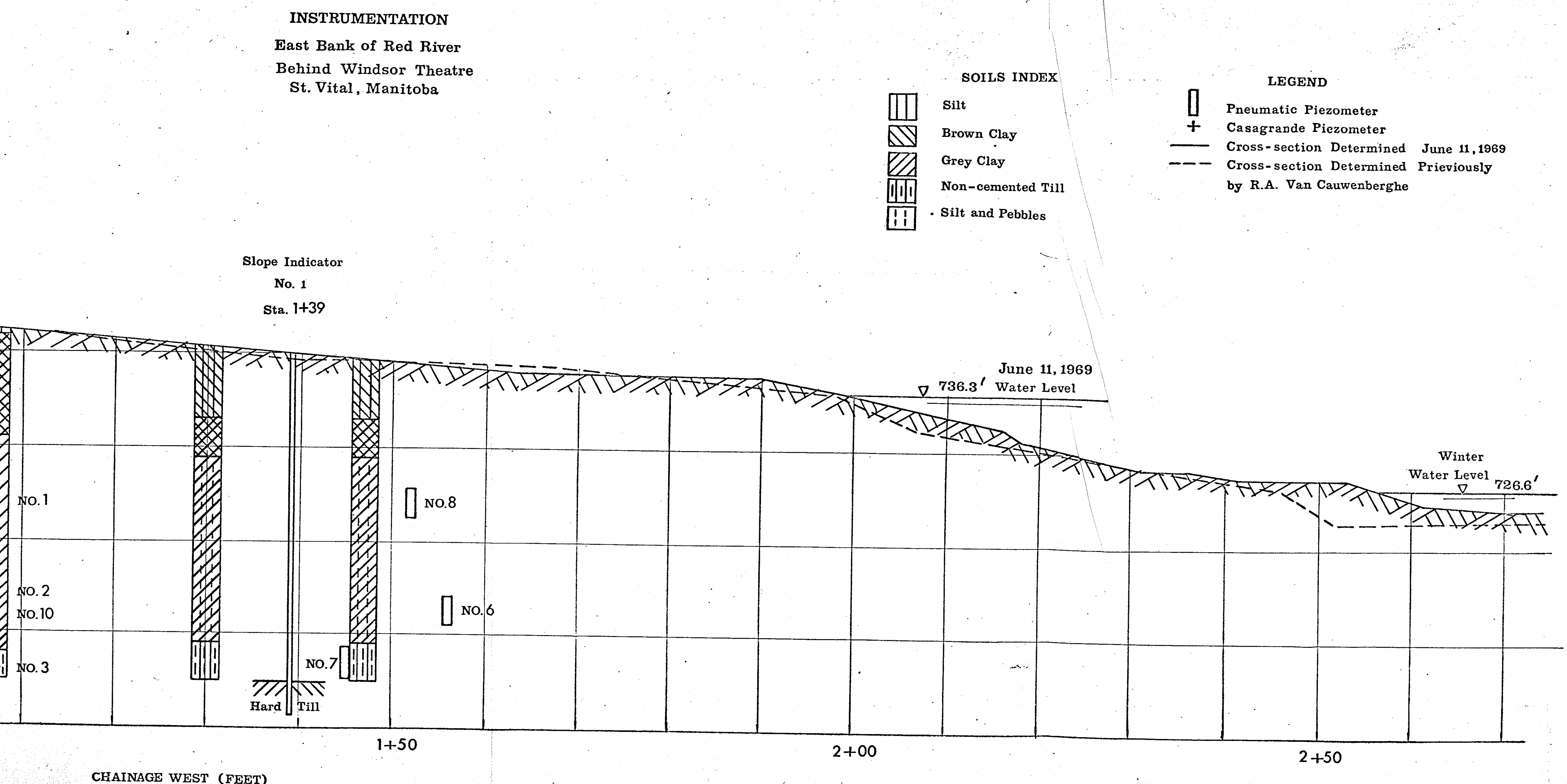
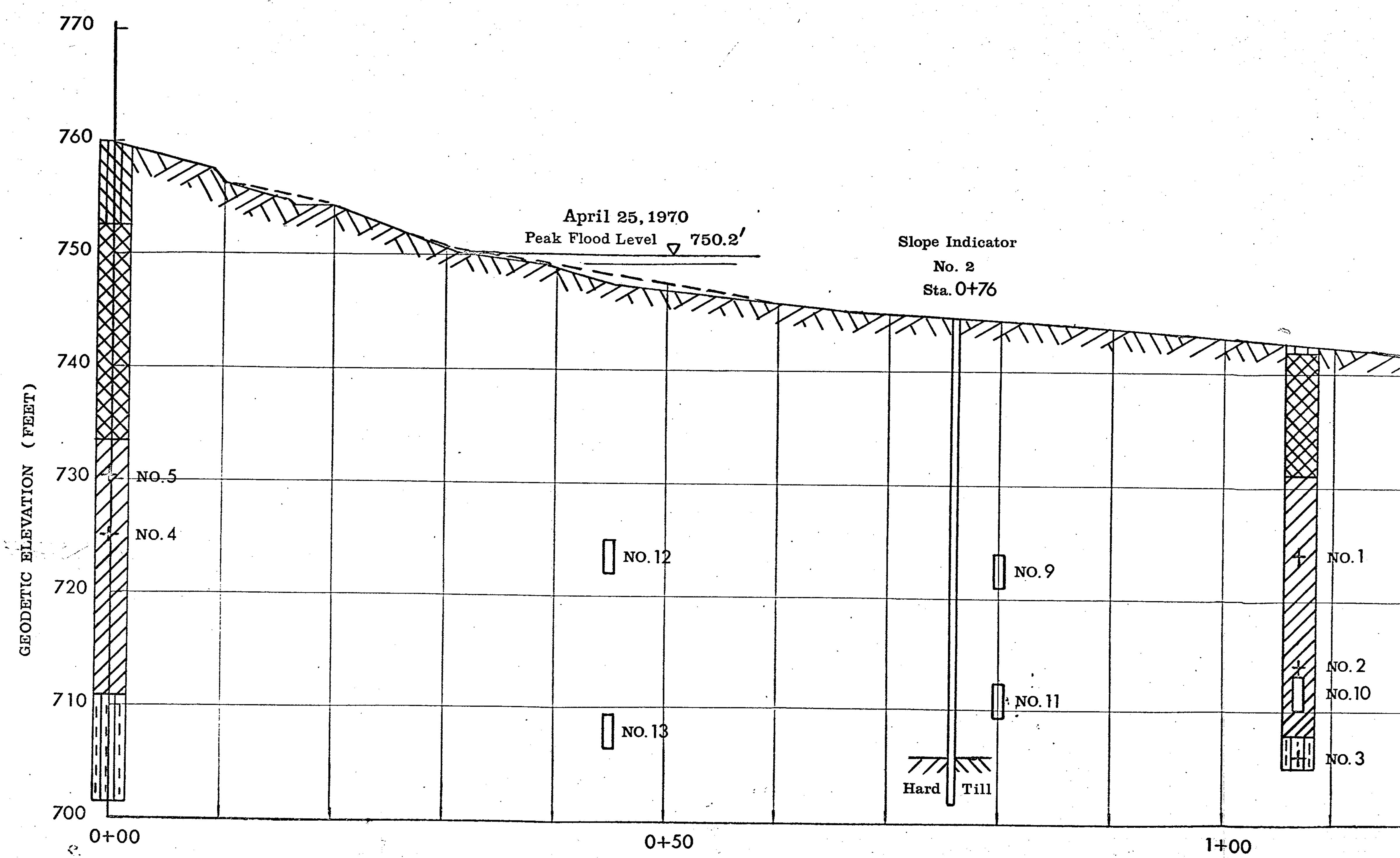
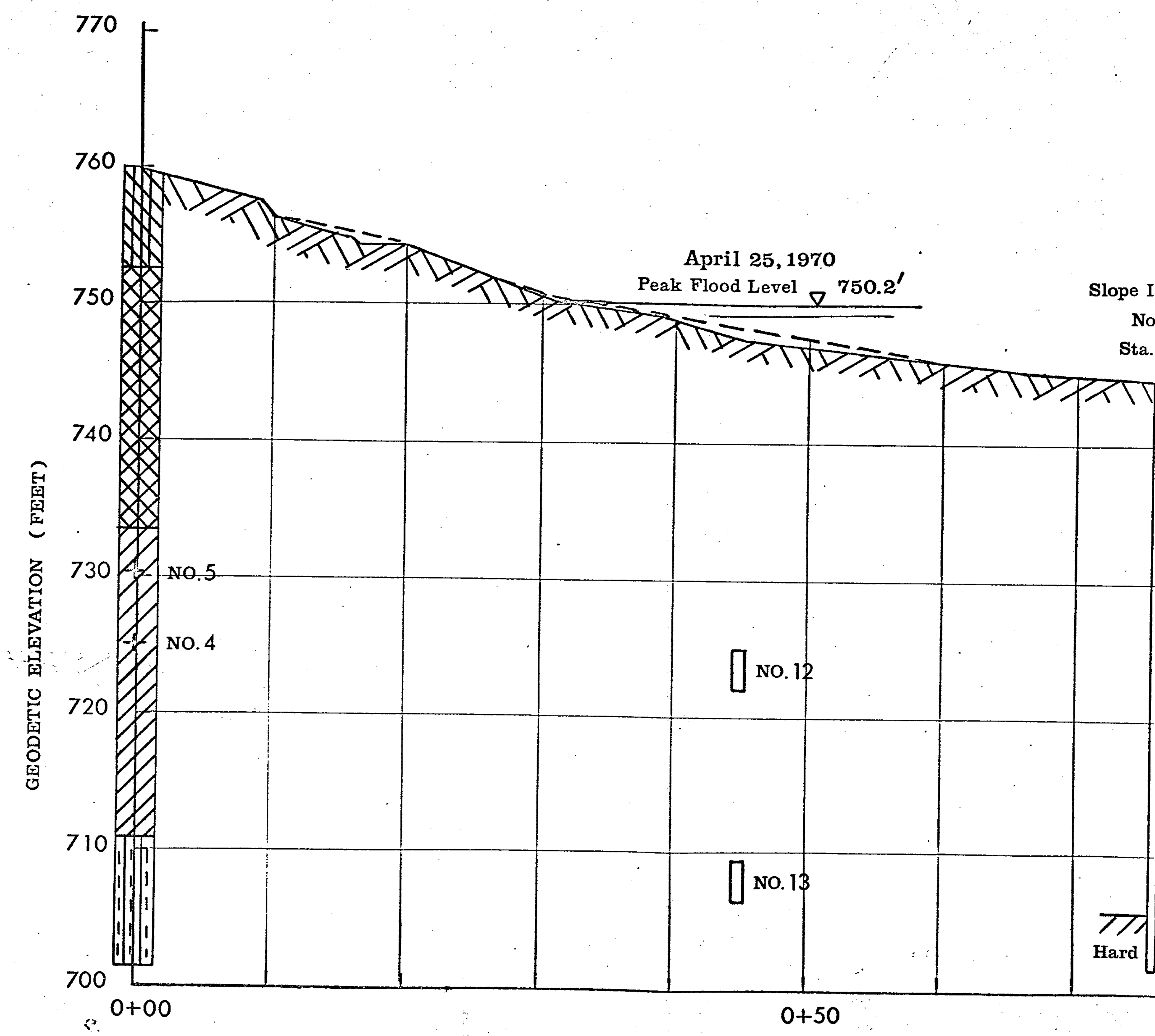

INSTRUMENTATION
East Bank of Red River
Behind Windsor Theatre
St. Vital, Manitoba
SOILS INDEX
Silt
Brown Clay
Grey Clay
Non-cemented Till
Silt and Pebbles
LEGEND
Pneumatic Piezometer
Casagrande Piezometer
Cross-section Determined June 11, 1969
Cross-section Determined Prieviously by R.A. Van Cauwenberghe
Slope Indicator
No. 1
Sta. 1+39
June 11, 1969
736.3' Water Level
Winter
Water Level
726.6'
NO. 1
NO. 8
NO. 2
NO. 10
NO. 6
NO. 3
NO. 7
Hard Till
1+50
2+00
2+50
CHAINAGE WEST (FEET)

FIGURE 3



770
760
750
740
730
720
710
700
GEODETIC ELEVATION (FEET)
April 25, 1970
Peak Flood Level 750.2′
Slope Indicator
No. 2
Sta. 0+76
NO. 5
NO. 4
NO. 12
NO. 13
NO. 9
NO. 11
Hard Till
NO. 1
NO. 2
NO. 10
NO. 3
0+00
0+50
1+00



770
760
750
740
730
720
710
700
GEODETIC ELEVATION (FEET)
April 25, 1970
Peak Flood Level 750.2′
NO. 5
NO. 4
NO. 12
NO. 13
Hard
0+00
0+50

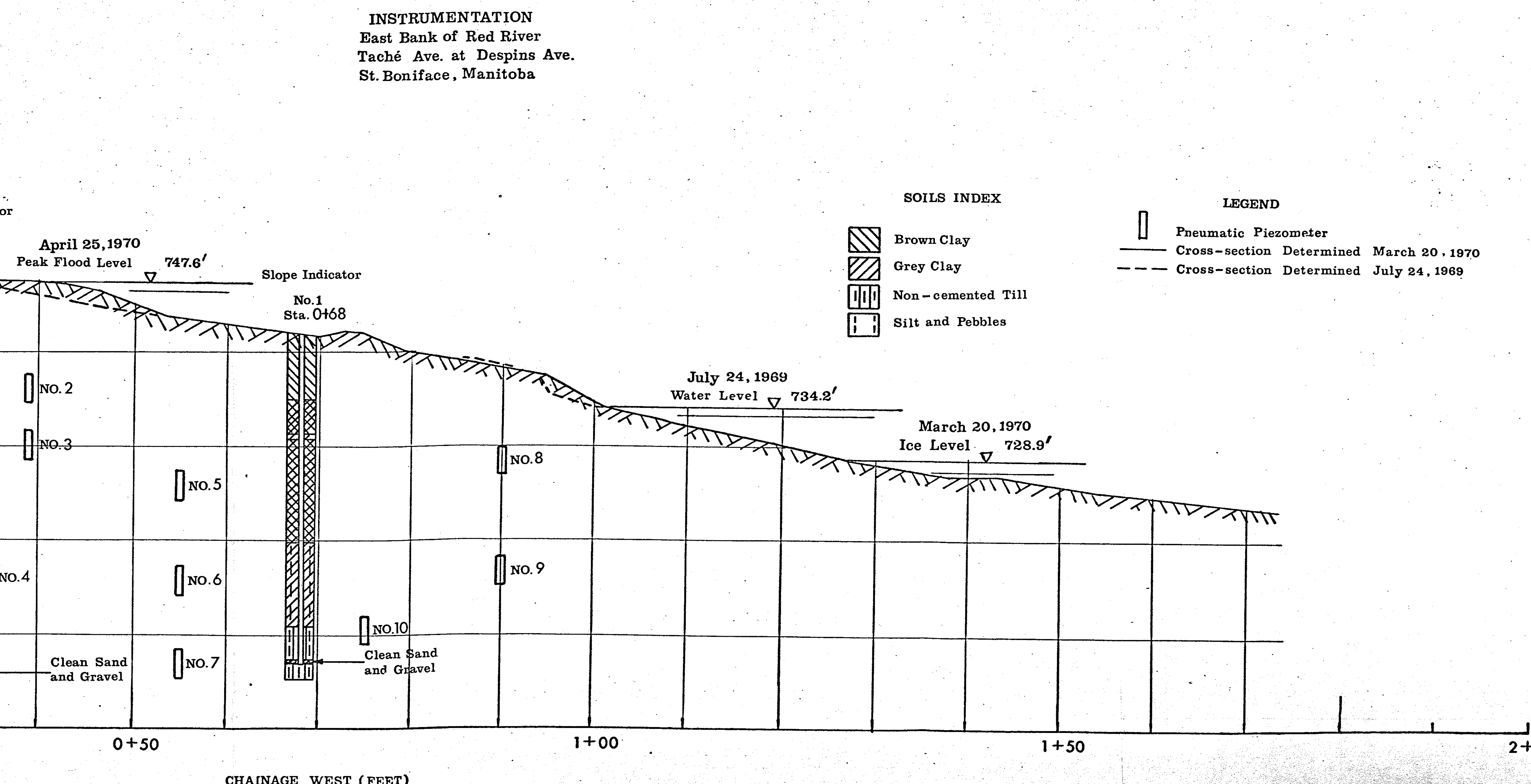

INSTRUMENTATION
East Bank of Red River
Taché Ave. at Despins Ave.
St. Boniface, Manitoba
SOILS INDEX
Brown Clay
Grey Clay
Non-cemented Till
Silt and Pebbles
LEGEND
Pneumatic Piezometer
Cross-section Determined March 20, 1970
Cross-section Determined July 24, 1969
or
April 25, 1970
Peak Flood Level 747.6′
Slope Indicator
No. 1
Sta. 0+68
July 24, 1969
Water Level 734.2′
March 20, 1970
Ice Level 728.9′
NO. 2
NO. 3
NO. 4
NO. 5
NO. 6
NO. 7
NO. 8
NO. 9
NO. 10
Clean Sand and Gravel
Clean Sand and Gravel
0+50
1+00
1+50
2+00
CHAINAGE WEST (FEET)

FIGURE 4

INSTRUMENTATION
East Bank of Red Riv
Taché Ave. at Despin
St. Boniface, Manitob

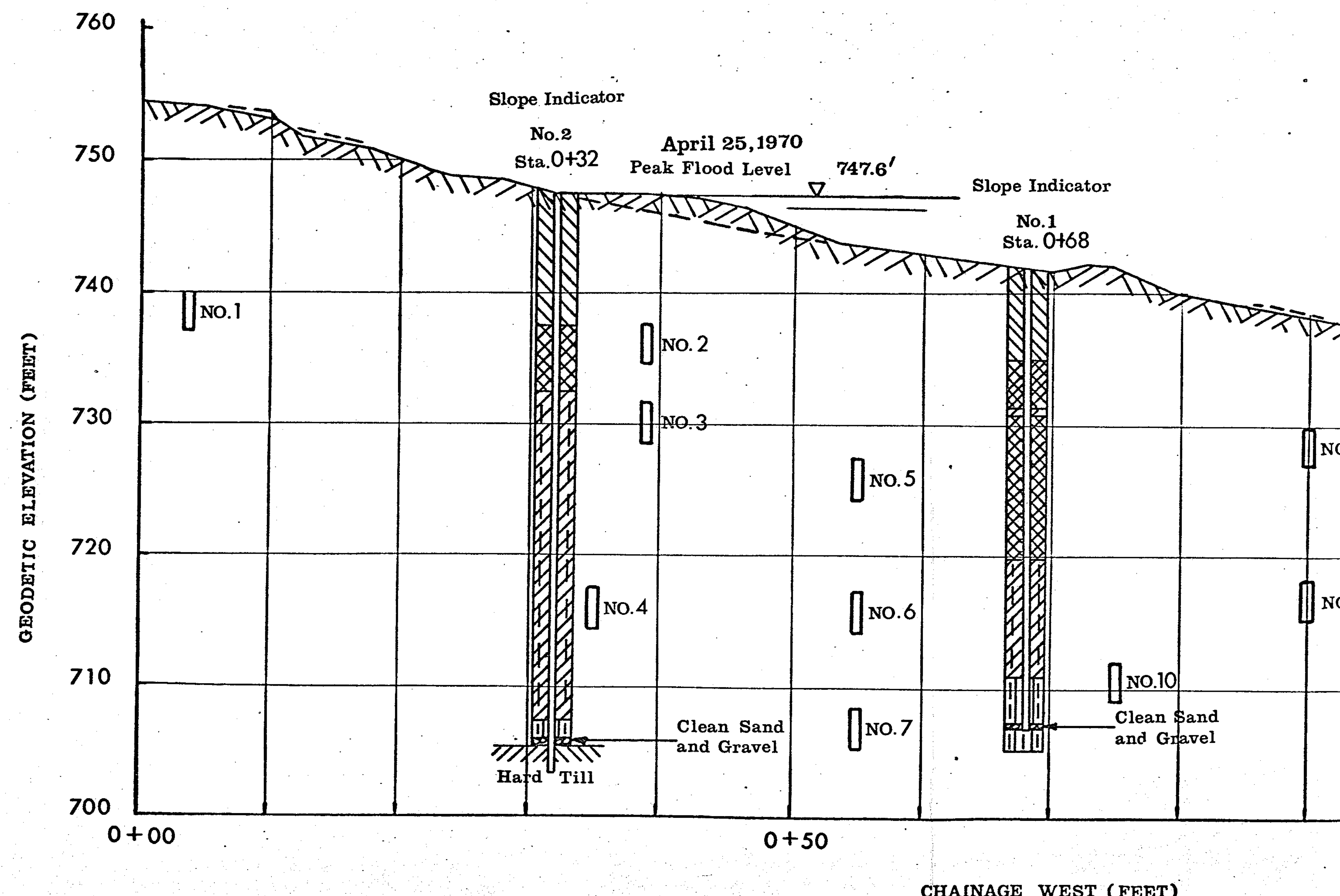


FIGURE 4

EQUIPOTENTIAL LINES
WINTER CONDITION; DECEMBER 20, 1969
ST. VITAL SITE

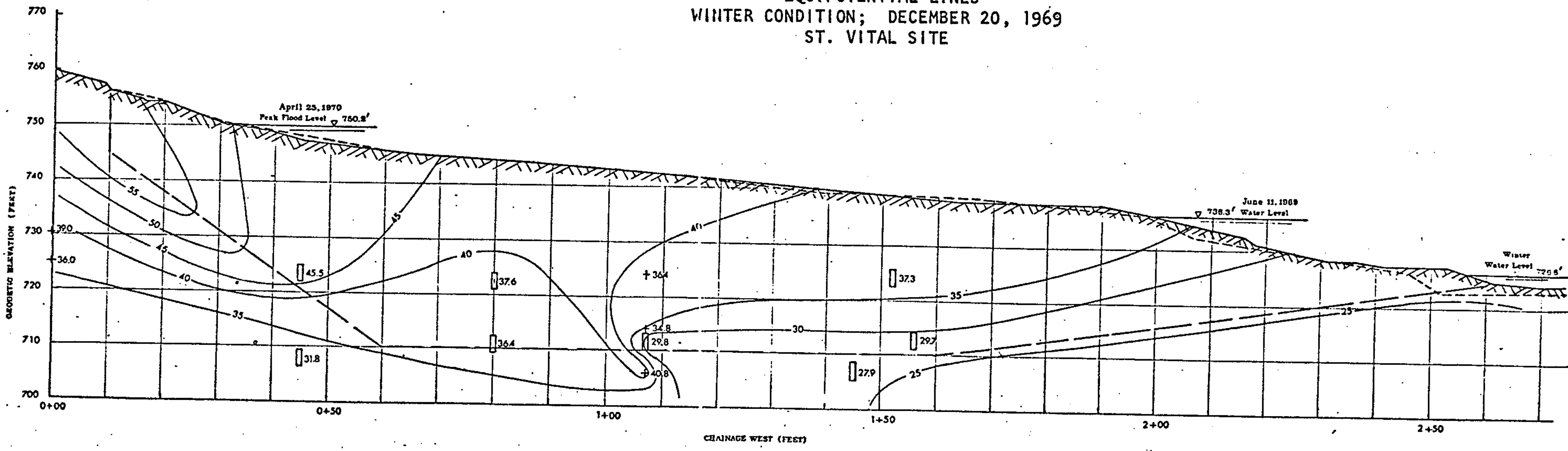


FIGURE 5.1

EQUIPOTENTIAL LINES
SPRING INUNDATION; APRIL 25, 1970
ST. VITAL SITE

LEGEND

Slip Line Assumed for Analysis
Equipotential Lines

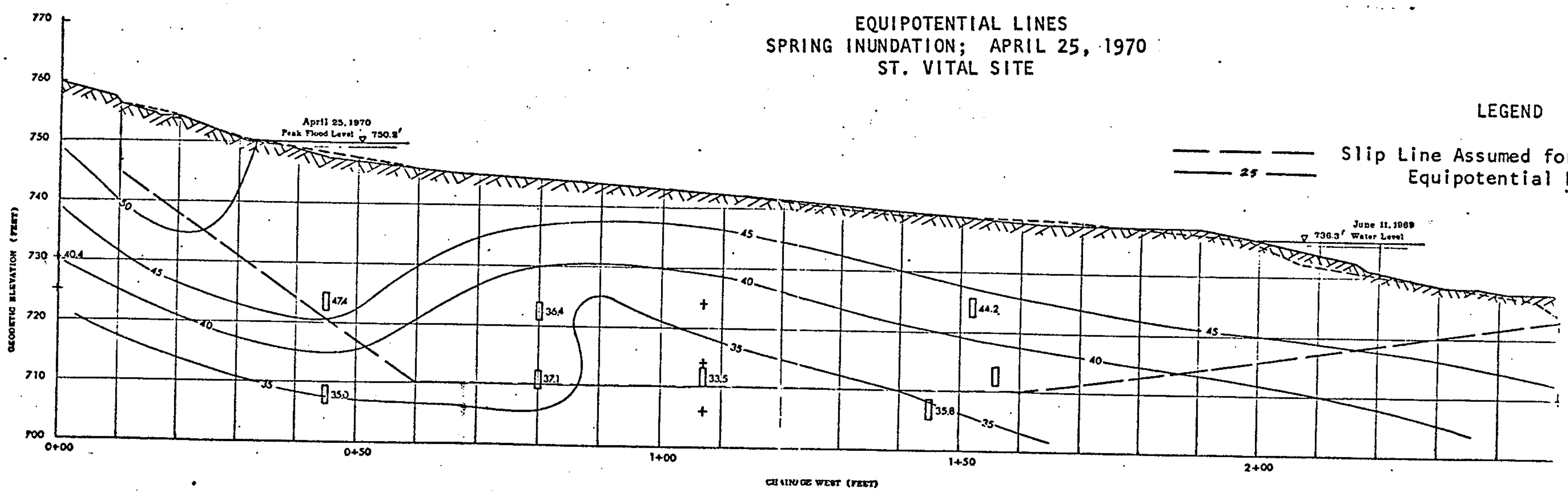


FIGURE 5.2

EQUIPOTENTIAL LINES
SPRING DRAWDOWN; MAY 22, 1970
ST. VITAL SITE

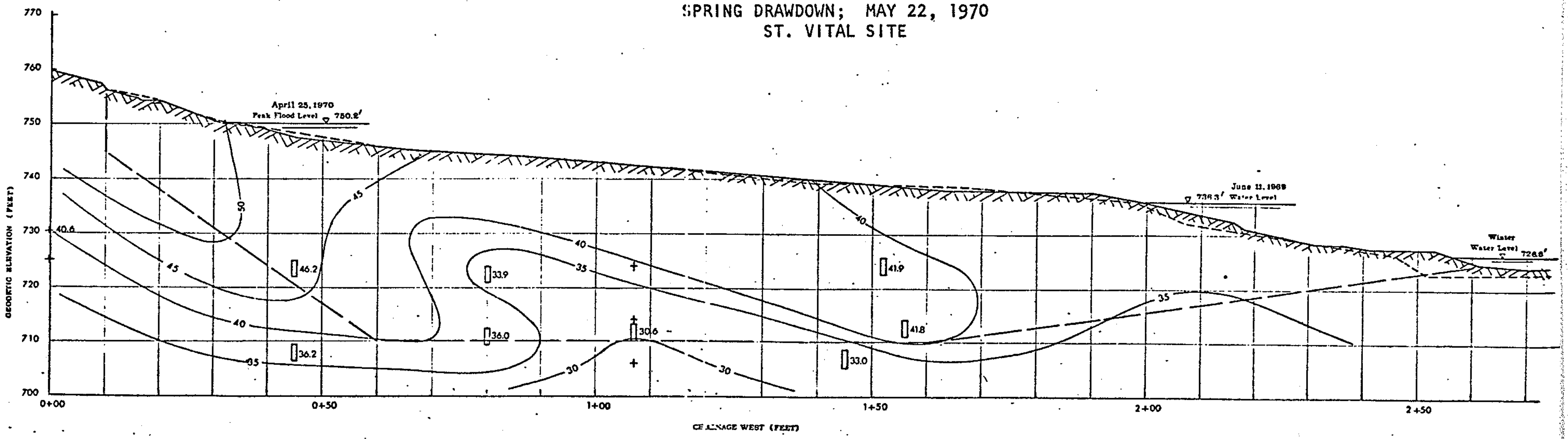


FIGURE 5.3

EQUIPOTENTIAL LINES
SUMMER CONDITION; AUGUST 7, 1970
ST. VITAL SITE

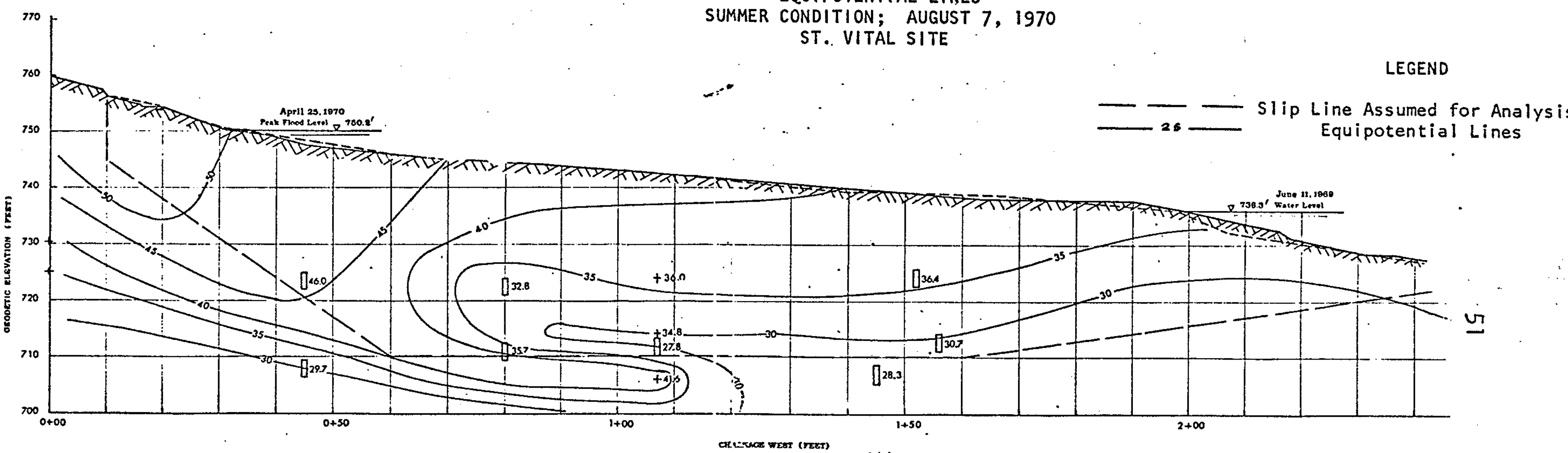


FIGURE 5.4

EQUIPOTENTIAL LINES
WINTER CONDITION; NOVEMBER 7, 1970
TACHE AVENUE SITE

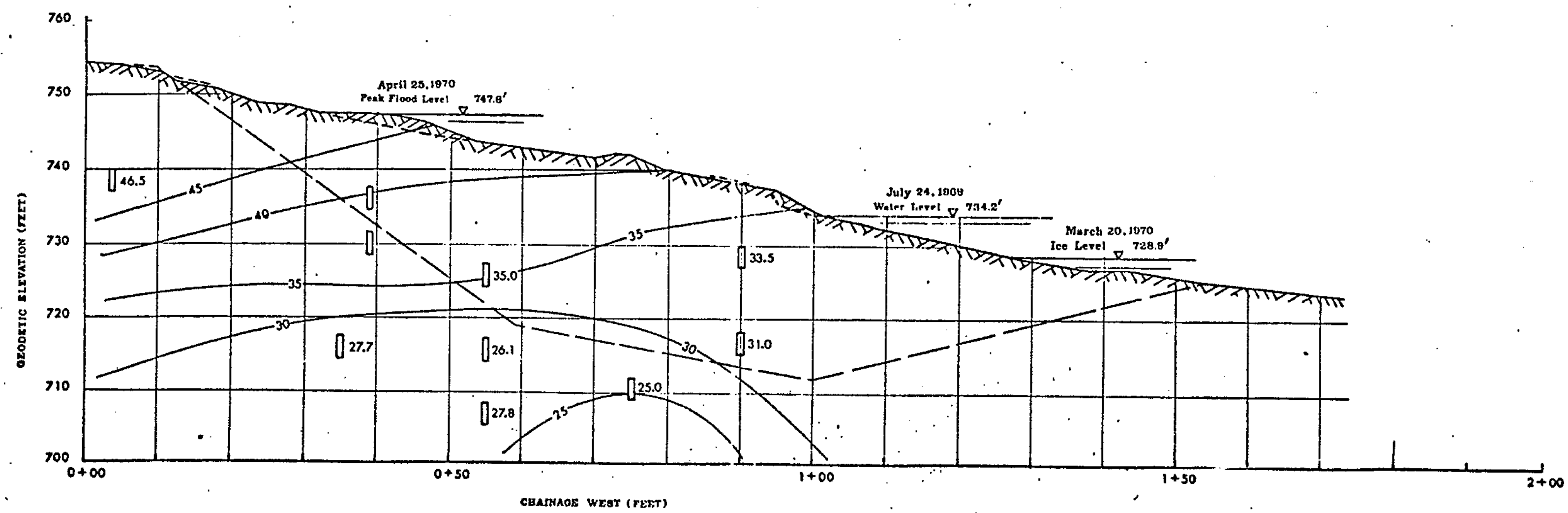


FIGURE 6.1

EQUIPOTENTIAL LINES
SPRING INUNDATION; APRIL 25, 1970
TACHE AVENUE SITE

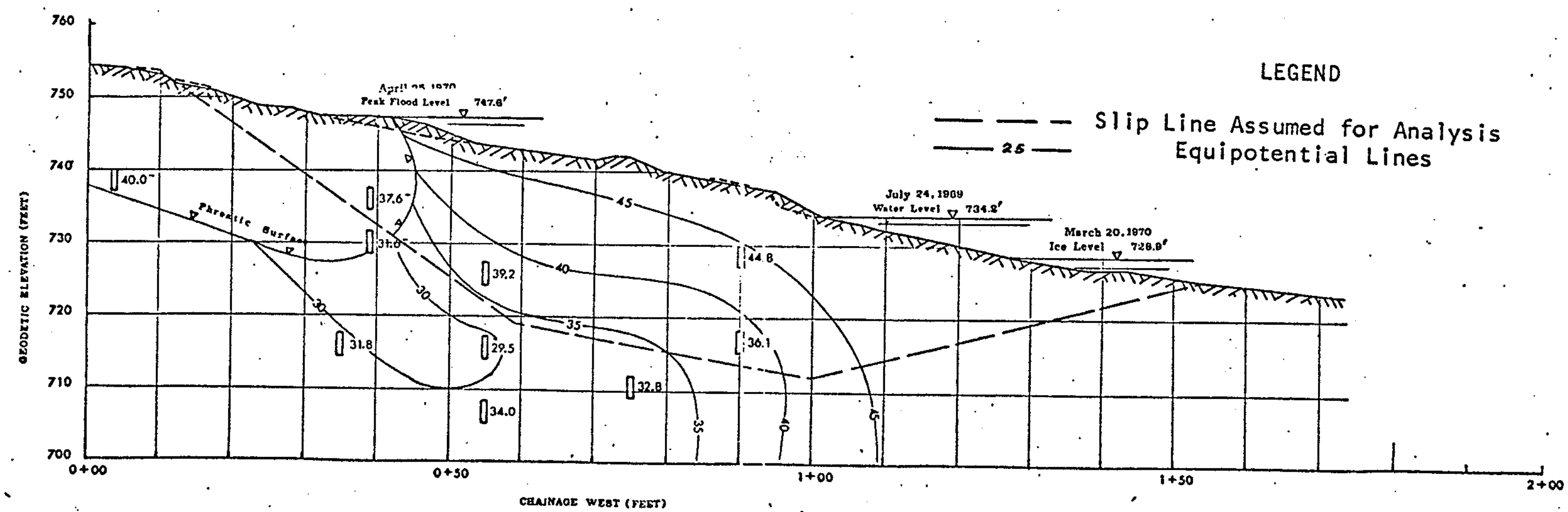


FIGURE 6.2

EQUIPOTENTIAL LINES
SPRING DRAWDOWN; MAY 22, 1970
TACHE AVENUE SITE

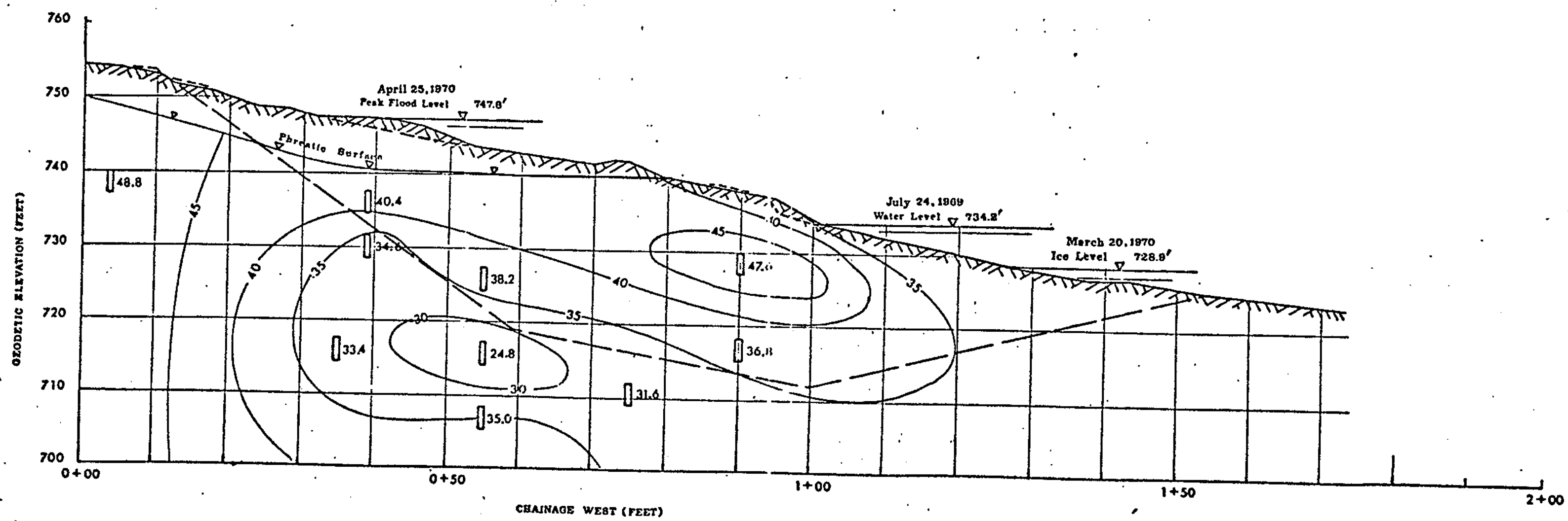


FIGURE 6.3

SUMMER CONDITION; AUGUST 17, 1970
TACHE AVENUE SITE

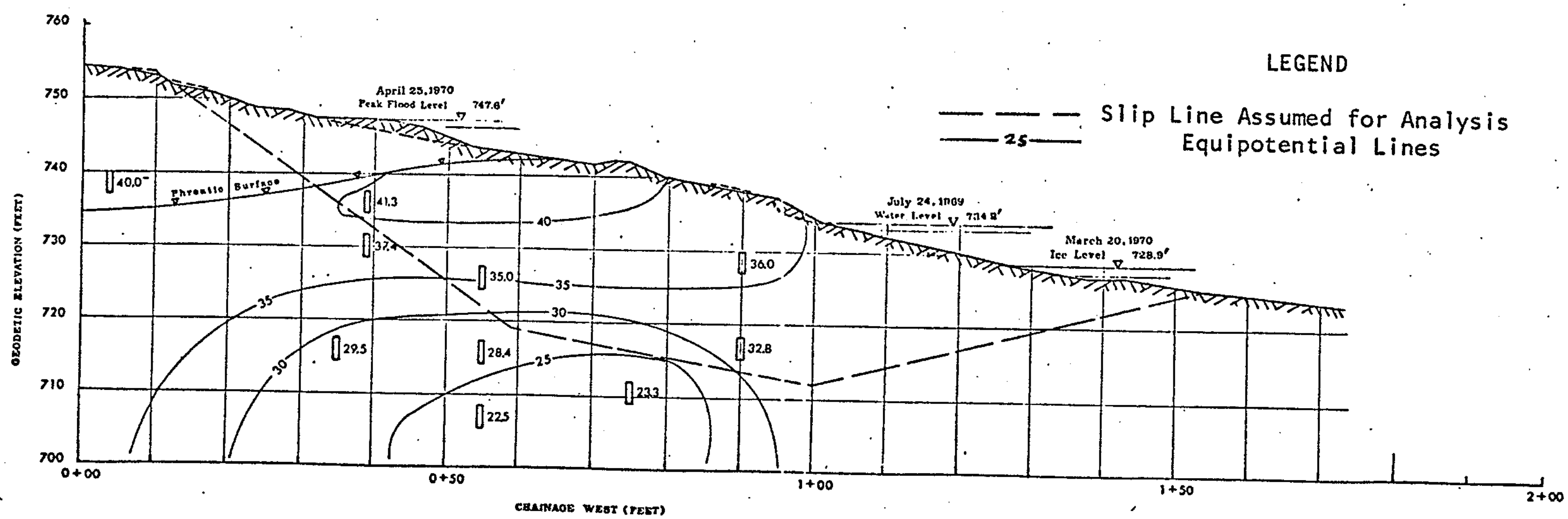


FIGURE 6.4

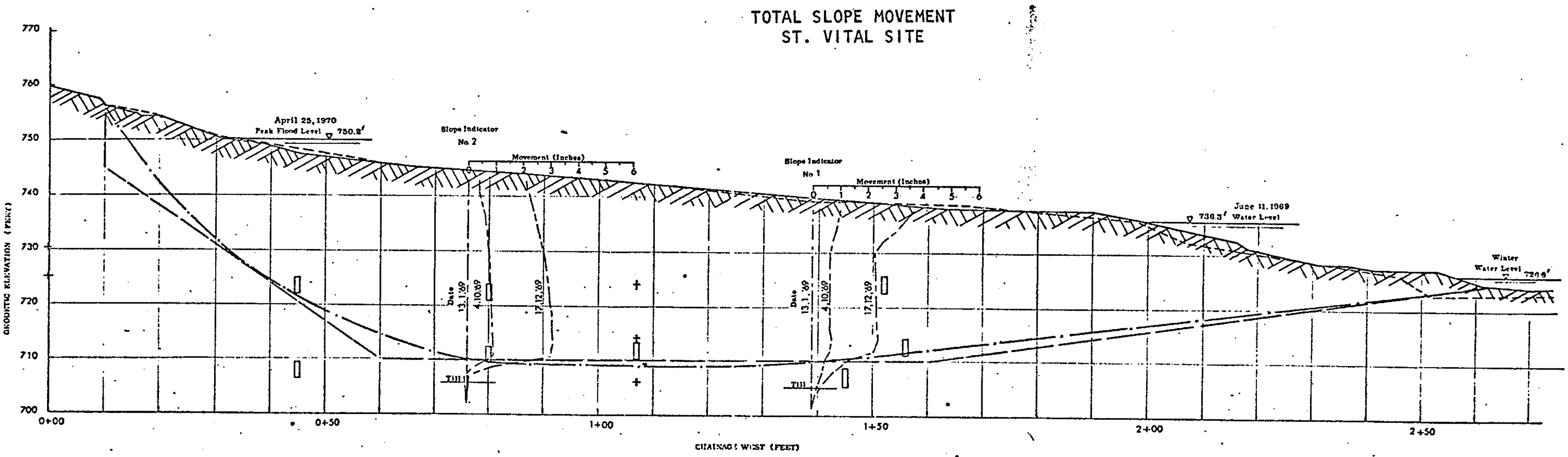


FIGURE 7

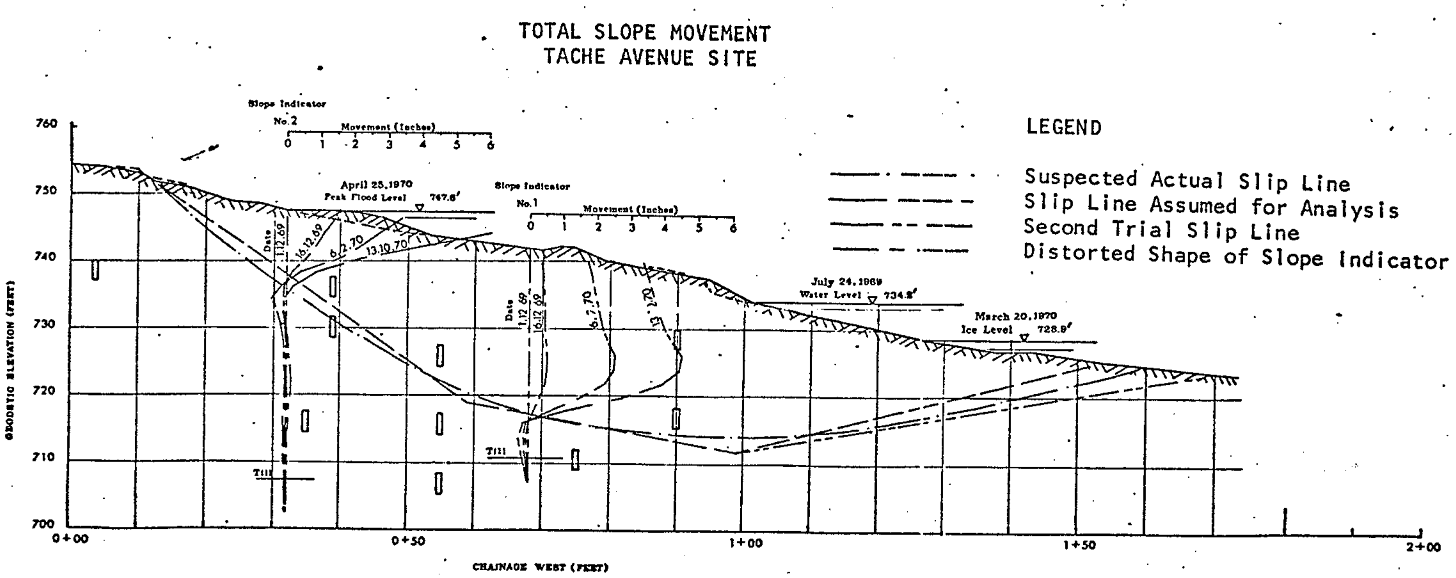
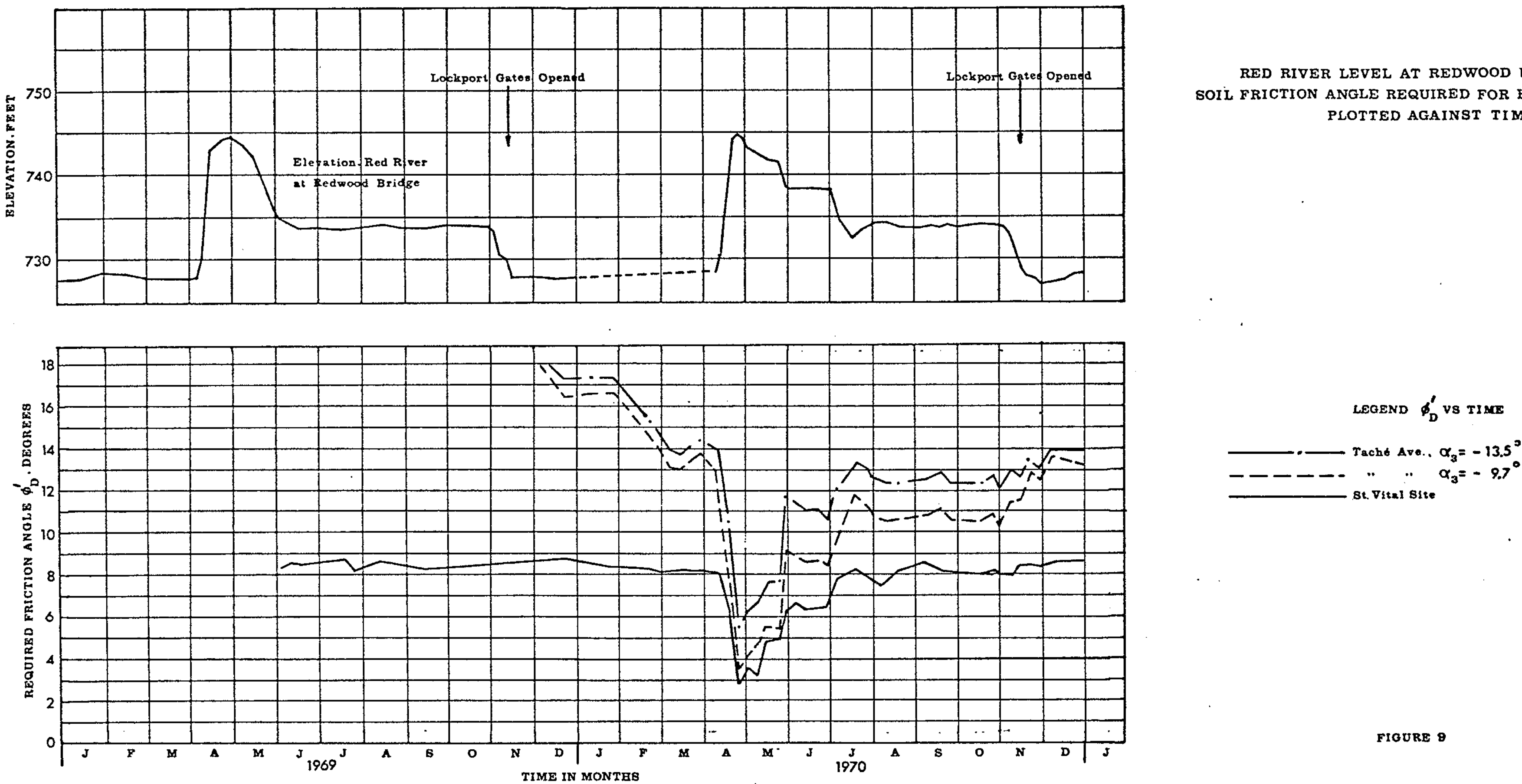
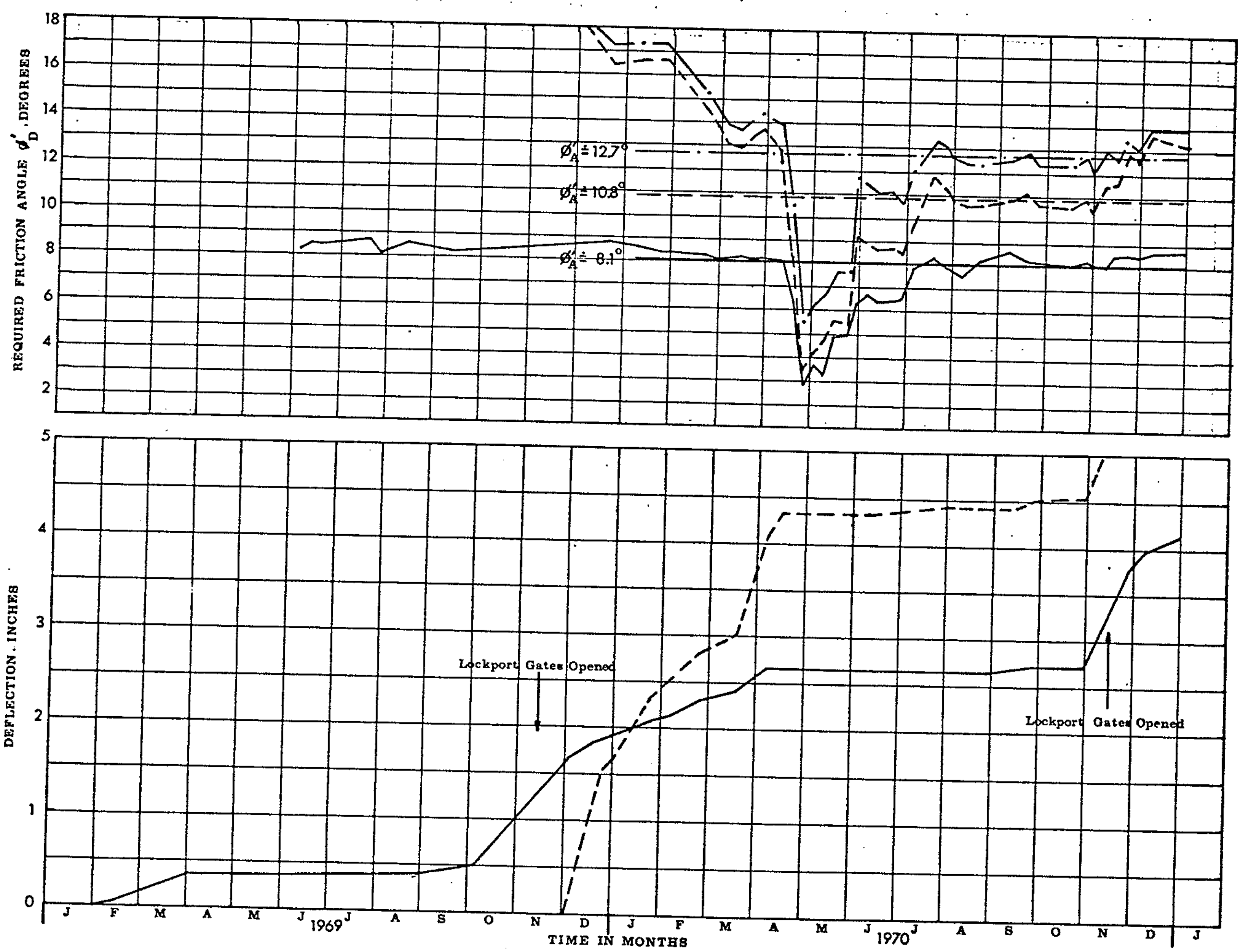


FIGURE 8



RED RIVER LEVEL AT REDWOOD BRIDGE AND SOIL FRICTION ANGLE REQUIRED FOR BANK STABILITY PLOTTED AGAINST TIME

FIGURE 9



SOIL FRICTION ANGLE REQUIRED FOR BANK STABILITY AND SLOPE HOLE MOVEMENT PLOTTED AGAINST TIME

LEGEND. ϕ'_D VS TIME

- · - · - Taché Ave.. $\alpha_3 = -13.5°$
- – – – " " $\alpha_3 = -9.7°$
- ——— St. Vital Site

NOTE

1. ϕ'_A Is the Estimated Available Soil Friction Angle.
2. ϕ'_D Is the Soil Friction Angle Required for Bank Stability, As Obtained from the Stability Analysis.
3. Slope Movement Impends When ϕ'_A and ϕ'_D Are Equal.
4. Banks Are Stable When ϕ'_D Is Less Than ϕ_A.

LEGEND. MOVEMENT VS TIME

- – – – Westward Deflection at the 16.3 ft. Depth; Taché Ave. Slope Indicator No. 1
- ——— Southward Deflection at the 26.0 ft. Depth; St. Vital Slope Indicator No. 1

FIGURE 10

LOCATION OF SLOPE INDICATORS

AT THE ST. VITAL SITE

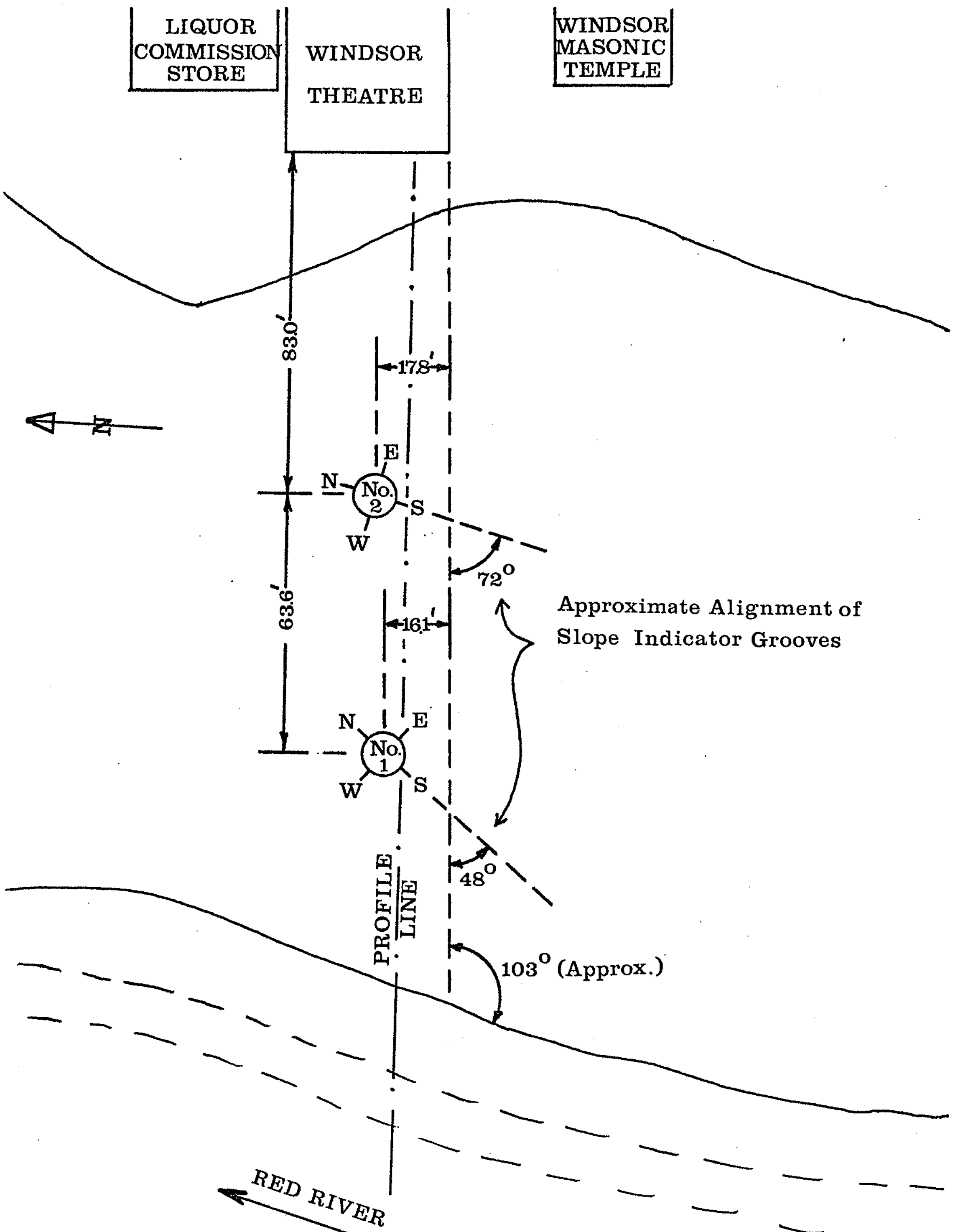


FIGURE 11

LOCATION OF SLOPE INDICATORS

AT THE TACHE AVE. SITE

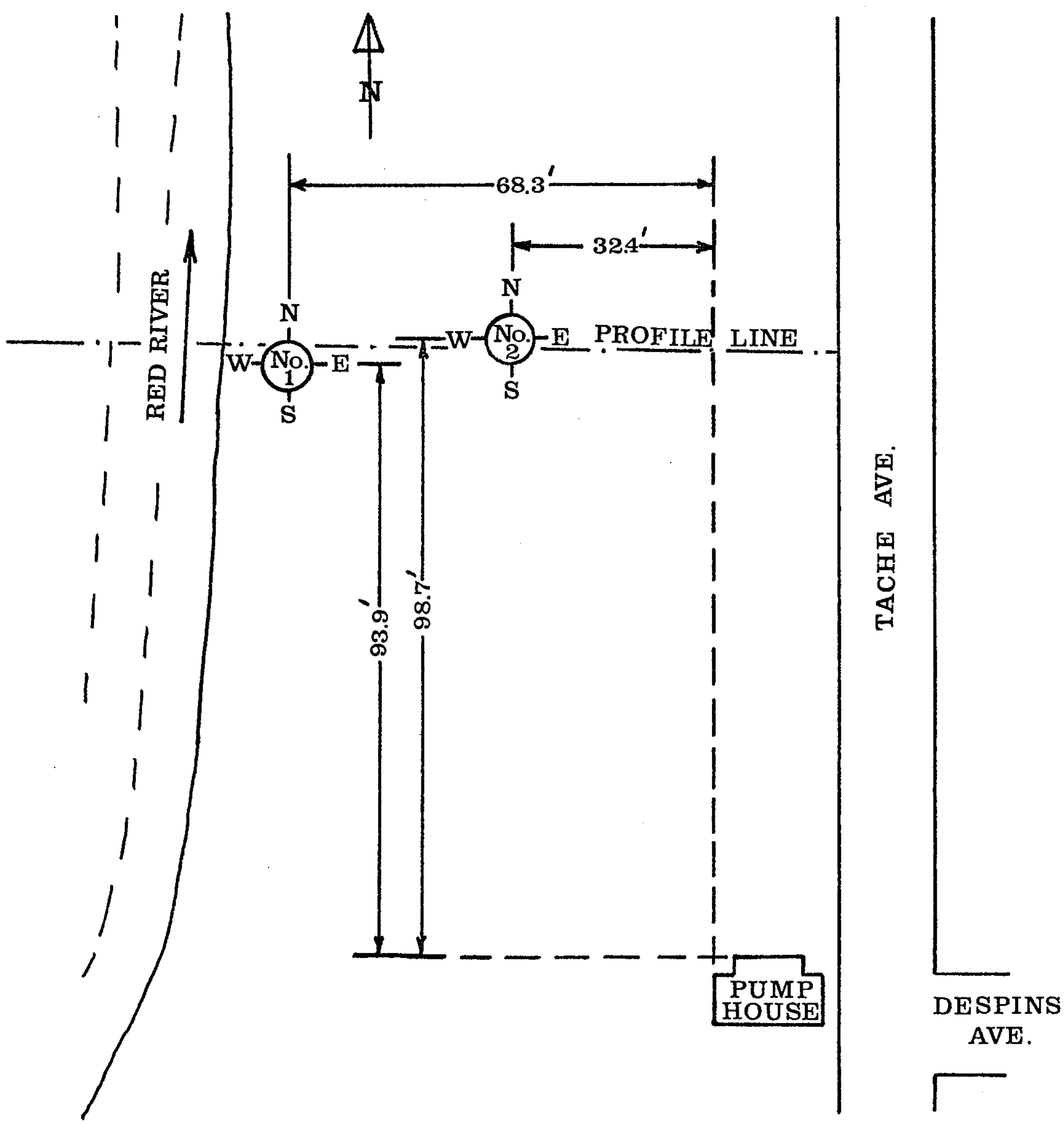
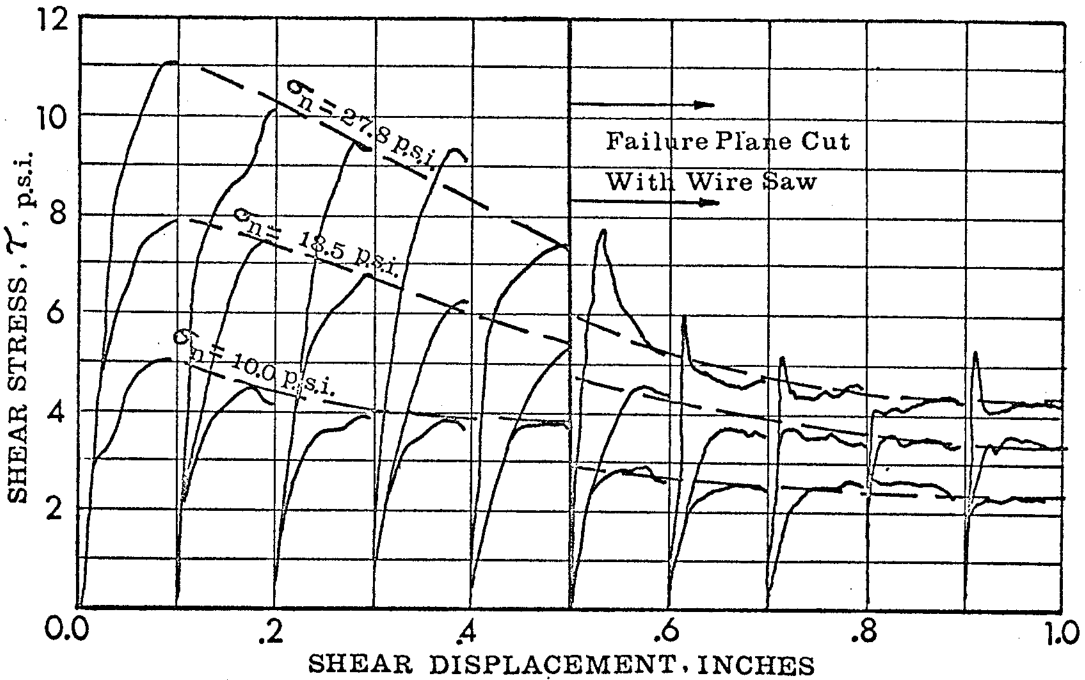


FIGURE 12

DIRECT SHEAR TEST RESULTS

SAMPLE TESTED: Highly Plastic Mottled Brown Clay at the 16 Foot Depth, Slope Indicator Hole No. 1, Taché Ave. Site



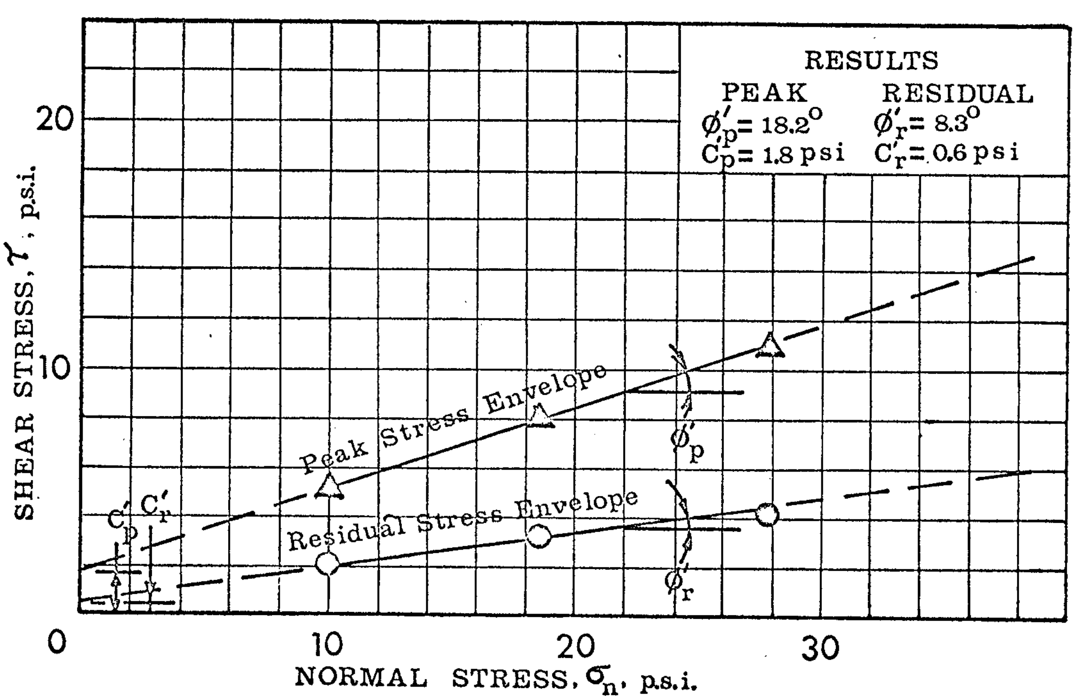


FIGURE 13

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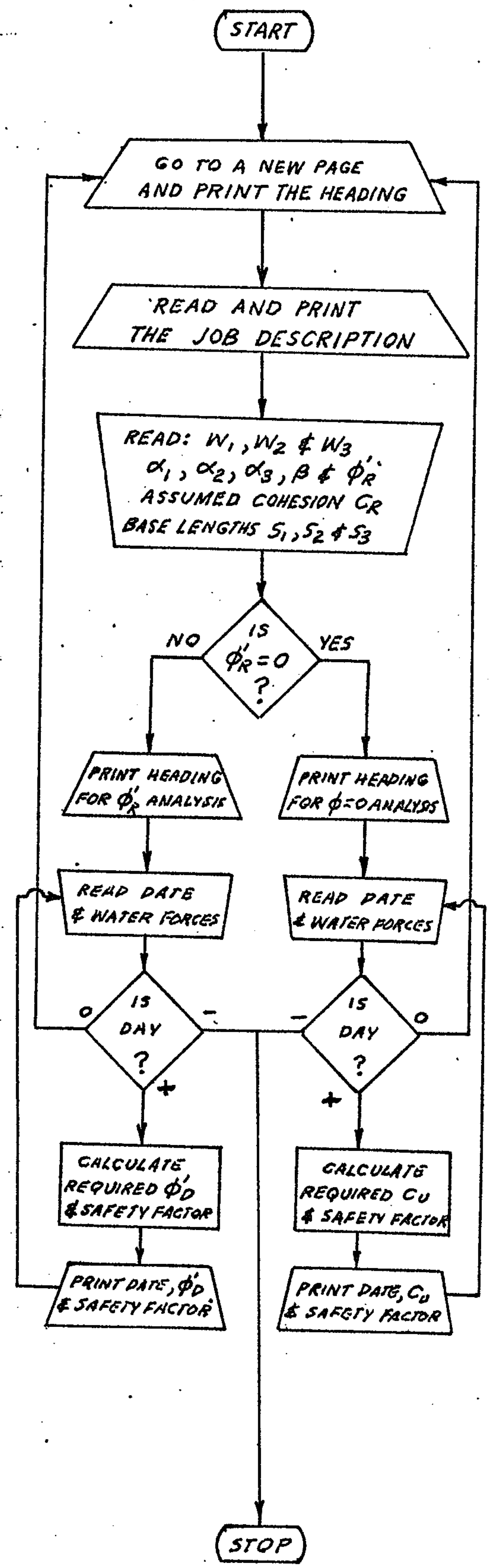
APPENDIX A. COMPUTER PROGRAM, INPUT DATA AND OUTPUT FOR SLIDING BLOCK SLOPE STABILITY ANALYSIS. STABILITY CALCULATION BY TAYLOR'S STABILITY NUMBER CHART.

TABLE 11

COMPUTER PROGRAM AND FLOW CHART FOR SLIDING BLOCK SLOPE STABILITY ANALYSIS

```
$JOB   WATFIV    PAUL JANZEN
  101 IMPLICIT REAL*8(A-H,O-Z)
  102 SIN(X)=DSIN(X)
  103 COS(X)=DCOS(X)
  104 ATAN(X)=DATAN(X)
  105 TAN(X)=DTAN(X)
  114 PRINT115
  115 FORMAT(1H1,'      SLIDING BLOCK SLOPE STABILITY ANALYSIS ')
  111 READ 112
  112 FORMAT('SITE DESCRIPTION OR HEADING
     )       ')
  113 PRINT 112
    5 READ(5,6)W1,W2,W3
    6 FORMAT(3F5.1)
    7 READ(5,8)AP1,AP2,AP3,BETA,FIR
    8 FORMAT(5F5.1)
   81 READ(5,82)CR,S1,S2,S3
   82 FORMAT(4F6.1)
  901 CC=.017453
  902 IF(FIR)903,903,3
  903 PRINT 904
  904 FORMAT(' ',' DATE  REQUIRED UNDRAINED "C",K.S.F.  SAFETY FACTOR ')
  905 READ(5,10)LDAY,MC,LYR,UB1,UB2,UB3,UT,UL1,RL1
  906 IF(LDAY)32,114,907
  907 CU=(UL1+RL1-UT*SIN(BETA*CO)+UT*COS(BETA*CO)*TAN(AP3*CO)+W1*TAN(AP1
     1*CC)+W2*TAN(AP2*CC)+W3*TAN(AP3*CC))/(S1*(COS(AP1*CC)+TAN(AP1*CC)*S
     2IN(AP1*CO))+S2*(COS(AP2*CC)+TAN(AP2*CO)*SIN(AP2*CO))+S3*(COS(AP3*C
     3C)+TAN(AP3*CC)*SIN(AP3*CC)))
  908 SFAC=CR/CU
  909 WRITE(6,910)LDAY,MC,LYR,CU,SFAC
  910 FORMAT('0'3I3,2F20.4)
  911 GO TO 905
    3 PRINT 4
    4 FORMAT(' ',' DATE          REQUIRED PHI (DEGREES)  SAFETY FACTOR ')
   91 C1X=CR*S1*COS(AP1*CO)
   92 C1Y=CR*S1*SIN(AP1*CC)
   93 C2X=CR*S2*COS(AP2*CC)
   94 C2Y=CR*S2*SIN(AP2*CO)
   95 C3X=CR*S3*COS(AP3*CO)
   96 C3Y=CR*S3*SIN(AP3*CC)
   97 W1=W1-C1Y
   98 W2=W2-C2Y
   99 W3=W3-C3Y
    9 READ(5,10)LDAY,MC,LYR,UB1,UB2,UB3,UT,UL1,RL1
  991 RL1=RL1-C1X-C2X-C3X
   10 FORMAT(3I3,6F6.1)
   11 IF(LDAY)32,114,16
   16 A=UB1*SIN(AP1*CC)+UB2*SIN(AP2*CC)+UB3*SIN(AP3*CO)
   18 B=W3-UB3*COS(AP3*CC)+UT*COS(BETA*CO)
   19 C=W2-UB2*COS(AP2*CC)
   20 D=W1-UB1*COS(AP1*CC)
   21 E=RL1+UL1-UT*SIN(BETA*CO)
   22 U=TAN(AP1*CO)
   23 V=TAN(AP2*CO)
   24 W=TAN(AP3*CC)
   25 Q=-A*U*V*W+B*U*V+C*U*W+D*V*W-E*U*V*W
   26 R=-A*(U*W+V*W+U*V)+B*(V+U-U*V*W)+C*(W+U-U*V*W)+D*(V+W-U*V*W)-E*(V*
     1W+U*V+U*W)
  261 S=-A*(W+U+V)+B*(1.-V*W-U*W)+C*(-V*W-U*V+1.)+D*(1.-U*W-U*V)-E*(V+W+
     1U)
  262 T=-A-B*W-C*V-D*U-E
  263 P1=R/Q
  264 C1=S/Q
  265 R1=T/Q
  266 A1=(3.*C1-(P1**2 ))/3.
  267 B1=(2.*(P1**3 )-9.*P1*C1+27.*R1)/27.
 2671 TESTR=(B1**2)/4.+(A1**3)/27.
 2672 IF(TESTR)367,2673,2680
 2673 IF(B1)2674,2674,2676
 2674 Y1=2.*((-B1/2.)**(1./3.))
 2675 GO TO 2677
 2676 Y1=-2.*((B1/2.)**(1./3.))
 2677 Y2=-Y1/2.
 2678 Y3=Y2
 2679 GO TO 272
 2680 IF(-B1/2.+(TESTR)**.5)2681,2681,2683
 2681 Y1=-((B1/2.)-(TESTR)**.5)**(1./3.)-((B1/2.)+(TESTR)**.5)**(1./3.)
 2682 GO TO 2684
 2683 Y1=((-B1/2.)+(TESTR)**.5)**(1./3.)+((-B1/2.)-(TESTR)**.5)**(1./3.)
 2684 Y2=Y1
 2685 Y3=Y2
 2686 GO TO 272
  367 CFI=(-B1/2.)/(((-A1**3)/27.)**.5)
 3671 IF(CFI)3672,3675,268
 3672 AL=((1.-CFI**2)**.5)/DABS(CFI)
 3673 FI=(180.*CC)-ATAN(AL)
 3674 GO TO 269
 3675 FI=90.*CC
 3676 GO TO 269
  268 AL=((1.-CFI**2)**.5)/CFI
  368 FI=ATAN(AL)
  269 Y1=((-A1/3.)**.5)*(COS(FI/3.))*2.
  270 Y2=((-A1/3.)**.5)*(COS(FI/3.+120.*CC))*2.
  271 Y3=((-A1/3.)**.5)*(COS(FI/3.+240.*CC))*2.
  272 FID1=ATAN(Y1-P1/3.)/CO
  273 FID2=ATAN(Y2-P1/3.)/CO
  274 FID3=ATAN(Y3-P1/3.)/CO
 2741 IF(DABS(FID1)-DABS(FID2))2742,2742,2743
 2742 IF(DABS(FID1)-DABS(FID3))284,284,288
 2743 IF(DABS(FID2)-DABS(FID3))286,286,2742
  284 FID=FID1
  285 GO TO 289
  286 FID=FID2
  287 GO TO 289
  288 FID=FID3
  289 SFAC=TAN(FIR*CC)/TAN(FID*CO)
  290 WRITE(6,291)LDAY,MC,LYR,FID,SFAC
  291 FORMAT('0'3I3,2F20.2)
   29 GO TO 9
   32 STOP
      END

$ENTRY
```



(Continued on following page)

TABLE II (CONTINUED)

Notes on the computer program:

1. The program will do a residual strength analysis using $c'_R = 0$ if the data card for cohesion is left blank.
2. A small cohesion , in kips per square foot, may be included in the analysis. In such a case, base lengths of blocks, in feet, must also be included.
3. A total stress ($\Phi = 0$) analysis may be performed. In such a case the value of Φ'_R in the data must be given as zero. The assumed undrained cohesive strength, c_U, in kips per square foot and base lengths in feet must also be included. The computer then solves for the cohesion required for static equilibrium, c_D, and calculates the safety factor as shown.

$$\text{Safety Factor} = \frac{c_U}{c_D}$$

4. The last data card must always have a "sentinel" negative number in the "day" column. This ends the program.
5. Several input decks can be used consecutively in the same run, provided that the last data card in each intermediate data deck is a blank "sentinel".

TABLE III

INPUT DATA FOR SLIDING BLOCK ANALYSIS TACHE AVENUE SITE, $\alpha_3 = -9.7°$

JOB DESCRIPTION CARD

TACHE AVE. SITE , ALPHA 3 = -9.7 DEGS.,C=0

W_1	W_2	W_3
61.4	109.9	85.0

α_1	α_2	α_3	β	ϕ'_R
35.0	9.5	-9.7	9.0	10.8

C	S1	S2	S3
0.0			

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	R_{L1}
26	11	69	39.4	53.1	63.5	11.0	.	.
20	12	69	31.5	49.3	57.2	10.6	0.0	0.0
10	1	70	30.4	46.6	61.7	10.6	0.0	0.0
24	1	70	38.7	38.9	61.7	10.6	0.0	0.0
21	2	70	24.6	38.9	55.0	10.6	0.0	0.0
28	2	70	25.0	32.9	52.7	10.6	0.0	0.0
4	3	70	25.3	31.4	43.8	10.6	0.0	0.0
14	3	70	18.1	35.2	46.1	10.6	0.0	0.0
21	3	70	18.1	32.4	54.1	10.6	0.0	0.0
28	3	70	22.8	32.4	54.1	10.6	0.0	0.0
11	4	70	16.3	37.6	61.7	18.0	0.0	0.0
18	4	70	10.1	40.7	100.8	59.0	0.0	0.0
25	4	70	10.9	54.5	125.3	106.9	0.0	0.0
2	5	70	5.8	49.2	122.9	97.6	0.0	0.0
9	5	70	3.6	44.0	118.5	90.3	0.0	0.0
16	5	70	4.3	53.7	118.5	86.0	0.0	0.0
22	5	70	16.3	49.3	102.6	64.9	0.0	0.0
29	5	70	20.3	46.6	93.1	56.3	0.0	0.0
12	6	70	19.2	44.1	90.8	57.5	0.0	0.0
19	6	70	15.9	48.0	90.5	57.6	0.0	0.0
26	6	70	18.1	38.1	86.9	56.3	0.0	0.0
3	7	70	21.7	38.9	84.1	47.2	0.0	0.0
17	7	70	18.1	38.9	75.2	30.6	0.0	0.0
24	7	70	21.0	33.7	79.6	34.7	0.0	0.0
31	7	70	19.9	40.1	74.2	37.6	0.0	0.0
7	8	70	20.3	39.4	74.2	28.6	0.0	0.0
17	8	70	14.5	35.0	77.9	35.7	0.0	0.0
8	9	70	16.3	34.7	82.7	37.0	0.0	0.0
18	9	70	18.1	36.3	82.7	37.0	0.0	0.0
23	9	70	18.1	33.7	79.6	37.0	0.0	0.0
17	10	70	18.1	37.8	74.0	37.0	0.0	0.0
25	10	70	19.2	41.5	74.0	37.0	0.0	0.0
31	10	70	16.3	36.9	68.8	36.0	0.0	0.0
7	11	70	18.1	36.3	70.2	30.0	0.0	0.0
13	11	70	14.5	28.8	55.6	18.9	0.0	0.0
20	11	70	10.9	36.3	51.9	11.6	0.0	0.0
27	11	70	10.5	31.1	49.2	9.3	0.0	0.0
7	12	70	15.2	38.9	50.1	9.2	0.0	0.0
11	12	70	15.2	36.3	54.6	12.5	0.0	0.0
0	0	0	0.0	0.0	0.0	0.0	0.0	0.0

JOB DESCRIPTION CARD

TACHE AVE. SITE , ALPHA 3 = -9.7 DEGS.,C=0.6 PSI

W_1	W_2	W_3
61.4	109.9	85.0

α_1	α_2	α_3	β	ϕ'_R
35.0	9.5	-9.7	9.0	10.8

C	S_1	S_2	S_3
.0864	56.4	40.4	72.0

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	R_{L1}
26	11	69	39.4	53.1	63.5	11.0	.	.
20	12	69	31.5	49.3	57.2	10.6	0.0	0.0

(AS FOR THE C=0 CASE)

JOB DESCRIPTION CARD

TACHE AVE. SITE , ALPHA 3 =-9.7 DEG.,UNDRAINED "C"=0.5 K.S.F.

W_1	W_2	W_3
61.4	109.9	85.0

α_1	α_2	α_3	β	ϕ'_R
35.0	9.5	-9.7	9.0	0.0

C	S_1	S_2	S_3
.500	56.4	40.4	72.0

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	R_{L1}
26	11	69	39.4	53.1	63.5	11.0	.	.
20	12	69	31.5	49.3	57.2	10.6	0.0	0.0

(AS FOR THE C=0 CASE)

NOTE:

1. W1,W2&W3 are block weights in KIPS.
2. Angles α1,α2,α3,β & ϕ'_R are measured in DEGREES.
3. C is the assumed cohesion in KIPS/SQ.FT.
4. S1,S2 & S3 are the base lengths of the blocks, in FEET.
5. UB1, etc. are measured in KIPS.
6. Day=0 , as shown on the last data card for the C=0 case , implies that more jobs follow & prepares the computer to read the next job data deck.

TABLE IV

INPUT DATA FOR SLIDING BLOCK ANALYSIS TACHE AVENUE SITE, α_3 = -13.5°.

JOB DESCRIPTION CARD

TACHE AVE. SITE , ALPHA 3 =-13.5 DEGS.,C=0

W_1	W_2	W_3
61.4	109.9	68.3

α_1	α_2	α_3	β	ϕ'_R
35.0	9.5	-13.5	9.0	12.7

C	S_1	S_2	S_3
0.0			

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	R_{L1}
26	11	69	39.4	53.1	43.5	2.7	.	.
20	12	69	31.5	49.3	38.7	2.5	0.0	0.0
10	1	70	30.4	46.6	42.1	2.5	0.0	0.0
24	1	70	38.7	38.9	42.1	2.5	0.0	0.0
21	2	70	24.6	38.9	37.0	2.5	0.0	0.0
28	2	70	25.0	32.9	35.3	2.5	0.0	0.0
4	3	70	25.3	31.4	28.4	2.5	0.0	0.0
14	3	70	18.1	35.2	30.2	2.5	0.0	0.0
21	3	70	18.1	32.4	36.3	2.5	0.0	0.0
28	3	70	22.8	32.4	36.3	2.5	0.0	0.0
11	4	70	16.3	37.6	42.1	6.3	0.0	0.0
18	4	70	10.1	40.7	72.0	32.3	0.0	0.0
25	4	70	10.9	54.5	90.8	69.3	0.0	0.0
2	5	70	5.8	49.2	89.0	62.2	0.0	0.0
9	5	70	3.6	44.0	85.6	56.7	0.0	0.0
15	5	70	4.3	53.7	85.6	53.2	0.0	0.0
22	5	70	16.3	49.3	73.4	52.1	0.0	0.0
29	5	70	20.3	46.6	66.1	30.1	0.0	0.0
12	6	70	19.2	44.1	64.4	30.9	0.0	0.0
19	6	70	15.9	48.0	64.1	31.0	0.0	0.0
26	6	70	18.1	38.1	61.4	30.1	0.0	0.0
3	7	70	21.7	38.9	59.3	24.0	0.0	0.0
17	7	70	18.1	38.9	52.4	13.7	0.0	0.0
24	7	70	21.0	33.7	55.8	16.2	0.0	0.0
31	7	70	19.9	40.1	51.6	18.1	0.0	0.0
7	8	70	20.3	39.4	51.6	18.7	0.0	0.0
17	8	70	14.5	35.0	54.5	16.8	0.0	0.0
8	9	70	16.3	34.7	58.2	17.8	0.0	0.0
18	9	70	18.1	36.3	58.2	17.8	0.0	0.0
23	9	70	18.1	33.7	55.8	17.8	0.0	0.0
17	10	70	18.1	37.8	51.5	17.8	0.0	0.0
25	10	70	19.2	41.5	51.5	17.8	0.0	0.0
31	10	70	16.3	38.9	47.6	17.1	0.0	0.0
7	11	70	16.1	36.3	48.6	13.4	0.0	0.0
13	11	70	14.5	28.8	37.4	6.9	0.0	0.0
20	11	70	10.9	36.3	34.6	3.1	0.0	0.0
27	11	70	10.5	31.1	32.5	2.1	0.0	0.0
7	12	70	15.2	38.9	30.8	1.9	0.0	0.0
31	12	70	15.2	36.3	36.7	3.5	0.0	0.0
0	0	0	0.0	0.0	0.0	0.0	0.0	0.0

JOB DESCRIPTION CARD

TACHE AVE. SITE , ALPHA 3 =-13.5 DEGS.,C=0.6 PSI

W_1	W_2	W_3
61.4	109.9	68.3

α_1	α_2	α_3	β	ϕ'_R
35.0	9.5	-13.5	9.0	12.7

C	S_1	S_2	S_3
.0864	56.4	40.4	56.0

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	R_{L1}
26	11	69	39.4	53.1	43.5	2.7	.	.
20	12	69	31.5	49.3	38.7	2.5	0.0	0.0

(AS FOR THE C=0 CASE)

TABLE V
INPUT DATA FOR SLIDING BLOCK ANALYSIS
ST. VITAL SITE

JOB DESCRIPTION CARD

ST VITAL SITE , C=0

W_1	W_2	W_3
134.	372.	185.

α_1	α_2	α_3	β	ϕ'_R
35.0	0.0	-8.0	10.0	8.1

C	S_1	S_2	S_3
0.0			

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	RL_1
04	06	69	88.0	137.2	99.8	26.9	3.5	.
11	6	69	90.0	137.2	96.1	22.1	3.5	0.0
16	6	69	87.9	137.2	97.5	21.0	3.5	0.0
17	7	69	76.0	157.0	96.1	20.2	3.5	0.0
23	7	69	67.0	137.2	98.0	21.0	3.5	0.0
13	8	69	80.4	146.6	101.1	22.0	3.5	0.0
10	9	69	69.1	140.5	98.6	20.6	3.5	0.0
20	12	69	78.5	140.5	70.5	4.9	3.5	0.0
10	1	70	74.0	145.2	62.4	4.9	3.5	0.0
24	1	70	77.8	124.8	64.9	4.9	3.5	0.0
31	1	70	74.8	124.8	68.6	4.9	3.5	0.0
21	2	70	71.5	132.0	64.2	4.9	3.5	0.0
28	2	70	74.8	123.0	64.2	4.9	3.5	0.0
14	3	70	75.6	118.5	68.6	4.9	3.5	0.0
21	3	70	65.1	123.6	70.4	4.9	3.5	0.0
28	3	70	69.6	124.2	65.5	4.9	3.5	0.0
11	4	70	67.3	124.8	71.7	8.9	3.5	0.0
18	4	70	67.2	134.1	137.3	62.7	3.5	0.0
25	[illegible]	70	65.8	146.7	155.9	142.7	3.5	0.0
2	5	70	74.8	147.3	149.7	127.2	3.5	0.0
9	5	70	50.5	143.6	146.7	129.2	3.5	0.0
16	5	70	79.5	137.2	149.7	98.9	3.5	0.0
22	5	70	82.2	151.6	128.0	96.9	3.5	0.0
29	5	70	77.8	131.7	122.2	62.2	3.5	0.0
5	6	70	83.8	140.3	132.8	62.2	3.5	0.0
12	6	70	77.0	137.2	124.2	62.2	3.5	0.0
19	6	70	74.8	146.8	122.2	62.3	3.5	0.0
26	6	70	74.8	156.0	119.1	62.3	3.5	0.0
3	7	70	91.6	162.3	117.9	45.6	3.5	0.0
17	7	70	74.8	149.6	85.5	17.7	3.5	0.0
24	7	70	74.8	128.5	94.9	21.6	3.5	0.0
31	7	70	67.3	147.5	97.3	23.8	3.5	0.0
7	8	70	63.5	141.0	91.7	25.5	3.5	0.0
17	8	70	72.6	153.5	95.4	22.7	3.5	0.0
8	9	70	80.0	166.6	92.8	23.1	3.5	0.0
18	9	70	71.5	150.0	95.5	23.1	3.5	0.0
23	9	70	63.6	156.1	97.3	23.1	3.5	0.0
17	10	70	65.4	156.0	90.4	23.1	3.5	0.0
25	10	70	64.3	156.0	93.0	22.4	3.5	0.0
31	10	70	63.6	153.4	94.2	22.4	3.5	0.0
7	11	70	58.0	147.2	86.1	16.8	3.5	0.0
13	11	70	67.3	143.5	75.5	10.1	3.5	0.0
20	11	70	63.5	147.2	68.0	5.5	3.5	0.0
27	11	70	63.6	137.2	68.0	4.4	3.5	0.0
7	12	70	63.6	149.9	64.3	3.9	3.5	0.0
31	12	70	61.0	157.8	73.0	7.2	3.5	0.0
-1	0	0	0.0	0.0	0.0	0.0	0.0	0.0

JOB DESCRIPTION CARD

ST. VITAL SITE PHI=0 , UNDRAINED "C"=0.5 K.S.F.

W_1	W_2	W_3
134.	372.	185.

α_1	α_2	α_3	β	ϕ'_R
35.0	0.0	-8.0	10.0	0.0

C	S_1	S_2	S_3
.500	60.6	100.0	101.7

DAY	MO.	YEAR	U_{B1}	U_{B2}	U_{B3}	U_T	U_{L1}	RL_1
04	06	69	88.0	137.2	99.8	26.9	3.5	.
11	6	69	90.0	137.2	96.1	22.1	3.5	0.0

(AS FOR THE C=0 CASE)

NOTE:

Days=-1 , as shown on the last card for the C=0 case implies that there are no more jobs & brings the computer to a stop.

TABLE VI

RESULTS OF THE SLIDING BLOCK ANALYSIS ASSUMING C = 0 TACHE AVENUE SITE, $\alpha_3 = -9.7°$ AND $\alpha_3 = -13.5°$

SLIDING BLOCK SLOPE STABILITY ANALYSIS
TACHE AVE. SITE , ALPHA 3 = -9.7 DEGS.,C=0

DATE	REQUIRED PHI (DEGREES)	SAFETY FACTOR
26 11 69	18.72	0.56
20 12 69	16.47	0.65
10 1 70	16.61	0.64
24 1 70	16.72	0.64
21 2 70	14.53	0.74
28 2 70	13.87	0.77
4 3 70	13.06	0.82
14 3 70	12.96	0.83
21 3 70	13.39	0.80
28 3 70	13.77	0.78
11 4 70	12.88	0.83
18 4 70	8.32	1.30
25 4 70	3.16	3.46
2 5 70	4.11	2.65
9 5 70	4.70	..32
16 5 70	5.48	1.99
22 5 70	5.28	2.07
29 5 70	9.17	1.18
12 6 70	8.67	1.25
19 6 70	8.64	1.26
26 6 70	8.33	1.30
3 7 70	9.81	1.10
17 7 70	11.84	0.91
24 7 70	11.29	0.96
31 7 70	10.74	1.01
7 8 70	10.56	1.02
17 8 70	10.66	1.01
8 9 70	10.85	1.00
18 9 70	11.07	0.98
23 9 70	10.70	1.01
17 10 70	10.58	1.02
25 10 70	10.87	0.99
31 10 70	10.38	1.04
7 11 70	11.42	0.94
13 11 70	11.59	0.93
20 11 70	12.77	0.84
27 11 70	12.60	0.85
7 12 70	13.57	0.79
31 12 70	13.14	0.82

SLIDING BLOCK SLOPE STABILITY ANALYSIS
TACHE AVE. SITE , ALPHA 3 =-13.5 DEGS.,C=0

DATE	REQUIRED PHI (DEGREES)	SAFETY FACTOR
26 11 69	19.52	0.64
20 12 69	17.27	0.72
10 1 70	17.28	0.72
24 1 70	17.39	0.72
21 2 70	15.23	0.83
28 2 70	14.57	0.87
4 3 70	13.85	0.91
14 3 70	13.71	0.92
21 3 70	14.02	0.90
28 3 70	14.43	0.88
11 4 70	13.97	0.91
18 4 70	10.55	1.21
25 4 70	5.51	2.34
2 5 70	6.22	2.07
9 5 70	6.7[illegible]	1.92
16 5 70	7.71	1.67
22 5 70	7.72	1.66
29 5 70	11.73	1.09
12 6 70	11.16	1.14
19 6 70	11.13	1.15
26 6 70	10.65	1.20
3 7 70	12.07	1.05
17 7 70	13.49	0.94
24 7 70	13.06	0.97
31 7 70	12.68	1.00
7 8 70	12.53	1.01
17 8 70	12.39	1.03
8 9 70	12.60	1.01
18 9 70	12.8[illegible]	0.99
23 9 70	12.47	1.02
17 10 70	12.42	1.02
25 10 70	12.78	0.99
31 10 70	12.20	1.04
7 11 70	13.02	0.97
13 11 70	12.66	1.00
20 11 70	13.46	0.94
27 11 70	13.10	0.97
7 12 70	13.93	0.91
31 12 70	13.90	0.91

TABLE VII
RESULTS OF THE SLIDING BLOCK ANALYSIS ASSUMING C=0, ST. VITAL SITE

SLIDING BLOCK SLOPE STABILITY ANALYSIS
ST VITAL SITE , C=0

DATE	REQUIRED PHI (DEGREES)	SAFETY FACTOR
4 6 69	8.19	0.99
11 6 69	8.45	0.96
16 6 69	8.50	0.95
17 7 69	8.65	0.94
23 7 69	8.07	1.00
13 8 69	8.53	0.95
10 9 69	8.21	0.99
20 12 69	8.78	0.92
10 1 70	8.62	0.94
24 1 70	8.37	0.97
31 1 70	8.38	0.97
21 2 70	8.36	0.97
28 2 70	8.27	0.98
14 3 70	8.28	0.98
21 3 70	8.20	0.99
28 3 70	8.21	0.99
11 4 70	8.07	1.00
18 4 70	6.37	1.28
25 4 70	2.94	2.77
2 5 70	3.62	2.25
9 5 70	3.30	2.47
16 5 70	4.89	1.66
22 5 70	4.94	1.65
29 5 70	6.31	1.29
5 6 70	6.69	1.21
12 6 70	6.40	1.27
19 6 70	6.47	1.26
26 6 70	6.56	1.24
3 7 70	7.95	1.02
17 7 70	8.42	0.96
24 7 70	7.98	1.01
31 7 70	8.09	1.00
7 8 70	7.71	1.05
17 8 70	8.34	0.97
8 9 70	8.69	0.93
18 9 70	8.23	0.98
23 9 70	8.21	0.99
17 10 70	8.12	1.00
25 10 70	8.18	0.99
31 10 70	8.15	0.99
7 11 70	8.09	1.00
13 11 70	8.40	0.96
20 11 70	8.51	0.95
27 11 70	8.39	0.96
7 12 70	8.58	0.94
31 12 70	8.66	0.93

TABLE VIII

RESULTS OF THE SLIDING BLOCK ANALYSIS ASSUMING C= 0.6 P.S.I.

TACHE AVENUE SITE, α_3 = -9.7° AND α_3= -13.5°

SLIDING BLOCK SLOPE STABILITY ANALYSIS
TACHE AVE. SITE , ALPHA 3 = -9.7 DEGS.,C=0.6 PSI

DATE	REQUIRED PHI (DEGREES)	SAFETY FACTOR
26 11 69	12.14	0.89
20 12 69	10.63	1.02
10 1 70	10.71	1.01
24 1 70	10.81	1.00
21 2 70	9.33	1.16
28 2 70	8.90	1.22
4 3 70	8.38	1.29
14 3 70	8.30	1.31
21 3 70	8.58	1.27
28 3 70	8.83	1.23
11 4 70	7.96	1.36
18 4 70	3.55	3.08
25 4 70	-1.32	-8.27
2 5 70	-0.29	-38.06
9 5 70	0.39	28.14
16 5 70	0.83	13.21
22 5 70	0.77	14.28
29 5 70	4.08	2.67
12 6 70	3.78	2.89
19 6 70	3.76	2.90
26 6 70	3.70	2.95
3 7 70	4.90	2.23
17 7 70	6.79	1.60
24 7 70	6.30	1.73
31 7 70	5.86	1.86
7 8 70	5.71	1.91
17 8 70	5.89	1.85
8 9 70	5.93	1.84
18 9 70	6.06	1.80
23 9 70	5.86	1.86
17 10 70	5.79	1.88
25 10 70	5.95	1.83
31 10 70	5.72	1.90
7 11 70	6.58	1.65
13 11 70	7.12	1.53
20 11 70	8.13	1.34
27 11 70	8.09	1.34
7 12 70	8.74	1.24
31 12 70	8.34	1.30

SLIDING BLOCK SLOPE STABILITY ANALYSIS
TACHE AVE. SITE , ALPHA 3 =-13.5 DEGS.,C=0.6 PSI

DATE	REQUIRED PHI (DEGREES)	SAFETY FACTOR
26 11 69	13.41	0.95
20 12 69	11.80	1.08
10 1 70	11.79	1.08
24 1 70	11.90	1.07
21 2 70	10.35	1.23
28 2 70	9.90	1.29
4 3 70	9.41	1.36
14 3 70	9.29	1.38
21 3 70	9.50	1.35
28 3 70	9.79	1.31
11 4 70	9.29	1.38
18 4 70	5.83	2.21
25 4 70	0.98	13.13
2 5 70	1.81	7.12
9 5 70	2.41	5.36
16 5 70	3.05	4.22
22 5 70	3.16	4.09
29 5 70	6.65	1.93
12 6 70	6.27	2.05
19 6 70	6.25	2.06
26 6 70	6.03	2.13
3 7 70	7.21	1.78
17 7 70	8.63	1.49
24 7 70	8.23	1.56
31 7 70	7.90	1.62
7 8 70	7.77	1.65
17 8 70	7.76	1.65
8 9 70	7.84	1.64
18 9 70	8.02	1.60
23 9 70	7.77	1.65
17 10 70	7.75	1.66
25 10 70	7.98	1.61
31 10 70	7.64	1.68
7 11 70	8.34	1.54
13 11 70	8.39	1.53
20 11 70	9.08	1.41
27 11 70	8.87	1.44
7 12 70	9.46	1.35
31 12 70	9.37	1.37

TABLE IX.

RESULTS OF THE SLIDING BLOCK ANALYSIS ASSUMING C=0.5 K.S.F. TACHE AVE. SITE; $\alpha_3 = -9.7°$ AND ST. VITAL SITE

SLIDING BLOCK SLOPE STABILITY ANALYSIS
TACHE AVE. SITE , ALPHA 3 =-9.7 DEG.,UNDRAINED "C"=0.5 K.S.F.

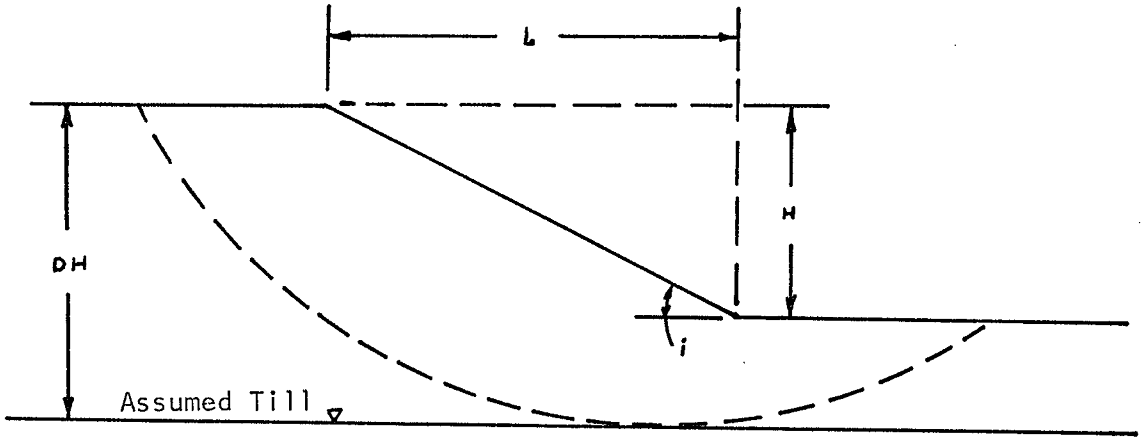
DATE	REQUIRED UNDRAINED "C",K.S.F.	SAFETY FACTOR
26 11 69	0.2367	2.1127
20 12 69	0.2374	2.1064
10 1 70	0.2374	2.1064
24 1 70	0.2374	2.1064
21 2 70	0.2374	2.1064
28 2 70	0.2374	2.1064
4 3 70	0.2374	2.1064
14 3 70	0.2374	2.1064
21 3 70	0.2374	2.1064
28 3 70	0.2374	2.1064
11 4 70	0.2242	2.2300
18 4 70	0.1513	3.3051
25 4 70	0.0661	7.5665
2 5 70	0.0826	6.0516
9 5 70	0.0956	5.2297
16 5 70	0.1033	4.8423
22 5 70	0.1052	4.7523
29 5 70	0.1561	3.2034
12 6 70	0.1540	3.2478
19 6 70	0.1538	3.2515
26 6 70	0.1561	3.2034
3 7 70	0.1723	2.9024
17 7 70	0.2018	2.4777
24 7 70	0.1945	2.5706
31 7 70	0.1893	2.6406
7 8 70	0.1876	2.6657
17 8 70	0.1927	2.5943
8 9 70	0.1904	2.6258
18 9 70	0.1904	2.6258
23 9 70	0.1904	2.6258
17 10 70	0.1904	2.6258
25 10 70	0.1904	2.6258
31 10 70	0.1922	2.6015
7 11 70	0.2029	2.4647
13 11 70	0.2226	2.2461
20 11 70	0.2356	2.1223
27 11 70	0.2397	2.0861
7 12 70	0.2399	2.0845
31 12 70	0.2340	2.1368

SLIDING BLOCK SLOPE STABILITY ANALYSIS
ST. VITAL SITE PHI=0 , UNDRAINED "C"=0.5 K.S.F.

DATE	REQUIRED UNDRAINED "C",K.S.F.	SAFETY FACTOR
4 6 69	0.2275	2.1982
11 6 69	0.2329	2.1471
16 6 69	0.2341	2.1357
17 7 69	0.2350	2.1275
23 7 69	0.2341	2.1357
13 8 69	0.2330	2.1461
10 9 69	0.2346	2.1316
20 12 69	0.2523	1.9820
10 1 70	0.2523	1.9820
24 1 70	0.2523	1.9820
31 1 70	0.2523	1.9820
21 2 70	0.2523	1.9820
28 2 70	0.2523	1.9820
14 3 70	0.2523	1.9820
21 3 70	0.2523	1.9820
28 3 70	0.2523	1.9820
11 4 70	0.2478	2.0181
18 4 70	0.1871	2.6727
25 4 70	0.0969	5.1625
2 5 70	0.1143	4.3732
9 5 70	0.1121	4.4612
16 5 70	0.1463	3.4188
22 5 70	0.1485	3.3668
29 5 70	0.1876	2.6646
5 6 70	0.1876	2.6646
12 6 70	0.1876	2.6646
19 6 70	0.1875	2.6662
26 6 70	0.1875	2.6662
3 7 70	0.2064	2.4229
17 7 70	0.2378	2.1023
24 7 70	0.2334	2.1419
31 7 70	0.2310	2.1649
7 8 70	0.2290	2.1831
17 8 70	0.2322	2.1534
8 9 70	0.2317	2.1576
18 9 70	0.2317	2.1576
23 9 70	0.2317	2.1576
17 10 70	0.2317	2.1576
25 10 70	0.2325	2.1502
31 10 70	0.2325	2.1502
7 11 70	0.2388	2.0934
13 11 70	0.2464	2.0292
20 11 70	0.2516	1.9873
27 11 70	0.2528	1.9776
7 12 70	0.2534	1.9732
31 12 70	0.2497	2.0026

TABLE X

CALCULATION OF SAFETY FACTORS USING TAYLOR'S STABILITY NUMBERS



(Assumed γ = 115 P.C.F.)

DESCRIPTION	ST. VITAL SITE	TACHE AVENUE SITE
H (FT.)	29	26
DH (FT.)	51	43
L (FT.)	220	120
Tan i	0.132	0.217
i°	7.5	12.2
γH (P.S.F.)	3340	2990
D	1.76	1.65
$C_d/\gamma H$ (From Chart)*	0.089	0.122
C_d (P.S.F.)	297	365
Safety Factor (Assumed C_u=0.5 K.S.F.)	1.68	1.37

* From Taylor's Stability Number Chart for the ϕ=0, limited depth case.

APPENDIX B. RESULTS OF PIEZOMETER READINGS
(TOTAL HEAD ELEVATIONS).

TABLE XI

TOTAL HEAD IN FEET, ST. VITAL SITE
(700 FOOT DATUM ELEVATION)

DATE	1962																		
PIEZ. NO.	JAN 20	31	FEB 2	3	5	6	7	8	9	13	16	28	MAR 5	14	28	APR 11	16	17	18
1	27.8	27.6									28.7	27.7		27.9	27.7	27.7			28.8
2	26.9	26.9								27.0		26.9		27.0	27.0	26.2			28.1
3	26.8	26.8								26.9		26.8		27.0	26.8	26.7			27.9
4	DRY	DRY	51.5	42.9	39.1	38.1	37.4	36.9	36.0	35.2		33.1	33.1	33.6	33.8	33.1			35.2
5																	DRY	31.9	31.8

DATE	1962																		
PIEZ. NO.	APR 19	20	21	22	23	24	25	26	27	28	30	MAY 1	2	3	5	8	9	12	14
1	29.0	29.2	29.9	31.1									32.4	32.1	31.6	31.0	30.7	30.4	30.5
2	28.2	28.4	29.0	29.8									31.5	31.3	30.8	29.9	29.6	29.3	29.2
3	28.0	28.4	29.3	30.5									31.0	30.9	30.3	29.5	29.2	28.9	28.9
4	33.3	33.5	33.5	33.5	33.5	33.5	33.7	33.6	33.7	33.8	33.8	33.7	33.9	33.8	33.8	33.5	33.5	33.5	33.5
5	32.0	32.5	32.6	33.0	33.2	33.8	34.1	34.1	34.6	34.9	35.3	35.6	35.8	36.0	36.4	36.4	36.7	37.0	37.1

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE
(700 FOOT DATUM ELEVATION)

DATE / PIEZ. NO.	1962 MAY 15	16	17	22	24	29	31	JUNE 1	7	13	19	22	27	JULY 4	11	18	25	AUG 2	8
1	30.7	30.7	30.9	32.1	32.4	32.8		33.3	33.4	33.1	33.5	33.4	33.3	32.7	32.1	31.9	32.2	31.6	31.3
2	29.2	29.3	29.4	30.0	30.1	30.8		31.2	31.5	31.4	31.7	31.8	31.6	30.9	30.1	30.2	30.3	29.7	29.3
3	29.0	29.1	29.2	29.5	29.7	30.5		30.9	31.2	31.1	31.4	31.4	31.2	30.6	29.7	29.8	29.9	29.3	28.8
4	33.4	33.5	33.8	33.8	33.6	34.0	33.9		33.8	34.0	34.0	34.4	34.4	34.4	34.3	34.2	34.2	34.5	34.2
5	37.2	37.3	37.6	37.9	37.8	38.4	38.4		38.5	38.5	38.7	38.9	38.9	38.9	38.9	38.8	38.7	38.8	38.6

DATE / PIEZ. NO.	1962 AUG 30	SEPT 7	12	21	OCT 20	NOV 7	DEC 4	1963 FEB 11	MAR 15	APR 6	MAY 15	23	30	JUNE 20	28	JULY 12	24	AUG 7	NOV 10	23
1	31.3	31.1	31.1	31.0	30.6	30.3	30.1	29.2	29.3	30.1	30.3	30.3	30.3	30.3	30.6	30.2	30.0	29.7	BLOCKED	
2	29.3	29.2	29.2	29.2	29.2	28.8	28.4		BLOCKED					29.4	29.4	28.9	28.7	28.5	BLOCKED	
3	28.8	28.7	28.8	28.8	28.7	28.4	28.1	BLOCKED		29.0	BLOCKED			29.3	29.1	28.5	28.4	28.0	BLOCKED	
4	34.4	34.6	34.5	34.4	34.5	34.1	34.5	34.4	35.3	35.1	35.1	35.0	34.1	35.1	35.4	35.3	35.1	35.0	35.1	34.7
5	38.6	38.7	38.5	38.5	38.4			BLOCKED							39.6	39.3	39.1	39.0	38.8	38.6

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE

(700 FOOT DATUM ELEVATION)

DATE	1964									1965									
PIEZ. NO.	FEB 1	29	MAR 21	MAY 1	JUNE 11	25	JULY 21	OCT 16	NOV 8	MAY 3	13	JULY 29	SEPT 11	OCT 2	NOV 6	DEC 11	MAR 19	APR 2	10
1				BLOCKED						SUBMERGED		35.6	35.1	34.6	34.9	35.3	35.3	SUBMERGED	
2				BLOCKED						SUBMERGED		34.7	34.2	33.7	33.7	34.3	34.5	SUBMERGED	
3				BLOCKED						SUBMERGED			28.0	27.9	28.5	28.5	29.0	SUBMERGED	
4	34.7	35.3	35.0	35.3	35.4	35.5	35.3	34.9	34.8	35.8	36.0	35.3	35.4	35.5	35.2	35.3	35.8	35.8	35.9
5	38.6	39.2	38.9	39.5	39.5	39.3	39.4	39.0	38.8	40.0	40.1	39.5	39.2	39.2	39.0	39.1	39.8	39.7	40.0

DATE	1966															
PIEZ. NO.	APR 12	14	16	20	26	30	MAY 4	7	28	JUNE 21	JULY 22	OCT 15	NOV 5	12	DEC 10	31
1				SUBMERGED					36.5	36.1	36.0	35.9	36.2	35.2	35.7	35.6
2				SUBMERGED					35.4	35.0	35.0	34.7	35.2	36.3	34.7	34.7
3				SUBMERGED					33.0	31.0	28.3	28.8	29.5	29.3	29.3	29.4
4	36.3	36.6	36.9	36.9	36.6	36.3	36.6	36.2	36.3	36.5	36.2	35.7	35.9	35.6	35.4	35.9
5	40.3	40.4	40.7	40.8	40.7	40.3	40.6	40.3	40.1	40.0	39.8	39.2	39.3	39.0	39.0	39.4

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE

(700 FOOT DATUM ELEVATION)

DATE	1967																1968		
PIEZ. NO.	JAN 14	28	MAR 4	25	APR 1	8	MAY 12	27	JUNE 16	JULY 8	AUG 12	SEPT 9	23	30	OCT 28	NOV 4	JAN 13	MAR 30	APR 26
1	35.6	35.7	35.3	35.1		SUBMERGED			38.2	35.9	36.0	35.5	35.5	35.5	35.8	35.7	34.9	34.9	35.1
2	34.7	34.7	34.6	34.5		SUBMERGED			35.1	34.8	34.8	34.5	34.5	34.4	34.7	34.7	33.9	33.9	34.3
3	29.4	29.4	29.5	29.5		SUBMERGED			41.6	41.0	39.8	39.1	38.7	38.4	37.7	37.4	35.1	35.1	34.9
4	35.8	35.6	36.2	36.3	35.9	36.3	36.1	36.0	36.2	36.3	36.1	35.8	35.8	35.9	36.0	35.6			
5	39.3	39.4	40.0	40.1	39.8	39.7	39.9	39.9	40.0	40.1	39.9	39.5	39.5	39.6	39.5	39.2	40.1	40.1	40.0

DATE	1968			
PIEZ. NO.	JUNE 12	JULY 6	24	AUG 22
1	36.0	36.2	36.2	
2	34.8	34.8	34.7	
3		33.9	33.3	
4	36.2	36.4	36.2	38.3
5	40.2	40.2	40.2	40.1

(CONTINUED)

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE

(700 FOOT DATUM ELEVATION)

DATE	1968 OCT											NOV						DEC
PIEZ. NO.	3	4	8	10	11	15	16	18	22	25	30	7	8	11	12	22	29	2
1	36.7		35.8				36.4	36.2	36.8		36.6	36.2	36.1		35.9	35.8		35.8
2	35.4		35.2				35.2	35.0	36.0		35.0	34.9	34.9		34.7	34.6		34.6
3	42.0		41.8				41.4	41.4	41.3		41.1	40.9	40.9		40.8	40.6		40.7
4	36.0		35.9				35.2	35.1	35.5		36.1	35.8	35.7		35.8	36.0		35.6
5	DRY		39.8				DRY	39.8	39.7		39.9	39.7	39.6		39.7	39.8		39.6
6	38.9	37.9	27.6	28.5	26.4	27.6	26.7	28.3	28.5	26.2	25.5	30.6	31.6	28.7	27.3	27.3	32.8	26.4

DATE	1968 NOV	DEC			1969 FEB	APR			MAY			
PIEZ. NO.	29	2	10	23	22	13	19	26	3	11	19	28
6	32.8	26.4										31.7
7		17.0	31.3	29.0	29.0	30.1	35.4	35.9	36.6	36.1	36.4	31.0
8	52.5	39.8	43.0	38.6	37.5	38.8	43.9	40.5	43.0	41.4	43.7	40.0
9												
10		22.2	38.8	33.9	33.7	29.3	34.8	35.8	35.6	36.0	34.6	31.9

(CONTINUED)

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE

(700 FOOT DATUM ELEVATION)

DATE	1969								1970											
PIEZ. NO.	JUNE 4	11	16	JULY 17	23	AUG 13	SEPT 10	DEC 20	JAN 10	24	31	FEB 21	28	MAR 14	21	28	APR 11	18	25	MAY 2
1				42.3	36.7	36.9	35.9	36.4	36.3		35.8	36.1	36.3	35.3	35.5	35.7	35.5			
2			35.8	35.8	35.2	35.8	35.3	34.8			34.7	34.9	34.7	34.3	34.4	34.6	34.7			
3			37.1	36.8	42.7	41.7		40.8	35.0											
4	36.5	36.7	36.5	36.4	36.5	36.8	36.4	36.0	36.0	36.1	36.4	36.0	36.0	36.2						
5	40.5	40.8	40.5	40.4	40.6	40.7	40.2	39.0	40.1	40.1	40.3	40.2	40.2	40.5	40.8	40.6	40.4	40.3	40.7	40.6
6	30.9	29.7	31.1	31.1	33.0	33.7	33.7	29.7		28.6										
7	30.5	30.5	30.5	33.5	29.8	28.9	28.6	27.9	27,9	27.5	27.5	26.6	27.5	27.5	27.0	26.5	26.8	30.7	35.8	33.5
8	39.2	39.4	38.2	37.5	37.3	37.3	37.1	37.3	37.3	36.6		35.5	32.7	32.7	34.3	33.2	34.3	38.2	44.2	42.2
9				33.8	37.1	36.0	33.2	37.6	36.9	36.9		36.2	34.1	36.0	34.8	33.7	34.8	34.1	36.4	35.7
10	31.2	31.2	30.7	34.2	30.0	29.8	29.8	29.8	29.8	29.8		29.8	29.1	29.1	29.1	29.3	28.4	29.3	33.5	32.5
11				59.6		32.2	37.1	36.4	36.4	35.9		35.4	34.8	34.1	35.2	35.5	35.2	35.2	37.1	36.6
12				44.5	39.8	44.4	39.8	45.5	47.2	47.2		47.4	46.6	47.8	46.0	49.0	46.7	46.7	47.4	47.4
13				36.2	33.4	39.2	38.6	31.8	32.9	32.9		31.3	31.0	31.5	31.1	29.5	30.1	32.4	35.0	35.5

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE
(700 FOOT DATUM ELEVATION)

DATE 1970 PIEZ. NO.	MAY 9	16	22	29	JUNE 5	12	19	26	JULY 3	17	24	31	AUG 7	17	SEPT 8	18	23	OCT 17	25	31
1				36.8	37.2	37.0	37.0	37.0	36.8	36.6	36.4	36.4	36.0	35.9	35.9	36.1	36.5	36.8	36.7	36.7
2				35.3	35.7	35.6	35.5	35.5	35.5	35.0	35.0	35.0	34.8	34.7	34.7	34.7	34.8	35.8	35.8	35.0
3							42.5	44.4	42.4	41.9	41.7	41.7	41.6	41.4	41.6	40.8	40.6	40.2	40.2	40.2
4																				
5	40.5	40.5	40.6	40.5	40.5															
6		42.5	41.8	38.5	40.4	39.2	37.3	39.5	36.7	31.4	33.0	33.0	30.7	34.4	28.3	29.1	30.0	30.7	30.0	30.0
7	34.2	32.8	33.0	30.6	31.5	31.3	31.5	30.2	31.7	28.3	27.4	29.2	28.3	29.3	28.8	29.3	29.6	29.9	29.5	29.5
8	42.4	42.2	41.9	39.2	39.6	37.8	35.9	38.5	38.0	36.4	36.6	37.3	36.4	36.6	36.4	36.4	36.2	36.9	36.6	36.6
9	30.2	33.7	33.9	33.9	33.7	33.0	33.3	31.9	33.0	31.6	33.0	33.7	32.8	33.7	33.5	35.1	34.0	35.3	34.9	34.9
10	32.1	30.8	30.6	29.2	30.1	29.7	26.2	28.5	28.3	25.5	28.5	28.5	27.8	29.7	31.0	33.4	35.0	36.1	36.8	36.6
11	36.2	36.0	36.0	35.5	36.1	35.5	36.0	36.0	36.2	35.9	35.7	36.6	35.7	36.4	35.5	35.9	35.7	35.9	35.9	35.4
12	36.8	47.4	46.2	46.0	46.7	46.0	45.1	46.0	46.2	45.5	45.5	46.0	46.0	44.8	46.0	45.8	44.9	45.8	45.3	45.3
13	35.7	37.1	36.2	34.5	35.1	33.6	35.0	33.6	33.0	29.9	29.9	31.1	29.7	29.7	28.5	29.2	29.9	31.3	31.5	29.4

TABLE XI (CONTINUED)

TOTAL HEAD IN FEET, ST. VITAL SITE

(700 FOOT DATUM ELEVATION)

DATE / PIEZ. NO.	1970 NOV 7	13	20	27	DEC 7	31
1	36.9	36.9	36.7	36.4	36.4	36.4
2	35.1	35.1	35.1	34.8	34.8	35.2
3	39.9	39.8	39.8	39.6	39.4	39.1
4						
5						
6	28.4	28.4	27.6	27.6	29.7	
7	28.8	28.3	26.0	26.9	28.3	28.3
8	35.7	35.0	33.9	34.8	36.8	36.8
9	33.4	32.8	32.5	33.0	35.5	36.2
10	35.4	35.0	33.8	33.6	36.1	35.6
11	34.5	34.5	33.4	33.4	35.7	35.7
12	45.8	44.2	43.9	43.2	45.8	45.8
13	29.5	30.2	27.6	27.6	29.9	29.9

TABLE XII

TOTAL HEAD IN FEET, TACHE AVENUE SITE
(700 FOOT DATUM ELEVATION)

DATE	1969		1970																	
PIEZ. NO.	NOV 26	DEC 20	JAN 10	24	FEB 21	28	MAR 4	14	21	28	APR 11	18	25	MAY 2	9	16	22	29	JUNE 12	19
1	55.7	53.2	53.6	53.2	43.7	40.0̄	40.7	41.6	43.5	47.9	50.9	42.5	40.0̄	40.0̄	40.0̄	48.3	48.8	49.7	45.9	40.0̄
2					41.5	41.5	42.5	38.5	37.6̄	37.6̄	37.6̄	37.6̄	37.6̄	37.6̄	37.6̄	37.6̄	40.4	39.9	39.2	38.9
3			43.6	48.7	47.2	48.0	46.2		46.6			31.6̄	31.6̄	31.6̄	31.6̄	31.6̄	34.6	35.1	36.0	36.8
4		43.9	54.3	56.5	52.9	50.7	49.5	47.7	35.1		22.1	25.6	31.8	33.7	33.7	34.1	33.4	33.4	33.2	30.2
5	35.7	33.6	35.3	36.0	35.9	35.3	35.7	35.7	32.7		34.8	35.3	39.2	29.9	31.1	37.3	38.2	37.6	38.1	37.1
6	36.5	32.8	30.5	29.5	23.1	21.2	23.1	17.3̄	19.8		24.9	25.9	29.5	30.0	29.7	29.8	29.8	29.1	29.3	28.6
7	36.2	39.4	37.8	35.6	32.9	32.4	32.9	32.9	33.1		32.4	32.2	34.0	33.8	34.5	35.0	35.0	34.5	31.7	27.8
8	49.4	45.5	32.8	30.0̄	30.0̄	30.0̄	30.0̄	30.0̄	30 0̄		30.0̄	30.0̄	44.8	48.0	47.6	47.6	47.6	46.2	44.3	43.4
9	35.4	33.8	33.8	33.6	30.1	30.8	31.9	31.9	32.2		32.2	34.9	36.1	38.4	36.8	36.8	36.8	35.4	34.0	33.8
10	39.8	38.8	37.2	34.8	31.9	30.9	30.7	30.5	29.3	28.9	27.9	27.7	32.8	31.9	30.9	32.1	31.6	31.4	30.5	30.0

TABLE XII (CONTINUED)

TOTAL HEAD IN FEET, TACHE AVENUE SITE
(700 FOOT DATUM ELEVATION)

DATE	1970																		
	JUNE	JULY				AUG		SEPT			OCT			NOV				DEC	
PIEZ. NO.	26	3	17	24	31	7	17	8	18	23	17	25	31	7	13	20	27	7	31
1	42.7	44.4	47.9	49.5	47.9	47.9	40.0	41.6	46.9	47.4	46.7	46.2	46.7	46.5					47.9
2	39.2	43.4	39.4	41.5	41.5	40.8	41.3	41.8	41.5	41.5	41.5		38.5						
3	36.2	36.5	37.2	37.2	37.4	37.4	37.4	37.4	38.1	37.4	39.5								
4	31.8	31.6	31.1	30.7	30.0	29.5	29.5	28.4	28.4	28.6	28.6	28.8	28.4	27.7	27.2	27.2	27.0		27.7
5	36.9	37.1	35.7	35.5	35.5	35.0	35.0	34.6	35.0	34.4	34.6	35.3	35.0	35.0	33.9	32.2	31.8	32.9	32.9
6	29.1	28.4	29.1	28.0	29.3	29.3	28.4	28.4	28.2	27.9	27.3	27.5	27.0	26.1	25.6	25.4	24.9	26.5	25.6
7	28.3	26.8	25.0	24.7	23.6	22.7	22.5	23.2	25.0	26.4	28.3	27.8	27.8	27.8	26.7	26.7	25.5	27.6	28.7
8	40.9	41.1	37.9	36.9	37.4	37.4	36.0	35.5		34.6	34.6	34.6	34.2	33.5	31.2				
9	32.6	33.1	33.5	32.3	32.8	32.8	32.8	32.8	32.8	32.8	32.2	39.9	32.2	31.0	30.6	30.6	30.3	32.6	31.9
10	28.0	26.8	25.9	24.7	25.0	24.3	23.3	23.1	23.5	23.5	24.9	25.8	25.9	25.0	24.7	24.7	24.7	26.8	26.1

APPENDIX C. RED RIVER ELEVATIONS.

TABLE XIII

RED RIVER ELEVATION AT
JAMES AVENUE PUMPING STATION

		1967		1968								
DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT
1				26.2			32.7			35.0		34.1
2							33.1				33.9	
3							33.7					
4												
5							34.0	33.7			34.4	
6		25.4										34.2
7												
8											34.0	
9												
10								33.7			31.0	34.2
11												
12								35.2	34.9			
13												
14												
15			25.8			26.2	31.7			33.7		
16												
17										34.1		
18												
19												
20							30.6	33.8				
21												
22						26.4				34.3		
23												
24										34.6		33.7
25						27.3						
26						27.5				35.1		
27						27.8						
28						28.4		33.6				33.6
29						29.9						
30	33.9					31.1	33.4					
31						32.0						

NOTE: Add 700 to gauge height to obtain elevation of river in feet.

TABLE XIII (CONTINUED)

RED RIVER ELEVATION AT

JAMES AVENUE PUMPING STATION

DAY	OCT	NOV	1968 DEC	1969 JAN	FEB	MAR	APR	MAY	JUNE
1		33.5	27.6	27.8	28.1	27.8	28.1		36.0
2		33.0							
3		32.5						44.2	
4		31.9							
5		31.4					27.9		
6		30.8							
7		30.3							
8		28.5							
9		27.9							
10		27.8							
11		27.8						44.0	34.6
12		27.7							
13		27.5					43.6		
14		27.7					44.6	43.8	
15	34.0		27.7	27.9	27.9	27.8	45.8		
16									
17								43.4	
18									
19							44.6		
20								42.1	
21									
22									
23								39.8	
24									
25									
26							45.8	38.1	
27									
28								37.0	
29									
30									
31	33.7								

NOTE: Add 700 to gauge height to obtain elevation of river in feet.

TABLE XIV

RED RIVER ELEVATION AT REDWOOD BRIDGE, MEASURED BY DEPARTMENT OF ENERGY, MINES AND RESOURCES
OBTAINED FROM MANITOBA WATER CONTROL AND CONSERVATION BRANCH

(DAY)	1966 OCT	NOV	DEC	1967 JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEPT
1	~~28.75~~	31.55	28.11	28.21	28.13	28.57		43.13	33.64		33.61	33.71
2	~~31.85~~	31.85	28.11	28.22	28.15	28.56		42.72	33.37		33.66	
3	33.69	31.85		28.22	28.15	28.53	35.19	42.38		33.14	33.68	
4	33.76	30.30		28.24			36.45	42.21		33.21	33.67	33.73
5	32.86		28.12	28.24			37.04	42.02	32.80	33.[illegible]		33.69
6	32.96		28.12		28.13	28.53	37.76		32.66	33.75		33.66
7	32.96	28.33	28.12		28.13	28.53	38.74		32.95	34.13	33.69	33.66
8		28.74	28.14		28.12	28.53		42.33	33.62		33.70	33.66
9		27..4	28.14	28.24	28.12	28.52		42.35	33.74		33.69	
10	33.71	27.25		28.25	28.11	28.52	43.63	42.22		33.98	33.80	
11	33.76			28.26			43.68	41.93		33.84	33.93	33.74
12	33.61		27.91	28.25			43.52	41.66	33.40	33.85		33.76
13	33.69		27.94	28.24	28.10	28.50	43.75		33.56	33.87		33.84
14	33.71	26.95	28.00		28.10	28.50	42.80		33.63	33.88	33.95	33.86
15		26.99	28.00		28.11	28.52		40.66	33.71		33.99	33.9
16		26.95	27.84	28.23	28.12	28.51		40.24	33.63		34.04	
17	33.73	26.98		28.23	28.11	28.50	41.91	39.68		33.86	33.91	
18	33.74	26.98		28.23			42.40	39.13		33.84	33.48	33.9
19	33.73		27.80	28.24			41.50	33.55	33.70	33.86		33.97
20	33.74		27.80	28.24	28.19	28.72	41.75		33.77	33.87		
21	33.76	27.29	27.80		28.23	28.75	44.04		33.53	33.98	33.65	33.9
22		27.35	27.83		28.36	28.76			33.51		33.80	33.9
23		27.41	27.84	28.24	28.36	28.76		36.33	33.70		33.99	33.8
24	32.82	27.43		28.23	28.37		44.86	35.92		33.94	33.82	
25	32.85	27.45		28.24			46.04	35.50	33.64	33.88	33.55	33.5
26	33.09		28.12	28.24			45.75	35.14	33.65	33.69		33.9
27	33.25		28.14	28.26	28.43	27.95	45.35		33.27	33.71		33.7
28	33.65	27.77	28.14		28.51	27.92	44.78			33.78	33.73	33.7
29		27.77	28.17			27.82		34.36	33.26		33.79	33.5
30		27.78	28.17	28.10		27.58		34.12	33.17		33.85	
31	32.85			28.13		30.62		33.92		33.68	33.93	

XXXXXXXXX F... XXXXXXXXX 746.04 feet x on April 25.

XXXXXXXXX ... XXXXXXXXX 726.95 feet x on November 14 & 16

Com:
Chec 87
Date

ADD 700.00 TO GAUGE HEIGHT to obtain elevation in feet above mean sea level (Geodetic Survey of Canada Datum adjustment Publication No. 21).

DEPARTMENT OF ENERGY, MINES AND RESOURCES – INLAND WATERS BRANCH – WATER SURVEY OF CANADA

TABLE XIV (CONTINUED)

DAILY GAUGE HEIGHTS

Station Name RED RIVER AT WINNIPEG Station No 05OJ001

Daily Elevations or Gauge Heights in Feet for the Year 1968

Day	January	February	March	April	May	June	July	August	September	October	November	December
1		726.96	726.98	732.43	733.82			733.66		733.93	733.25	
2	726.32	726.97		733.10	734.07		734.10	733.62	733.72	734.09		727.56
3	726.49			733.23	734.00	734.07	734.04		733.73	734.08		727.55
4	726.69		726.95	733.24		734.07	733.98		733.72	734.08	731.89	727.54
5	726.76	726.95	726.95	733.25			733.97	733.22	733.86		731.33	727.56
6		726.95	726.94		733.94	734.10		732.62	734.16		730.63	727.54
7		726.96	726.91		733.88	734.16		731.62		733.70	729.98	
8	726.79	726.97	726.92	732.26	733.89	734.20	733.84	731.63		733.85	728.25	
9	726.83	726.97		732.65	733.89		733.67	733.72	734.16	733.93		727.54
10	726.89			733.06	733.77	734.25	733.77		734.14	733.96		727.53
11	726.93		726.92	733.96		734.30	733.79		733.95	734.01	728.05	727.54
12	726.98	726.97	726.93			734.49	733.87	733.78	733.81		727.95	727.54
13		726.96	726.93		734.97+	734.91		733.84	733.64		727.84	727.55
14		726.95	726.92		734.97	734.88		733.83		734.21	727.54	
15	726.96	726.94	726.92	731.64	734.96		733.93	733.84		734.12	727.44	
16	726.95	726.94		731.37	734.96		734.66	733.85	734.00	733.99		727.54
17	726.94			731.03	734.96	733.95	734.17		734.04	733.95		727.54
18	726.95		727.05	730.63		733.82	733.93		734.10	733.85	727.42	727.55
19	726.94		727.10	730.47		733.87	733.92	734.08	734.15		727.42	727.54
20		726.92	727.36		734.62	733.98		734.57	734.09		727.41	727.54
21		726.93	727.52		734.42	733.78		733.11		733.99	727.42	
22	726.94	726.94	727.63	730.40	734.06		733.83	733.80		734.05	727.43	
23	726.95	726.93		730.37	734.06		733.77	733.91	734.03	734.01		727.55
24	726.95	726.94		730.26	734.22	733.99	733.93		734.07	733.99		727.54
25	726.97		727.65	730.13		734.10	734.66		733.93	733.95	727.46	727.54
26	726.97	726.94	727.66	730.49		734.10	734.63	735.65	733.92		727.49	727.55
27		726.96	728.10		733.79	734.20		734.91	733.91		727.52	727.55
28		726.97	728.57		733.80	734.52		734.41		733.94	727.54	
29	726.94	726.98	730.56	730.54	733.82		734.22	734.21		733.92	727.56	
30	726.94			732.10	733.80		734.96	733.73	733.93	733.32		
31	726.95				733.79		733.42			733.69		

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Summary For the Year				
Maximum daily elevation	734.97	ft at –	on May 13 & 14	Computed by A.H. Date Feb. 11/69
Minimum daily elevation	726.32	ft at –	on Jan. 2	Checked by WMB Date Feb. 12/69

DEPARTMENT OF ENERGY, MINES AND RESOURCES -- INLAND WATERS BRANCH – WATER SURVEY OF CANADA

TABLE XIV (CONTINUED)

DAILY GAUGE HEIGHTS

Station Name ~~RED RIVER AT~~ ~~BRIDGE~~ WINNIPEG Station No. 05OJ001

Daily Elevations ~~or Gauge Heights~~ in Feet for the Year 1969

Day	January	February	March	April	May	June	July	August	September	October	November	December	Day
1	727.54			727.74	744.62		733.83	733.95	733.90	734.17		727.60	1
2	727.54			727.74	744.49	735.13	733.73		733.90	734.17		727.60	2
3	727.55	728.45	727.74	727.75		734.95	733.63		733.92	734.17	732.94	727.61	3
4		728.46	727.74	727.75		734.81	733.27	733.54	733.93		731.84	727.53	4
5		728.46	727.74		743.91	734.68		733.21	734.03		730.92	727.53	5
6	727.54	728.46	727.75		744.44	734.58		733.55			730.[illegible]		6
7	727.54	728.47	727.75	727.97	744.18		733.96	733.57		734.16	730.[illegible]		7
8	727.55			728.85	743.95		733.65	733.61	733.93	734.15		727.54	8
9	727.54			730.25	743.53	734.28	733.57		733.96	734.12		727.53	9
10	727.55	728.48	727.76	733.12		733.98	734.12		733.98	734.08	730.92	727.53	10
11		728.49	727.79	735.19		733.89	733.89	733.95	734.00	734.08	729.50	727.54	11
12		728.49	727.81		742.91	733.69		733.98	734.03		729.91	727.54	12
13	727.59	728.49	727.87		742.99	733.45		734.01		734.08	727.97		13
14	727.61	728.49	727.89	742.63	742.54		733.59	734.15		734.09	727.93		14
15	727.65			742.78	742.32		733.61	734.18	733.88	734.07		727.54	15
16	727.81			743.45	742.11	733.84	733.72		733.93	734.07		727.54	16
17	727.95	728.49	727.81	743.34		732.98	733.78		734.05	734.08	727.91	727.54	17
18		728.49	727.78	743.95		732.84	734.00	733.94	734.05		727.90	727.54	18
19		728.49	727.74		741.55	732.71		733.70	734.08		727.90	727.55	19
20	727.85	728.49	727.74		740.95	732.64		733.82		734.08	727.91		20
21	727.85	728.51	727.74	743.81	740.19		733.82	733.90		734.09	727.90		21
22	727.86			743.93	739.47		733.91	733.94	734.04	734.08		727.55	22
23	727.86			743.99	738.87	732.74	733.95		734.12	734.07		727.56	23
24	727.86	727.75	727.74	744.15		732.83	733.89		734.15	734.08	727.90	727.57	24
25		727.75	727.74	744.35		732.94	733.82	733.78	734.15		727.89	727.60	25
26		727.75	727.74	744.35	737.23	732.93		733.81	734.16		727.89	727.60	26
27	727.99	727.75	727.74		736.84	733.44		733.84		734.05	727.90		27
28	727.99	727.74	727.74	744.57	736.38		733.64	733.87		734.04	727.90		28
29	728.28			744.64	735.95		733.72	733.90	734.16	734.04		727.60	29
30	728.35			744.66	735.65	733.93	733.70		734.17	733.92		68	30
31	728.41		727.74				733.82			733.91			31

Summary For the Year ~~For the Period~~ (January 1 to December 31)					Computed by	Checked by
	Maximum ~~instantaneous~~ daily elevation, ~~gauge height~~,	744.66	ft at —	on April 30	el	
	Minimum ~~instantaneous~~ daily elevation, ~~gauge height~~,	727.53	ft at —	on Dec. 4,5,9,10	Date July 2, 1970	Date

F45 (Dec. 67)

PROVINCE OF MANITOBA

TABLE XIV (CONTINUED)

WATER CONTROL AND CONSERVATION BRANCH

n Daily Gauge Height in Feet / n Daily Discharge in Second-Feet } of RED Lake / River / Creek at / near REDWOOD BRIDGE

for Year ending September 30, 1970

OCT.	NOV.	DEC.	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEPT.	Day
734.24		727.31				8.33	23.45	18.24	17.82		13.81	1
.24	733.68	.31				.33		18.29	16.82		13.81	2
.24	.45	.52				.33		18.59	16.32	13.41	13.79	3
	.11	.51					22.19	18.99		13.62	13.78	4
.27	732.85						22.21	19.23		13.52		5
.23	.58					.35	22.25		14.82	13.45		6
.22		.49				.36	22.28		14.39	13.84	13.98	7
.19		.49				.39	22.99	19.24	14.11		13.99	8
.15	731.36	.48				.39		19.41	13.99		14.11	9
.15	.15	.48				.39		19.87	13.81	14.35	14.11	10
		.45					22.54	19.24		14.12	14.15	11
	730.05						22.15	19.53		14.11		12
.15	729.29					10.15	21.95		13.45	13.98		13
.16		.45				12.27	21.75		13.15	13.64	14.05	14
.14		.46				14.83	21.99	18.26	12.05		14.08	15
.14	728.99	.46				16.65		18.15	12.99		13.64	16
.15	.24	.47				18.49		18.04	12.98	13.78	13.75	17
	729.99	.46					21.98	18.16		13.85	13.68	18
	.85						21.98	18.18		13.95		19
.15	.65					24.13	21.99		12.99	13.99		20
.16		.48				23.91	21.97		13.28	14.15	14.25	21
.15		.99				23.91	21.52	18.98	13.51		14.21	22
.14	727.63	728.12				24.15		18.85	13.75		14.01	23
.14	.59	.12				24.94		18.92	13.45	14.11	13.63	24
	.21						21.15	18.92		13.98	13.65	25
	726.81						20.99	19.05		13.85		26
.12	.81					23.35	19.85		13.35	13.88		27
.11		.15				23.41	19.65		13.91	13.89	14.35	28
.11						23.41	18.25	18.51	14.01		13.99	29
733.99	.99	.16				23.42		18.17	14.15		13.75	30
.98									14.21	13.84		31
												Tot
734.24												Mea

NOTE ADD 720.0 Feet to convert Gauge height to River elevation

Computed ________

Checked ________

APPENDIX D. RESULTS OF SLOPE INDICATOR READINGS (HORIZONTAL MOVEMENT)

TABLE XV

ST. VITAL SITE SLOPE INDICATOR NO. 1

NORTHWARD DEFLECTION IN INCHES

DEPTH (FT.)	1969 JAN 30	FEB 12	APR 2	JUNE 2	SEPT 3	OCT 4	DEC 1	17	1970 FEB 6	23	JUL 31	AUG 28	SEPT 25
2	-0.186	-0.168	-0.582	-0.534	-0.672	-0.906	-3.228	-3.498	-4.074	-4.458		-4.974	-5.148
4	-0.150	-0.126	-0.528	-0.504	-0.636	-0.846	-2.904	-3.138	-3.612	-3.954		-4.404	-4.548
6	-0.114	-0.072	-0.468	-0.468	-0.582	-0.756	-2.502	-2.688	-3.072	-3.372		-3.780	-3.888
8	-0.060	-0.036	-0.408	-0.402	-0.498	-0.642	-1.986	-2.142	-2.454	-2.706		-3.120	-3.186
10	-0.030	-0.018	-0.354	-0.336	-0.384	-0.534	-1.650	-1.788	-2.058	-2.292		-2.640	-2.676
12	-0.030	-0.036	-0.366	-0.360	-0.408	-0.540	-1.674	-1.806	-2.094	-2.310		-2.682	-2.712
14	-0.030	-0.036	-0.360	-0.354	-0.402	-0.522	-1.662	-1.806	-2.100	-2.304		-2.682	-2.718
16	-0.018	-0.030	-0.366	-0.354	-0.402	-0.510	-1.644	-1.800	-2.094	-2.286		-2.664	-2.706
18	-0.018	-0.036	-0.372	-0.360	-0.402	-0.504	-1.368	-1.806	-2.106	-2.280		-2.658	-2.706
20	-0.006	-0.042	-0.372	-0.372	-0.408	-0.504	-1.638	-1.812	-2.118	-2.286		-2.658	-2.718
22	0	-0.042	-0.360	-0.372	-0.402	-0.492	-1.632	-1.806	-2.112	-2.274		-2.646	-2.712
24	-0.006	-0.048	-0.360	-0.384	-0.414	-0.498	-1.650	-1.818	-2.130	-2.292		-2.670	-2.736
26	-0.006	-0.054	-0.360	-0.390	-0.420	-0.504	-1.656	-1.824	-2.142	-2.304		-2.664	-2.724
28	0	-0.042	-0.306	-0.330	-0.348	-0.438	-1.434	-1.584	-1.842	-1.986		-2.274	-2.328
30	0	-0.030	-0.174	-0.204	-0.204	-0.270	-0.786	-0.846	-0.942	-1.206		-1.182	-1.218
32	+0.006	-0.012	-0.086	-0.132	-0.156	-0.174	-0.414	-0.426	-0.456	-0.492		-0.588	-0.618
34	+0.006	+0.006	-0.036	-0.060	-0.078	-0.090	-0.180	-0.168	-0.180	-0.192		-0.234	-0.252
36	-0.006	-0.006	-0.024	-0.036	-0.048	-0.048	-0.078	-0.060	-0.066	-0.072		-0.084	-0.090
39	0	0	0	0	0	0	0	0	0	0	0	0	0

(Slope indicator installed January 13, 1969)

TABLE XV (CONTINUED)
ST.VITAL SLOPE INDICATOR NO. 1
NORTHWARD DEFLECTION IN INCHES

DEPTH (FT.)	1970 OCT 29	NOV 26	DEC 3	10	30
2	-5.232	-8.888	-9.144	-9.240	-9.582
4	-4.632	-7.686	-7.938	-8.022	-8.310
6	-3.966	-6.366	-6.570	-6.648	-6.888
8	-3.252	-4.835	-5.010	-5.088	-5.274
10	-2.742	-3.762	-3.900	-3.972	-4.152
12	-2.784	-3.804	-3.942	-4.014	-4.182
14	-2.784	-3.822	-3.966	-4.026	-4.194
16	-2.766	-3.798	-3.942	-4.014	-4.164
18	-2.766	-3.780	-3.930	-4.008	-4.164
20	-2.766	-3.774	-3.930	-4.008	-4.158
22	-2.748	-3.750	-3.912	-3.996	-4.146
24	-2.760	-3.756	-3.924	-4.014	-4.132
26	-2.724	-3.750	-3.906	-4.008	-4.140
28	-2.334	-3.276	-3.384	-3.456	-3.582
30	-1.212	-1.572	-1.566	-1.584	-1.584
32	-0.606	-0.696	-0.702	-0.714	-0.696
34	-0.252	-0.228	-0.234	-0.246	-0.228
36	-0.906	-0.066	-0.066	-0.066	-0.066
39	0	0	0	0	0

TABLE XVI
ST. VITAL SITE SLOPE INDICATOR NO. 1
EASTWARD DEFLECTION IN INCHES

DEPTH (FT.)	1969 JAN 30	FEB 12	APR 2	JUNE 2	SEPT 3	OCT 4	DEC 1	17	1970 FEB 6
2	-0.126	-0.168	-0.708	-0.666	-0.750	-1.013	-3.884	-3.675	-4.488
4	-0.096	-0.150	-0.630	-0.624	-0.684	-0.935	-3.072	-3.363	-4.092
6	-0.066	-0.138	-0.552	-0.570	-0.630	-0.846	-2.718	-2.991	-3.648
8	-0.030	-0.114	-0.486	-0.510	-0.558	-0.732	-2.286	-2.535	-3.132
10	-0.036	-0.102	-0.450	-0.462	-0.498	-0.648	-2.010	-2.241	-2.796
12	-0.042	-0.114	-0.456	-0.486	-0.510	-0.654	-2.040	-2.277	-2.820
14	-0.048	-0.120	-0.462	-0.492	-0.522	-0.654	-2.046	-2.295	-2.820
16	-0.054	-0.126	-0.480	-0.498	-0.534	-0.660	-2.052	-2.307	-2.826
18	-0.060	-0.144	-0.486	-0.510	-0.546	-0.672	-2.064	-2.319	-2.826
20	-0.060	-0.144	-0.492	-0.516	-0.558	-0.672	-2.064	-2.325	-2.826
22	-0.072	-0.156	-0.486	-0.510	-0.566	-0.660	-2.052	-2.316	-2.814
24	-0.084	-0.162	-0.492	-0.528	-0.582	-0.678	-2.070	-2,334	-2.826
26	-0.096	-0.168	-0.512	-0.564	-0.618	-0.708	-2.112	-2.376	-2.856
28	-0.096	-0.156	-0.474	-0.516	-0.570	-0.666	-1.950	-2.196	-2.610
30	-0.066	-0.096	-0.306	-0.360	-0.408	-0.474	-1.206	-1.338	-1.548
32	-0.036	-0.066	-0.192	-0.246	-0.288	-0.342	-0.690	-0.762	-0.870
34	-0.018	-0.036	-0.096	-0.144	-0.180	-0.216	-0.336	-0.366	-0.420
36	-0.012	-0.012	-0.030	-0.054	-0.078	-0.090	-0.126	-0.132	-0.150
39	0	0	0	0	0	0	0	0	0

DEPTH (FT.)	1970 FEB 23	JUL 31	AUG 28	SEPT 25	OCT 29	NOV 26	DEC 3	10	30
2	-4.740	-5.196	-5.364	-5.412	-5.490	-9.756	-10.110	-10.230	-10.572
4	-4.308	-4.728	-4.860	-4.908	-4.988	-8.586	-8.880	-8.994	-9.306
6	-3.834	-4.224	-4.314	-4.356	-4.416	-7.254	-7.494	-7.596	-7.884
8	-3.300	-3.666	-3.744	-3.744	-3.804	-5.676	-5.910	-6.000	-6.252
10	-2.958	-3.246	-3.330	-3.300	-3.378	-4.602	-4.812	-4.878	-5.118
12	-2.982	-3.294	-3.372	-3.360	-3.414	-4.674	-4.902	-4.962	-5.214
14	-2.982	-3.312	-3.384	-3.378	-3.426	-4.698	-4.938	-4.998	-5.250
16	-2.976	-3.324	-3.390	-3.378	-3.426	-4.692	-4.938	-4.998	-5.244
18	-2.982	-3.336	-3.396	-3.390	-3.436	-4.692	-4.944	-4.998	-5.244
20	-2.976	-3.348	-3.402	-3.396	-3.420	-4.692	-4.944	-5.004	-5.238
22	-2.952	-3.354	-3.408	-3.390	-3.408	-4.686	-4.926	-5.004	-5.214
24	-2.964	-3.384	-3.444	-3.420	-3.438	-4.710	-4.950	-5.040	-5.238
26	-3.012	-3.444	-3.492	-3.468	-3.492	-4.752	-4.986	-5.088	-5.280
28	-2.748	-3.132	-3.162	-3.138	-3.156	-4.350	-4.560	-4.626	-4.812
30	-1.620	-1.836	-1.836	-1.818	-1.836	-2.304	-2.322	-2.328	-2.364
32	-0.888	-1.026	-1.026	-1.014	-1.020	-1.146	-1.158	-1.164	-1.164
34	-0.420	-0.498	-0.492	-0.480	-0.492	-0.486	-0.486	-0.480	-0.480
36	-0.150	-0.186	-0.180	-0.174	-0.180	-0.156	-0.156	-0.156	-0.144
39	0	0	0	0	0	0	0	0	0

TABLE XVII
ST. VITAL SITE SLOPE INDICATOR NO. 2
DEFLECTION IN INCHES

NORTHWARD

DEPTH (FT.)	1969 JAN 30	FEB 12	APR 2	JUNE 2	SEPT 3	OCT 4	DEC 1	17	1970 FEB 6	23
2	-0.120	-0.168	-0.198		-0.138	-0.246	-0.336	-0.384	-0.306	-0.378
4	- 0	-0.030	-0.078		-0.018	-0.060	-0.162	-0.156	-0.174	-0.246
6	+0.012	- 0	-0.048	-0.030	- 0	-0.036	-0.144	-0.132	-0.162	-0.222
8	+0.030	+0.024	-0.024	-0.042	+0.018	-0.018	-0.126	-0.114	-0.156	-0.204
10	+0.036	+0.036	-0.012	-0.036	+0.024	-0.006	-0.120	-0.108	-0.156	-0.192
12	+0.042	+0.036	-0.006	-0.036	+0.018	- 0	-0.126	-0.120	-0.162	-0.210
14	+0.030	+0.024	-0.018	-0.042	+0.006	-0.012	-0.162	-0.150	-0.204	-0.246
16	+0.030	+0.018	-0.024	-0.048	-0.006	-0.018	-0.180	-0.168	-0.234	-0.264
18	+0.030	+0.012	-0.030	-0.066	-0.018	-0.024	-0.192	-0.198	-0.246	-0.276
20	+0.024	- 0	-0.036	-0.072	-0.030	-0.030	-0.204	-0.210	-0.258	-0.288
22	+0.018	-0.006	-0.048	-0.072	-0.042	-0.042	-0.216	-0.228	-0.270	-0.300
24	+0.006	-0 018	-0.066	-0.084	-0.066	-0.066	-0.234	-0.252	-0.288	-0.324
26	+0.006	-0.012	-0.060	-0.078	-0.066	-0.066	-0.234	-0.252	-0.288	-0.324
28	+0.006	-0.012	-0.060	-0.078	-0.066	-0.072	-0.240	-0.258	-0.300	-0.330
30	+0.012	-0.006	-0.054	-0.072	-0.066	-0.072	-0.240	-0.264	-0.306	-0.330
32	+0.006	-0.012	-0.054	-0.078	-0.078	-0.084	-0.252	-0.282	-0.330	-0.348
34	- 0	-0.024	-0.060	-0.078	-0.084	-0.090	-0.258	-0.294	-0.342	-0.354
36	+0.012	- 0	-0.006	-0.018	-0.024	-0.024	-0.090	-0.108	-0.126	-0.126
38	+0.012	+0.006	+0.012	+0.006	+0.006	+0.012	- 0	-0.006	- 0	- 0

EASTWARD

DEPTH (FT.)	1969 JAN 30	FEB 12	APR 2	JUNE 2	SEPT 3	OCT 4	DEC 1	17	1970 FEB 6	23
2	+0.048	-0.078	-0.528	-	-0.324	-0.396	-1.758	-2.112	-3.198	-3.450
4	+0.072	-0.078	-0.510	-	-0.390	-0.408	-1.938	-2.154	-3.054	-3.312
6	+0.036	-0.084	-0.522	-0.474	-0.486	-0.516	-2.058	-2.292	-3.000	-3.228
8	+0.006	-0.090	-0.534	-0.552	-0.576	-0.612	-2.172	-2.412	-2.976	-3.162
10	-0.024	-0.090	-0.552	-0.612	-0.642	-0.684	-2.262	-2.514	-3,006	-3.180
12	-0.060	-0.120	-0.588	-0.672	-0.702	-0.744	-2.358	-2.616	-3.096	-3.270
14	-0.060	-0.126	-0.600	-0.690	-0.720	-0.768	-2.412	-2.682	-3.174	-3.354
16	-0.072	-0.138	-0.624	-0.714	=0.750	-0.792	-2.490	-2.784	-3.306	-3.492
18	-0.078	-0.144	-0.636	-0.732	-0.774	-0.816	-2.562	-2.874	-3.414	-3.612
20	-0.084	-0.150	-0.642	-0.744	-0.798	-0.834	-2.604	-2.934	-3.486	-3.684
22	-0.090	-0.156	-0.648	-0.762	-0.882	-0.852	-2.634	-2.976	-3.522	-3.726
24	-0.102	-0.162	-0.660	-0.780	-0.846	-0.876	-2.652	-3.006	-3.540	-3.756
26	-0.102	-0.168	-0.666	-0.786	-0.858	-0.888	-2.676	-3.024	-3.570	-3.786
28	-0.108	-0.180	-0.672	-0.792	-0.870	-0.894	-2.700	-3.048	-3.594	-3.810
30	-0.108	-0.186	-0,684	-0.804	-0.876	-0.924	-2.706	-3.060	-3.606	-3.822
32	-0.120	-0.192	-0.696	-0.816	-0.900	-0.948	-2.748	-3.108	-3.654	-3.876
34	-0.126	-0.198	-0.696	-0.810	-0.894	-0.936	-2.664	-3.018	-3.570	-3.774
36	-0.036	-0.042	-0.198	-0.256	-0.282	-0.276	-0.870	-0.990	-1.140	-1.194
38	- 0	+0.012	-0.006	-0.006	-0.024	+0.012	-0.036	-0.042	-0.054	-0.054

Note: Slope indicator was installed January 13, 1969
and failed after February 23, 1970.

TABLE XVIII

TACHE AVENUE SLOPE INDICATOR NO. 1

DEFLECTION IN INCHES

DEPTH (FT.)	NORTHWARD 1969 DEC 8	16	EASTWARD 1969 DEC 8	16	24	1970 FEB 6	23	JUL 31	AUG 14	28	SEPT 14	25	OCT 13	29	NOV 12
2.2	0.036	0.138	-0.342	-0.732	-1.056	-1.752	-1.992	-3.120	-3.072	-3.054	-3.048	-3.372	-3.390	-3.396	-3.396
4.2	0.054	0.150	-0.360	-0.774	-1.134	-1.842	-2.070	-3.270	-3.296	-3.240	-3.234	-3.474	-3.510	-3.510	-4.050
6.2	0.066	0.174	-0.384	-0.816	-1.212	-1.938	-2.154	-3.426	-3.426	-3.438	-3.426	-3.588	-3.624	-3.624	-4.170
8.2	0.054	0.156	-0.414	-0.852	-1.260	-2.004	-2.220	-3.564	-3.564	-3.570	-3.570	-3.714	-3.744	-3.738	-4.290
10.2	0.060	0.162	-0.438	-0.888	-1.308	-2.088	-2.316	-3.708	-3.708	-3.714	-3.720	-3.846	-3.876	-3.864	-4.428
12.2	0.072	0.168	-0.462	-0.930	-1.362	-2.202	-2.436	-3.882	-3.876	-3.882	-3.894	-4.008	-4.038	-4.020	-4.602
14.2	0.072	0.174	-0.480	-0.984	-1.440	-2.346	-2.592	-4.122	-4.116	-4.110	-4.128	-4.224	-4.254	-4.242	-4.830
16.2	0.078	0.186	-0.504	-1.044	-1.536	-2.538	-2.802	-4.422	-4.404	-4.404	-4.416	-4.494	-4.518	-4.524	-5.118
18.2	0.078	0.192	-0.498	-1.032	-1.518	-2.502	-2.760	-4.350	-4.326	-4.326	-4.332	-4.416	-4.434	-4.434	-5.046
20.2	0.066	0.162	-0.456	-0.942	-1.386	-2.250	-2.490	-3.882	-3.864	-3.870	-3.882	-3.942	-3.954	-3.966	-4.530
22.2	0.012	0.066	-0.318	-0.660	-0.984	-1.542	-1.716	-2.634	-2.610	-2.628	-2.640	-2.652	-2.682	-2.682	-3.066
24.2	-0.018	-0.006	-0.132	-0.288	-0.456	-0.660	-0.738	-1.092	-1.074	-1.092	-1.122	-1.098	-1.134	-1.110	-1.284
26.2	-0.012	-0.018	0.018	0.036	0	0.072	0.072	0.234	0.234	0.222	0.186	0.234	0.198	0.234	0.216
28.2	0.006	0.012	0.048	0.078	0.060	0.150	0.162	0.342	0.318	0.324	0.288	0.336	0.312	0.336	0.330
30.2	0.012	0.024	0.036	0.066	0.048	0.114	0.132	0.276	0.246	0.252	0.234	0.270	0.240	0.270	0.264
32.2	-0.006	0.006	0.024	0.042	0.030	0.060	0.078	0.180	0.156	0.162	0.150	0.174	0.144	0.174	0.162
34.2	-0.006	0	0.006	0.012	0.006	0.018	0.024	0.078	0.060	0.066	0.060	0.072	0.066	0.072	0.072

Note: Slope indicator was installed Dec. 1, 1969 and failed after Nov 12, 1970 by becoming blocked (pinched off) at the 25 foot depth.

TABLE XIX
TACHE AVENUE SLOPE INDICATOR NO. 2
DEFLECTION IN INCHES

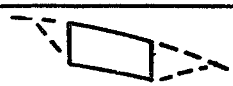
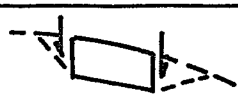
DEPTH (FT.)	NORTHWARD 1969 DEC 8	16	EASTWARD 1969 DEC 8	16	24	1970 FEB 6	23	APR 27	MAY 1	7
1	0.003	0.384	-0.516	-1.368	-2.064	-3.390	-3.822	-5.178	-5.760	-6.060
3	-0.003	0.276	-0.402	-1.116	-1.680	-2.802	-3.240	-4.096	-4.440	-4.626
5	-0.021	0.168	-0.300	-0.870	-1.308	-2.220	-2.634	-3.018	-3.114	-3.186
7	-0.033	0.060	-0.204	-0.588	-0.900	-1.554	-1.936	-1.914	-1.830	-1.734
9	-0.057	0.006	-0.048	-0.174	-0.258	-0.444	-0.696	-0.798	-0.768	-0.690
11	-0.057	-0.012	0.036	0.048	0.096	0.150	-0.024	-0.120	-0.114	-0.042
13	-0.057	-0.012	0.060	0.114	0.210	0.330	0.180	0.150	0.150	0.222
15	-0.051	0	0.048	0.090	0.186	0.288	0.138	0.156	0.168	0.240
17	-0.057	0.006	0.024	0.042	0.120	0.180	0.024	0.042	0.072	0.150
19	-0.057	-0.006	0.018	0.012	0.084	0.108	-0.042	-0.024	0	0.072
21	-0.048	-0.006	0.018	-0.012	0.060	0.072	-0.072	-0.066	-0.048	0.024
23	-0.036	0	0.012	-0.012	0.042	0.054	-0.078	-0.078	-0.066	0.006
25	-0.018	0.024	0.012	-0.018	0.018	0.030	-0.078	-0.078	-0.066	0.006
27	-0.006	0.042	0.024	-0.018	0.012	0.036	-0.060	-0.060	-0.048	0.024
29	-0.006	0.036	0.036	-0.006	0.036	0.060	-0.036	-0.036	-0.024	0.042
31	-0.006	0.030	0.042	0.006	0.054	0.078	-0.012	-0.012	-0.006	0.054
33	-0.012	0.024	0.048	0.012	0.054	0.084	0	0	0.006	0.060
35	-0.012	0.018	0.048	0.012	0.048	0.078	0	-0.006	0.006	0.048
37	-0.012	0.012	0.048	0	0.042	0.066	-0.006	-0.024	-0.012	0.024
39	-0.018	0	0.030	-0.006	0.024	0.036	-0.018	-0.024	-0.018	0.006
41	-0.030	-0.012	0.018	-0.006	0.012	0.024	-0.024	-0.030	-0.024	-0.006
43	-0.024	-0.006	0	-0.006	0	0.006	-0.024	-0.030	-0.012	-0.012
45	-0.006	0	-0.006	-0.006	-0.006	-0.006	-0.018	-0.030	-0.006	0

DEPTH (FT.)	1970 MAY 22	JUNE 12	JULY 31	EASTWARD AUG 14	28	SEPT 14	25	OCT 13	29	NOV 12	19
1	-6.492	-6.606	-7.218	-7.098	-7.128	-7.470	-7.980	-8.058	-8.004	-8.880	-11.100
3	-4.914	-4.962	-5.478	-5.370	-5.394	-5.598	-6.060	-6.012	-6.054	-6.744	-8.874
5	-3.348	-3.330	-3.762	-3.648	-3.702	-3.738	-4.152	-4.152	-4.116	-4.620	-6.594
7	-1.776	-1.818	-2.046	-1.932	-1.980	-1.872	-2.244	-2.190	-2.172	-2.496	-4.170
9	-0.732	-0.756	-0.888	-0.774	-0.828	-0.606	-0.930	-0.906	-0.864	-0.948	-1.374
11	-0.078	-0.120	-0.210	-0.084	-0.150	-0.102	-0.186	-0.174	-0.138	-0.120	-0.030
13	0.186	0.144	0.060	0.180	0.120	0.354	0.084	0.090	0.144	0.132	0.276
15	0.210	0.174	0.084	0.198	0.138	0.354	0.096	0.096	0.150	0.114	0.180
17	0.138	0.108	0.024	0.120	0.060	0.270	0.018	0.018	0.072	0.024	0.012
19	0.063	0.024	-0.048	0.030	-0.030	0.168	-0.054	-0.054	-0.012	-0.060	-0.060
21	0	-0.024	-0.108	-0.048	-0.078	0.114	-0.090	-0.096	-0.060	-0.108	-0.102
23	-0.024	-0.048	-0.120	-0.066	-0.096	-0.078	-0.108	-0.114	-0.078	-0.126	-0.120
25	-0.036	-0.060	-0.114	-0.078	-0.102	-0.060	-0.114	-0.120	-0.072	-0.126	-0.126
27	-0.024	-0.042	-0.096	-0.066	-0.084	0.066	-0.096	-0.114	-0.060	-0.108	-0.114
29	0.006	-0.012	-0.060	-0.048	-0.060	0.084	-0.066	-0.078	-0.036	-0.066	-0.066
31	0.030	0.006	-0.036	-0.030	-0.036	0.090	-0.036	-0.042	-0.012	-0.030	-0.030
33	0.036	0.018	-0.024	-0.012	-0.024	0.096	-0.102	-0.030	-0.006	-0.012	-0.006
35	0.030	0.018	-0.018	-0.012	-0.024	0.084	-0.012	-0.024	0	-0.006	0
37	0.012	0	-0.030	-0.024	-0.048	0.054	-0.024	-0.024	-0.018	-0.012	-0.006
39	0.006	-0.012	-0.048	-0.042	-0.060	0.018	-0.030	-0.036	-0.036	-0.030	-0.024
41	-0.006	-0.012	-0.054	-0.030	-0.060	0	-0.018	-0.048	-0.042	-0.030	-0.024
43	-0.006	-0.018	-0.042	-0.024	-0.048	-0.006	-0.012	-0.042	-0.036	-0.024	-0.024
45	-0.006	-0.024	-0.024	-0.030	-0.024	-0.006	-0.012	-0.030	-0.024	-0.018	-0.018

Note: Slope indicator was installed Dec 1, 1969 and failed after Nov 19, 1970 by becoming blocked (pinched off) at the 8 foot depth.

TABLE XX

DEVIATION IN RESULTS OF ANALYSES ASSUMING
VARIOUS DISTRIBUTIONS OF TOTAL WEIGHT BETWEEN BLOCKS

CASE	ASSUMED DIRECTION OF VERTICAL FORCES ON SIDES OF CENTER BLOCK	ST. VITAL SITE ADJUSTED BLOCK WEIGHTS, KIPS			$\Delta\Phi'_A$*, DEGS.	TACHE AVENUE SITE ADJUSTED BLOCK WEIGHTS, KIPS			$\Delta\Phi'_A$, DEGS.
		W_1	W_2	W_3		W_1	W_2	W_3	
ORIGINAL		134	372	185	0.0	61.4	109.0	85.0	0.0
1		121	404	166	0.7	55.3	123.6	76.5	0.1
2		121	366	204	1.4	55.3	106.6	93.5	1.7
3		147	340	204	0.7	67.5	94.4	93.5	0.1
4		147	378	166	1.4	67.5	111.4	76.5	1.9

* NOTE: $\Delta\Phi'_A$ is the deviation in Φ'_A from that found using unadjusted (original) block weights.

APPENDIX E. DEFINITIONS OF TERMS USED

DEFINITIONS OF TERMS USED

The following terms are commonly used in the field of soil engineering.

Cohesion That component of shearing resistance which is independent of the normal forces acting on a shear plane.

Direct Shear Test A strength test in which the soil specimen is enclosed in a split box and is forced to shear along the plane dividing the two halves of the box.

Effective Stress The component of stress which is transmitted through the soil skeleton by means of grain-to-grain contact. (a simplified definition)

Failure Zone The soil mass which moves as the result of a slope failure.

Friable Having the property of being readily crumbled into small nuggets.

Hydrostatic Force The force exerted on a soil surface by free water.

Intergranular Frictional Force The force transmitted through frictional interaction of soil grains due to an induced tendency to shear.

Neutral Force The force exerted on a soil by its pore water.

Neutral Stress The component of stress which is transmitted through the water contained between soil particles.

Nuggety Consisting of discrete small polygonal fragments.