THE UNIVERSITY OF MANITOBA

A REVIEW STUDY OF RIVER BANK STABILITY AND SLOPE MOVEMENTS IN THE WINNIPEG AREA

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

WINNIPEG, MANITOBA

OCTOBER, 1975

SUMMARY

The study of previous investigations of riverbank stability in the Winnipeg area was extended using up-to-date field instrumentation data. A review of earlier investigations and approaches, and comparisons of slope stability analyses results with previous investigations were included. In the present study, residual strength parameters were used in effective stress analyses, using the simplified Bishop's Method of Slices for a circular slip surface and a modified rigorous Janbu solution for a non-circular failure surface. For both methods, the porewater pressure distributions were incorporated in a computer program as piezometric levels. Improved results of factors of safety computations were obtained and values of computed safety factors for the given porewater pressure conditions were more reasonable.

A preliminary study on riverbank movement based on slope indicator data obtained from the sites was also made. Observations of the slope movement data indicated that the rate of slope movement varied according to the groundwater conditions and reflected the state of riverbank stability.

ACKNOWLEDGEMENTS

The author wishes to express his gratitude to the advisor Professor A. Baracos of the University of Manitoba for his guidance, valuable suggestions and patience during the preparation of this thesis. Grateful thanks are given to Dr. L. Domaschuk of the University of Manitoba for the helpful discussions and his comments on the thesis. Mr. R. A. Van Cauwenberghe, P. Eng. is also thanked for reviewing the thesis as the External Examiner of the thesis committee.

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LIST OF SYMBOLS

b = width of slice

c' = cohesion intercept

C'_R = effective residual cohesive strength

C_u = undrained cohesive strength

E_n, E_{n+1} = horizontal interslice forces of the n th, $n+1$th slice

F = factor of safety

I_L = liquidity index

$L = b \sec\alpha$ = length of base of slice

P = total reactive force acting normal to base of slice

P' = effective reactive force acting normal to base of slice

r_u = pore-pressure ratio

S_m = mobilized shear strength

T_n, T_{n+1} = vertical interslice force of the n th, $n+1$th slice

U = total water force

u = unit porewater pressure

α = angle between failure surface and horizontal

β = slope angle

ϕ = undrained frictional angle

ϕ' = effective angle of shearing resistance

ϕ'_R = effective residual angle of friction

A REVIEW STUDY OF RIVER BANK STABILITY AND SLOPE MOVEMENTS IN THE WINNIPEG AREA

CHAPTER 1

INTRODUCTION

The flooding hazard of the Red River and the slope stability of its banks in Metropolitan Winnipeg area have been of major concern to urban development and construction in the Winnipeg area. The flooding hazard has been alleviated in recent years by the construction and operation of the Red River Floodway, which diverts the flood waters and greatly reduces the peak flood level. Nevertheless, the existence of the unstable slopes remains and renders river bank construction difficult or impossible.

It has been realized that natural slopes represent the ultimate long term equilibrium state of a profile formed by geological process (BISHOP and BJERRUM[1]). A characteristic feature of a natural slope is that the pore pressures show values corresponding to certain groundwater conditions, and that changes in pore pressures are affected by both extreme and relatively small seasonal variations (BJERRUM and KJAERNSLI[2]).

1.1 SCOPE OF STUDY.

The field instrumentations installed by the Civil Engineering Department, University of Manitoba, and the

Manitoba Water Control and Conservation Branch, at two selected riverbank sites were used for monitoring the porewater pressure and riverbank movements. The instrumentations have provided necessary data for slope stability analyses and a good indication of the stability under field conditions. In the study, knowledge of the actual porewater pressure distribution made it possible to use the effective stress approach.

The porewater pressure data collected in this study during three consecutive years from June 1969 to November 1972, were utilized in determining the groundwater regimes of the river banks. The location and configuration of existing failure surfaces were estimated from the slope indicator data. The rate of slope movement was also measured.

The Janbu and Bishop's simplified methods were employed in the slope stability analyses for these riverbanks.

1.2 SOIL CONDITIONS.

The geological origin and the soil profile in the Metropolitan Winnipeg area and common to the Red River Valley, have been described in detail in previous papers (MACDONALD[3], RIDDEL[4], BARACOS[5]). The soil profile consists of topsoil, clayey silt, tan silt, clay, and "hardpan". The clay stratum, consists predominantly of highly plastic, lacustrine laminated clay deposits of the glacial Lake Agassiz. A further breakdown of the clay deposits reveals the occurrence of three types of clay layers in sequence of increasing depth, namely, the brown clay, "mixed" clay, and the grey clay.

A six to twelve-inch layer of highly organic topsoil overlies the clay-silt layer which ranges from several inches to several feet in thickness. A thin layer of tan silt lies between the clay-silt layer and the clay stratum. The tan silt and clayey silt layers are frost susceptible and have been subjected to weathering. The shallow deposits have been preconsolidated by desiccation, which has also produced fissures. Occasionally, soil preconsolidated by desiccation may be found to a depth of about thirty feet below ground surface.

The brown clay layer, locally known as "chocolate" clay, usually has easily discernible laminations except in the Winnipeg area where it is thinly laminated (ELSON[6]). QUIGLEY[7] reported that the percentage of clay size particles decreased from 80% to 60% with depth and contained about 80% monmorillonite-illite within the clay size fraction. BARACOS and BOZOZUK[8] estimated that the clay fraction consists of approximately 30% monmorillonite, and the remainder is practically all illite.

The intermediate "mixed" layer of mottled brown and grey silty clay forms a transition zone between the brown and grey clays and is not always encountered.

The bottom layer of softer and siltier "grey" clay is less layered, and is normally underlain by a shallow layer of soft non-cemented till material. Pockets of silt, and pebbles can sometimes be found in this layer.

The uncemented till material sometimes mixes with the overlying few feet of grey clay, and has an unconfined compressive strength less than 1000 lbs per sq ft.

The dense till layer consisting of silt, coarse sand, and pebbles, underlies the grey clay or the uncemented till material. This layer is hard and cemented, and is commonly known as "hardpan". Its thickness varies from several feet to a maximum of sixteen feet and it is underlain by limestone bedrock. Occasionally, the till layer may be absent and the grey glacial clay is underlain by bedrock.

CHAPTER 2

LONG TERM STABILITY OF RIVERBANKS

2.1 PREVIOUS INVESTIGATIONS.

Various stability analyses have been made by previous investigators (JANZEN[9], VAN CAUWENBERGHE[10]) of the riverbanks under study in order to assess a realistic and reliable analytical method. The uncertainties arising from several unknown factors rendered some earlier approaches less successful. The problems have been gradually solved with the availability of the current data related to the uncertainties, namely, the actual porewater pressure distribution, shear strength parameters and the slip surface configurations.

The common method of analysis of slopes used to be the total stress ($\phi = 0$) method, where the available shearing strength was assumed equal to one-half the unconfined compressive strength. SUTHERLAND[11] reported that the calculated factor of safety was substantially over-estimated when applying the undrained strength to both the short term and the long term conditions. BARACOS[5] and MISHTAK[12] reported that for failed slopes in Winnipeg area, the factors of safety usually had values greater than unity, using the total stress analysis. Through a rational and semi-empirical approach, both BARACOS[5] and SUTHERLAND[11] indicated that a reduced value of the undrained shearing strength based on laboratory testing had to be adopted to give reliable factors of safety. BARACOS[5] suggested the use of a lower shearing strength of about 300 to 500 lbs per sq ft for estimated slip surface passing through the old failure zone.

Effective stress analysis is an alternative method for checking the riverbank stability, provided that, the proper choice of shear strength parameters and the groundwater conditions have been incorporated in the analysis. SUTHERLAND[11] found that in applying effective stress analysis to short term condition of slopes in Winnipeg, the factors of safety were also over-estimated. The present riverbank sites have been studied by JANZEN[9] and VAN CAUWENBERGHE[10] using porewater pressure and slope movement data obtained from field instrumentation. Direct shear tests of Winnipeg clays were conducted by SUTHERLAND[13], MUIR[14], and JANZEN[9] in assessing both peak and residual shear strength parameters. Residual strength parameters were used to account for the visible and detectable old slip scars. In his study, JANZEN[9] developed a simplified analysis using a three-block system and used the piezometric readings for water force calculations. VAN CAUWENBERGHE[10] assumed a phreatic surface for the critical condition and incorporated the porewater pressure calculation in the conventional and rigorous methods of analysis. Slip surface configurations were determined from points of known slip established with the slope indicators. In both Janzen's and Van Cauwenberghe's investigations, computer programs developed by the investigators facilitated computations in the analysis. Although the factors of safety were slightly over-estimated and in excess of unity for actual failure, the results obtained by these methods of analyses can be used as good approximations for actual field conditions, as they were based on instrumentation data. The discrepancies in results may be attributed to the inadequate data collected over a short period of time.

2.2 LIMITATIONS OF PREVIOUS APPROACHES.

When applying various classical methods of analysis to river banks in the Winnipeg area, the results were sometimes misleading, showing relatively high factors of safety. The methods adopted were basically circular arc methods, such as Taylor's friction circle method and methods of slices, which were used in conjunction with both total stress analysis and effective stress analysis.

(A) TOTAL STRESS ANALYSIS.

In applying this approach, the undrained strength parameters for the existing field conditions should be determined by laboratory testing. The $\phi = 0$ analysis is based on the assumption that the soil behaves as a perfectly cohesive material (TAYLOR[15]). This concept is valid according to SKEMPTON[16] for fully saturated clays, for which the angle of shearing resistance is zero when there is no water content change under the applied stress. However, partially saturated clays do not exhibit an angle of shearing resistance equal to zero when tested under condition of no overall water content change. Skempton also pointed out that the no water content change was only true for the immediate end-of-construction stage, and in time, the shear strength of the clay would progressively alter from that used in a total stress analysis. For natural slopes, the general tendency may be a reduction in strength. In general, the $\phi = 0$ analysis will not lead to a correct prediction of the actual shear surface. The conditions of no drainage are seldom satisfied in natural slopes. Usually the stability analysis requires different strength parameters than those obtained from laboratory tests, in order to obtain a safety factor of unity for a failed slope.

The introduction of a reduced value of the undrained strength, as recommended by BARACOS[5] and SUTHERLAND[11], may give conservative results or may over-estimate the factor of safety because the assumption of the reduced strength value is chosen on the basis of correlation to past failures. If no reduced strength was assumed, the factors of safety would be unreasonably high in stiff clay slopes, and the stability would tend to be under-estimated in soft clay slopes (BJERRUM and KJAERNSLI[2]). The apparent discrepancy between the field behaviour of natural slopes and the analytical results based on shear strength obtained by undrained triaxial tests, in general, has indicated the inappropriateness of the total stress analysis for long-term stability as documented by

BJERRUM and KJAERNSLI[2].

(B) EFFECTIVE STRESS ANALYSIS.

For the effective stress analysis, knowledge of porewater pressure distribution along the assumed failure plane is the key factor in assessing a reliable factor of safety. In many practical engineering situations, in-situ effective stress is calculated by assuming hydrostatic conditions below the water table. This is not the case when there is seepage within the soil mass, such as in natural slopes. Underestimating the porewater pressure will yield a factor of safety on the unsafe side. As the porewater pressure is an independent variable in natural slopes (PECK[17]), a preferred method of collecting data on the actual porewater pressure distribution is by installing piezometers.

Choice of shear strength parameters is another determining factor in the analysis. The values of the shear strength parameters may fall within the limits defined by the peak shear strength and the residual or ultimate shear strength. The maximum shear strength developed during the process of deformation is called the peak shear strength. At large displacements, the minimum constant value retained by the soil is called the residual shear strength. The difference between the peak and residual values of shear strength for over-consolidated clays is usually quite significant, whereas, the peak and residual shear strengths for normally-consolidated clays are essentially the same (Figure 1).

For peak strength values, the parameters can be evaluated by means of triaxial compression or direct shear tests on undisturbed soil samples. Direct shear tests should be used to determine the residual strength parameters because these tests permit the large strains required. To adopt peak or residual strength parameters for analysis depends on whether loss of strength with time has occurred on the slope or not. Based on extensive field studies and laboratory test results, SKEMPTON[18] gave an account of the reduction of soil strength to residual value for the long-term case and proposed several possibilites causing this phenomenon. The peak and residual strength envelopes based on drained direct shear tests are shown in Figure 2.

2.3 STABILITY OF NATURAL SLOPES.

It is often extremely difficult to determine how much shear strength is actually available within the slope whether total stress or effective stress analysis is used.

It was observed by PECK[17] that for the undrained analysis, an empirical reduction factor related to the liquidity index could be applied to the unconfined compressive strength to be used in the stability analysis. The reduction factor is applied as follows (notations defined after Table 2):

$$C_u \text{ (reduced)} = C_u \times \frac{I_L}{100} \times \text{Reduction Ratio}$$

In which, Reduction Factor = $\frac{I_L}{100} \times$ Reduction Ratio

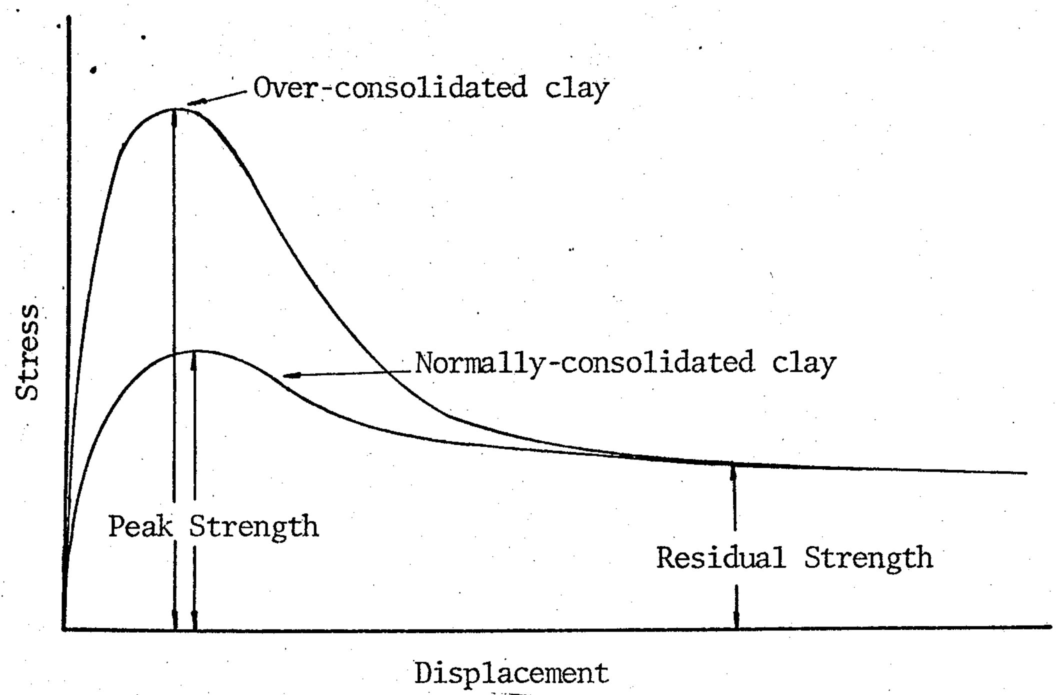


Figure 1 Stress-Strain Curves for Cohesive Soils

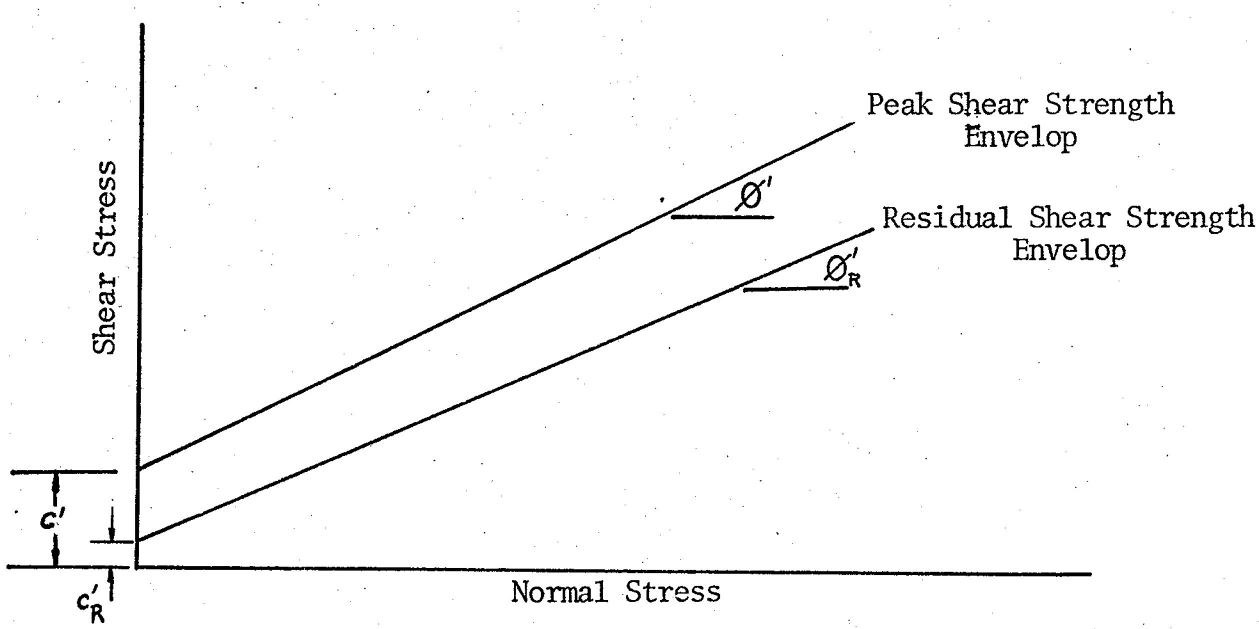


Figure 2 Mohr-Coulomb Shear Strength Envelopes Showing Peak and Residual Conditions

(After SKEMPTON[18])

TABLE 1

PROPERTIES OF GREATER WINNIPEG GLACIAL LAKE CLAYS

(AFTER BARACOS[5])

	Brown "Chocolate" Clay				Mixed Brown and Grey Clay				Grey "Blue" Clay			
	Max.	Ave.	Min.	No. of Tests	Max.	Ave.	Min.	No. of Tests	Max.	Ave.	Min.	No. of Tests
Depth to top of stratum - ft.	16	1	2	147	28	20	6	176	35	25	15	154
Depth to bottom of stratum - ft.	40	25	11	147	35	25	8	176	62	45	15	154
Moisture Content - %	57	48	27	76	63	56	31	57	61	41	27	44
Dry Density - p.s.f.	99	77	64	73	87	69	53	51	102	79	63	39
Moist Density - p.c.f.	125	109	95	83	114	108	98	51	130	112	101	42
Saturation - %	100	97	86	73	100	98	89	50	100	98	89	32
Unconfined Compression Strength - p.s.f.	4750	2054	865	87	3790	2169	112	49	3570	2182	1188	44
Plastic Limit	40	30	14	36	36	30	26	9	32	25	16	17
Liquid Limit	117	89	37	36	110	93	70	9	95	76	37	17
Plasticity Index	88	59	23	36	75	63	51	9	68	50	20	17

TABLE 2

REDUCTION FACTORS FOR UNCONFINED COMPRESSIVE STRENGTH BASED ON LIQUIDITY INDEX TO PERMIT STABILITY ANALYSIS (WINNIPEG CLAYS)

		BROWN "CHOCOLATE" CLAY	MIXED BROWN AND GREY CLAY	GREY "BLUE" CLAY
Based on average C_u	Reduction Ratio for C_u = 500 lb per sq ft	1.60	1.12	1.46
	Average C_u (p.s.f.)	1027	1035	1091
	Average I_L (%)	30.5	41.3	31.4
	Reduction Factor	0.488	0.461	0.458
Based on highest C_u	Reduction Ratio for C_u = 500 lb per sq ft	0.773	0.578	0.611
	Highest C_u (p.s.f.)	2285	1895	1785
	Highest I_L (%)	28.3	36.5	45.8
	Highest Reduction Factor	0.262	0.264	0.280

In which,

C_u = Undrained Shear Strength, lb per sq ft

I_L = Liquidity Index = $\frac{w_n - w_p}{w_l - w_p}$, in percentage

where,

w_n = Natural water content (average moisture content as shown in Table 1)

w_l = Liquid Limit (Average value as shown in Table 1)

w_p = Plastic Limit (Average value as shown in Table 1)

Based on the data compiled by BARACOS[5] (Table 1), the above empirical relation may be difficult to apply to the Winnipeg clays owing to the large range of values of unconfined compressive strength of the Winnipeg clays. Based on a reduced undrained strength of 500 lb per sq ft, which yields a safety factor of unity, the corresponding reduction factors are given in Table 2 for the clays, properties of which are listed in Table 1. Reduction factors were computed based on the average undrained strengths and on the maximum undrained strengths. The application of these reduction ratios would only be warranted for clays the strength of which fell between the maximum and minimum given in Table 1. If the field strengths of the soil samples for Winnipeg clay were significantly below the average value listed in Table 1, it would be preferable to use the recommended reduced undrained strength of 400 lb per sq ft rather than applying the reduction factors.

Taking into account the fact that the major portion of failure surfaces of slopes in Winnipeg pass through the relatively deep-seated grey clay layer, the average range of strengths for the entire assumed failure surface could be taken as that range of values for the grey clay layer with little error. This assumption is justified because the average values of the reduction factors for that small portion of failure surface passing through the upper brown clay and mixed clay layers are approximately the same as the average range for grey clay layer.

However, the above proposed range of reduction factors is dependent on the liquidity index, which decreases significantly with increasing stiffness of the soil, and extensive tests have to be carried out to confirm an empirical relationship and update the data listed in Table 1.

BISHOP[19] proposed that the difference between field strength and that of small samples was attributed partly to the difference in stress conditions between laboratory testing and in nature; partly to the difference in the time to failure; and partly to the anisotropy of the materials. However, the largest difference was considered to be a result of the small size of the samples. Other findings include the series of test results carried out by R. J. Conlon on the glacial lake clays from Winnipeg, which were reviewed by PECK[17] in his Terzaghi lecture. The phenomenon of the dependence of strain at peak strength on the normal pressure was associated with the nature of progressive failure.

Even in the effective stress analysis, there is no satisfactory way of predicting the average strength that could be mobilized between the limiting values of the peak strength and the residual strength. Failure in natural slopes is progressive in nature, and the strength that could be developed in the sliding mass could not be as high as the peak value along the entire surface of sliding, nor would the strength be as low as the residual value (SKEMPTON[18]). However, Skempton pointed out that the residual value had proved to give a correct estimate of the factor of safety in some instances, such as weathered clay slopes with fissures or joints. If a failure has already occurred, any subsequent movements on the existing slip surface will be controlled by the residual strength, no matter what type of clay is involved (SKEMPTON[18]). The complexity of the variation of the soil strength in natural slopes has not yet been fully understood.

On the other hand, variation of porewater pressure distribution could be measured by installing instrumentation to evaluate the long term equilibrium of the natural slopes. Previous investigations (JANZEN[9] and VAN CAUWENBERGHE[10]) of the riverbank sites, and the present study of the piezometric readings have indicated the groundwater flow pattern in Winnipeg river banks. These patterns varied with time of the year, and they have the following characteristics (VAN CAUWENBERGHE[10]):

(a) During drawdown periods and winter, the major component of the hydraulic gradient within the riverbank was downward.

(b) During spring inundation and summer, the major seepage flow was outward and downward from the river into the glacial till layer.

It can be seen that for natural slopes in Winnipeg, the fluctuating groundwater and river conditions affect the porewater pressure distribution, indicating that the distribution never corresponds to hydrostatic condition or steady seepage cases. This is in contrast to many cases found in natural slopes where the porewater pressures would have already reached an equilibrium pattern which can be ascertained from piezometer measurements in the field.

CHAPTER 3

FIELD INSTRUMENTATION

3.1 PIEZOMETERS.

The St. Vital site on the east bank of the Red River, is situated in St. Vital, a suburb of the City of Winnipeg. The site had instrumentation monitoring porewater pressure measurements for ten years dating back to 1962. Five Casagrande type porous tube piezometers, installed by the Manitoba Water Resources Branch, were placed at different levels in two boreholes (Figure 6). Several piezometers were placed at various depths in one borehole for space and economy reasons, using a slightly larger borehole diameter. Unfortunately, these piezometers were placed too far away from the edge of the river to be of much value in the stability analysis.

For the study of the St. Vital riverbank site by VAN CAUWENBERGHE[10], eight more pneumatic piezometers were installed in 1968. At the bank at Tache Avenue site (St. Boniface area in Winnipeg) studied by JANZEN[9], ten pneumatic type piezometers were installed. The pneumatic piezometers were installed because of their advantages over the Casagrande "open" type piezometers as will be presented later. In order to appreciate these advantages, a brief description of the structures of these two types of piezometers is given below.

The Casagrande type porous tube piezometer (HANNA[20]) consists of a porous element, usually referred to as a tip or well point, which is connected to a riser pipe and is placed in a layer of sand or gravel filter to make it porous. The depth to water in the piezometer is determined by an electrical probe.

The well-point equipped pneumatic type Thorpiezo piezometers (GRIFFIN[21]) employed at both sites uses air under pressure in a hydraulic balance system to provide porewater pressure readings with negligible time lag. The main body of the unit is constructed of acrylic plastic or polythelene. The main working components consist of a rolling diaphragm of neoprene and fibreglass and springs of silicon bronze with baked teflon coating, attached to a teflon disc check or valve. A neoprene O-ring is used for the valve seat. The piezometer unit is corrosion-proof as no exposed metal of any type is used, neither in the instrument nor in any of the external fittings. Nylon tubings are used for the lines connecting the embedded instrument to the ground surface, where the tube leads fitted with connectors may be conveniently coiled and protected in a central metering station. A schematic diagram of the piezometer tip is shown in Figure 3.

The portable control box (shown in Figure 4) of briefcase size manufactured by the Terra Tec Company, is made of light weight aluminium. It contains an air pressure tank and two pressure gauges. The pressure tank in the box is designed to accomodate 2000 lbs per sq in., thus eliminating the need for frequent replenishing of the air supply. The two leads of the

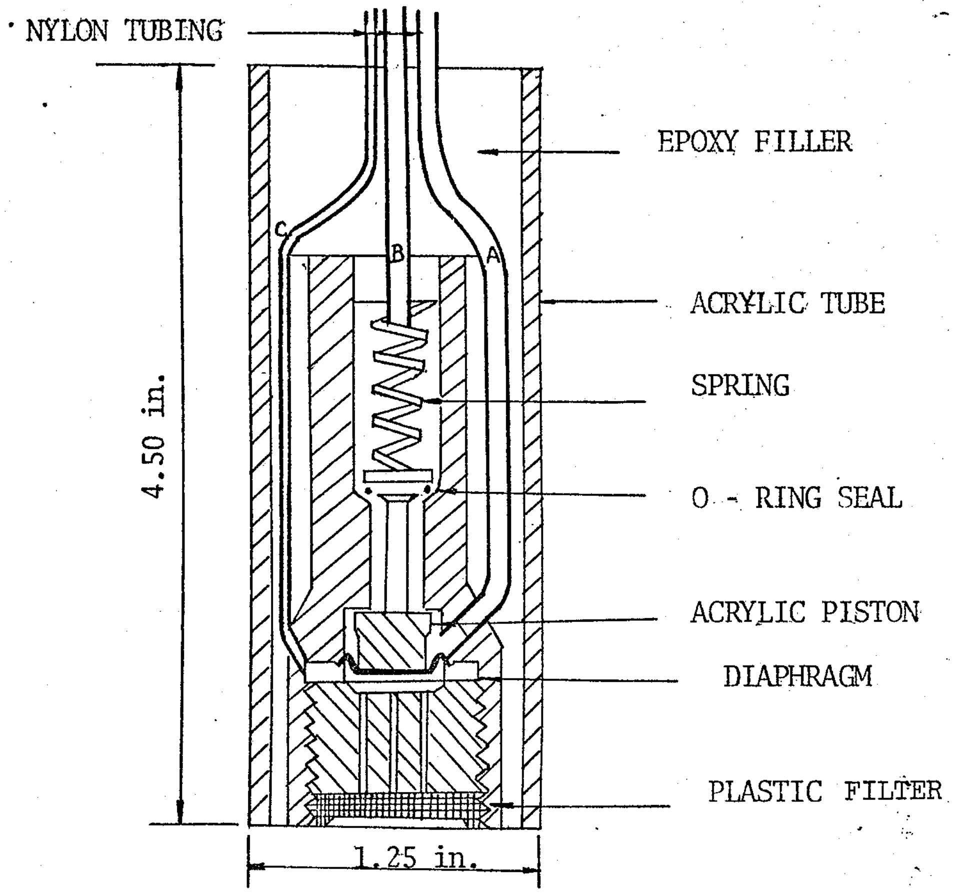


Figure 3 – Schematic Diagram of Air-activated Thorpiezo Piezometer Tip

Figure 4 – Portable Control Box For Air-Activated Piezometers

piezometer at the read-out terminal are easily connected to the control box by quick couplings, and the pressurized air from the control units activates the hydraulic balance system. For this study, helium was used as a drying agent to prevent freezing of moisture in the pneumatic lines.

The operating procedures of the hydraulic balance system have been described in details in the manufacturer's instruction sheet and by previous investigators (JANZEN[9] and VAN CAUWENBERGHE[10]), and are not repeated here.

The advantages of the Casagrande type of piezometer are corrosion-proof construction with non-metallic ceramic tip, and its "open" system which does not require de-airing. It has the following disadvantages:

1. Relatively longer equalization time is required to respond to porewater pressure changes.
2. The piezometer is affected by inundation.
3. The time required for installation.

The advantages of Thorpiezo pneumatic piezometer are ease of installation, convenience of obtaining readings, and minimal maintenance cost. It is superior to the Casagrande "open" type piezometers in performance because of the following features (GRIFFIN[21]):

1. It responds rapidly to porewater pressure fluctuations. For all practical purposes, it is a negligible time lag instrument with nondisplacement of water. (Only in

extremely impervious soils would the minute displacement of water cause any appreciable time lag in developing the actual porewater pressure reading).

2. It is less susceptible to malfunction due to freezing and flooding; the air-actuated system can withstand the severe winter condition, and readings for all lines can be taken at a central read-out terminal which is situated far up the slope and not be interfered with by flooding.

3.2 SLOPE INDICATORS.

Slope indicators, also known as inclinometers, are sensitive instruments for measuring horizontal ground movements. Seven slope indicator tubes, provided by the Civil Engineering Department of the University of Manitoba, have been installed at the two riverbank sites by the Manitoba Water Resources Branch since 1969. The slot-tube type Wilson slope indicators were manufactured by the Slope Indicator Company of Seattle. For each arrangement of the slope indicator installation a grooved aluminum casing with outside diameter of 3.18 inches was anchored to the firm glacial till layer, and observations were made by lowering the slope indicator down the casing with a water-proof cable containing wires and a stranded steel cable to support the probe.

The slope indicator employs a Wheatstone bridge type probe consisting of an electrically activated pendulum, of which the tip makes contact with a special precision-wound resistance coil, subdividing it into two resistances that form one half of the bridge circuit. The other half of the bridge is contained in a portable control box. The instrument (Figure 5) is approximately eighteen inches long and two inches in diameter. It supports two pairs of spring-mounted wheels which travel in the plane of the pendulum along four equally spaced longitudinal grooves, running vertically inside the guide casing or slot tube. In taking observations from the ground, the precision potentiometer contained in the control box indicates readings which are directly proportional to the component of the inclination in the plane of the pendulum. Readings of lateral slope movements are taken in two perpendicular directions, from which the two components of horizontal movement can be determined. The resultant inclination can be calculated by combining these two components. This arrangement of slot tube and slope indicator satisfies the requirements in obtaining accurate observations (TERZAGHI and PECK[22]) such that the slots permit reliable observations of the instrument in successive surveys with the same orientation and at the same point in the casing.

The casings were installed vertically in boreholes drilled to the firm layer for proper seating. The void left between the hole and casing was backfilled by applying bentonite mud around the casing. As the bottom of the casing has been keyed into the dense till layer, the base is virtually free from translation and a reference point of zero movement can be assumed.

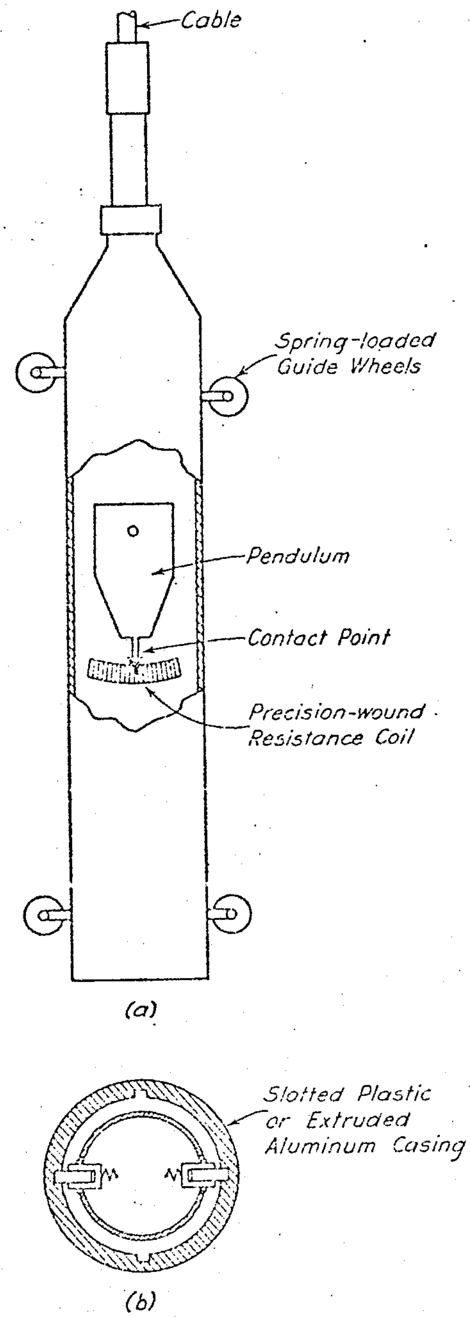


Figure 5 - Diagrammatic sketch of Wilson Slope Indicator. (a) View of instrument. (b) Cross section showing instrument in slotted casing. (After TERZAGHI and PECK[22])

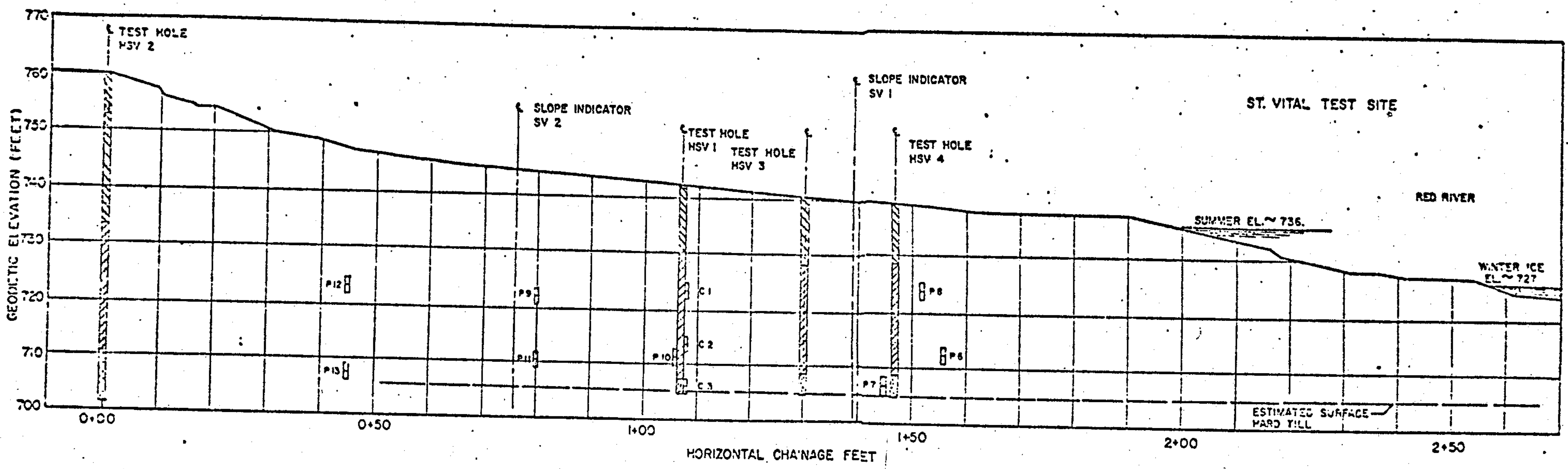
CHAPTER 4

INSTRUMENTATION PERFORMANCE AND OBSERVATIONS

4.1 RIVERBANK CONDITIONS AND GEOHYDROLOGY.

The soil profiles as described in the first chapter are fairly consistent with the general soil conditions in the Winnipeg area. The soil profile of the riverbanks, situated on the concave edges of curves; consists of a shallow layer of topsoil, modified by vegetation and weathering, silty clay, and glacio-lacustrine clay underlain by "hardpan" or bedrock. The varved glacial lake clays (usually with fewer laminations than the glacio-lacustrine clays found in other parts of the Red River Valley outside Winnipeg -- ELSON[6]), are overlain by predominantly silty clay. Disturbance of the glacial clays may have been caused to some extent by previous slides, and by creep which is still taking place. The stability of the river banks is lessened by the constant scouring at the toe of the banks by river currents and by drawdown. Riverbank movement along the old slides due to decrease of effective stress and reduction of shear strength caused by infiltration of rainfall also contributed to the instability.

Both the St. Vital and St. Boniface sites are situated on the concave side of the Red River. The former site has a gentle slope with an over-all slope angle of about six degrees; and the latter site has an over-all slope angle of about twelve



P7 Pneumatic Piezometer

C1 Casagrande Piezometer

Figure 6 - Riverbank Cross-Section, instrumentation (St. Vital Site)
(Courtesy of Civil Engineering Department, University of Manitoba)

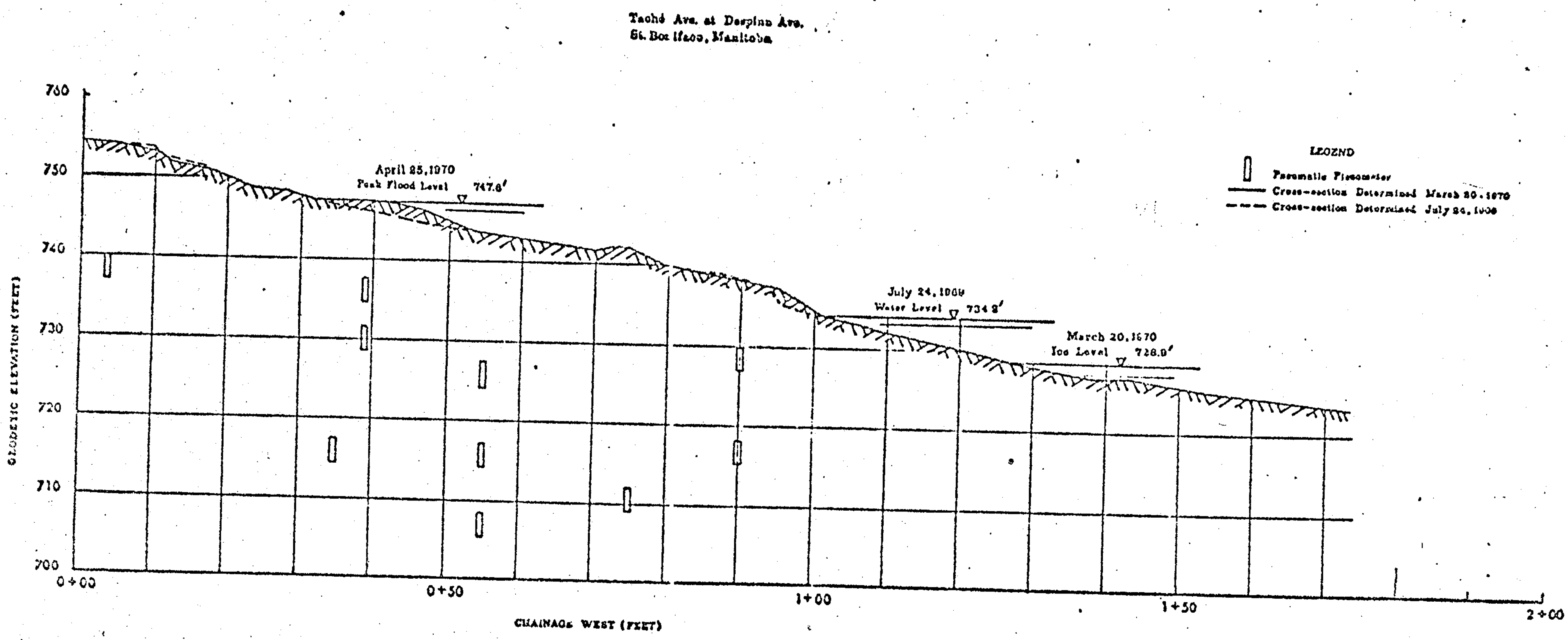


Figure 7 - Riverbank Cross-section, instrumentation (St. Boniface Site No. 1)
(Courtesy of Civil Engineering Department, University of Manitoba)

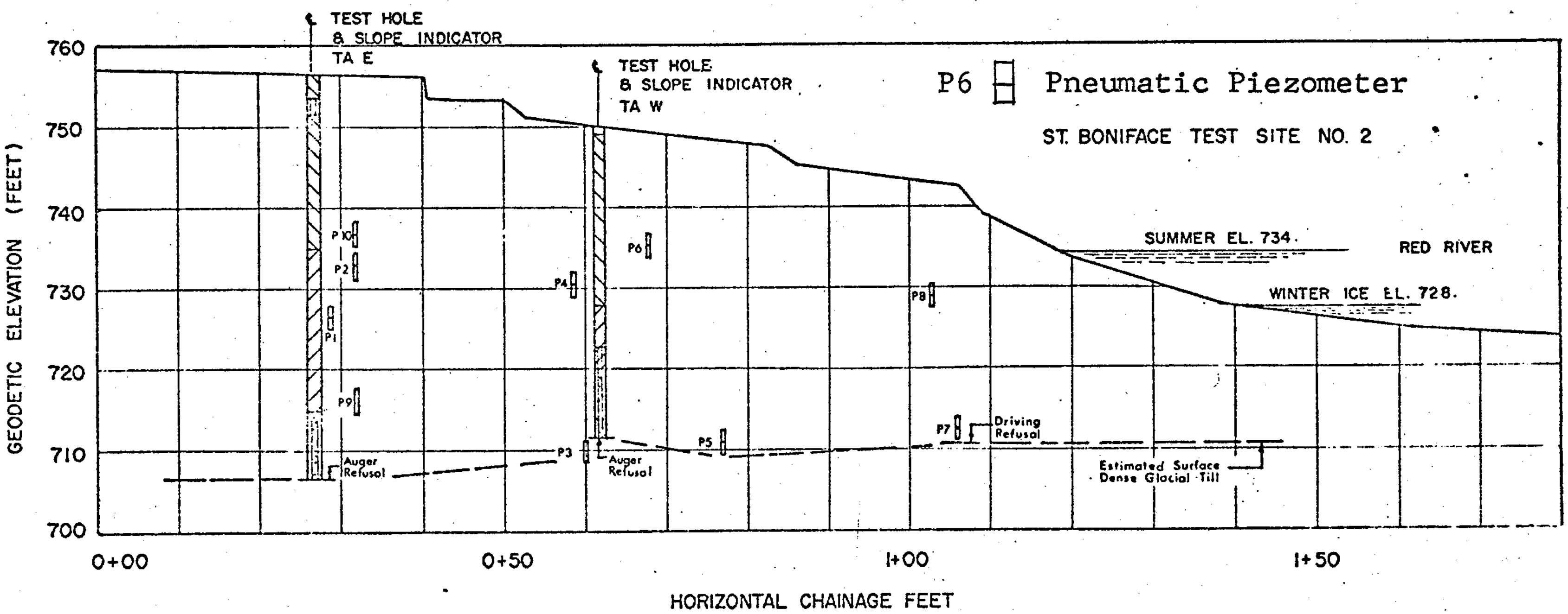


FIGURE 8. – BANK CROSS-SECTION, TEST HOLES, INSTRUMENTATION

(Courtesy of Civil Engineering, University of Manitoba) degrees (JANZEN [9]). These riverbank sites had failed previously and were later partly stabilized. The profile of the riverbank sites, the logs of boreholes and the locations of the field instrumentations are shown in Figures 6, 7, and 8. It is noted that on the St. Vital bank, a thin layer of a more recent siltier deposit of two to eight feet covers the glacial laminated clays. Free water is found at times in the tension cracks or fissures on the site, resulting from percolation of surface water into the upper silt layer. The amount of free water is generally small and prolonged dry weather results in its disappearance (BARACOS [5]).

The geohydrology of metropolitan Winnipeg area was well documented by RENDER [23]. Although his investigation did not indicate the direct connection of the Upper Carbonate aquifer in the limestone bedrock with the glacial till, forming a continuous supply of groundwater, some of his findings provided good correlation with the piezometric levels of the riverbanks. BARACOS [5] pointed out that with the exception of the top six to twelve feet, almost the entire depth of clay generally showed complete saturation indicating a substantial zone of capillary rise. Below the capillarity zone, porewater pressures in the saturated clay layer, especially between the clay-glacial till interface, are not hydrostatic, as confirmed by the piezometric readings obtained at the riverbank sites.

VAN CAUWENBERGHE [10] suggested that the porewater pressure in the riverbank might be in part, influenced by the piezometric level of the Upper Carbonate aquifer. Some evidence in the investigation of RENDER[23] substantiates the possibility of contact of the aquifer with the clay-glacial till interface through fissures in the limestone bedrock and the "hardpan". He reported that hair line joints in the cemented glacial till had been observed and these probably accounted for most of its permeability, which was found to be in the order of 10^{-6} cm per sec. The artesian pressures of the aquifer are due to the upland recharge from the Birds Hill area, north-east of Winnipeg, where an aquifer complex is situated at a higher geodetic elevation.

4.2 PIEZOMETRIC READINGS.

The total head (expressed in terms of Canadian Geodetic Elevations) is equal to the sum of the elevation head and the piezometric head, since the velocity head is negligible. The piezometric heads are based on the measured readings without applying corrections to the meter readings. Earlier corrections were based on the re-calibration (using air) of the pneumatic piezometers at the University of Manitoba (VAN CAUWENBERGHE[10]). Subsequent calibrations were taken for measuring pressures exerted by water (porewater pressures), showing that corrections based on air-calibration were not necessary.

The porewater pressure data for St. Vital site were collected from June 1969 to November 1972, and those of St. Boniface Site No. 1 were for the period from November 1969 to

November 1972. Data obtained from October 1970 to August 1972 from another site at St. Boniface (Site No. 2, with piezometers supplied by the Department of Public Works, Government of Canada) are also included to supplement this study. Piezometric levels and river elevations in terms of geodetic elevations for these sites are shown in Figures 10 to 12.

(A) St. Vital Site:

The five Casagrande piezometers (labelled Number 1 to 5 inclusive) were installed in two boreholes that were 107 feet apart. Piezometers 4 and 5 were placed near the top of the upper slope of the riverbank. The former piezometer became plugged at a depth of 23 feet in April, 1970, and the latter piezometer became blocked at 18.7 feet in June, 1970, making further readings impossible. The remaining three Casagrande piezometers continued to function until autumn 1971. Piezometric readings for these three piezometers were not available during the spring inundation periods of 1970 and 1971, when the caps of the piezometer collars were not accessible due to submergence below river level. A comparison of the performance of the Casagrande type piezometer (Number 2) and the pneumatic piezometer (Number 10) is shown in Figure 9. These two closely located piezometers were installed at approximately the same elevation, but information during flooding and winter periods could not be recorded by the Casagrande piezometer. The pneumatic piezometer had good response to the porewater pressure

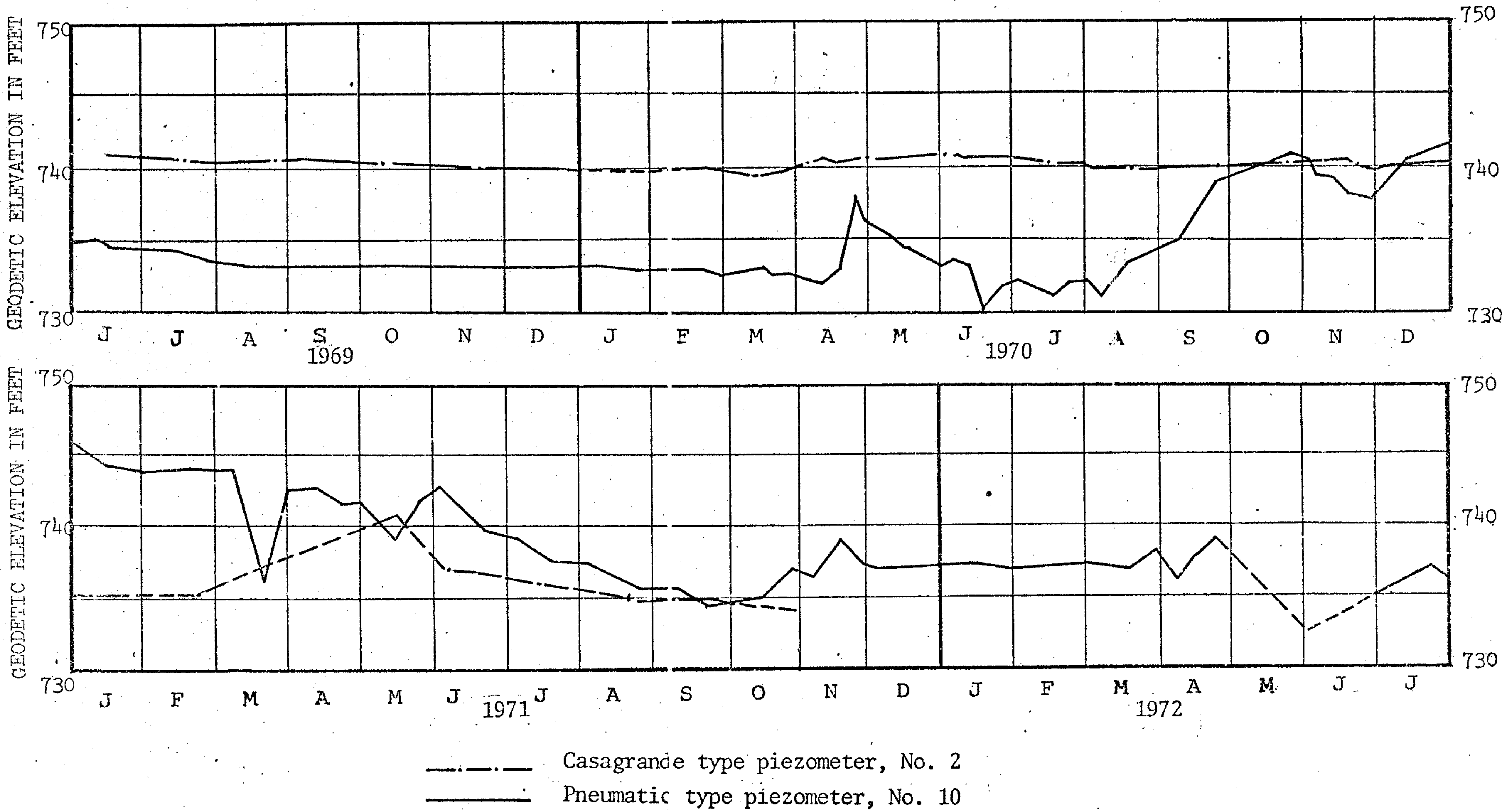


Figure 9 - Comparison of response of Thorpiezo with Casagrande-type piezometer for porewater pressure measurement (St. Vital Site).

fluctuations, showing peaks during the spring inundation and fall drawdown periods.

The total head elevations versus time graphs for the other pneumatic piezometers (Numbers 6, 7, 11, and 13) are shown in Figure 10. River elevations are also plotted on the same graph. These piezometers were selected because they were situated near the clay-till interface. It may be noted that sharp peaks as high as 30 feet of piezometric head (piezometer Number 7) were recorded for the spring inundation periods in April, and high total head elevations were maintained during the winter months through the spring thaw in March until spring drawdown. Piezometer Number 7 was in operation throughout the entire period from June 1969 to November 1972; its performance provided a continual record of the porewater pressure variations.

(B) St. Boniface Site No. 1:

The ten piezometers installed were all of the pneumatic type. Piezometers labelled as B-2, B-3, B-5, B-6, and B-9 were situated at various elevations in the clay layers and in the non-cemented till. The total head elevations versus time graphs are shown in Figures 11A and 11B. It may be noted that piezometers B-6 and B-9 demonstrated dips in total head elevations during the spring thaw period in March just prior to the spring inundation. This is possibly due to the relief of porewater pressure following the thaw of frozen soil cover of the river bank. For all the piezometers, only gentle rises to moderately

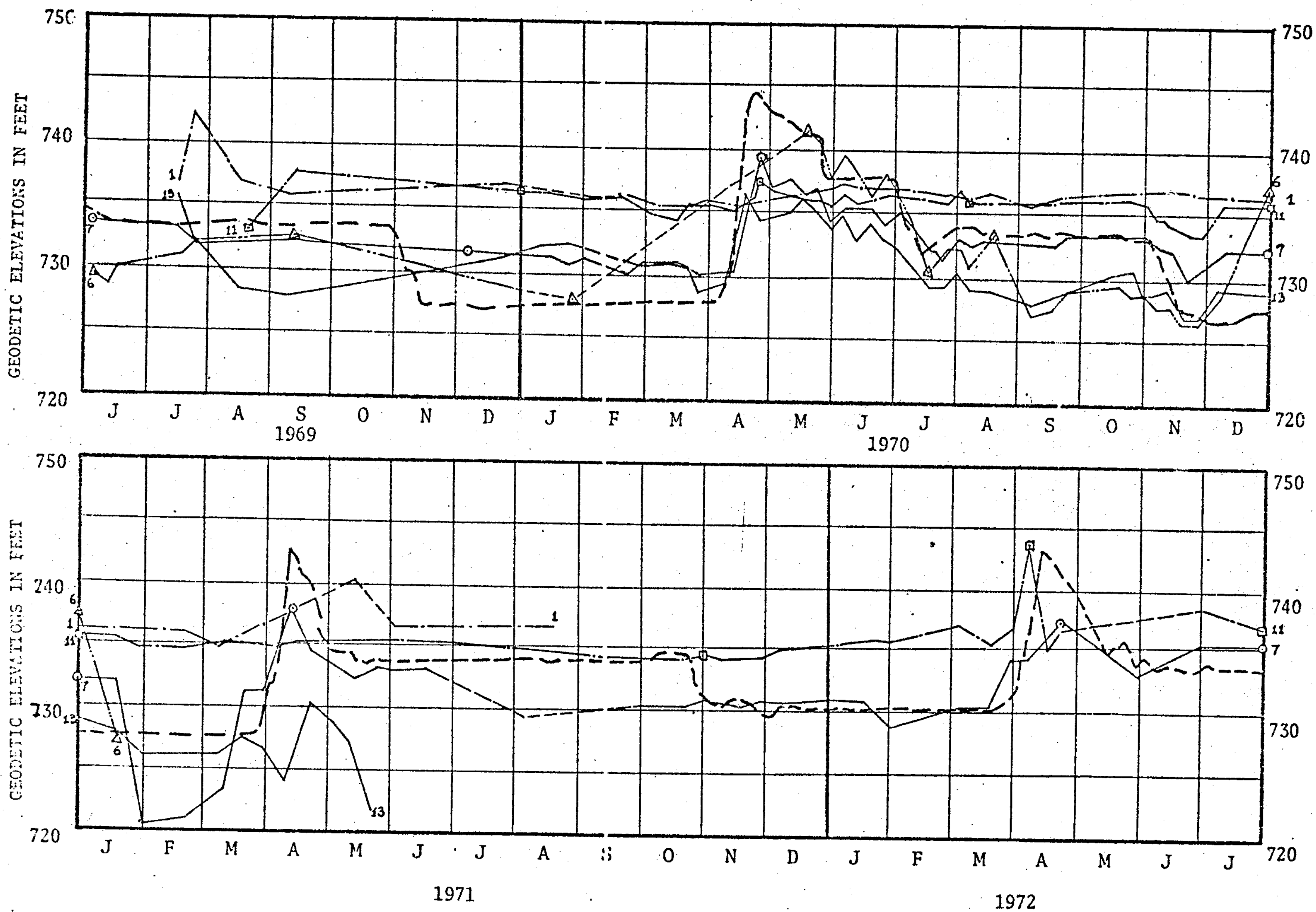


Figure 10 - Total Head Elevations vs. Time Graphs for Piezometer Nos. 1, 6, 7, 11 and 13 (St. Vital Site).

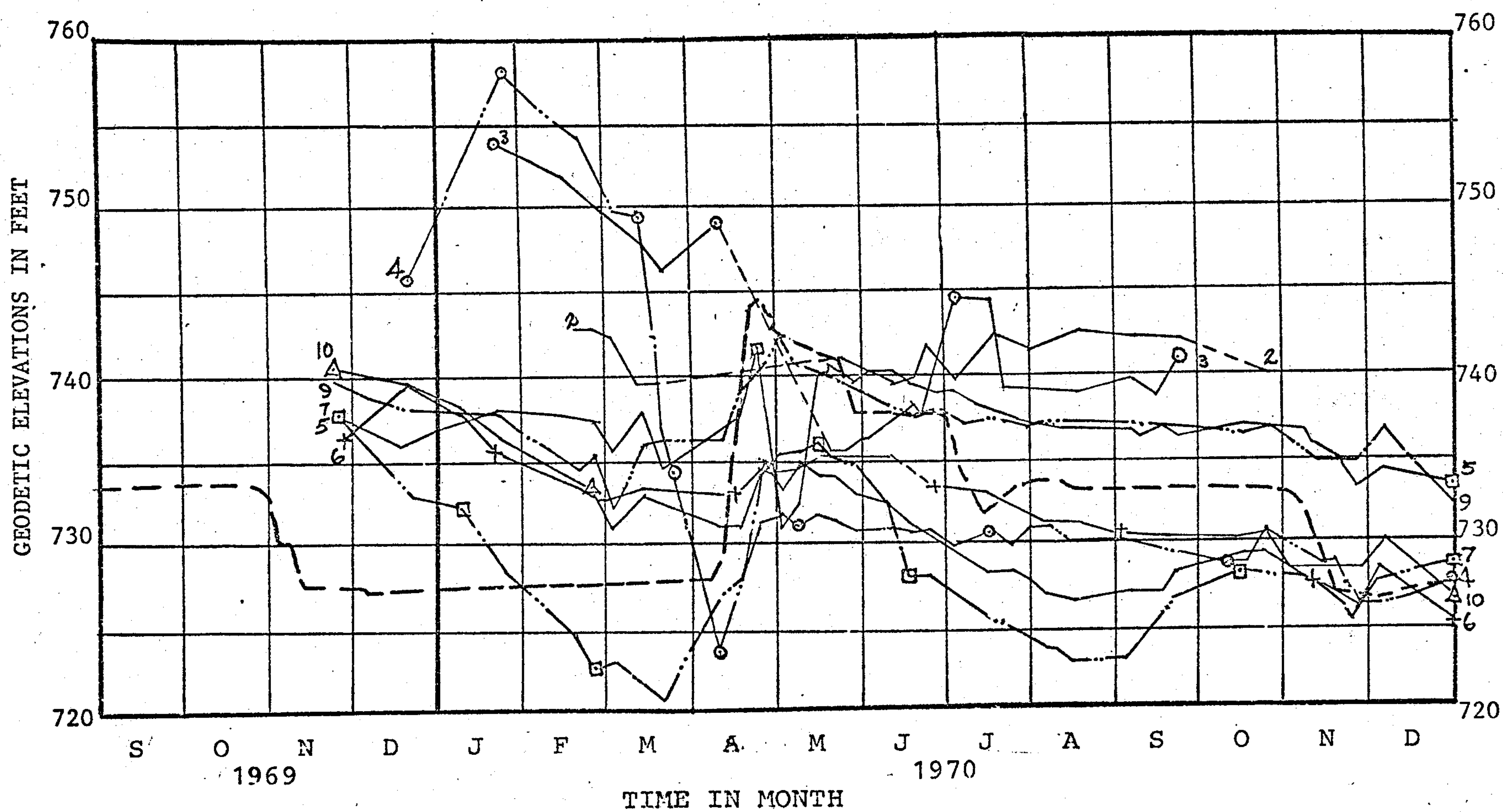


Figure 11A - Total Head Elevations vs. Time Graphs for Piezometer Nos. 2, 3, 4, 5, 6, 7, 9, and 10. (St. Boniface Site No. 1)

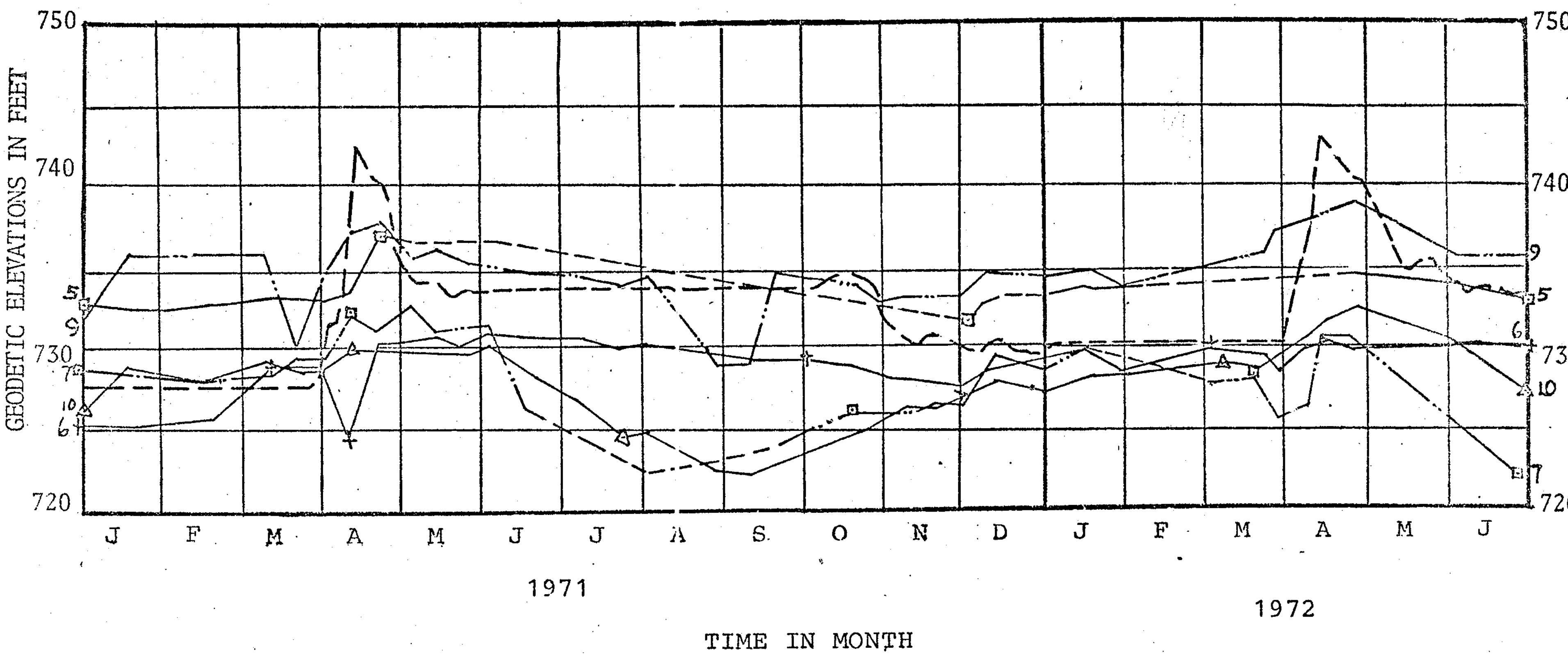


Figure 11B - Total Head Elevations vs. Time Graphs for Piezometer Nos. 2, 3, 4, 5, 6, 9, and 10. (St. Boniface Site No. 1) high peaks were indicated on the graphs for the spring inundation periods in 1971 and 1972. It is of interest to note that piezometers B-7 and B-10, which were embedded near the surface of the dense till layer, demonstrated dips in total head elevations for the summer months in 1970 and 1971. This phenomenon could be a geohydrological effect in the metropolitan Winnipeg area as explained below.

RENDER[23] commented that the groundwater withdrawals had created a major drawdown cone in the central industrial area of metropolitan Winnipeg. The riverbank site on Tache Avenue is situated in the St. Boniface district which is the central industrial area. It is also known that yield due to pumping of groundwater for air-conditioning use during summer months exceeds the amount of groundwater recharge, and thus depresses the piezometric level during that period. This situation may imply that the low piezometric surface in the Upper Carbonate aquifer in the Central Winnipeg area (RENDER[23]) could be responsible for the drop in porewater pressures as recorded by the piezometers.

The other piezometers were embedded in the clay layer, and the summer piezometric levels were maintained at about the same total head elevations after the spring drawdown due to the relatively high and constant summer river level.

(C) St. Boniface Site No. 2:

This site is situated at about 600 feet north of site No. 1 on Tache Avenue, St. Boniface. Piezometers labelled P-3 and

P-5 were situated near the clay-till interface at approximately the same depth. Piezometer P-3 was embedded in the dense glacial till layer where the borehole log indicated auger refusal; whereas piezometer P-5 always had a total head elevation of two to twelve feet lower than that of piezometer P-3, although the former was one foot higher in elevation head, could also be due to the influence of geohydrology. This influence could be of a nature similar to that of piezometers B-7 and B-10, as described in the previous section.

Piezometer P-2 was in operation for only one month, and became blocked due to freezing of the lines. Two other piezometers labelled P-9 and P-10 were installed near piezometer P-2, down to depths of 40 feet and 20 feet from the slope surface, respectively, in October 1971. The total head elevations versus time graphs are shown in Figures 12A and 12B. During the prolonged fall drawdown period in 1971, a substantial rise to as high as 65 feet of piezometric head was recorded for these two piezometers. These piezometric heads lasted for a short period and then dropped off rapidly. The general pattern was similar to that demonstrated by piezometer P-4, which only showed a small peak in total head elevations and then declined steadily. One of the possibilities of forming such extraordinary piezometric heads, assuming no faulty readings, could be due to a leak in a nearby water main. However, owing to the short interval of time in which porewater pressure could be collected, the

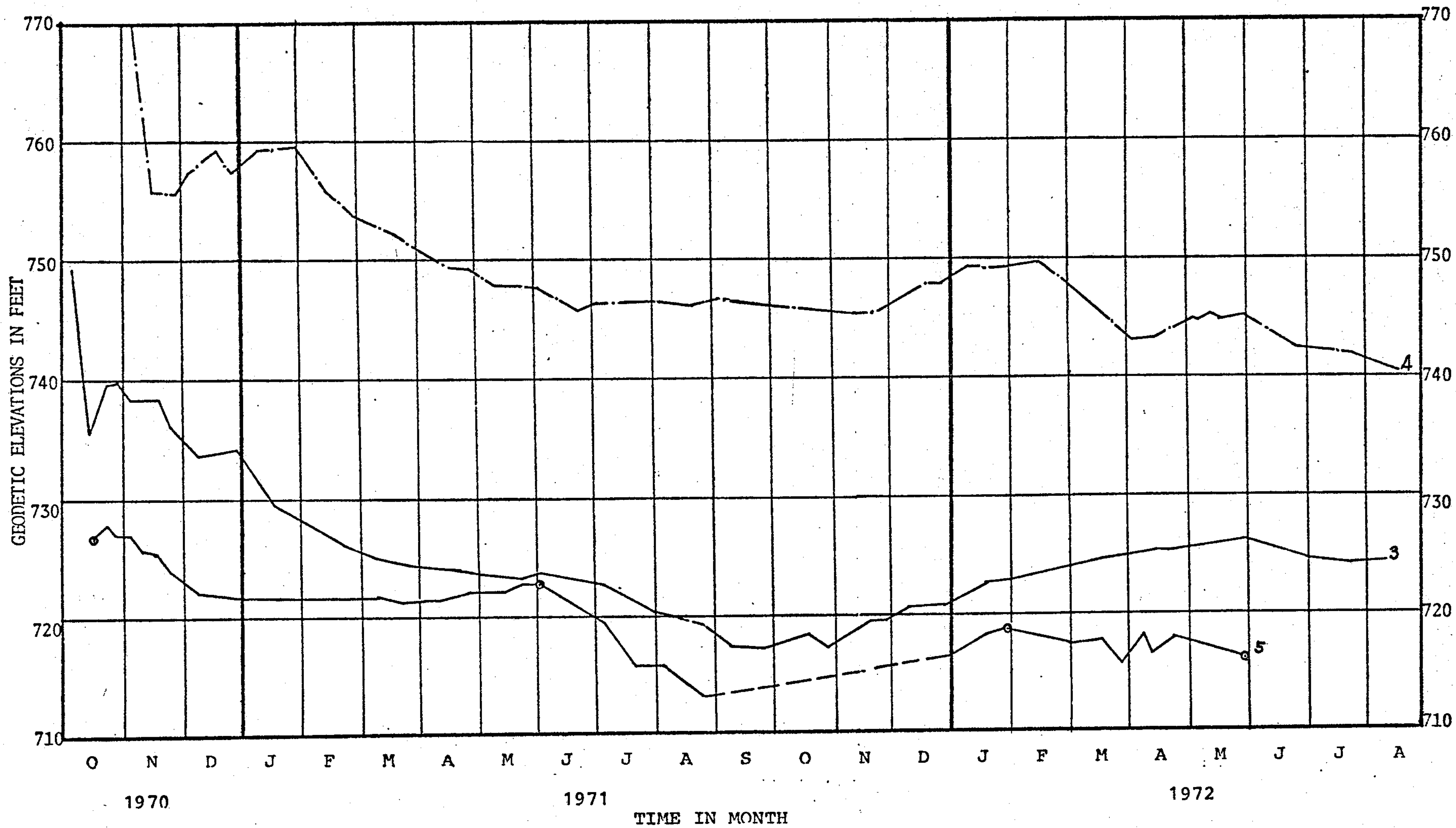


Figure 12A - Total Head Elevations vs. Time Graphs for Piezometer Nos. 3, 4, and 5.
(St. Boniface Site No. 2)

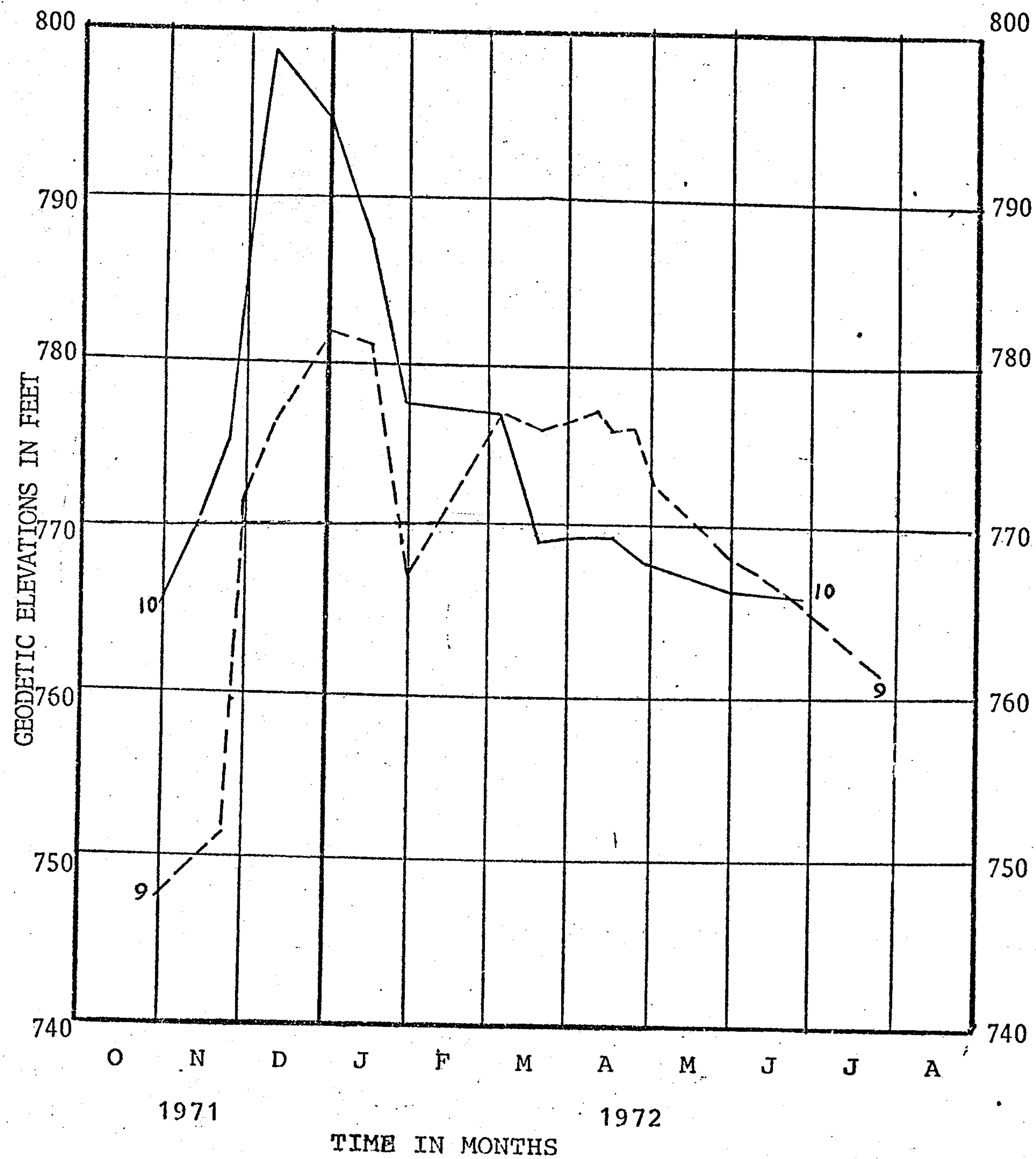


Figure 12B - Total Head Elevations vs. Time Graphs for Piezometer Nos. 9 and 10. (St. Boniface Site No. 2) complex nature of the pattern of total head elevations for these two piezometers was not fully understood. The extraordinary readings for piezometers P-9 and P-10 had to be discounted in considering a representative piezometric regime for the bank site, especially for the critical periods.

4.3 SLOPE MOVEMENT DATA.

Slope indicator data were used to estimate the locations of the slip surfaces for the three sites. Two slope indicators were installed at each site and readings were taken at two-foot intervals of depth down to the bottom of the hole. Lateral deflections recorded from the slope indicators were plotted against depth of test holes in feet. Slope movements at selected points, as measured by slope indicators installed in these sites, are shown in Figure 13. Location plans for all slope indicators as well as the points selected for the movements are shown in Figures 14 to 16.

(A) St. Vital Site:

The slope movement of each indicator is shown in Figure 14. Slope indicator SV-1 was in operation from February 1969 to March 1972. The deflection point at the 28 foot-depth indicates a total of 5.74 inches slope movement in the southward direction over a period of three consecutive years. Slope indicator No. SV-2 was in operation from January 1969 to April 1970 until it became constricted at a depth of about 33 feet,

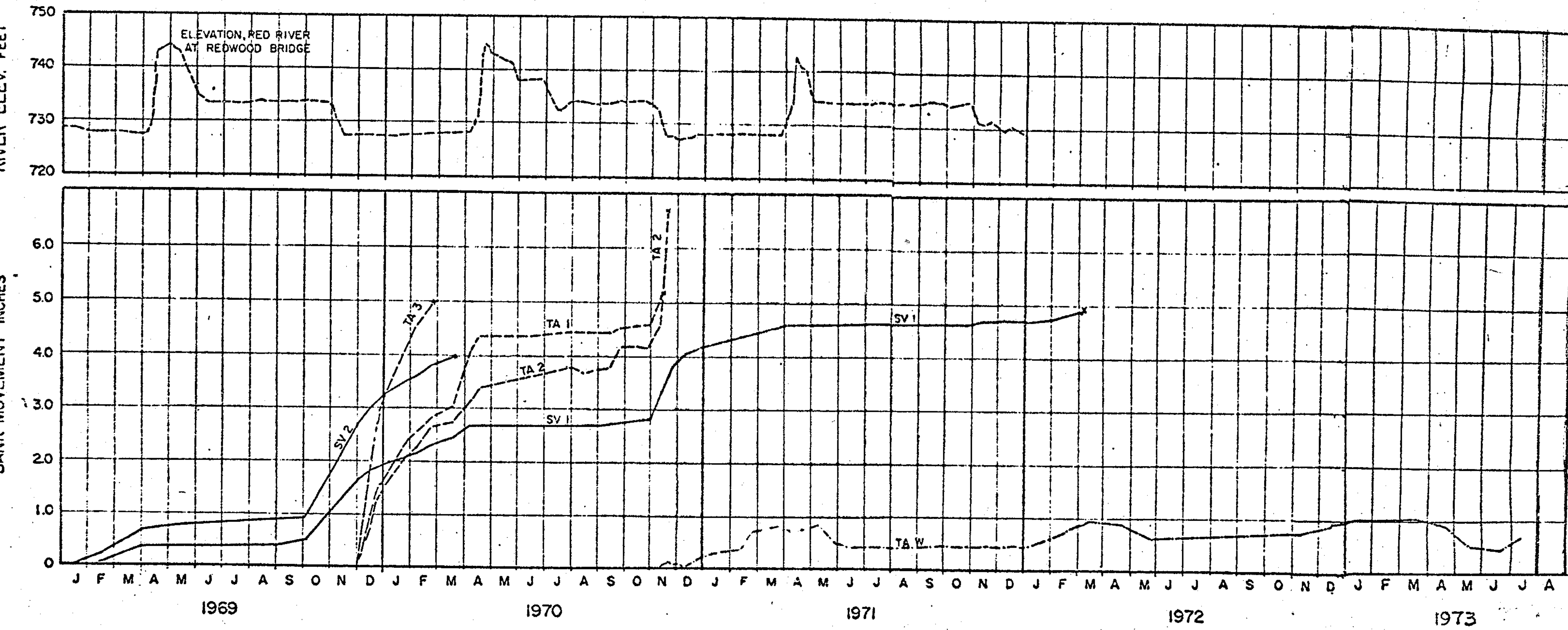


FIGURE 13 RIVER ELEVATIONS, TYPICAL RIVER BANK, MOVEMENTS AS MEASURED BY SLOPE-INDICATORS
RED RIVER, WINNIPEG, MANITOBA

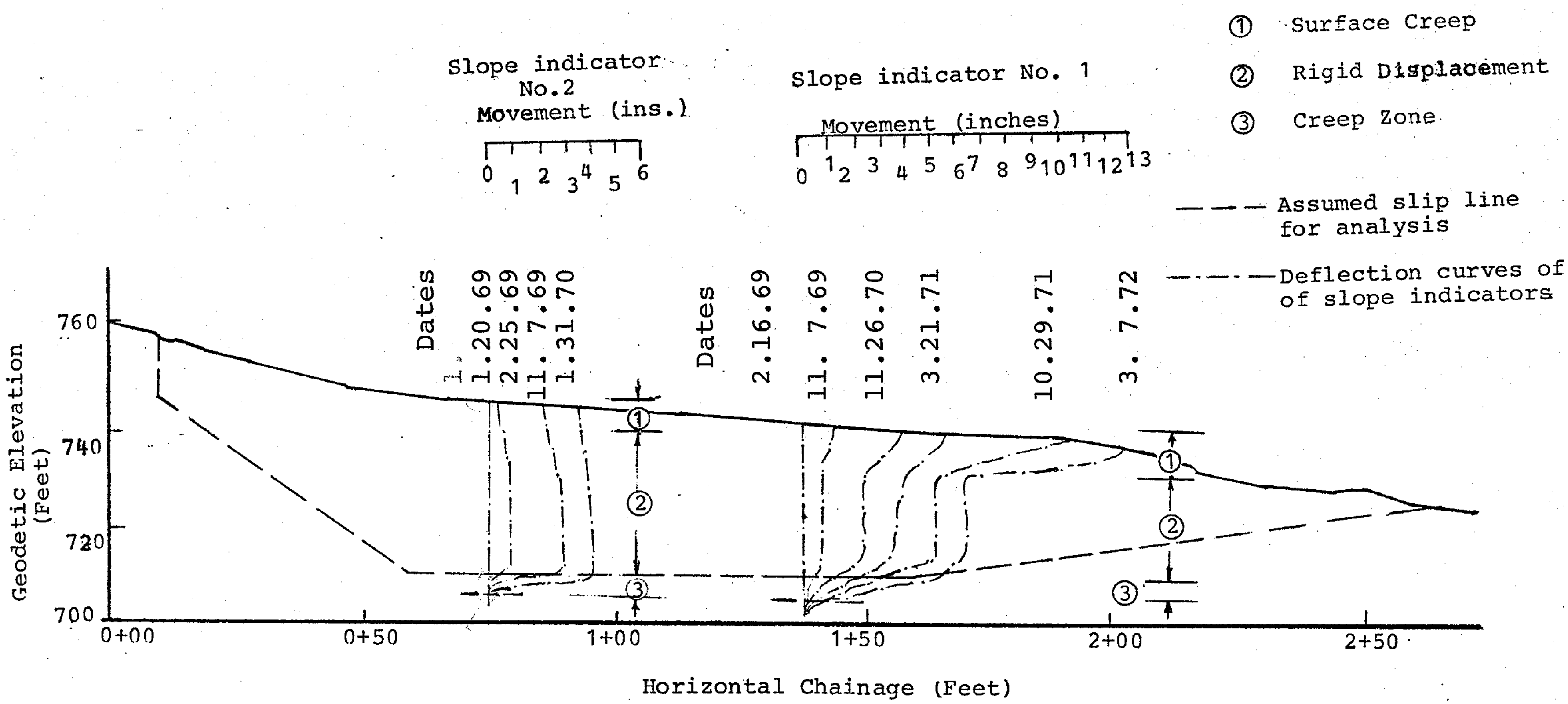


Figure 14 A - Deflection Curves for Slope Movement at St. Vital Site.

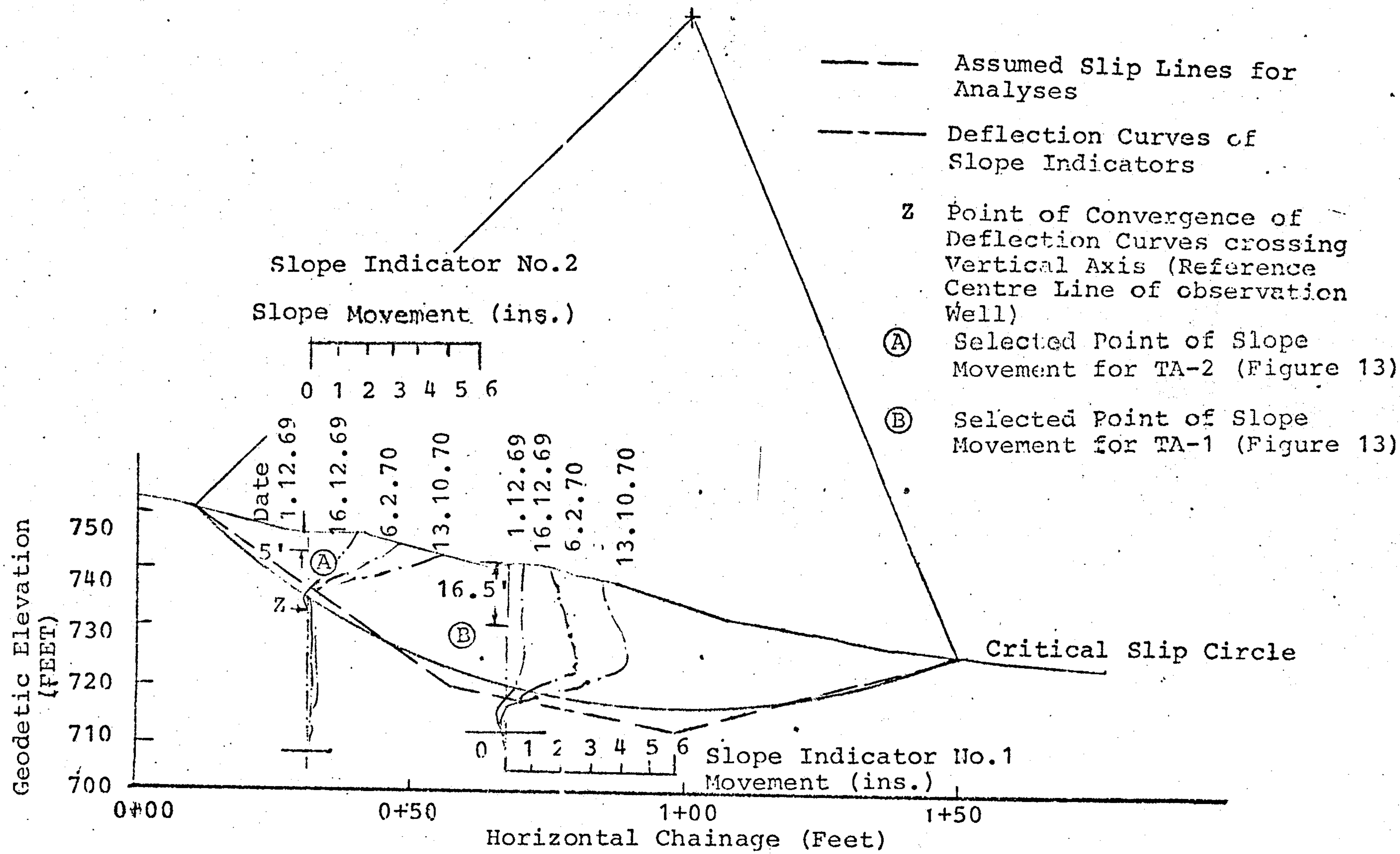


Figure 14B - Deflection Curves for Slope Movement at St. Boniface Site No.1

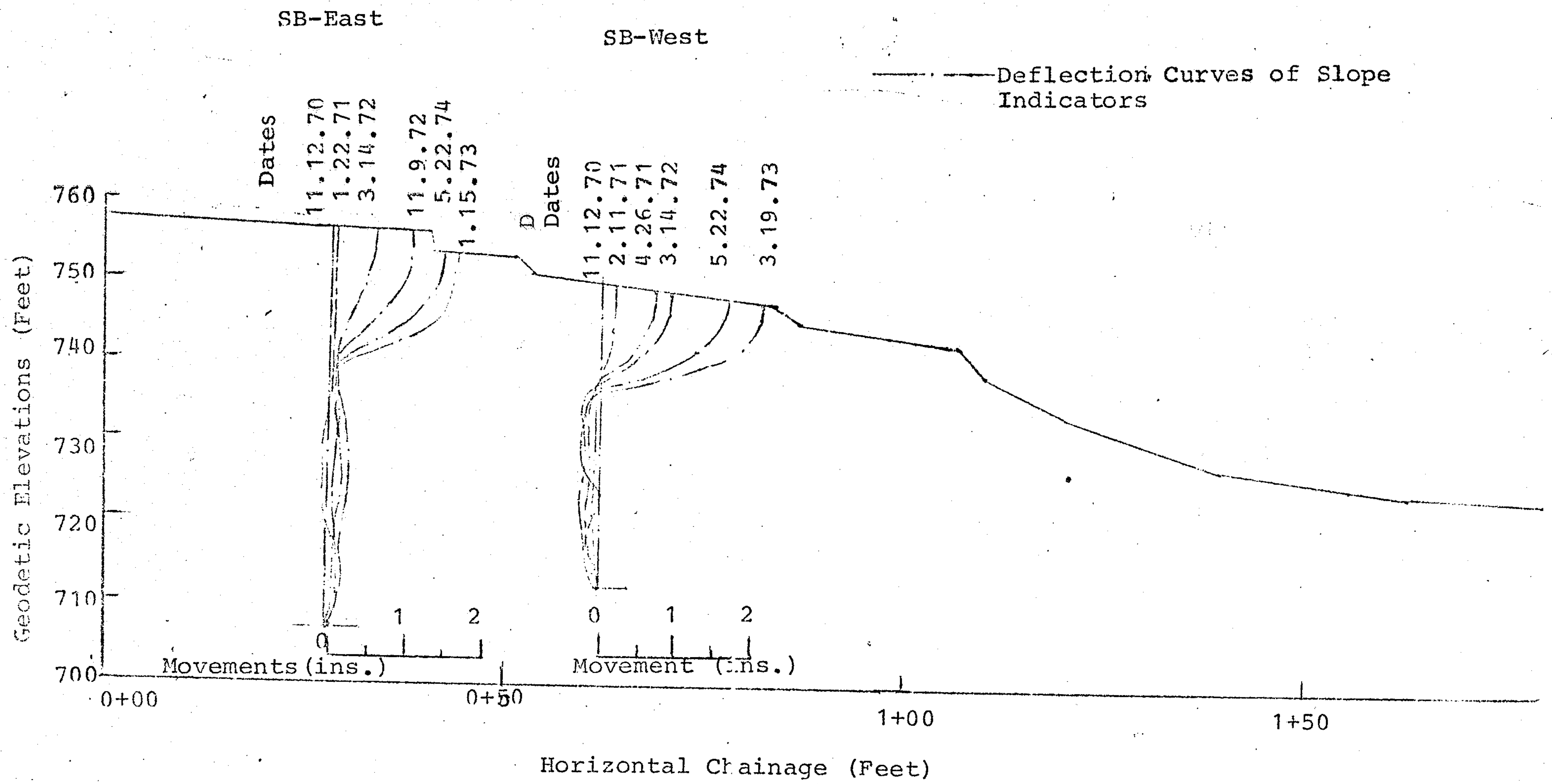


Figure 15 - Deflection Curves for Slope Movements at St. Boniface Site No. 2

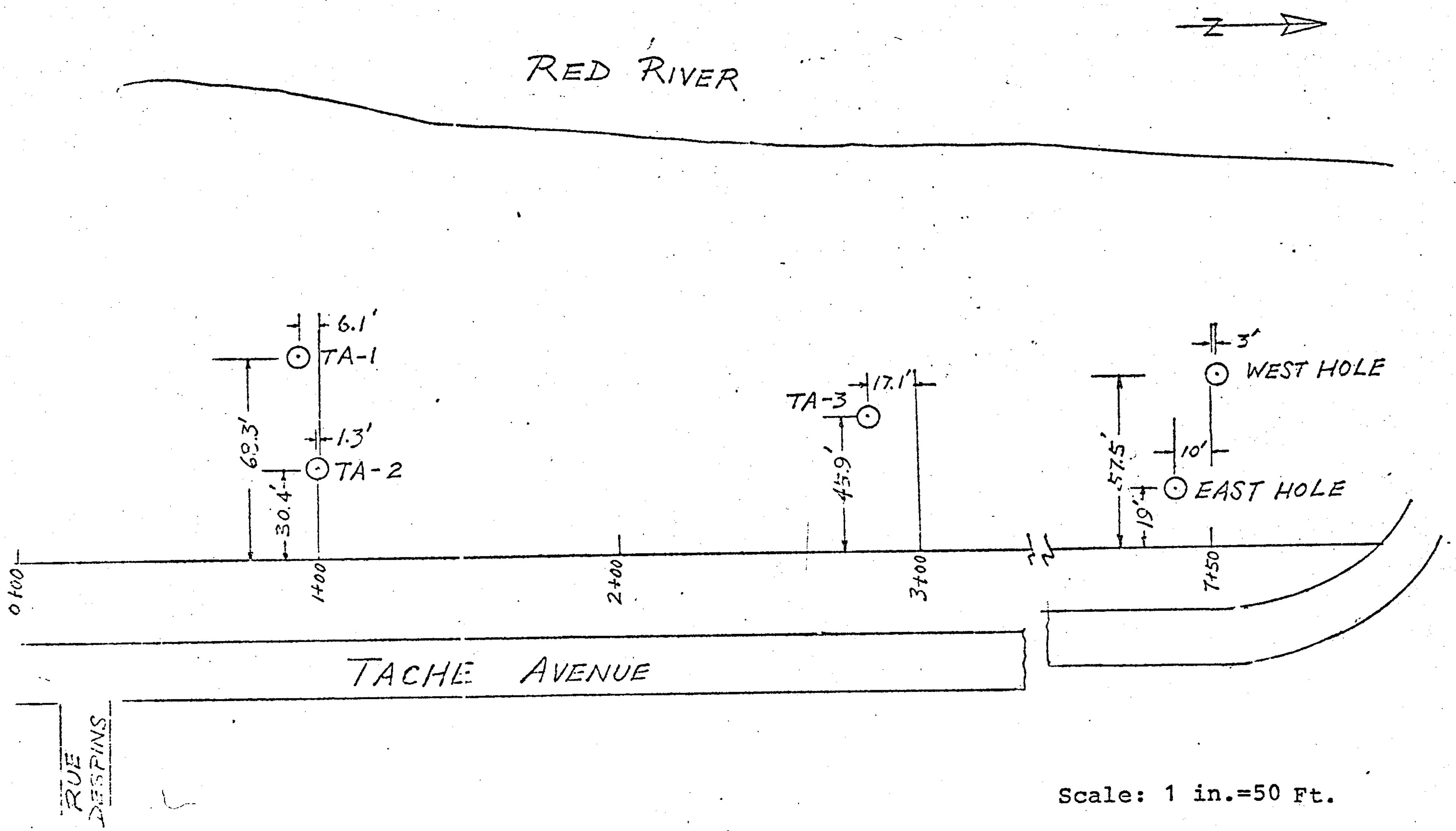


Figure 16 - Location Plan of Slope Indicators at St. Boniface Sites.

and the probe could not be lowered beyond that depth. The deflection point at the 34-foot depth plotted in Figure 13 indicates a total of 3.95 inches slope movement in the westward direction in 15 months.

The profile of slope movement revealed that large components of ground movement had taken place immediately above the clay-till interface. This indicates that the location of the failure zone lies in a segment bound by the two boreholes. The major direction of lateral motion was westward and the river bank moved obliquely towards the river. VAN CAUWENBERGHE[10] determined the positions of several points on the slip surface based on the data of the two slope indicators, the sub-soil profile and a ground surface profile.

(B) St. Boniface Site No. 1:

The slope indicator Nos. TA-1, TA-2, and TA-3 were installed in December 1969. Slope indicator TA-1 operated until November 1970, at which time it failed by being blocked at the 25-foot depth. During the high water period, submergence of slope indicator TA-1 prevented readings from being taken during the last half of April and May of 1970.

Slope indicator TA-2 also failed in November 1970 after becoming pinched off at the 8-foot depth. Slope indicator TA-3, which was situated about 282.9 feet north of the other two slope indicators, operated only until February 1970 when the slope indicator was constricted at a depth of 23 feet. The total amount of slope movement of the deflection points, as plotted in Figure 13, and their major movement directions are summarized in Table 3:

TABLE 3

SUMMARY OF ST. BONIFACE SITE NO. 1 SLOPE MOVEMENTS AT SELECTED POINTS

Slope Indicator No.	Depth (FT.)	Movement Direction	Lateral Deflection (IN.)	Time Interval (MONTHS)
TA - 1	16.2	Westward	5.12	11.5
TA - 2	5.0	Westward	6.59	11.5
TA - 3	21.0	Westward	4.94	3.0

The rates of movement depend on the depth at which the points were situated, and other factors such as groundwater conditions within the slope, duration of the submergence period, and frequency and magnitude of precipitation. The installation of additional slope indicators near the toe of the slope would be desirable, but cost considerations prevented installation.

Each lateral deflection curve of the two slope indicators (Figure 14B) indicates a deflection point at the level showing a reversal in slope movement direction, with the curve crossing the vertical axis of the observation well at a point of convergence with other deflection curves. Observations based on the subsoil profile and the slope indicator data showed that, for this site, these points of deflection were located near the clay-till interface or in the non-cemented till layer. The significane feature revealed by this observation was that for slope indicator TA-1, the failure zone existed immediately above the clay-till interface; whereas for slope indicator TA-2, data showed that the failure zone passed through the clay layer near the slope surface. The estimated slip line, nearly tangential to the deflection points of the deflection curves, was assumed to emerge at the toe as no information was available regarding the slip line location near the toe. This assumed slip surface appears to be circular.

(C) St. Boniface Site No. 2:

After the slope indicator installed at St. Boniface Site No. 1 had failed, two slope indicators designated as "SB-East" and "SB-West" were installed in October 1970. The slope indicators belonged to the Civil Engineering Department of the University of Manitoba, were supplied by the Department of Public Works of Government of Canada, and were installed by the Manitoba Water Control and Conservation Branch. The two slope indicators at this site were still monitoring movement data in June 1974.

A plot of the deflection point at the depth of 5 feet versus time for slope indicator SB-West (Figure 13) shows a total of 1.09 inches of slope movement in the westward direction in 29 months. The slope indicator SB-East recorded a total of 0.75 inch slope movement at the 5-foot depth in 27 months. This slow rate of slope movement may indicate that the slope indicators are located on the slope that is more stable near the upper part of the bank. The riverbank site has been somewhat stabilized by flattening the slopes to per permit installation of piezometers and slope indicators.

CHAPTER 5

SLOPE MOVEMENT STUDY

5.1 OBSERVATIONS OF SLOPE MOVEMENT DATA.

For natural slopes that have developed as a result of the natural processes of weathering, including river banks, materials near the slope surface often undergo a slow downhill movement referred to as "creep" (WILSON[24]). The rate of soil creep depends on many factors of which the three most significant variables are the inclination of the slope, the type of soil, and the amount and frequency of rainfall. The slope indicator data provide a record of the rate and amount of slope movement in the field, before and after the failure of a slope.

In determining the locations of the failure surfaces at the St. Vital site from slope indicator data, it was found that the estimated slip line, with a somewhat flattened bottom, passed through the clay-till interface region. This non-circular failure surface, which began from the bottom of an observed tension crack and extended down to the interface, could be fairly well defined because of the known slip which had occurred previously. For the other two sites situated in St. Boniface area, the potential failure surfaces that pass through the deep deposits of glacio-lacustrine clays are likely to be circular, as estimated from the observed data.

An examination of the slope movement profile shows that at the St. Vital site, the zone of movement is well defined.

Slope indicators SV-1 and SV-2 indicated large displacements of 6.27 inches and 3.88 inches, respectively, near the clay-till interface, and that the bank was moving westward towards the river. A larger displacement was recorded by slope indicator SV-1 because it operated two years longer than did slope indicator SV-2 and also due to differences in the observation locations. Data showed that during the period while both slope indicators were in operation, the displacement near the slope surface recorded by SV-2 was smaller than that of SV-1. This would imply that the soil mass near SV-1, being further down the slope, had a total displacement equal to the sum of the translational component due to the upper slope movement, and the relative displacement component due to independent movement, or creep of the soil mass. This independent movement may also imply the existence of separate or independent slip surfaces other than the one estimated to exist near the clay-till interface. However, the sharply defined slip surface would probably be the same failure plane that had developed along the old sliding surface.

For slope indicator SV-2, data indicated that the slope movement in the southward direction was about one-tenth of the westward movement, and hence was negligible. During the three years operation of slope indicator SV-1, 12.37 inches slope movement in the westward direction, and 11.82 inches southward movement near the slope surface was recorded. The movements in these two directions were of the same order, and the resultant movement of the soil mass near the slope surface in an oblique direction towards the river. Total slope movement of such magnitude as recorded by slope indicator SV-1 would imply that the ground movement near the slope surface may be due to creep, as slope movements within the soil mass at deeper soil layers were of lesser magnitudes.

The slope movement profiles of the two adjacent sites in St. Boniface area have different features although the sites are only 600 feet apart. For site No. 1, slope indicator TA-2 indicated a total of 8.88 inches slope movement in the westward direction near the slope surface, and a corresponding movement of 3.94 inches was recorded by slope indicator TA-1. This implies that slope movements of the soil mass near TA-2, which was situated near the upper slope, and that near TA-1 could be due to rotation or independent movements. The major movement near TA-2 took place within the upper fifteen feet from the slope surface and as a result of the creep of soil mass within this zone, a maximum displacement of 11.1 inches in 11 months towards the river was recorded at the slope surface (Figure 14B). Further observations showed that at all times, there existed a transition zone extending from twelve feet to nineteen feet below the slope surface, and within the mixed grey clay and non-cemented till layer, in which a small movement of about 0.2 inch over a period of 50 weeks was measured (point Z, Figure 14B). This eastward movement may be explained by the fact that the slot tube for guiding the slope indicator became loose in the observation well.

For the soil mass near slope indicator TA-1, the slope movement profile showed that major movement had taken place near the clay-till interface, with a maximum displacemnt of 5.12 inches in 11.5 months (Table 3). The smaller displacement of 3.94 inches recorded over a period of 11.5 months (Figure 14B) near the slope surface also implied that existence of independent slip surfaces. Another slope indicator, TA-3, installed at approximately the same elevation and about 190 feet north of slope indicator TA-2, recorded an approximately same rate of movement and had a similar slope movement profile when both were operating.

The movement profiles of the two slope indicators (SB-East and SB-West) at the St. Boniface Site No. 2 were similar to those of slope indicators TA-2 and TA-3 of Site No. 1, but the rate of movement was only about half an inch per year. The eastward ground movement and the westward movement towards the river appeared erratic each month of the year. The eastward movement may have been the result of the slot tube becoming loose in the observation well. However, data showed that the slope movement was towards the river during spring inundation and fall drawdown, followed by a period of eastward movement, and then westward towards the river again. The data indicated that the slope movement near SB-East and SB-West consisted of a major translational component and a relatively small displacement component due to creep. The small amount of slope movement recorded at this bank site may be due to stabilization by flattening the slope to permit installation of piezometers. As the slope indicators SB-East and SB-West are still monitoring slope movement, future data may help in assessing a more definite movement profile.

5.2 RATE OF SLOPE MOVEMENT.

The rate of slope movement varied with time of the year and the depth of soil mass. For the bank sites studied, the rates of movement at ground surface and/or the maximum displacements recorded are summarized in Table 4.

An example of a rapid rate of movement that deserves study is the surface movement near slope indicator TA-2 of St. Boniface Site No. 1, where a displacement of 2.22 inches in one week was recorded between November 12th and November 19th, 1970. An examination of the Red River elevation record indicated that this period corresponded to the fall drawdown. Piezometric readings indicated that during that period, the porewater pressure within the soil mass was still very high, causing a reduction in shear strength. Unfortunately, slope indicator TA-2 failed (as described previously) after November 19th, 1970, but a comparison of data can be made with the rapid rate of movement recorded at St. Vital site. Data indicated that for slope movement at surface, slope indicator TA-2 recorded a total of 3.1 inches in 21 days (October 29th to November 19th, 1970), and for St. Vital site, slope indicator SV-1 recorded a total of 4.27 inches of slope movement in 28 days (October 29th to November 26th, 1970).

5.3 STUDY OF CREEP DEFORMATION.

The study of the stress-strain relationship of the soil

TABLE 4

TYPICAL ANNUAL MOVEMENTS REGISTERED

BY RIVER BANK SLOPE INDICATORS

(Unless otherwise stated, movement is in the westward direction)

SITE	SLOPE INDICATOR NO. AND DEPTH	MOVEMENT IN INCHES FOR THE YEAR 1969	1970	1971	1972	1973
St. Vital Site	SV-1 (S) Ground Surface	3.76	5.82	1.82	0.42* (3M)	----
	SV-1 Ground Surface	3.95	6.62	1.83	0.33* (3M)	----
	SV-2 Ground Surface		3.49** (6M)	----	----	----
	SV-2 At Depth 32 Ft.		3.94**	----	----	----
St. Boniface Site No. 1	TA-1 Ground Surface	1.13	2.81* (11M)	----	----	----
	TA-1 At Depth 16.5 Ft.	1.64	3.48* (11M)	----	----	----
	TA-2 Ground Surface	2.29	8.81* (11M)	----	----	----
	TA-3 Ground Surface	4.16* (3M)	-----	----	----	----
	TA-3 At Depth 21 Ft.	4.94* (3M)	----	----	----	----
St. Bon. Site # 2	SB-East Ground Surface	----		0.52+ (14M)	1.15	0.35* (5M)
	SB-West Ground Surface	----		1.29+ (14M)	0.70	0.56* (5M)

* Partial Year, movement for number of months shown

** Movement for 6 months, winter of 1969-1970

\+ Movement for 14 months, 1970-1971

S - Southward Movement

M - Time in Month

mass within a slope at impending failure could provide a better knowledge of the soil deformation due to creep in relation to slope stability. However, this approach would require an estimation of the amount and rate of shear deformations, together with evaluation of the boundary conditions and soil properties. The amount and rate of shear deformations could be obtained from observation of slope indicator data, but the evaluation of boundary conditions for stability analysis would require theoretical considerations based on the theory of rheology. The failure of soil due to creep had been studied by some investigators, among whom were SUKLJE[25] and YEN[26].

Creep deformation is plastic deformation which does not necessarily cause failure. The continuous creep of a slope in a state of impending failure, is a case in which the down-slope component of the weight of soil causes a slow, continuous yield (YEN[26]). A delicate balance of forces is represented by this state of slow yielding and sliding can be triggered off at any instant once the resisting forces have been overcome. Other prime factors such as slope geometry, pore-water pressure, and soil constitutive equations should be taken into account in explaining the behaviour and stability of the slope. The soil constitutive equations define the stress-strain relations of the soil mass, depending on the basic assumptions and the approach adopted. For theory of elasticity, the constitutive equations are based on Hooke's Law, and for the theory of plasticity, the stress-strain relationship must satisfy the yield or failure criterion, and these equations usually involve plastic strain rates.

YEN[25] adopted an infinite slope approach for long slopes because of the relatively simpler mathematical treatment. The present riverbank sites depart from the infinite slope assumption in that the bedrock or dense till layer and water table are not parallel to the slope surface. Other idealized assumptions of uniform acceleration of slope movement and steady-state velocity profile also do not exist in nature. The actual slope movement is a function of stress and strain affected by such independent variables as floods, drawdown, and the rate of slope movement varies from time to time during the year.

CHAPTER 6

RIVERBANK STABILITY STUDY

6.1 SLOPE STABILITY THEORY.

The prevalent approach for slope stability analysis is based on a limiting equilibrium analysis using potential slip surfaces. The shear strength of the soil is a governing factor. A factor of safety computation is the essence of the analysis, providing a standard of comparison in determining the stability of the slope in question. The most useful and widely adopted definition of the factor of safety is taken as the ratio of the total shear strength available on the slip surface to the total shear strength mobilized in order to maintain equilibrium.

The common analytical method being widely used is basically the circular arc method. This method developed by Petterson (SUKLJE[26]), is also known as the Swedish circle method. A cylindrical rupture surface with a normal axis to the plane of the slope cross-section is assumed, and the soil mass rests on a rigid, impenetrable base. The factor of safety computation was taken as the ratio of the resulting opposing forces developed on the failure surface to the sum of the tangential working forces (tangential component of the weight of soil mass above the failure surface) along the slip arc. This early version of circular arc method was merely a curvilinear generalization of the slip plane approach. The complete generalization for curved slides was later modified and the normal component of the weight reacting perpendicularly to the slip surface was taken into account. This modified method is still in use and is known as the conventional or Fellenius method; it can be conveniently formulated as the method of slices in which the failure mass within the slip surface is divided into thin, finite slices. BISHOP[27] further developed the method of slices, taking into account the normal forces acting on the vertical sides of the slices (or horizontal interslice forces) and the tangential shear forces (or vertical interslice forces). A simplified version of Bishop's method neglecting the vertical interslice forces is commonly used, and the error in factor of safety computation of this approximate solution is within 15% of the value obtained by the refined version of Bishop's method (JANBU et al.[28]). The common feature to all these approximate methods, is that the safety factor computation is defined in terms of moments of weights of the failure mass (sum of the driving moments) to the moments of mobilized shear strength along the failure arc (sum of the resisting moments).

The friction-circle method developed by TAYLOR[15] involves the application of stability numbers from prepared tables and charts which can be used to find safety factors with respect to soil strength. A safety factor defined with respect to cohesion and a safety factor defined with respect to the frictional angle (tan ϕ) are first estimated by assuming a value of the frictional angle with the aid of Taylor's charts. By trial-and-error procedures, the value of the safety factor with respect to strength is obtained when a value of safety factor which applies equally to both cohesion and friction is found.

For non-circular failure surfaces, several methods of analysis have been developed by some investigators, such as JANBU[29], MORGENSTERN and PRICE[30], and NONVEILLER[31]. The application of these methods will be discussed in a later section.

6.2 ASSUMPTIONS AND THEORETICAL CONSIDERATIONS.

The inherent assumptions for the above-mentioned methods of stability analysis, whether for circular or non-circular failure surfaces, are as follows:

1) that the soils within the failure zone are ideally rigid-plastic materials;

2) that the failure soil mass along the assumed rupture surface is in plastic equilibrium and has rigid-plastic deformation properties;

3) that failure due to progressive mechanism does not occur;

4) that the soil mass is homogeneous and satisfies the Coulomb-Mohr failure criterion.

Certain simplifying assumptions are also made, depending on the method of analysis used.

The choice of shear strength parameters in the effective stress analysis, as mentioned in the previous section, and the selection of the porewater pressure distribution in the real problem, govern the accuracy of the evaluation of the factor of safety. Analytical study of slope stability provides a determination of a margin of safety in the design of earth structures, and a correlation between field observations and the results of stability analyses based on measured properties of the materials involved in the mass-movement. The common methods of analysis can be applied to both first-time slides, i.e., slides in previously unsheared material, and slides due to reactivation of movement along a pre-existing surface.

For natural slopes or riverbanks, the long-term stability is characterized by a driving moment which remains constant or changes in an extremely slow manner. Failure of the slope is brought about by the gradual and permanent decrease in the resisting moment, and frequently, sudden decrease in the resisting moment is augmented by decrease of effective stress caused by increase of porewater pressure due to seasonal variations. When a first-time slide occurs, the material underlying the newly exposed failure surface begins to soften and the process continues until another slide occurs. When the slope fails along a pre-existing rupture surface, the soil strength has reached the ultimate or residual value. Basic types of landslides on clay slopes and their mechanisms have been classified in details by BISHOP and HUTCHINSON[32], and they are not dealt with in the present study. In general, the long-term failure is the result of a gradual decrease in shearing resistance, which is usually caused by decrease in the effective stress due to development of excess porewater pressure or a rise in groundwater table.

6.3 COMPUTER PROGRAMME FOR SLOPE STABILITY ANALYSIS.

In this study, effective stress methods of analysis were applied to both circular and non-circular slip surfaces. The results were then compared with those of total stress analysis with undrained strength parameters, which had been obtained by some previous investigators. The emphasis of this study was on the effect of porewater pressure distribution, and the computations of the stability analyses were facilitated by a computer program incorporating the Simplified Bishop's method, and Janbu's method for non-circular failure surfaces.

The computer program for the Bishop's method consists of slope stability analyses of many trial slip circles with different locations of circle centres, and by an iterative procedure, a factor of safety for each trial circle satisfying all conditions of static equilibrium, is found. The centres are located on a grid pattern, and a value of safety factor is obtained for each trial slip circle of a trial centre. By drawing contour lines of equal safety factors, the critical centre providing the lowest factor of safety can be found.

For the Janbu's method, the geometry of the assumed composite (including circular arc) slip surface can be approximated by a series of straight lines, and the entire soil mass within the failure zone is divided into an assigned number of thin, finite slices. A similar procedure of isolating the failure mass for analysis was described by BELL[33]. The feature of the present program used is that the slope geometry, river level, slip surface, thrust line and the piezometric level are all represented by a series of straight lines.

The porewater pressure domain was described by means of a piezometric level. Based on the piezometric readings taken at approximately weekly or fortnightly intervals, equipotential lines were drawn connecting points of equal total heads expressed in terms of geodetic elevations. As the porewater pressure distribution near the assumed slip surface was of primary interest, the piezometric levels along this surface were established by erecting vertical lines from the points of intersection of equipotential lines and the slip lines, to the height indicating the level to which piezometers placed at each individual intersection point would rise. The piezometric surface was then defined by a series of straight lines joining the piezometric levels (Figure 17). For both circular and non-circular slip surfaces, the method of slices was adopted, and the computed unit porewater pressure for each finite slice was taken to be the height of water from the piezometric surface to the base of the slice, situated on the assumed slip surface.

In the analyses, the average residual values of the shear strength parameters, based on the test results of JANZEN[9] and SUTHERLAND[13] on Winnipeg clay, were used. For the St. Vital riverbank site, the average residual strength parameters of Winnipeg clay are $\phi'_R = 11^\circ$, and $c'_R = 0$ p.s.i. based on the

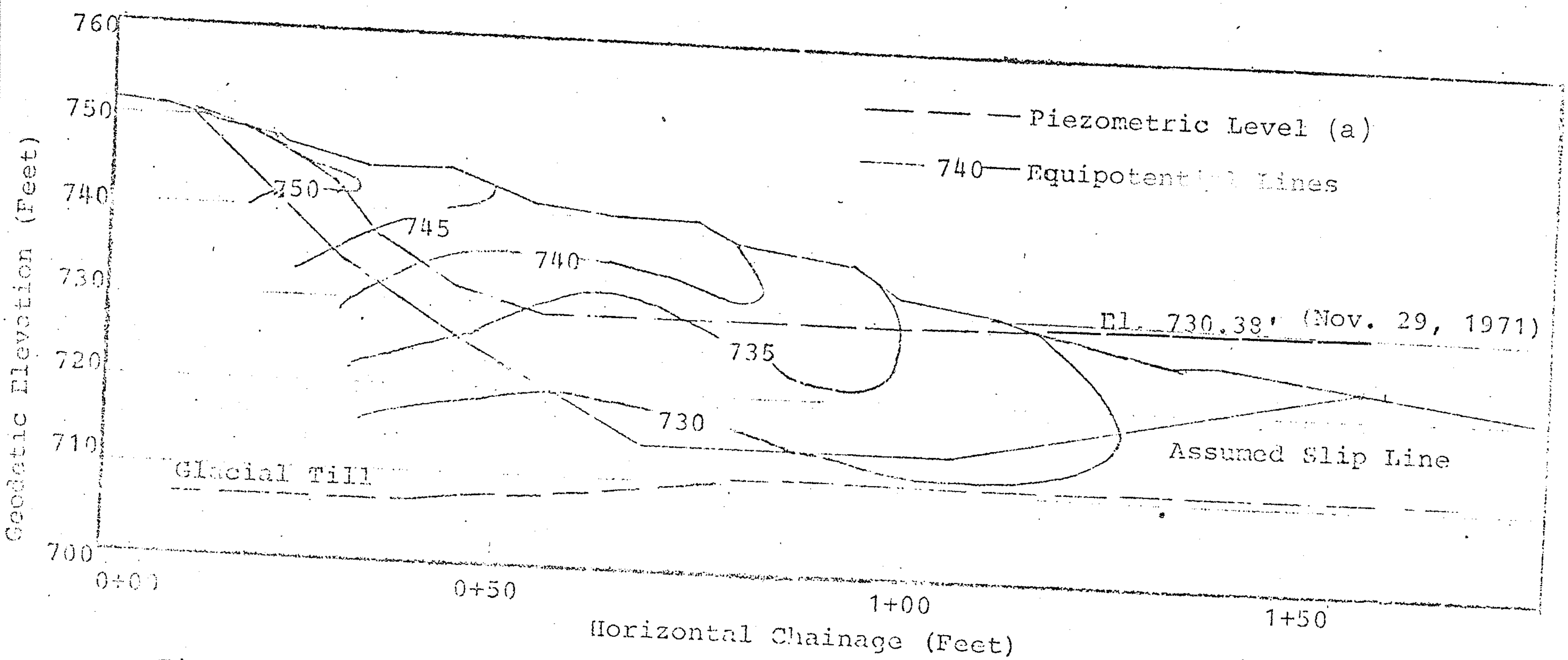


Figure 17 - Piezometric Level and Equipotential Lines for Fall Drawdown Condition (St. Boniface Site No. 1)

(a) For determining pore pressures on slip plane results of SUTHERLAND[13]. For the two riverbank sites (St. Boniface and St. Vital) values for ϕ'_R of clay ranged between 8° and 13°, and for c'_R, between 0 p.s.i. and 1.5 p.s.i. for the effective residual angle of friction and effective cohesive strength, respectively (JANZEN[9]).

6.4 RIVERBANK STABILITY ANALYSES WITH CIRCULAR SLIP SURFACE.

The method of slices, assuming circular slip surfaces, has been applied to the three riverbank sites for slope stability analysis. VAN CAUWENBERGHE[10] studied the St. Vital bank, and the two St. Boniface riverbank sites are included in the present study. The Fellenius and Simplified Bishop's methods were used to determine the factor of safety for each groundwater condition.

(A) FORMULATION OF THE CIRCULAR ARC METHOD.

The forces acting on a typical slice are shown in Figure 18. Figure 19 illustrates the forces considered in the Fellenius and simplified Bishop's method of slices. In the Fellenius method the resultant of all side forces is assumed to act parallel to the slope. A projection line is drawn perpendicular to the base of slice, and the normal effective forces are summed along the direction of this projection line. In this way, the forces acting on the sides of any slice are assumed to have zero resultant in the direction normal to the failure arc for that slice. On the other hand, for the simplified Bishop's method, the resultant of all side forces is assumed to act perpendicular to the side of the slice, and the normal effective forces are summed along the projection on the vertical. In this way the interslice forces are neglected.

The pertinent forces which act on a typical single slice shown in Figure 18, and their notations are as follows:

1) The total weight of the slice, W.
2) The total normal force acting on the base of slice, P, having two components:
   (i) the effective normal reaction, P', and
   (ii) the total force due to porewater pressure, U, which is equivalent to uL or ub sec α.

where:

u = unit porewater pressure,

b = width of slice,

L = b sec α = length of base of slice,

α = angle between failure surface and horizontal.

3) The horizontal interslice forces, E_n and E_{n+1}.
4) The vertical interslice forces, T_n and T_{n+1} ; where n denotes the nth slice.
5) In terms of effective stress, the mobilized shear force satisfying Coulomb's shear strength equation is given by:

$$S_m = \frac{c'L}{F} + \frac{P' \tan \phi'}{F}$$

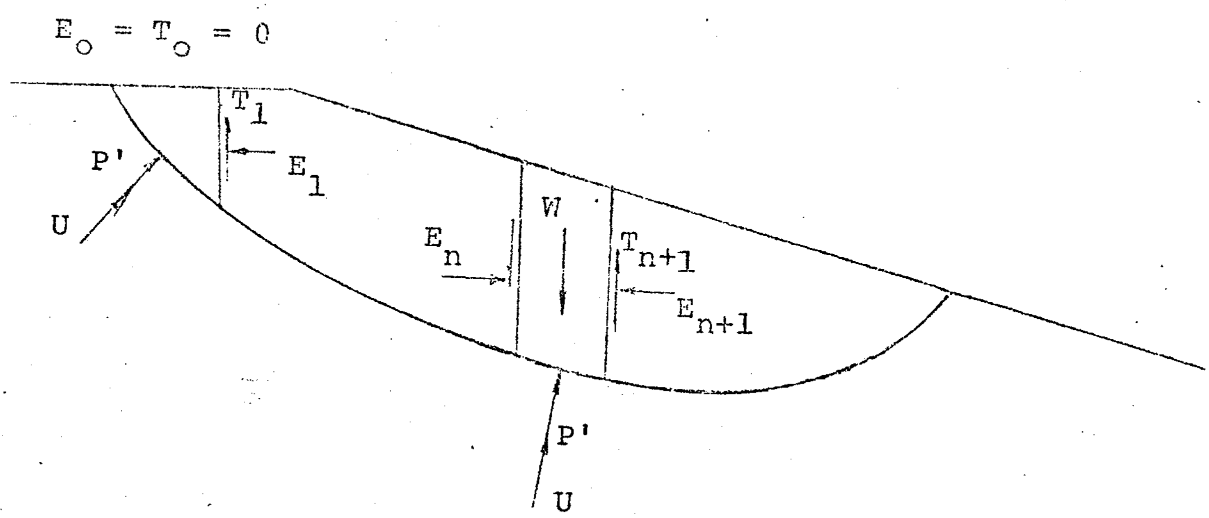


Figure 18 Complete System of Forces Acting on Slope

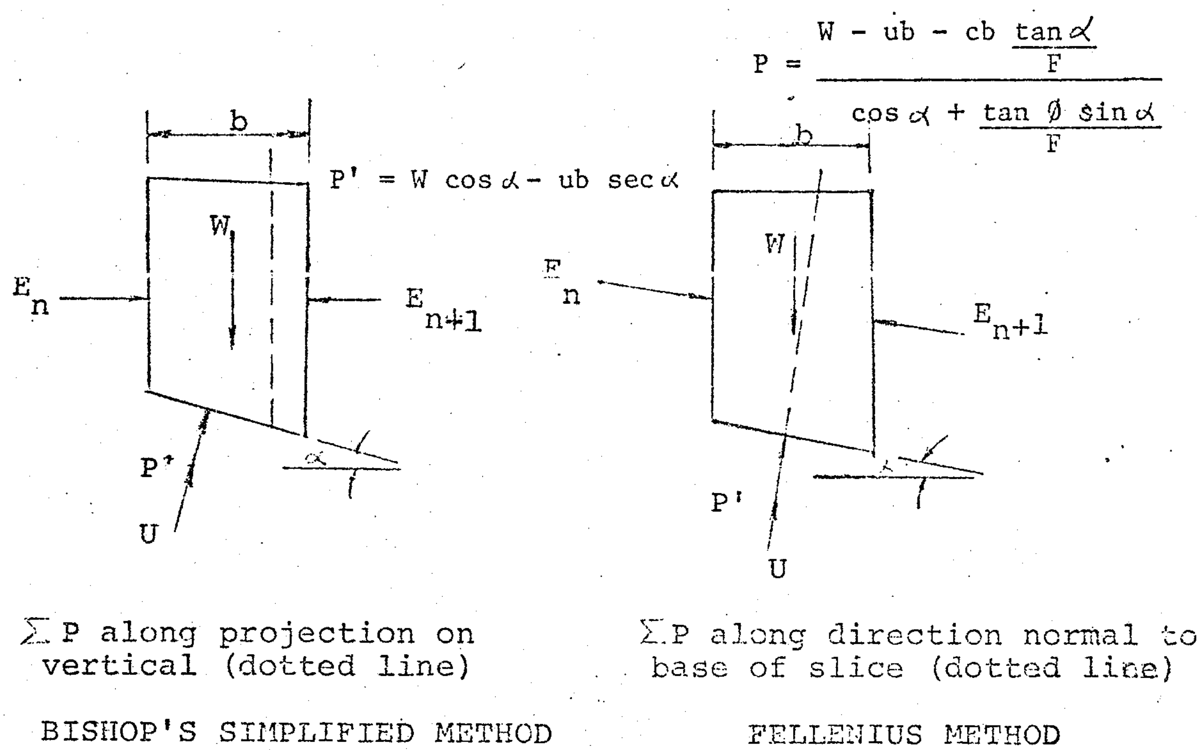


Figure 19 Forces Considered in Method of Slices
(After WHITMAN and BAILEY [34])

where:

c' = cohesion intercept,

ϕ' = angle of shearing resistance

F = the factor of safety for the stability of the slope as defined in §6.1.

For the Fellenius method of slices, the equation for the factor of safety is:

$$F = \frac{\sum\left[c'L + (W\cos\alpha - uL)\tan\phi'\right]}{\sum W\sin\alpha} \qquad \ldots\ldots (6.1)$$

The stress distribution is not known in advance, and in calculation of the effective normal stress distribution, the slice or free body is not placed in complete equilibrium. The system of slices is statically indeterminate and can only be solved if some simplifying assumption is made. The factor of safety calculation based on Fellenius method is generally underestimated. The shortcomings of this method was discussed in detail by WHITMAN and BAILEY[34].

The equation for the factor of safety in Bishop's method of slices is given by:

$$F = \frac{\sum\left\{c'b + \left[(W - ub) + (T_n - T_{n+1})\right]\tan\phi'\right\}(1/m_\alpha)}{\sum W\sin\alpha} \qquad \ldots(6.2)$$

where m_α is a function of α and ϕ':

$$m_\alpha = \cos\alpha\left(1 + \frac{\tan\alpha\tan\phi'}{F}\right)$$

The simplified version of this method was developed by assuming horizontal interslice forces acting on the vertical sides of a slice and neglecting the components of these forces in the vertical direction. By assuming $\sum\left[(T_n - T_{n+1}) \tan \emptyset'\right]$ to be zero, the equation for the factor of safety is then simplified to:

$$F = \frac{\sum\left[c'b - (W - ub) \tan \emptyset'\right] (1/m_\alpha)}{\sum W \sin \alpha} \qquad \ldots\ldots (6.3)$$

The omission of vertical components of the interslice forces introduces a slight error without loss of accuracy in the computation of the factor of safety. For Bishop's method of slices, the shear strength distribution along the assumed failure arc is again not known in advance, and the free body is also not placed in complete equilibrium in the calculation of the effective normal stress distribution. Fortunately, the additional iterative procedure, facilitated by the computer which iterates and adjusts by trial-and-error procedure until the safety factor is balanced on both sides of the equation (6.3), yields calculated safety factors which are fairly accurate, despite the fact that this method fails to satisfy statics completely.

(B) RESULTS OF THE ANALYSES.

The stability analyses with circular slip surfaces by both Fellenius and simplified Bishop's methods were applied to the two adjacent riverbank sites at Tache Avenue in St. Boniface. Average values of strength parameters : $c'_r = 0.6$ p.s.i. and $\emptyset'_r = 12.7^{\circ}$ were used according to the test results of soil samples obtained from these sites (JANZEN[9]). A comparison of the analytical results of the present study and those obtained by JANZEN[9] is given in Table 5.

The results of the stability analyses using the Fellenius method are summarized in Table 6.

Errors involved in both Fellenius and simplified Bishop's methods were discussed in details by WHITMAN and BAILEY[34]. Serious errors are introduced in Bishop's method when deep failure surfaces are assumed with high porewater pressure condition. Fortunately the present riverbank sites under study have gentle slopes, and the potential failure surfaces are not so deep-seated as to be affected by the critical porewater pressure conditions. On the other hand, the Fellenius method is in greater error in the safety factor computation than the simplified Bishop's method. The assumed direction for the effective normal forces acting on the base of each slice, P', has a noticeable effect on the safety factor computation. As indicated on Figure 19 (a), the direction of the acting forces leads to an equation for P' given by:

$$P' = W \cos \alpha - ub \sec \alpha \qquad \ldots\ldots (6.4)$$

The following results were discussed by WHITMAN and BAILEY[34]:

1) For slopes with no porewater pressure involved, the error in safety factor computation increases with increasing central angle subtending the failure arc. This error

TABLE 5

COMPARISON OF RIVERBANK STABILITY ANALYSES RESULTS

FOR ST. BONIFACE SITES

Site	Date	PRESENT STUDY ‡				JANZEN** (Safety Factor)		
		Safety Factor (A)	Critical Circle	Centre Co-ordinates	Radius (Feet)	Effective Stress	Undrained Analysis	Comment
St. Boniface Site #1	Dec. 20, 1969 Jan. 24, 1970	0.956 0.925	0.923 0.903	(90,825) (90,825)	111 111	1.08 1.07	2.11 2.11	Winter Condition
	Apr. 25, 1970	1.528	1.315	(95,765)	51	13.13	7.56	Spring Inundation
	May 22, 1970	1.021	0.973	(95,835)	123	4.09	4.75	Spring Drawdown
	Aug. 17, 1970	1.054	1.054	(99,812)	100	1.65	2.59	Summer Condition
	Nov. 26, 1969 Nov. 7, 1970	0.932 1.041	0.900 0.985	(95,825) (93,835)	123 123	0.95 1.54	2.11 2.46	Fall Drawdown
* St. Bon. Site #2	Nov. 7, 1970	* 0.860 (1)	Suspect piezometric data					Fall Drawdown
	Dec. 11, 1970	* 0.896						Winter Condition
	Apr. 23, 1971	* 0.787						Spring Inundation
	Aug. 6, 1971	* 0.777						Summer Condition
	Dec. 10, 1971	* 0.868 (2)						Winter Condition

(A) Assumed slip circle with centre at (100', 799') and radius 85.5 feet

† Factor of safety **computed by Bishop's simplified method with porewater pressure** calculations based on piezometric levels.

** Factor of safety computed by sliding block analysis with resultant water force computation based on equipotential lines.

* Piezometric data suspect, analyses with piezometric level based on site No. 1 piezometric readings yielded factors of safety of:

(1) 1.028 for Nov. 7, 1970
(2) 0.921 for Dec. 10, 1971

Assumed slip plane with centre at (96', 800.5') and radius 87 feet

TABLE 6

STABILITY ANALYSES BY FELLENIUS METHOD

SITE	DATE	FACTORS OF SAFETY		Critical slip circle	
		Assumed slip plane with centre at (0+99 Ft., 815 Ft.); radius=100 Ft.	Critical Circle	Centre Coordinates	Radius (Feet)
St. Boniface Site No. 1	December 20, 1969	0.856	0.770	(80, 794)	85.5
	January 24, 1970	0.805	0.740	(85, 789)	80.5
	April 25, 1970	1.390	1.253	(120, 809)	80.5
	May 22, 1970	0.851	0.851	(100, 799)	85.5
	August 17, 1970	0.953	0.953	(95, 799)	85.5
	November 7, 1970	0.961	0.866	(90, 789)	80.5
	November 29, 1971	1.051	0.984	(80, 799)	85.5
St. Boniface Site No. 2	November 7, 1970	0.781	-----	- - -	----
	December 11, 1970	0.753	-----	- - -	----
	April 23, 1971	0.657	-----	- - -	----
	August 6, 1971	0.648	-----	- - -	----
	December 10, 1971	0 731	-----	- - -	----
	April 15, 1972	0.651	-----	- - -	----

results from underestimating the normal force, P, along the steeply inclined portion of the failure surface.

2) For slopes with large porewater pressure, the term "ub" in Eq. (6.4) will be nearly equal to the W value. To prevent P' from becoming negative, P' is usually set equal to zero for such slices. The shear resistance and hence the safety factor is under-estimated because generally there is some value for the effective stress acting at the base of these slices in reality. The larger the porewater pressure, the greater the number of slices that will be affected in this way, and hence the larger the error.

Comparison of the results obtained by Fellenius and simplified Bishop's methods indicated that in the present study, the former method gave values of factors of safety much lower than those obtained by the Bishop's method. For St. Boniface Site No. 1, the approximate values of safety factors obtained by Fellenius method were close to those obtained by the simplified Bishop's method. For St. Boniface Site No. 2, owing to the existence of high porewater pressure, the computed safety factors given by Fellenius method were so low that they indicated that failure of the riverbank had occurred.

On the other hand, the simplified Bishop's method with porewater pressure calculations based on piezometric levels yielded a safety factor approximately equal to unity assuming a circular slip surface. The failed slopes have safety factors nearly equal to unity, indicating that they are at impending failure or limiting equilibrium. Values of factors of safety less than unity suggests that the slope is in a state of movement, although imminent failure may not occur abruptly. ZARUBA[35] suggested that the factors of safety with values less than 0.95 would indicate rapid movement of the cohesive slope. When applying the effective stress analysis using the simplified Bishop's method to the St. Boniface riverbank sites, the factor of safety obtained from each analysis based on the given ground-water condition was compared to the slope movement versus time graph (Figure 13). The analyses for all groundwater conditions during a year cycle for site No. 1 were satisfactory, giving factors of safety having values greater than unity for the period when the riverbank was stable, and yielding values less than unity for the period when the riverbank was in a state of movement. A closer look at the resulting factors of safety for the circular slip surface based on slope indicator data, indicated that these values were not the lowest safety factors for the theoretically most critical slip circles. By drawing contour lines of equal factors of safety, the most critical circle was found to be flatter and extended further up the slope than the circle representing the pre-existing failure surface as shown in Figures 21 and 22. This deviation may be due to the inadequacy of present methods of analyses or a change in the geometry of the slope from the time of the failure to the present.

As pointed out in chapter 4, the piezometric regime of St. Boniface Site No. 2 was disturbed by extraordinary groundwater conditions which could be the result of leaks from water mains in the neighbourhood, and the resulting piezometric levels yielded low factors of safety. The piezometric data collected were in suspect for this site because no corresponding significant slope movement data were recorded. In order to obtain a more reasonable stability analysis of this riverbank site, the piezometric regime for the porewater pressure calculation was based on the piezometric readings obtained from St. Boniface Site No. 1 during the corresponding period. This adjusted analysis is partly justified because the two test sites in St. Boniface are adjacent. The piezometric regime and circular slip surfaces used in this riverbank site are shown in Figures 20(a) and 20(b).

6.5 RIVERBANK STABILITY ANALYSIS WITH COMPOSITE SLIP SURFACE.

For the great majority of cases, using the assumption of a circular slip surface and the simplified Bishop's method does not lead to substantial error. Circular slip surface analysis was not applied to the St. Vital site in the present study because VAN CAUWENBERGHE[10] had compared the circular with that of the non-circular slip surface analysis based on the slope indicator data was more satisfactory. When there are large variations in the soil conditions which depart markedly from

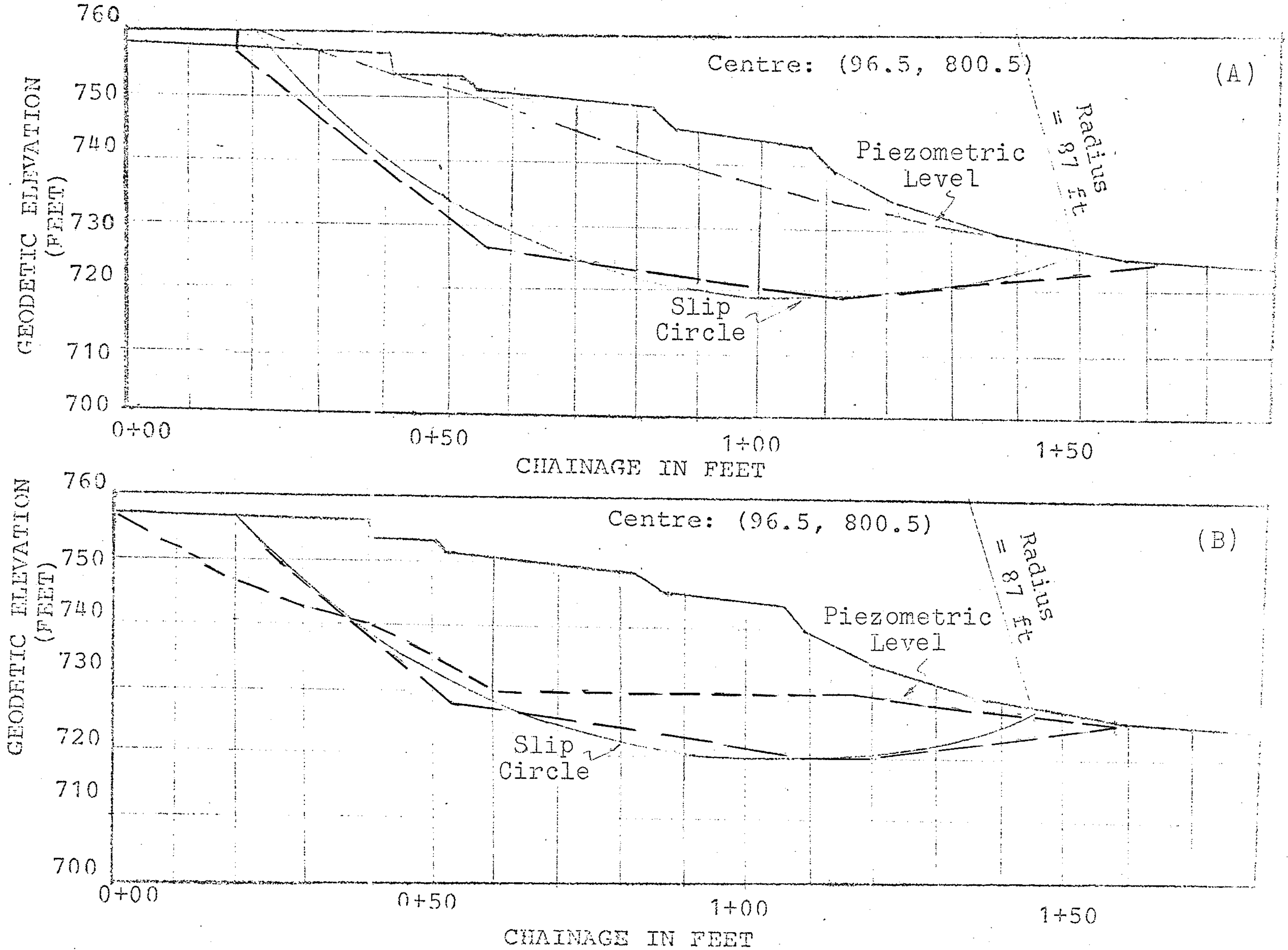


Figure 20 - Piezometric Levels and Circular Slip Surfaces used in Stability Analysis For St. Boniface Site No. 2. (A) Piezometric Level Based on Site No. 2 Data. (B) Piezometric Level Based on St. Boniface Site No. 1 Data

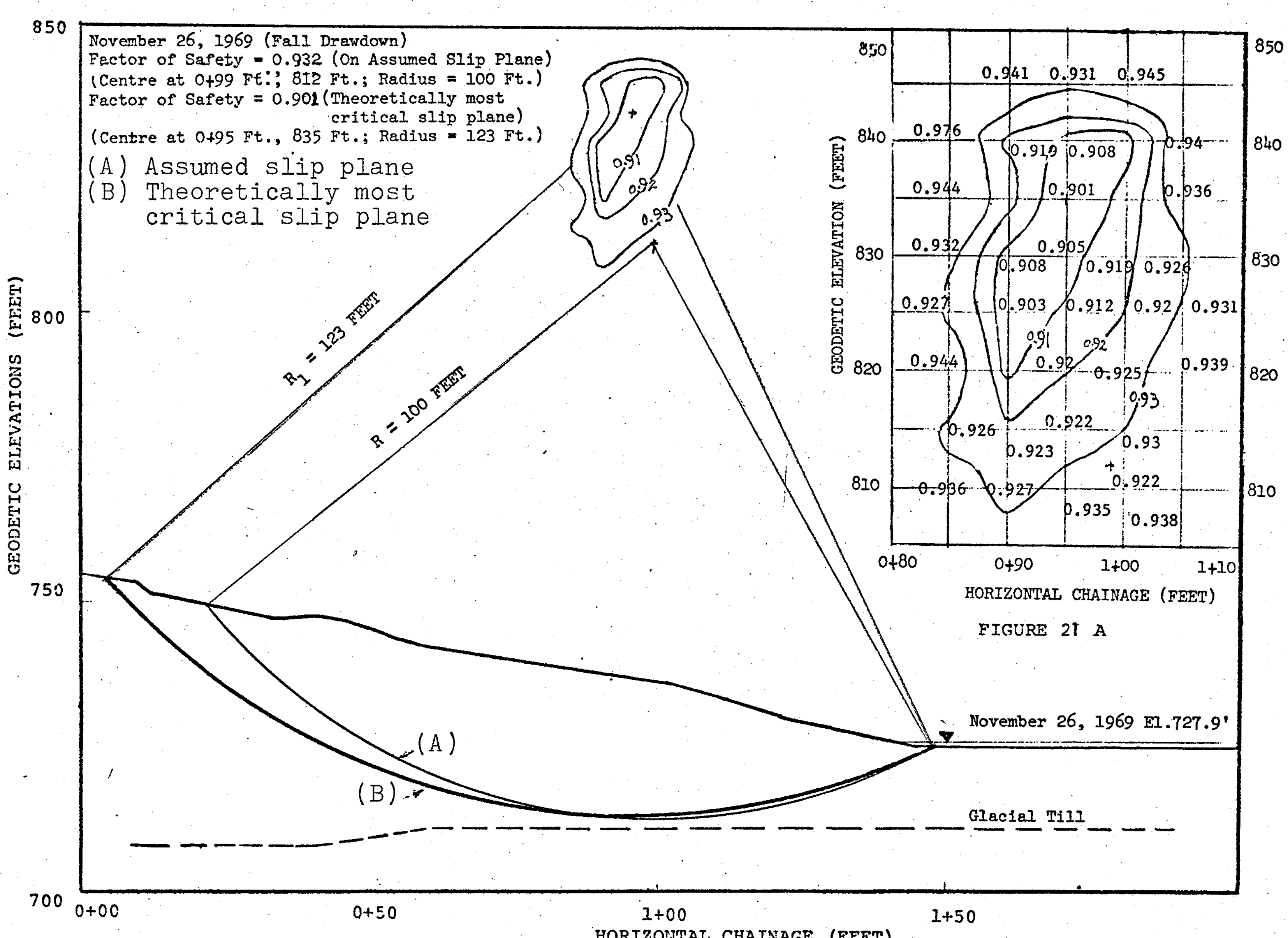


Figure 21 Contours of Safety Factor and Location of Slip Circles
(St. Boniface Site No. 1)
Figure 21A - Enlarged Inset of Safety Factor Contours

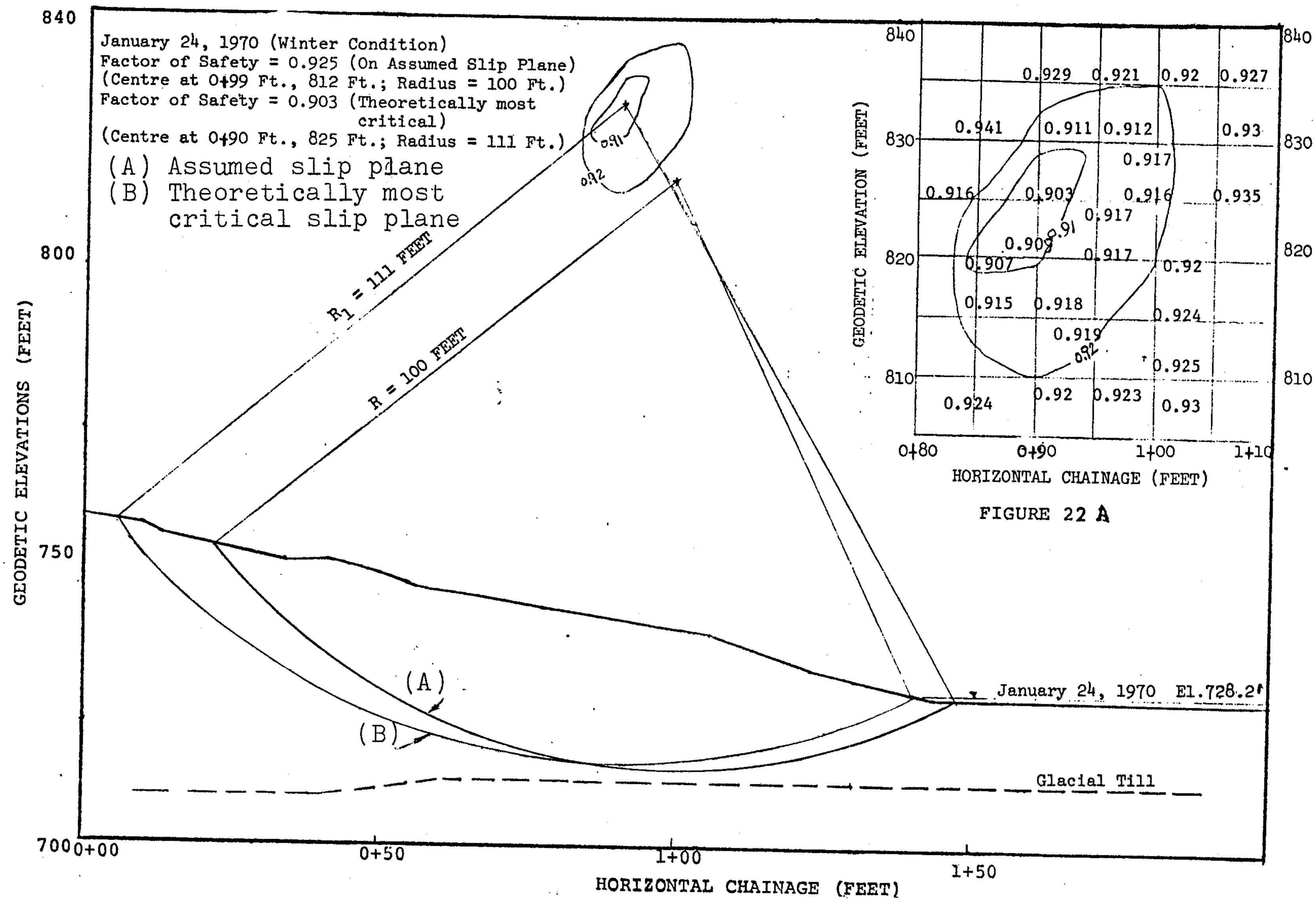


Figure 22 Contours of Safety Factor and Location of Slip Circles
(St. Boniface Site No. 1)
Figure 22A - Enlarged Inset of Safety Factor Contours

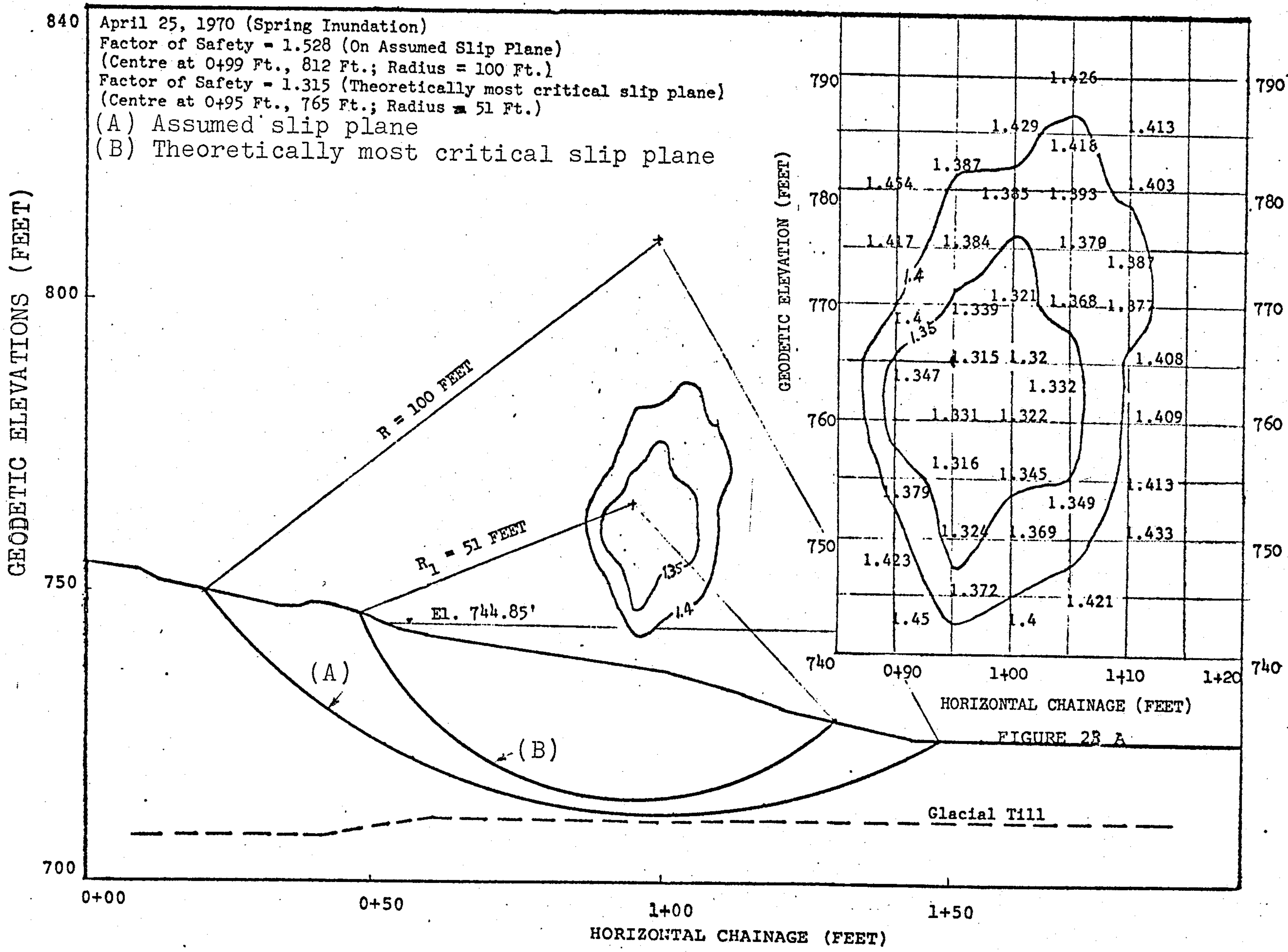


Figure 23 Contours of Safety Factor and Location of Slip Circles (St. Bon. Site #1)
Figure 23 A _ Enlarged Inset of Safety Factor Contours

the homogeneous, isotropic assumptions, then the true rupture surface cannot be approximated by the circular form because substantial errors would be introduced in the analysis. When this occurs, an analysis using a non-circular surface which approximates more closely to the form of the true failure surface is necessary. A discussion of several analytical methods is given below.

(A) REVIEW OF SOME ANALYTICAL METHODS.

In the non-circular slip surface analysis, assumptions are made to make the problem statically determinate. Some analytical methods were proposed by such investigators as JANBU[29], MORGENSTERN and PRICE[30], NONVEILLER[31] and others. The common characteristics of these proposed methods is that interslice forces are taken into account, and the ratio of the resultant vertical interslice forces (T_n - T_{n+1}) and the resultant horizontal interslice forces or thrusts (E_n - E_{n+1}) acting on each individual slice are taken as some specified constants. The complexity of the iterative procedure involved virtually makes the use of a digital computer necessary as an aid for analysis.

Morgenstern and Price method of slices satisfies both the force equilibrium and moment equilibrium conditions. The procedure assumes that the position of the reaction force is known and the interslice forces are assumed to be governed by an expression of the form (in terms of effective stress values):

$$X = \lambda f(x) E' \qquad \ldots\ldots (6.5)$$

where X is the slope of the internal forces in vertical sections, f(x) is some specified function of the horizontal dimension x, and λ is a coefficient which may be modified as necessary during the iterative process. In this method, the equilibrium conditions are successively applied from slice to slice and the moment condition is related to the centre of the base of each slice. The variables f(x), λ, as well as the safety factor F, appearing in the failure criterion, must satisfy the appropriate boundary conditions. The statically determinate solution obtained does not appear to be very sensitive to the arbitrarily chosen function f(x), which may be given any reasonable form.

The method theoretically satisfies all equilibrium equations, but it does not free the engineer from judging whether the solutions are reasonable. It is necessary to ensure that the chosen form of f(x) does not imply the existence of inappropriate conditions such as tension in the soil or X forces which exceed the shear strength of the soil.

A modified method involving the use of a "line of thrust" and the computation of the required angle of frictional resistance was proposed by WHITMAN and BAILEY[34]. The mobilized frictional resistance on the vertical sides of slices should be less than the available frictional resistance everywhere in the failure mass. The line of thrust is the line which passes through the points of action of the interslice forces. BISHOP[27] pointed out that the positions of the lines of thrust between the slices should be reasonable, ensuring that no unbalanced moment should be implied in any slice. Besides using a trial-and-error method to adjust the position of thrust lines, SPENCER[37] proposed a thrust line criterion for stability analysis, which is analogous to MORGENSTERN and PRICE[30] method of slices.

For non-circular slip surfaces, the JANBU[29] method of slices for a composite surface could be used for stability analysis. In this method, the overall condition of equilibrium in a horizontal direction is taken as the stability criterion. The derived equation for the factor of safety is given by:

$$F = \frac{\sum\left\{c'b + \left[(W - ub) + (T_n - T_{n+1})\right]\tan\phi'\right\}(1/m_\alpha)}{\sum\left[W + (T_n - T_{n+1})\right]\sin\alpha} \quad \ldots\ldots (6.6)$$

Also no simplifying assumption is made in the derivation regarding the shape of the potential failure surface, the method can be applied to any specified or chosen surface. A more rigorous solution was proposed by VAN CAUWENBERGHE[10] by positioning the line of thrust and incorporating the method of WHITMAN and BAILEY[34] to ensure that no physical imbalance would be implied in the solution.

An analytical solution was derived independently by NONVEILLER[31] in which the moment of all slices were related to an arbitrary pole. The reaction force P' was assumed to act at the centre of the slice. The tangential components T of the effective internal or interslice forces were chosen in such a way that their differences ($T_n - T_{n+1}$) satisfied the equilibrium conditions. The working equation for calculating the factor of safety was proven to be the same as that derived earlier by JANBU[29]. The resulting equations are suited for computations by digital computer. However, the porewater pressure distribution term is expressed in terms of excess porewater pressure with respect to the water level. It is suitable for partially submerged and dry slopes, but not for the case in which porewater pressures are measured by piezometers.

(B) RESULTS OF ANALYTICAL SOLUTIONS FOR NON-CIRCULAR SURFACES.

The non-circular slip surface analysis was applied to the St. Vital site because the failure surface could be approximated by a sliding block described by a series of straight lines. The slip line was based on the slope indicator readings. As the present site had undergone movement in the past, the strength parameters used were the residual values, $\phi'_r = 11^o$ and $c'_r = 0$. The results of the analyses using the Janbu solution are summarized in Table 7, and the results for the critical porewater pressure conditions are given below:

(1) Winter Condition (December 31st, 1970) : S.F. = 1.02

(2) Spring Inundation (April 12th, 1971) : S.F. = 2.12

JANZEN[9] and VAN CAUWENBERGHE[10] analyzed the same slope with groundwater condition corresponding to the drawdown and

TABLE 7

COMPARISON OF RIVERBANK STABILITY SAFETY FACTOR ANALYSES FOR ST. VITAL SITE

DATE	RESULTS OF PRESENT STUDY† (c'_r = 1 p.s.i., ϕ'_r = 9.5°)	JANZEN'S RESULTS		COMMENT
		Effective Analysis*	Undrained Analysis‡	
Nov. 20, 1970	1.199	0.95	1.98	Fall Drawdown
Nov. 19, 1971	1.023	----	----	
Dec. 20, 1969	1.044	0.92	1.98	Winter Condition
Dec. 31, 1970	0.998	----	----	
Apr. 12, 1971	2.116	----	----	Spring Inundation
May 24, 1971	1.220	----	----	Spring Drawdown
Aug. 6, 1971	1.373	----	----	Summer Condition

Notes:

† Non-circular slip surface

* Three-block system analyses with effective strength parameters: C'_r = 0 p.s.i. and ϕ'_r = 8.1°

‡ For undrained analysis, strength parameters used: C_u = 0.5 k.s.f., ϕ_u = 0°

winter conditions. By calculating the resultant water force based on equipotential lines, JANZEN[9] obtained a factor of safety of 0.93 for the winter condition in his analysis. VAN CAUWENBERGHE[10] used a phreatic surface based on the piezometric readings, and obtained a factor of safety of 1.39, using the rigorous Janbu solutions for the same groundwater condition. By using a higher phreatic surface, a lower value of 1.23 was obtained. The factor of safety for the sliding slope should be less than unity.

In the absence of piezometric readings, the porewater pressure terms of the derived equations in previous sections could be replaced by the product of the pore-pressure ratio, r_u, and the overburden weight of the material above a point or plane under consideration (usually along the failure plane). For stability analysis using the method of slices, r_u, is a dimensionless parameter which is equal to the ratio of pore-water pressure to the weight of the material composing the slice, above the centre of the base of a slice. Based on the piezometric readings, the average pore-pressure ratio for the winter condition of the St. Vital riverbank site was found to be 0.54 and the resulting analysis using the rigorous Janbu solution yielded a factor of safety of 0.97.

As the factor of safety of a slope at impending failure is known to be unity, another analysis was made using BISHOP and MORGENSTERN[36] method to estimate the porewater pressure condition. The major portion of the flattened slip surface of St. Vital site was found to be nearly parallel to the long, gentle slope and consequently, a semi-infinite slope assumption was made. The simplifying assumptions along with the residual strength parameters $c'_r = 0$ and $\phi'_r = 11^\circ$ lead to the following equation for factor of safety (BISHOP and MORGENSTERN[36]):

$$F = \frac{\tan \phi'}{\tan \beta} (1 - r_u \sec \beta) \qquad \ldots\ldots (6.7)$$

where:

β = slope angle (from horizontal)

r_u = pore-pressure ratio with respect to the failure plane

For the St. Vital site, the effective angle of shearing resistance and slope angle are known to be 11° and 6°, respectively. Setting the factor of safety for the sliding slope to unity, the resulting pore-pressure ratio required to satisfy this limiting equilibrium condition was found to be 0.45. Using this value for pore-pressure ratio, an analysis based on the rigorous Janbu solution gave a factor of safety of 1.13. However, the actual pore-pressure ratio had an average value of 0.58 for the winter condition, and the factor of safety was found to be about 0.97 as indicated earlier in this section. A revised estimation using 0.58 for r_u and substituting into equation (6.7) gave a factor of 0.85. The deviation in the estimated factor of safety and that obtained by the more accurate analysis is due to the simplified assumption of a semi-infinite slope, and the relatively low value of angle of shearing resistance ($\phi'_r \approx 11^\circ$) for a "cohesionless" soil. However, the BISHOP and MORGENSTERN[36] method gives a satisfactory approximate analysis if the actual pore-pressure ratio is known.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATION

7.1 CONCLUSIONS.

The following conclusions were arrived at from the study of the riverbank sites in the Winnipeg area:

(1) The long-term stability of the riverbanks depends on:

a) the resultant shear strength τ, given by,

$$\tau = \bar{\sigma} \tan \phi'_r + c'_r$$

where $\bar{\sigma}$, ϕ'_r, and c'_r are the usual notations for effective normal stress, residual frictional angle, and residual cohesive strength, respectively;

b) the porewater pressure condition which can be high during winter months because of slow pore pressure dissipation;

c) fluctuation in river levels.

(2) Piezometric readings indicated that the porewater pressure was not hydrostatic for the riverbank sites and that the porewater pressure fluctuation was affected by such factors as river level, groundwater pumping, snow thaw and precipitation.

(3) Based on the investigations of previous investigators, the residual values of the shear strength parameters should be used for slopes or riverbanks having pre-existing slip surfaces. The present study also confirmed that effective stress analysis of slope stability using drained shear

strength parameters was appropriate for the long-term stability analysis of a failure occurring for the first time.

(4) The simplified Bishop's method gave a good approximation of factors of safety computations for a slope having circular slip surfaces. The effective stress analysis for the critical winter condition at St. Boniface Site No. 1 yielded a factor of safety of 0.96 along the slip plane, the location of which was based on slope indicator data.

(5) The rigorous Janbu solution gave an accurate and reliable analysis for the St. Vital slope having non-circular slip surfaces. The effective stress analysis for critical winter condition at this site yielded a factor of safety of 1.02, indicating that for practical purposes, the slope was in a state of failure.

(6) The rate of slope movements varied according to the groundwater conditions. The slope movements accelerated during periods of porewater buildup when the effective soil strength was reduced. The slope movements were slower after dissipation of the excess porewater pressure, and the slopes appeared to be stationary when they were stabilized by the high river level.

7.2 RECOMMENDATION FOR FURTHER STUDY.

The trend in studying slope stability is to emphasize the stress-strain relationship of the soil mass before and just prior to failure. It would be appropriate in future studies of the riverbank stability to study the stress-strain relationship and to take into account the development of progressive failure, existence of several failure surfaces within the soil mass, etc. The soil may be treated as a strain-softening material and an appropriate mathematical model could be set up for the study.

The use of finite element technique as a tool is recommended for studying the stress-strain relationship. The finite method is a numerical approach which possesses certain characteristics that take advantage of the special facilities offered by the high-speed computers. In particular, the method can be systematically programmed to accomodate such complex and difficult problems as non-homogeneous materials, non-linear stress-strain behavious, and complicated boundary conditions. These complexities are difficult to be accomodated in other methods.

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