THE UNIVERSITY OF MANITOBA

SLOPE STABILITY CONSIDERATIONS

OF A

RIVERBANK IN METROPOLITAN WINNIPEG

by

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A THESIS

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ABSTRACT

Pneumatic piezometers and slope indicators were installed, in a riverbank along the Red River in Metropolitan Winnipeg, to monitor porewater pressures and possible riverbank movements. The instrumented riverbank had failed previously and was later partly stabilized. The performance of the pneumatic piezometers was successful as they were capable of rapidly measuring the porewater pressures developed in the relatively impervious Lake Agassiz Clay. Piezometric conditions were determined for the period November 1968 to May 1969, inclusive.

The porewater pressure data indicated that the major component of the hydraulic gradient was in a downward direction. The most critical piezometric condition in terms of slope stability was not determined as the time of measurement was too short. The two slope indicators presented data which indicated that the riverbank was in a slow state of movement. The slope indicator data also indicated that a major portion of the slip zone was immediately above the clay-till interface, and that the movement of the riverbank was predominantly in a lateral direction. The best approximation of the shape of the slip surface was found to be that defined by the sliding block analysis.

Employing effective "residual" shear strength parameters for the soils within the slip zone, slope stability analyses were conducted by the Fellenius, Simplified Bishop's and Janbu Methods. The Janbu method was considered to be the superior method of analysis because its non-circular slip surface feature enabled the theoretical slip surface to have a geometry very similar to the observed slip surface.

ACKNOWLEDGMENTS

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TABLE OF CONTENTS

PAGE

PAGE

LIST OF FIGURES

FIGURE PAGE

LIST OF TABLES

LIST OF SYMBOLS

α = angle between base of slice and horizontal,

α_t = angle between interslice force and horizontal,

b = slice width,

c' = cohesion in terms of effective stress,

c'_R = residual cohesion in terms of effective stress,

d = depth or height of slice,

F = the Factor of Safety for the stability of the slope. The definition of the Factor of Safety is as follows:

$$F = \frac{\text{available shear strength}}{\text{shear strength required for equilibrium}}.$$

γ_t = total unit weight of a material.

H_n = horizontal interslice force,

H_{n+1} = horizontal interslice force,

L = length of base of slice

P = total reaction normal to the base of the slice,

P' = effective reaction normal to the base of the slice,

ϕ' = angle of shearing resistance in terms of effective stress

$\phi'_{available}$ = angle of shearing resistance in terms of effective stress which is available on the vertical side of the slice,

ϕ'_R = angle of shearing resistance in terms of effective stress,

$\phi'_{req'd}$ = angle of shearing resistance in terms of effective stress which is required on the vertical side of the slice,

S_m = mobilized shear force,

U = porewater pressure,

u = unit porewater pressure,

LIST OF SYMBOLS

V_n = vertical interslice force,

V_{n+1} = vertical interslice force,

W = total weight of the slice.

SLOPE STABILITY CONSIDERATIONS

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CHAPTER I

INTRODUCTION

1.1 OBJECTIVES

Pneumatic piezometers were installed in a riverbank along the Red River in Metropolitan Winnipeg to monitor porewater pressures. This type of instrumentation had not previously been used on riverbanks in the Winnipeg area. Therefore a prime objective was to evaluate the installation techniques and actual performance of this instrument in the local riverbank soils.

It was known that a slip surface or slip zone existed within the riverbank because the riverbank had failed previously and was later partly stabilized. Therefore further objectives were to determine more accurately the location and geometry of the slip surface, and to determine the rate of riverbank movement, if in fact it was presently in a state of movement. The determination of these latter objectives were aided by two "slope indicator" installations.

The porewater pressure data obtained was limited to the period November 1968 to May 1969, inclusive, and therefore possible porewater pressure conditions are only partly represented. The continuation of obtaining data was a part of another investigation.

In order to utilize the porewater pressure data obtained, computer programs were developed based on the Fellenius[1], Bishop's

Simplified[2], and Janbu[3] methods of slope stability analysis. The results obtained from each of these slope stability analyses were compared with field failure conditions to determine which method was most applicable.

1.2 PREVIOUS INVESTIGATIONS

The subject of the slope stability of the riverbanks of the Metropolitan Winnipeg area has received considerable consideration and study due to their relatively unstable characteristics. These riverbanks generally consist of plastic clays of Glacial Lake Agassiz, which are generally underlain by glacial till.

MISHTAK[4] in a 1960 survey reported that very few banks along the Red and Assiniboine Rivers were stable. He found only six stable slopes of the 141 slopes examined along the riverbanks. The unstable slopes had visible tension cracks, sloughing or toe erosion. They either had suffered failures in the last two years or were in the state of creep. The failure mechanisms appeared to be relatively complex as indicated by the fact that the shear strength of the clays in those slopes tended to reduce with time (Ref. 4). BARACOS[5] found that toe erosion on the concave portions of the river caused a gradual deterioration in the long term stability of these riverbanks. Factors such as surface drainage, internal porewater pressures, and the "rapid drawdown" of the rivers have also contributed to these instabilities.

The predominant soil profile common to the Red River Valley has previously been described by MACDONALD[6], RIDDELL[7], and BARACOS[5]. Basically the soil profile consists of an upper layer of brown clay, occasionally an intermediate layer of a brown and grey clay, and an underlying generally somewhat siltier grey layer known as "blue" clay. These glacial lake clays are generally covered by a layer of more recent siltier and organic deposits ranging in thickness from two to sixteen feet. Table 1, compiled by BARACOS[5] is a list of the depth of occurrence of these clays and some of their pertinent properties. Note that where these clays occurred in riverbanks they generally showed some considerable disturbance and lower unconfined strengths. A further list of properties and shear strength parameters of these silts and clays is presented in Table 2. It was compiled by SUTHERLAND[8] from tests conducted at the University of Glasgow on samples extracted from the Winnipeg Floodway site.

The analysis employed to determine stability of these slopes has generally been the classical "Fellenius Method of Slices" with a circular slip surface assumption. The total stress ($\phi = 0$) analysis has been used extensively, but appeared to be successful only when used with lower shear strength values than those determined from unconfined compression tests. BARACOS[5] and MISHTAK[4] found that the unconfined compressive strength of these clays averaged about 2000 lbs./sq. ft., but when this value was applied to a failed slope a safety factor greater than unity usually resulted. In 1950, A. BARACOS reported to the Greater Winnipeg Dyking Board that he found that the calculated factor of safety for slopes in the Winnipeg Clays was over-estimated. He indicated that it would be more realistic to use a value of 800 to 1200 lbs./sq. ft. for the unconfined compressive strength. SUTHERLAND[9] also recommended the application of a reduced value of shear strength when using the $\phi = 0$ analysis.

This apparent discrepancy between analytical and observed

<u>TABLE 1</u>

<u>Some Properties of Greater Winnipeg Glacial Lake Clays</u>

(From BARACOS, Ref. 5)

	Brown "Chocolate" Clay				Mixed Brown and Grey Clay				Grey "Blue" Clay			
	Max.	Av.	Min.	No. of Tests	Max.	Av.	Min.	No. of Tests	Max.	Av.	Min.	No. of Tests
Depth to top of stratum - ft.	16	11	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 - lb/cu/ft.	99	77	64	73	87	69	53	51	102	79	63	39
Moist density - lb/cu/ft.	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 - lb/sq/ft.	4570	2054	865	87	3790	2169	112	49	3570	2182	1188	44
Plastic Limit	40	30	14	36	36	31	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 II

Recent Test Results of Winnipeg Glacial Lake Clays

(From SUTHERLAND, Ref. 8)

Material	Depth	Water Content			Liquid Limit	Plastic Limit	Wet Density	Preconsol. Pressure	Residual Strength Angle	Peak Shear Strength Parameters				
		Avg.	Range	No. Obs.						Stress Range	Bulk Clay		On Laminations	
											c' p.s.i	Ø'	c' p.s.i.	Ø'
	ft.	%			%	%	p.c.f.							
Tan Silt	12	34.4	29-38	44	53	22	120	4 to 5 tsf	25	20 - 70 psi	4.2	24½	--	--
								60-80 psi		3 - 20 psi	1.6	31	--	--
										0 - 3 psi	0.8	40	--	--
										0 - 12 psi	---	--	0.4	32
										12 - 70 psi	---	--	4.5	17
Brown Clay	10	54.9	44-61	60	120	34	105	4.1-4.8 tsf	8	Complete	5.8	20.1	--	--
								60-70 psi		Complete	---	---	0.5	15
Grey Clay	25.5	45.9	40-59	83	72	25	111	2.7-3.6 tsf	10	Complete	2.0	23½	--	--
								42-56 psi		Complete	---	---	0	26½
Grey Plastic Clay	29.5	62.7	43-81	116	101	32	103	0.7 tsf	11	Complete	1.4	17.7	--	--
								11 psi		Below 10 psi	0.4	27.1	--	--
										Above 10 psi	1.7	17.3	--	--
										Complete	---	---	0.7	16

stability behaviors has generated study on possible alternative methods that should be employed to analyse the stability of a riverbank in the Red River Valley. Both BARACOS [5] and SUTHERLAND [9] have suggested that the effective stress method of analysis might provide a better correlation between analytical behavior and field performance for Winnipeg Clays.

CHAPTER 2

INVESTIGATION

2.1 ORGANIZATION OF INVESTIGATION

The steps that were undertaken in this investigation were as follows:

1. A riverbank with the following characteristics was selected:
   a) One that was potentially in danger of a slope failure, such that the installed instrumentation would monitor the activities preceding and during the anticipated failure.
   b) Riverbank site was to be readily accessible by equipment to assist the installation of required instrumentation and to obtain subsequent data.
2. A site survey was conducted to obtain a typical profile of the ground surface.
3. An extensive sub-surface investigation was conducted to determine and classify the soils of the bank, determine depth to a firm foundation material, depth of water table, and to extract a series of undisturbed samples of various soils for soil classification and laboratory shear strength determinations.
4. The following instruments were installed:
   a) A series of pneumatic piezometers, and
   b) two "slope indicators".

5. A continuous record of river levels were kept.
6. The following parameters were selected for each soil in order to conduct a slope stability analysis:
   a) Total unit weight, and
   b) Shear strength parameters.

2.2 SITE:

The riverbank under investigation is located on the Red River in the City of St. Vital within Metropolitan Winnipeg. A typical cross-section is shown in Figure 1. This riverbank is on the outside edge (concave side) of the river and therefore has been subjected to the higher velocity currents that generally prevail in these areas. Two characteristics of the site then follow:

1. It is not in a zone of river deposition and therefore the insitu soils are primarily of a glacial lake origin (lacustrine).
2. Its stability in the past was threatened by toe erosion both from river currents and Spring ice flows.

As indicated by Figure 1, the slope of the bank is relatively gentle, approximately 7 to 1. The whole section is virtually void of tree growth except for a narrow dense stand of willows along the river edge immediately North of the indicated cross-section. Therefore for the greater part the only existing vegetation is grass which extends to the river edge. Some rip-rap exists at the river edge but is not continuous. It consists of a random array of boulders and old concrete slabs ranging in size from approximately 1 to 3 ft. in diameter. Further up the slope there exists some buried rip-rap consisting of the previously mentioned

CROSS-SECTION OF RIVERBANK

at

ST. VITAL SITE

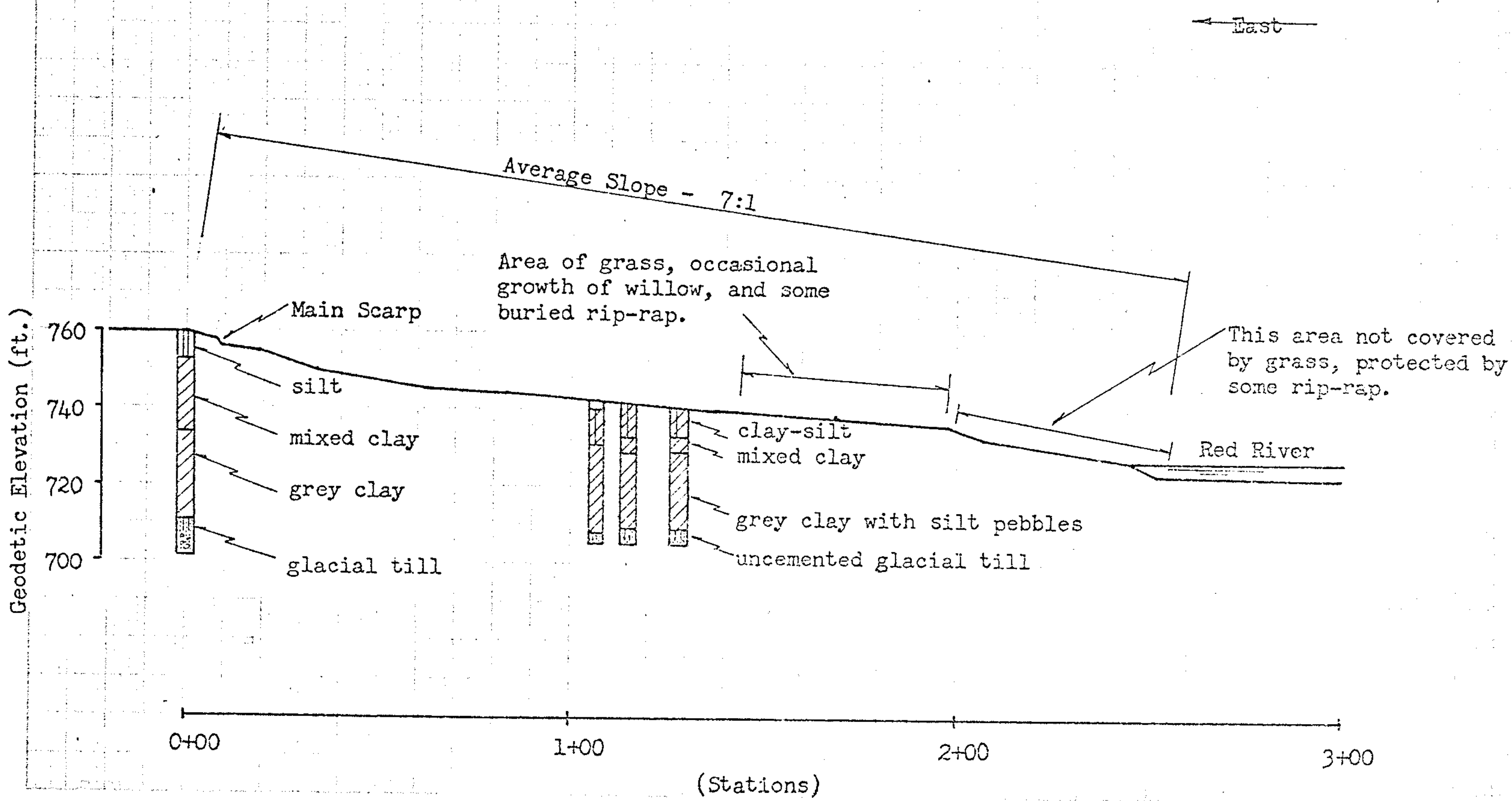

Figure 1

concrete slabs and boulders. Due to the sparce vegetation it is apparent that this rip-rap tends to retard toe-erosion during the period of Spring break-up when high water levels, currents of high velocity and movements of large blocks of ice greatly increase the potential forces of erosion.

The riverbank is a typical example of an unstable bank that has failed in the past and is now moving at a very slow rate. To stabilize this riverbank, the following measures were undertaken:

1. To provide rip-rap at the toe of the slope. Its main function was to provide resistance against erosion by the forces of river currents and ice flows. The large size of the rip-rap is important as the relatively heavy weights of the individual slabs have accounted for the partial success in resisting soil displacement by the erosion forces.
2. The steepness of the slope was reduced. The relatively gentle slope of 7:1 would normally be adequate.
3. To provide continuous grass vegetation on the slope. This measure facilitated surface water drainage and therefore minimized the quantity of water that would flow into the subsoil either by seepage or via tension cracks.

But since the time of the above mentioned stabilization measures, the slope has resumed its movement toward the river. Deterioration in stability is visibly evident as a significant scarp exists at Station 0 + 09 where an abrupt vertical drop of approximately 2 to 3 ft. exists in the ground surface profile. Further down the slope there are several ground surface cracks which run almost parallel to the main scarp. These might indicate the possibility of independent block movements within the slope.

2.3 SOIL PROFILE:

The predominate soil type of this riverbank is a highly plastic varved clay of glacial Lake Agassiz. Figure 2 illustrates several borehole soil logs as determined from the soil investigation conducted at the site. The 6 to 12 inches of topsoil consists of a highly organic material which supports the vegetative growth on the slope. Underlying the topsoil are three distinct clay layers which are readily identified by color. The upper clay layer is a light brown clay which extends to a depth of approximately 6 to 10 feet. The intermediate clay layer is a mixed brown and grey clay which ranges in thickness from approximately four feet at the bottom of the slope to almost 20 feet at the top. The bottom layer is a grey clay with numerous pebble-size silt inclusions. This clay layer rests on an almost horizontal bed of non-cemented glacial till, and has a maximum layer thickness of nearly 25 feet. These clays are classified as "CH" soils according to the Unified Classification system. They are highly plastic clays with plasticity index in the 50 to 60 percent range and wet densities of approximately 110 pounds per cubic foot.

The upper five feet of the underlying non-cemented glacial till consists predominately of a mixture of fine sand and silt with increasing content of coarser sand and pebbles with depth. The till was very dense as indicated by the penetration refusal of the sampling shelby tubes.

Existing at a depth near the bottom of the boreholes, although not actually determined in this investigation, is a carbonate rock known as the "Red River Formation" composed of dolomitic limestone and dolomite. Auger refusal at 5 to 8 foot depths into the till could be attributed either to a cemented glacial till or the limestone bedrock. According to

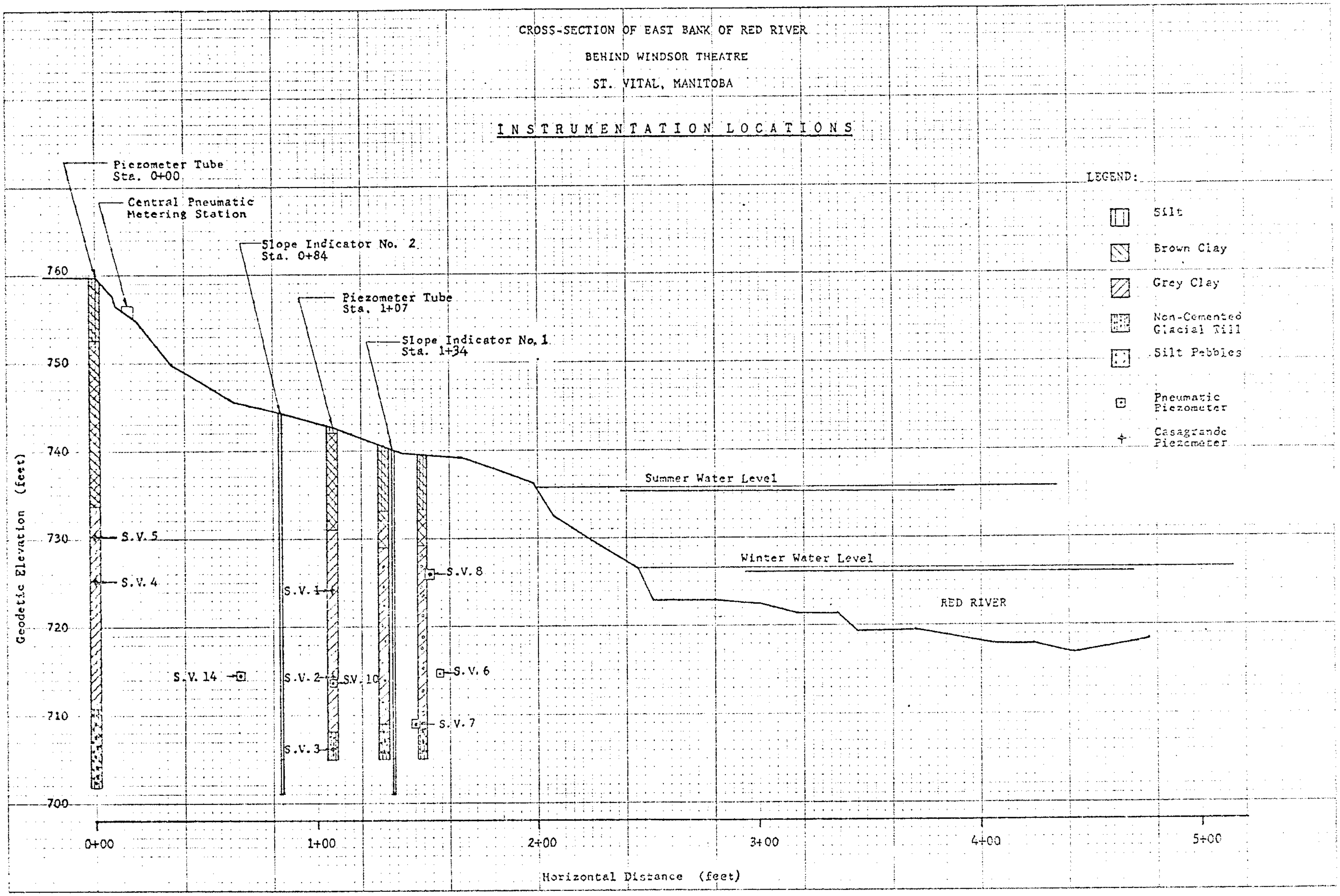
CROSS-SECTION OF EAST BANK OF RED RIVER
BEHIND WINDSOR THEATRE
ST. VITAL, MANITOBA
INSTRUMENTATION LOCATIONS
Piezometer Tube Sta. 0+00
Central Pneumatic Metering Station
Slope Indicator No. 2 Sta. 0+84
Piezometer Tube Sta. 1+07
Slope Indicator No. 1 Sta. 1+34
LEGEND:
Silt
Brown Clay
Grey Clay
Non-Cemented Glacial Till
Silt Pebbles
Pneumatic Piezometer
Casagrande Piezometer
Summer Water Level
Winter Water Level
RED RIVER
S.V. 5
S.V. 4
S.V. 1
S.V. 8
S.V. 14
S.V. 2
SV. 10
S.V. 6
S.V. 7
S.V. 3
760
750
740
730
720
710
700
Geodetic Elevation (feet)
0+00
1+00
2+00
3+00
4+00
5+00
Horizontal Distance (feet)

Figure 2

the topographic plan of the Bedrock Surface by RENDER,[10] bedrock in this particular area is in the vicinity of 700 foot elevation (Geodetic), which was the approximate elevation of the auger refusals.

CHAPTER 3

INSTRUMENTATION

3.1 PIEZOMETERS

The St. Vital site had been previously instrumented by the Manitoba Water Resources Branch with five "Casagrande" type piezometers. Initial examination indicated that these piezometers were insufficient in number, and possibly were too slow to respond to porewater pressure changes. To overcome these shortcomings, five pneumatic piezometers were installed in the autumn of 1968. These piezometers were placed at various positions on a cross-section perpendicular to the river and at various depths as indicated by Figure 2.

The pneumatic type of piezometer employed at the St. Vital site operates by measuring the air pressure required to close a hydra-pneumatic balance system. The main body of the unit is constructed of polyethlene, the main working parts are a belofram of dacron, Buna N. Springs of silicone bronze with baked teflon coating, and a neoprene O-ring. The tubing for the lines from the instrument to the terminal is constructed from heavy wall nylon enclosed in polyvinyl chloride. The two leads at the read-out terminal are easily connected to the control unit by quick couplings.

Figure 3 is a schematic diagram of the piezometer tip. The unit has a stiff diaphragm which is slightly displaced upward due to the pressure of the porewater. In order to measure the magnitude of the porewater pressure, air pressure from the pressure cylinder in the control unit is applied through Line 2A. During the pressure build-up, the air pressure flows from the control unit via Line 2A past the O-ring seal. The flow of air continues up Line 2 back up to the second lead of the control unit. The returning air pressure is recorded on a pressure gauge mounted on the control unit. When the supply air pressure equals the porewater pressure the diaphragm is displaced downward, closing the O-ring check. This closure creates a seal which precludes further flow of air into Line 2. The air pressure in Line 2 at closure is then equal to the porewater pressure and is recorded on the pressure gauge. The minute movement required to close the O-ring check causes a slight displacement of water. According to the manufacturer a 1/16 inch line, which is open to the atmosphere, allows this minute displacement of water to occur without movement of water into or out of the soil.

The pneumatic piezometric units were calibrated by the manufacturer prior to delivery in accordance to the required length of lead for each unit. These units were re-calibrated at the University of Manitoba prior to installation by directly connecting the units to variable air pressure lines incorporating sensitive air gauges. The calibration curves of the piezometers are shown in Figures 23 and 24 of Appendix "B" and were employed to correct all data monitored at the site. The calibration process indicated that great errors in readings resulted if the rate of air flow into Line2A was not regulated carefully. The input air supply valve had to be adjusted in such a manner as to allow a minimum rate of air pressure build-up. This technique prevented an air pressure build-up lag in Line 2 and therefore precluded readings which were less than actual.

The installation of these units was simplified by employing piezometers encased in well-points connected to one-inch pipe. The units were then driven into the soil either by a hammer or hydraulic force. Standard 1 inch water pipe sections were added until the required depth was reached.

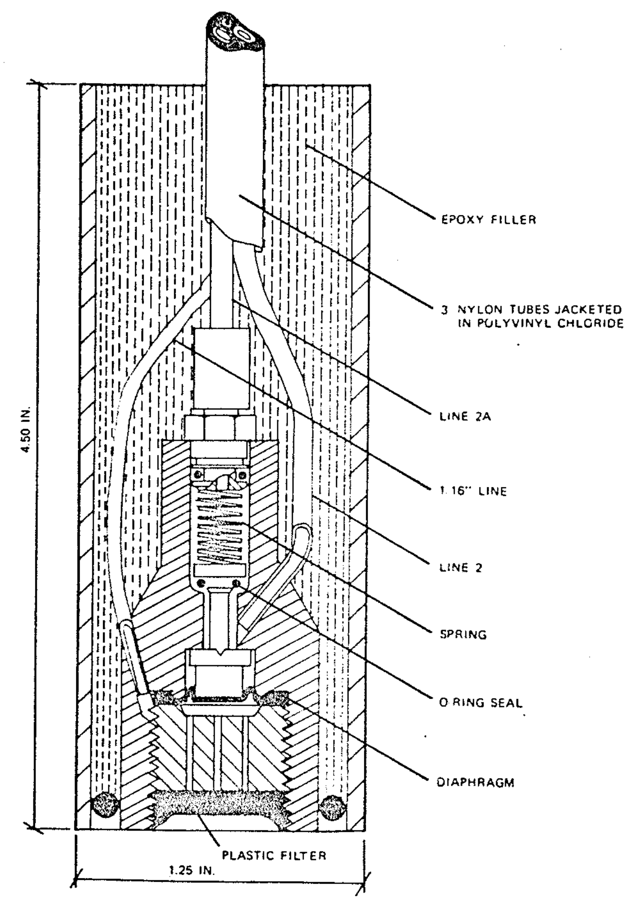

Figure 3: Schematic Diagram of the Piezometer Tip

The installation procedure was generally as follows:

1) A 4 inch diameter hole was advanced by means of flight augers, truck mounted drill, to within 10 feet of final desired depth of piezometer.

2) A well-point encased piezometer with adequate length of 1 inch water pipe was then positioned at the bottom of the hole.

3) Sufficient water was poured into the hole to completely submerge the well point.

4) The first piezometer installed was driven to the desired depth by allowing a 40 pound hammer to drop in a free fall of several feet unto the afore-mentioned pipe. A special adapter was connected to the pipe to guide the manual lifting and free fall of the hammer. For the subsequent installations the well-point was forced to the desired depth by employing the hydraulic pressure exerted by the drill truck.

5) The tubing from the piezometer unit extended vertically to within two to three feet from the ground surface, then traversed underground at a three foot depth parallel to ground surface to the Central Pneumatic Metering Station.

6) In an effort to assure that the unit would be sealed from the surface and overlying soil layers, multiple coats of fibre glass were applied to the 4 foot pipe section immediately above the unit. The fibre glass layers provided a pipe diameter slightly larger than the leading well-point unit and also furnished a rough shaped surface to create a tight pipe-soil contact.

The features of this unit that made it appealing for installation at this site are as follows:

1. The negligible time lag in developing the actual pore water pressure reading is mandatory for the relatively impervious insitu clays (coefficient of permeability is in the order of 10^{-9} to 10^{-11} cm/sec)[5] existing at the site.
2. Clay soil corrosion is eliminated by the non-metallic construction of the underground portions of the unit.
3. Two features inherent of the air system are: (1) elimination of the freezing of lines, and (2) allowance for the lines to be terminated at a single point to facilitate the collection of readings.
4. A convenient measuring procedure as facilitated by the ease of operation and movement of the light-weight portable control case.
5. Relative ease of installation as attained by employing piezometers encased in well-points.

Refer to Appendix "C" for a documentation on various types of piezometers that are presently being employed. Brief discussions are given of the operation, advantages, and limitations of each type of piezometer system.

3.2 SLOPE INDICATORS:

Slope Indicators were installed along the profile, as indicated by Figure 2, one at Station 0 + 84 and the other at Station 1 + 34. The instrument employed was the Series 200-B unit supplied by the Slope Indicator Co. of Seattle, Washington.

The instrument is lowered down an aluminum casing which has four equispaced longitudinal grooves, the grooves controlling the orientation of the instrument in the casing. The inclination of the instrument at any depth in the casing is determined by means of a pendulum activated electrical circuit. The actual value of the inclination is obtained from Wheatstone Bridge readings at the ground surface. The detailed operation of this instrument is presented in the "Instruction Manual" issued by the manufacturer. Inclination readings are taken at frequent intervals of depth and are subsequently converted to displacements. Consecutive readings at the same depth intervals at periodic intervals of time are used to determine depth and rate of ground movement. The instrument has a sensitivity of one part in 1000, which means that a tilt of as little as three minutes of arc can be detected. This corresponds to a lateral displacement of one inch in 100 feet of depth.

Aluminum casing was installed at the two locations as indicated above. The 3.18 inch outside diameter casings were lowered into holes drilled by flight augers. The casings were anchored into the firm till layer. The void on the outside of the casing was then backfilled with an expansive grout near the base of the hole, and with a sand from just above the base to the ground surface.

CHAPTER 4

OBSERVATIONS AND FIELD MEASUREMENTS

4.1 THE RED RIVER

Some of the important features of the Red River affecting the stability of its riverbanks have been cited by BARACOS[5] and are herein summarized as follows:

1. The Red River has cut a sinuous path within a relatively straight belt of approximately one mile width within the Metro Winnipeg area.

2. The terrain is relatively flat except where the riverbanks drop 30 to 50 feet to the river bed.

3. Depth of the river has been limited by the underlying firm glacial till or bedrock.

4. River velocities are generally low except in time of flooding when peak velocities range between 5 to 6 feet per second.

5. The river is subject to spring flooding with changes in level of over 30 feet occurring during major floods and frequently up to 18 feet.

Drawdown following spring peak river levels is not immediate, ranging from two to six weeks. Normal winter level of the river is at a Geodetic elevation of approximately 727 feet. The river has risen from 15 to more than 30 feet above normal winter level during Spring flooding. But since the construction of the Red River Flood Control Structure in 1969 it is not likely that the 30 foot level will be reached again. During the summer, the river level ranges from 6 to 8 feet above normal winter level as controlled by the locks located at Lockport, downstream from Winnipeg. This level is maintained throughout the summer, and then late in the Autumn the river is allowed to drop to its normal winter level.

4.2 GEOHYDROLOGY:

The geohydrology of this same area is thoroughly documented by RENDER.[10] This reference refers to a major aquifer underlying the Winnipeg area, known as the Upper Carbonate aquifer which occurs in the top fifty to one hundred feet of the Paleozoic limestones and dolomites and is confined above by the glacial drift. It is of interest to determine the effect this aquifer has on the piezometric regime of the riverbanks. The upper 25 feet of the carbonate rock of this aquifer is characterized by a network of fractures, joints, and bedding planes which provides the permeability to make it the zone of most water flow. The lower portion of the overlying glacial drift consists of boulder till and associated glaciofluvial deposits which are generally cemented.

The recharge of the Upper Carbonate aquifer occurs in three main segments:

1. From the east by infiltration in the glacial till upland east of the lacustrine plain and in the Birds Hill aquifer complex.

2. From the north-west via areas of thin glacial till.

3. From the south-west through a thin veneer of glacial till and fluvial deposits.

Several observation wells, located in the general area of the St. Vital Riverbank Test Site, are seated in the Upper Carbonate Aquifer and have recorded piezometric levels up to 735 feet (Geodetic Elevation). As the bedrock surface elevation at the test site is approximately 700