THE UNIVERSITY OF MANITOBA

STABILIZATION OF A TYPICAL CLAY RIVERBANK IN THE WINNIPEG AREA

by

Kenneth A. Buhr

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

WINNIPEG, MANITOBA

OCTOBER 1973

ACKNOWLEDGEMENTS

The writer would like to thank all those who assisted in the preparation of this thesis by providing the necessary data and encouragement. Professor A. Baracos of the Department of Civil Engineering provided advice and direction as supervisor. The Geotechnical Section of the Water Resources Branch, Provincial Department of Mines, Resources and Environmental Management under the direction of Mr. F. Penner provided the necessary engineering and technical assistance for obtaining the slope indicator and pin movement data used in the thesis. Mr. P. Janzen and Mr. G. Hunter provided the piezometric data while working as graduate students at the University of Manitoba.

TABLE OF CONTENTS

APPENDICES

APPENDIX | PAGE

LIST OF FIGURES

LIST OF TABLES

LIST OF PHOTOGRAPHS

SUMMARY

This thesis is an extension of previous studies of a typical failed clay riverbank in the Winnipeg area in order to recommend possible remedial measures once the causes of failure are known.

From the re-examination of previously reported data, more recent data, static water well levels in a nearby well, daily precipitation and earlier work by others, it concluded that:

1. residual shear strength values should be used in analyzing a failed river bank in clay and in the design of remedial measures,

2. a lowering of piezometric levels in the bank by drainage, especially at the toe, and improving the surface of the slope to facilitate the run off of surface water should result in improved stability,

3. stability of a clay bank may be maintained or improved by increasing the developed shear strength through decreasing the loads on the bank or increasing the resistance at the toe and preventing loss of soil at the toe.

A review of existing literature indicated a number of possible methods of stabilizing a clay riverbank, i.e. counterbalancing with a toe berm, reduction of the slope angle and/or height of bank, drainage of the embankment to reduce pore pressures, protection of the toe of the slope against scour, retaining structures, chemical stabilization and eliminating or reducing drawdowns. Where applicable

local examples of some of the above remedial measures are referred to. Also photographs were taken of a number of existing remedial measures along the rivers and streams in Winnipeg. The relative success of the various remedial works is appraised in light of the conditions at Tache Avenue.

Of the possible remedial and protective measures reviewed, counterbalancing the weight of the bank by means of a toe berm is considered most applicable. Chemical stabilization by means of electro-injection also appears applicable. However, because of the uncertainty of chemical stabilization in clays, it is recommended that it be initially considered for a test site. Counterbalancing and chemical stabilization are usually considered for relatively short reaches of riverbank. For a more general benefit to riverbanks throughout Winnipeg, it is recommended that consideration be given to a study of the feasibility of a fixed crest weir at Lockport to replace the present St. Andrews locks as a means of improving the stability of the riverbanks by eliminating the fall drawdown.

CHAPTER I

INTRODUCTION

The City of Winnipeg, like many older cities in Canada, had its origin on a major watercourse. Winnipeg, as a matter of fact, grew up along two major waterways, the Red and Assiniboine Rivers as shown on Figure 1. As the city grew in size, the property along the riverbanks became more and more attractive for private property, apartment blocks and parks. A major concern in construction along the riverbanks has been, and still is, the instability of the banks.

In his 1960 survey of 141 riverbanks along the Red and Assiniboine Rivers in Winnipeg, Mishtak (1) found that approximately six sites appeared to be stable, i.e. not showing signs of previous failures or evidence of recent movement. This fact was also noted in 1961 by Baracos (2) who stated, "The extent of unstable banks is such ... that practically every bank on an outside of a river curve has either been affected by an old slide, or is presently sliding, or has required stabilizing measures". Frequently banks have stood up for years only to move suddenly downslope with little warning. Others appear to have undergone a long history of creep followed by a sudden movement.

Considerable difficulty has been experienced in analyzing the failed banks and in determining the safety factor of a stable bank. Most of these difficulties stem from a limited knowledge of the internal forces causing and resisting failure. In an attempt to understand the failure-causing forces or loss of strength, studies were recently undertaken by Van Cauenberghe (3) and Janzen (4) to investigate the pore pressures in a number of typical riverbanks in the Winnipeg area and to analyze the stability of the banks using measured field pore pressures, slope indicator data, and strength parameters as determined by direct shear tests.

Three sites were investigated: a St. Vital site, and two Tache Avenue sites. An intensive program of instrumentation and analysis was undertaken at each of the sites by post-graduate students of the Civil Engineering Department of the University of Manitoba under the direction of Professor A. Baracos, with the help of both federal and provincial government agencies.

This thesis is an extension of previous theses by Van Cauenberghe (3) and Janzen (4) with particular reference to the Tache Avenue riverbank. A number of possible remedial measures will be presented and commented on with a view to implementation at the Tache Avenue riverbank.

CHAPTER II

HISTORY

Most of the riverbanks in the Winnipeg area have a history of failure and of attempts, some successful, some not, to prevent further movement. The Taché Avenue site, illustrated by Photograph P1, may be considered typical of many such sites.

Little recorded information was found regarding the site prior to 1950. A photograph (P2) of the riverbank taken in 1858 shows a steep, ungrassed bank without any trees. The ungrassed bank appears typical of banks having undergone recent movement.

There is evidence of more recent movements and the remedial measures employed in an attempt to stop it. A photograph(P3.1) of the bank taken in December of 1959 shows a row of vertical timber piles at the water's edge. The deterioration of the piles would indicate that they had been placed some time prior to the time of the photograph. A photograph (P3.2) taken of the site in 1971 indicates the apparent ineffectiveness of the piles to prevent further movement.

During the 1950 flood in Winnipeg, dykes were constructed along Tache Avenue and behind the St. Boniface Hospital to the south. After the flood waters receded the dykes were removed. However, work was soon started to construct a permanent dyke behind the hospital. Waste fill, which covered the area was removed and replaced by clay to final grade. Shortly after completion of the dyke the riverbank failed and the dyke sudsided several feet. To stabilize the failed portion of riverbank, up to ten feet of limestone rockfill was placed at the toe of the slope and extended well into the river. The dyke was then reconstructed with a series of narrow berms and brought up to grade with a reduced top width. This stabilized portion of the riverbank appears stable to-date.

In 1957 an investigation was carried out to study the feasibility of paving Tache Avenue in light of the instability of the riverbank. A consultants' report concluded that with proper precautions taken for the diversion of surface water there should be little problem in paving the street.

In 1959 the Water Control and Conservation Branch of the provincial Department of Agriculture investigated a report of new movements in the bank along Tache Avenue. Photograph, P4.1 shows the extent of the movement. The investigation was preliminary in nature and outlined a program of further investigation to be undertaken. However, no action appears to have been taken at this time. A photograph, P4.2 taken in March of 1971 shows the same bank with an established growth of grass and small trees. The curvature of the tree trunks is an indication of the continuous creep movement undergone by this bank.

In 1961 Tache Avenue was paved. Street drains were installed with pipes beneath the road, discharging out onto the slope at a lower elevation. At about the same time a new water main was installed along Tache Avenue.

In the spring of 1966, another flood of considerable magnitude required that temporary dykes again be constructed along Tache Avenue. No major movements were reported at this time.

In 1969 the Soils Section of the Civil Engineering Department of the University of Manitoba under Professor A. Baracos set up a program to investigate this site. With funds made available by the National Research Council a line of piezometers was installed approximately 100 feet north of Rue Despins and running perpendicular to the slope. Along the same line, two slope hole indicators were installed with technical assistance and engineering services provided by the Water Resources Branch of the provincial Department of Mines and Natural Resources. The Water Resources Branch also installed a grid of alignment pins to monitor surface movements in the slide area.

In 1970 additional instrumentation was undertaken by the University with the financial assistance of the federal Department of Public Works. Instrumentation at this site consisted of eight piezometers and two slope indicators.

In the winter of 1972-73 the City of Winnipeg implemented remedial measures along Tache Avenue in accordance with recommendations of consultants hired by the City.

Timber piles, closely spaced, were placed in pre-bored holes up to 30 feet deep and then driven into the ground an additional 6 feet, along the west edge of Tache Avenue from Rue Despins to Ave de la Cathedrale. The riverbank was reshaped from Ave de la Cathedrale to Provencher Avenue. A rock counterbalance was placed at the toe of the slope from Rue Despins to Provencher Avenue.

CHAPTER III

TOPOGRAPHY AND SOIL CONDITIONS

The topography at the Tache Avenue sites is shown on Figure 2. The contours are based on a 1951 survey and are shown to illustrate the topography in the general vicinity. Cross-sections were taken through the failure area in 1969 and are located as shown on Figure 2. A survey of the area in 1972 indicated that little or no change in the topography had taken place since 1951 except in the immediate vicinity of the failure.

The failure is on the outside of a bend in the river just downstream of where the Assiniboine River enters the Red River from the west. The width of the river at this point is approximately 750 feet from top of bank to top of bank. The top of bank is at approximately elevation 755. The lowest point in the river is at approximately elevation 718 and is approximately 450 feet from the east bank. The slope of the bank is roughly 5:1.

There is considerable evidence of past instability of the bank as is indicated in Photograph, P1. The photograph shows that in 1971 the failure extended from the pumping station at Rue Despins to almost the Provencher Avenue bridge, a distance of about 1500 feet along the top of the bank. A much older, well-grassed-in failure scarp may be noted in the photograph. Evidence of movement was also obvious from cracks in the pavement.

Further north towards the Provencher Avenue bridge, the former failures had also become well grassed-in; however, some fresh cracks were evident. Although the banks here are much steeper than the banks further south, no large movements had taken place.

A subsurface investigation was carried out in the failure area in 1969 and 1970 in conjunction with the instrumentation of the bank. The boreholes were logged and a general stratigraphy of the bank may be represented by three soil logs superimposed on a typical cross-section through the bank. The cross-section was taken at Line 3, located as shown on Figure 2. The typical cross-section is shown as Figure 3. The three boreholes used to describe the soil conditions at the top of the bank, on the slope and on the lower bank are TA-5, TA-3 and TA-1 respectively. The locations of the boreholes are shown on Figure 2.

Borehole TA-5 was taken as representative of the soil conditions in the "undisturbed" portion of the riverbank, back of the existing failure. The top of the borehole is at elevation 756.5. The soil log and properties are shown on Figure 3.

Borehole TA-3 was located within the failure itself. The top of the borehole is at elevation 740.0 and the bottom of the borehole at "auger refusal". The soil log and soil properties are shown on Figure 3.

The third borehole, TA-1 was located approximately one-third of the way up the slope from the toe and still within the failure. The top of the borehole elevation was 742.5. The log is shown on Figure 3.

Considerable laboratory testing was carried out on samples obtained in the subsurface investigation. Moisture contents were obtained on almost all samples with classification tests on representative samples. The greater part of the testing dealt with obtaining strength parameters by means of direct shear tests. These were carried out in the Soil Laboratory of the Civil Engineering Department of the University of Manitoba and detailed procedures are reported elsewhere, Janzen (4). Other testing was carried out by the Water Resources Branch, provincial Department of Mines, Resources and Environmental Management. These included Atterberg limits, moisture contents and unconfined compressive strengths. A summary of all the laboratory test results are presented in Table I.

It should be noted that in the summary of data of the till material, no effort was made to differentiate between the consolidated and unconsolidated till. This may help to explain some of the range of values.

In addition to the general soil conditions, it is of some interest to note some of the pecularities found in a number of the boreholes. In borehole, TA-1 it was found that the thin-walled tube sample taken at the 10 foot depth contained an abrupt demarcation at 45 degrees to the vertical near the bottom of the tube. The break was denoted by a shiny surface and the distinct difference in colour of the soil above and below the demarcation. Above the shiny surface the soil was mottled brown and grey, while below, the soil was grey. The thickness of the grey layer can only be estimated since the next sample was taken four feet below the previous sample and it contained only brown clay.

In the same borehole the highly plastic clay from a depth of 22 feet to 31 feet contained an extremely high moisture content, especially at the 25 foot depth.

In borehole, TA-5 it was noted that augering became very difficult at a depth of 30 feet below the ground surface. It was recorded as a "possible gravel seam"; however, this was never confirmed.

Another point that should be noted is the depth of fill as indicated by the boreholes. The thickness of the fill material varied from negligible in TA-5, back of the failure, to as much as 15 feet in TA-3, in the center of the failure.

CHAPTER IV

INSTRUMENTATION

Three types of instrumentation were employed at the Tache Avenue sites to monitor the pore pressures in the riverbank and the movements of the bank.

A. Piezometers

Two main lines of piezometers were installed in the section of riverbank under investigation in 1969-70. They are identified as Sites 1 and 2 in this thesis.

The first line of piezometers was installed near the southern edge of the failure. The site is shown on Figure 2. The location was chosen in order to obtain as long a period of observation as possible before the instruments were destroyed by the movements of the bank. Therefore, a site was chosen where movement was at a minimum, but some movement was definitely taking place. It was hoped that readings could be obtained over a period of time encompassing a flood, a spring drawdown and a fall drawdown.

This line of observation consisted of ten piezometers located in cross-section as shown on Figure 4.1. The piezometers that were installed are known as Thorpiezometers. The Thorpiezometers operate by measuring the air pressure required to close a hydraulic system within the unit, i.e. the pore pressure is measured by the air pressure required to close the system. The piezometers are claimed to be non-displacement, negligible time lag instruments. A more detailed description of the operation is presented by Van Cauenberghe (3).

The second line of piezometers was located at Site 2 on Figure 2. It consisted of eight piezometers driven to depths as shown on Figure 4.2.

All the peizometers were read from two to five times a month with the greater number of readings in the spring and the fall. The readings were recorded and plotted as total head versus time. A daily record of river levels in terms of elevation was also plotted versus time in order to compare the effect of the river levels on the piezometer levels. The data are presented in Appendix A.

B. Slope Indicators

A number of slope indicator pipes were installed in the bank at both the sites described previously. The slope indicator pipes have interior longitudinal grooves at 90 degree spacings and are placed into vertical boreholes. A precision instrument, one-half of a Wheatstone Bridge, is lowered down the pipe aligned with the grooves, and stopped at predetermined intervals. Readings are taken by means of the other half of the Wheatstone Bridge. A separate set of readings is taken for each of the grooves. The difference in readings between opposite grooves determines the inclination of the pipe in the ground. The lateral movement over each interval of depth is determined by multiplying the difference in readings for consecutive readings by a constant which is dependent on the depth interval and an instrument calibration factor. Total movement at any depth is the summation of the individual movements relative to some fixed point. The procedure for calculating the movement at some point is described in more detail in Appendix D. The plot of the movements relative to the initial readings is used to locate the plane or zone of movement in the bank.

Two slope indicator pipes were installed at Line 1 of Site 1 at the locations shown in plan on Figure 2 and in section on Figure 4.1. Both slope indicator pipes were anchored in the hard till beneath the clay and may be considered as fixed at the tip.

Another two slope indicator pipes were installed at Site 2. The locations in plan are shown on Figure 2 and in cross-section on Figure 4.2. These were also anchored in the hard till, and may be considered fixed at the tips.

The slope indicator pipes were read at least once a month, except during periods of high water, when some of them were submerged. During the fall drawdown, additional readings were taken. The observations have been plotted as deflection versus depth at specific time intervals and also as deflections versus time. The data are presented in Appendix C.

C. Alignment Pins

To check on the surface movements of the slide mass a grid of iron pins (one inch in diameter by three feet long) were installed in five lines at 100 foot intervals along the riverbank as shown on Figure 2. Each line consisted of five pins extending from back of the pavement on Tache Avenue, where movement was assumed to be negligible, to the toe of the slope. The first pin of each line was installed as a reference pin. The second pin was located between the pavement and the former failure scarp. The third pin of each line was located just below the failure scarp. The fourth pin was located above summer water level, and the fifth at the low water mark.

The elevation of each pin and the slope chainage between adjacent pins were taken at approximately the same time as the slope indicator pipe readings. The data were tabulated and plotted as movement versus time. The data are presented in Appendix E.

CHAPTER V

FIELD DATA

A considerable amount of data has been obtained from the test installations described in the previous chapter. The data were compiled and are presented in the Appendices. Plots of the data are presented as well as observations regarding apparent correlations between such factors as river levels, precipitation, bank movement and pore pressures.

A. Piezometer Data

The data obtained from the two lines of piezometers were plotted as total head versus time. River levels are also plotted over the same period of time. For the first line of piezometers, Site 1, data was obtained from the fall of 1969 to the winter of 1971. This included two spring floods, two spring drawdowns and two fall drawdowns. The data are presented in Appendix A and are plotted as shown on Figure 5.1. The piezometric head in the limestone bedrock as observed in a deep well is also plotted against time on Figure 5.1.

To study the possible correlations between precipitation and the total head of a shallow piezometer, data were obtained from the fall of 1969 to the fall of 1970 (Appendix B) and plotted on Figure 6.

The piezometric level of the shallow piezometer 1 rose quickly in the spring of 1970, prior to the rise in river level and then continued to fluctuate independently of the river level throughout the summer with a levelling off in the fall and winter. The rise can be attributed to snow-melt water in the spring of 1970. The fluctuations of the piezometric levels in the summer correspond surprisingly well with fluctuations in the daily precipitation readings in the Winnipeg area over the same period of time, although there was some time lag in the piezometric reaction to periods of heavier precipitation after periods of relatively little precipitation (Figure 6). The precipitation data are given in Appendix B. Piezometer 3 behaved similarly to piezometer 1. However, it failed after only one year of operation.

Piezometer 2 fluctuated with the river level throughout the year as did piezometers 4, 5 and 6. However, these piezometers showed an increase in piezometric head at the time of the fall drawdown, when a renewed movement of the banks took place.

Piezometers 7 and 10 also appear to have responded to fluctuations in the river level. However, in the fall before the river was drawn down, these two piezometers showed a substantial increase in piezometric head. This increase corresponded to an increase of piezometric head in the bedrock as observed in a well penetrating into the limestone and located at a nearby dairy. The piezometric head of these piezometers was also much lower in the summer than in the fall. reflecting the effects of pumping from the bedrock during the summer months. This would cause seepage into the bank during the summer and thereby possibly improving the stability of the bank.

Piezometer 8 also appeared to respond to fluctuations of the river levels. Its tip was approximately 10 feet below the ground surface and just below the summer river level. Its piezometric head was well above the ground surface and reflects artesian pressure at the toe. The piezometer failed before a second fall drawdown took place and no further observations were possible.

In addition to the above observations at Site 1, it was observed that considerable ponding of water occurred in some parts of the failed bank, particularly where a depression had formed at the back of a former failure. Also, during the spring thaw, considerable snow-melt water washed over the bank and seeped into cracks at the top of the slope. In the spring of 1971, water was observed discharging out of a failure crack at a point just downslope of a street drain inlet and at least six to eight feet above the outlet which was located further downslope.

The second line of piezometers was installed in the fall of 1970. The readings were taken over a period of about two years. A similar trend of piezometric fluctuations as observed at Site 1 appears evident at this site. Two additional piezometers 9 and 10, were installed in October of 1971 to replace 1 and 2 which became inoperative shortly after installation. Piezometers 9 and 10 recorded piezometric elevations well above the top of the bank. Piezometer 10 may be compared to Piezometer 1 at Site 1, but where Piezometer 1 had a total head below the top of the bank, Piezometer 10 showed piezometric levels in excess of 50 feet above the top of the bank. This high value could be the result of either a malfunction of the piezometer or the reflection of some other source of seepage such as a leaking water main.

B. Slope Indicator Data

The data obtained from the slope indicator readings are summarized in Appendix C. The movement was plotted against time as shown in Figure 8. The greatest movement was observed to be taking place between the months of November and April of the following year. Between April and October the amount of movement was negligible. A plot of river levels over the same period of time indicates that movement begins at about the same time as the fall drawdown of the river and continues until the flood waters raise the river level in spring. In the spring of 1970, a sudden and substantial movement was recorded. This corresponded to the increase in piezometric heads of piezometers 1 and 3, Site 1, which was attributed to snow-melt water entering the cracks in the bank.

Periodic deflections versus depth were plotted on Figure 4.1 and 4.2 to indicate the possible failure plane. It is of interest to note that in slope indicator pipe TA-2 the depth of fill on the bank at this point corresponds to the depth at which movement took place. In TA-1 the movement appears to take place over a 10 foot depth of soil which corresponds to the thickness of the high plastic clay layer at this point. The movement in both TA-1 and TA-2 was of such a degree that they failed approximately one year after installation. Slope indicator pipes, TA-4 and TA-5 showed only some minor movements as shown on Figure 4.2.

C. Alignment Pin Data

Data from the alignment pins are included in Appendix E. The data were plotted as vertical and downslope movement versus time time as shown on Figure 7.

Pins 1 were used as reference points for measuring the downslope movement of the other pins. The elevations of Pins 1 were related to a Geodetic benchmark. Figure 7.1 shows a negligible variation in elevation of Pins 1.

Pins 2, except for Pin 2, Line 1, showed an average drop in elevation of 1.5 feet with the maximum being at Line 3, the centre of the failure. The downslope movement ranged from 1.4 to 2.7 feet with the maximum again being at Line 3.

Pins 3 showed a drop in elevation of 0.3 to 1.0 feet with the maximum occurring at Line 5. The downslope movement varied from 0.5 to 2.2 feet with the maximum movement at Line 2. See Figure 7.3. Some comparison can be made between this pin and the slope indicator pipe TA-2, which was located less than five feet from the pin at Line 1. TA-2 showed a horizontal movement of 9.5 inches from December of 1969 to November of 1970, when the slope indicator pipe sheared-off due the large movement. The movement of Pin 3, Line 1, indicated only a downslope movement of 3.6 inches over the same period of time. The discrepancy is difficult to account for. However, one explanation may be that the failure mass consists of independent segments, each moving at a different rate. This may account for the shiny surface between the fill and the insitu clay noted in borehole TA-1.

Pins 4 showed lateral movements of 0.9 to 2.8 feet with the minimum movement at Line 1 and the maximum at Line 2. The elevations of the pins dropped only 0.1 to 0.2 feet. At Line 1 the slope indicator pipe TA-1 is within a few feet of the pin. The movement indicated by the slope indicator pipe was about 3.9 inches. The pin moved 9 inches during the same period of time.

Pins 5 averaged a rise in elevation of 0.3 feet with an average lateral movement of 1.8 feet and a maximum of 2.8 feet. This would confirm that the failure is moving out at the toe of the slope as shown in Photograph P5.

In summary it may be said that the major movements occurred about the middle of November and the middle of April. The movement in the fall corresponds to the lowering of the river level, while the movement in April may be attributed to an increase in piezometric levels due to seepage into the bank and cracks from snow-melt runoff. Pins 2 and 3 showed the most vertical movement, Pins 4 showed the most lateral movement and Pins 5 showed both lateral movement and vertically upward movement. This would tend to substantiate a failure plane as proposed by Janzen (4) from the slope indicator pipe deflections.

CHAPTER VI

PREVIOUS STABILITY ANALYSES

Stability analyses of the failure at Tache Avenue were performed by Janzen (4). In his results, he noted that the stability of the bank depended on the river level. The riverbank was stable when the river level was high and tended to be unstable when the river level was low. He reported that the bank moved when the developed angle of internal friction (effective angle required for static equilibrium) of the clay was more than some critical value, and a semblance of stability was observed when the developed angle was smaller than the critical value. This critical value ranged from 10.8 to 12.7 degrees at Tache Avenue and compared favourably with the residual values of the angle of internal friction as obtained by direct shear tests in similar clays (8.3 to 13.0 degrees). For his analysis Janzen (4) assumed the residual cohesion to be zero. The above would indicate that movement as observed by the pins and slope indicators have reduced the shear strength along the failure plane to the residual values of clay.

Janzen also stated that the piezometric levels indicate seepage into the bank during periods of high water. However, the pore pressures in the clay are not readily relieved when the river level drops and this results in piezometric levels higher than the river levels. This also results in a zone of high pore pressures at the toe of the bank after spring drawdown with resulting upward seepage at the toe and a local instability.

Therefore the instability of the Tache Avenue riverbank may be attributed to the fact that the effective shear strengths have been gradually reduced to their residual values through the movements of the bank.and the pore pressures, particularly at the toe of the bank, are not relieved quickly enough after the fall drawdown for stability to exist. Under these conditions the stability of the bank would have to be achieved by one or more of the following:

1. Increasing the shearing resistance of the soil,
2. Reducing the loads on the bank, and
3. Lowering the piezometric levels in the vicinity of the bank, particularly at the toe.

Remedial and protective measures must provide an adequate safety factor against further movement. A safety factor of 1.2 has been accepted by the Assiniboine River Advisory Board (5) for residual shear strength values in Lake Aggassiz clays. Locally a 20 percent increase in safety factor on residual shear strength values has been considered a design minimum where economically feasible. Therefore it is recommended that any remedial measures employed at the Tache Avenue site should result in an increase in safety factor of 20 percent.

CHAPTER VII

POSSIBLE REMEDIAL AND PROTECTIVE MEASURES

A number of possible remedial and protective measures were investigated consistent with the requirements for stability of a riverbank as set out in Chapter VI. Although there are many possibilities for stabilizing embankments, an attempt was made to review only those which appeared applicable for riverbanks in the Winnipeg area, particularly at Tache Avenue. Where available, local examples of remedial measures were examined. Of the possibilities investigated, the following lend themselves most readily to the local problem.

A. Counterbalancing

A conventional means of stabilizing a slope that is in a state of impending motion or has failed, is to increase the resistance to sliding by means of a counterbalance at the toe of the slope. The counterbalance usually consists of gravel, clay or rockfill placed on the toe of the slope and is of sufficient weight to increase the resistance to sliding. The counterbalance is placed so that the potential failure does not extend above or beyond the toe.

The stability of the existing slope may be determined by conventional stability analysis, assuming the worst conditions of pore pressure and river level. The shape of the failure plane for the analysis may be fairly accurately determined from slope indicator data. and from visual evidence such as oracks at the top of the slope (Photograph P5) and overthrusts at the toe (Photograph P6). A counterbalance of the desired shape (within right-of-way and causing the least interference with existing structures, roads and waterways) is superimposed on the failure and the new safety factor determined.

1. Gravel Buttress

Gravel buttresses have been recommended for stability problems by Long (6). The material at the toe of the slope is replaced with a free-draining material as shown on Figure 9.1. The reduction of pore pressures through the free-draining material results in increased effective weight of the toe.

The buttress is ideal for locations where right-of-way is limited. However, the difficulty in using the gravel buttress is in the placing of the material. If the slope is already at a state of impending failure, any further disturbance such as removing soil at the toe to make room for the gravel could result in a failure.

2. Soil Berm

Clay is usually used as a counterbalance where erosion problems are not severe. The effectiveness of a clay berm is demonstrated by an example of a dyke on a riverbank on the Morris River (Figure 9.2) as reported to the Geotechnical Section of the Winnipeg branch of the E.I.C. (7).

In conjunction with the protection of towns along the Red River, south of Winnipeg, dykes were constructed around a number of towns. At Morris the construction of dykes along the Morris River on the north side of town required the relocation of sections of the Morris River. At two locations, dykes were constructed over the original river bed. In the fall of 1968, after the final few feet of fill had been placed, a large crack appeared along the top of a section of dyke.

A portion of the top of the dyke (approximately five feet of fill) was immediately removed along the line of failure. Berms were placed in two stages at the toe of the slope for a total height of ten feet (Figure 9.2). The dyke was then brought back up to grade. Slope indicators were installed along the centreline of the failed section and monitored. Although movements continued for several months after, no further cracks were observed and present readings indicate that movements are steadily decreasing as the excess pore pressures dissipates. Todate the berms have withstood several inundations by flood waters and appear to be performing satisfactorily.

At another site on the Morris River where conditions for failure were similar to the above, a berm was placed prior to any visible signs of failure as an extra precaution. Although high pore pressures were observed during construction of the dyke and berm, no signs of failure were observed. Slope indicators installed in the area indicate that some movement is taking place but it appears to be decreasing with time.

3. Rock Berm

Another method of increasing the stability of an embankment is the use of rock fill as a berm. Such a material is recommended for stabilizing slopes where the toe is always submerged and subject to the erosive action of moving water. The size of the material required to resist displacement by flowing water may be determined as shown by the USBR (9). It should be noted that if a clay berm is used and extended into the river it would require riprapping. Therefore it may be more advantageous to use rock for the whole berm.

The effectiveness of this method is illustrated by the stabilization of the riverbank behind the St. Boniface Hospital, just upstream of the Tache Avenue failure. Baracos in 1951 (8) recommended to the Greater Winnipeg Dyking Board the use of rockfill as a stabilizing material in order that a dyke could be placed on the bank. Todate the bank has been stable. A sketch of a rock berm as a remedial measure for stabilizing a failure is shown on Figure 9.3.

Riprap is available in large quantities within a reasonable and economical hauling distance of Winnipeg. That it can be placed by dumping from a barge has also been demonstrated. Where access is not a problem the rock can be placed by machines from the bank.

B. Reduction of Slope Angle and Bank Height

An effective means of stabilizing a failed bank or improving the stability of any bank is to flatten the slope or to reduce the height of bank for the same slope. This reconstruction of the slope results in reducing the forces causing failure.

Flattening the slope results in a decrease of the soil shear strength required for stability by reducing the loads on the bank. Reconstruction of the slope for increased stability has been used on a number of riverbanks in the Winnipeg area. Photograph P7 taken of a bank on the Assiniboine River at the Maryland Bridge is a good example of improving stability by flattening the slope. Photograph P8 shows an example of slope flattening on the Red River in north Winnipeg.

A study of riverbanks in clays in the Winnipeg area by Mishtak (1) indicated that failed banks in clays 25 to 40 feet in height became stable at slopes between 4.5:1 and 6.75:1 although some flattened out to almost 10:1. This influenced the selection of slopes for the Red River Floodway around Winnipeg where 6:1 was used for the slopes.

Slope flattening is limited to areas having sufficient right-of-way. The slope should be cut back far enough to achieve the increase in safety factor of 20 percent as described in Chapter VI. The slope should be shaped to prevent ponding of water and seeded to prevent surface erosion. Any sources of moisture such as leaking sewer outfalls and street drains should be removed from the vicinity of the stabilized riverbank. Any loads placed on the bank must be set back from the top edge of the bank far enough to maintain the safety factor of the unloaded stable bank.

The slope may also be stabilized by unloading the existing bank, i.e. reducing the overall height of bank for a particular slope.

It is obvious that these methods are limited to locations where flattening of the slope does not exceed the allowable right-of-way and where lowering the bank is permissible with respect to high water levels. Flattening of the slope may not be applicable to areas where pore pressures are excessively high because of the resulting low effective strength. The ineffectiveness of slope flattening where pore pressures are relatively high is demonstrated at the outlet of the Seine River Diversion into the Red River, south of Winnipeg. The low flow channel, as designed, was five feet deep with 3:1 side slopes. Scour at the toe of the bank and degradation of the channel caused the bank to fail 12 years later. Calculations indicated that flattening of the slope to 6:1 would not appreciably increase the safety factor because of the influence of the pore pressures.

C. Drainage

The use of drains, both horizontal and vertical, is documented in most standard textbooks. The purpose of the drains is to stabilize the embankment by either reducing the seepage pressures or reducing the excess hydrostatic pressures.

The drains are probably most efficient where the zones of water bearing strata are well defined. They may also be applied where a relatively impermeable material is underlain by a zone with a high hydrostatic head.

Every process of drainage reduces the stress in the pore water of the soil without appreciably changing the total stress. Terzaghi (11) states that lowering the piezometric level beneath a clay stratum ultimately increases the effective pressure on a horizontal plane by an amount equal to the weight of the height of a column of water representing the reduction in pore pressure. This decrease in pore pressure results in an increase in the effective stresses on every potential surface of sliding within the soil and thereby increases the strength. The weight of the soil located above this surface is unaltered. Drainage increases the factor of safety with respect to sliding provided the external conditions for equilibrium remain unchanged.

Turnbull and Hvorslev (12) recommend internal drainage by means of horizontal drains, free flowing or pumped wells and vertical sand drains to increase the internal soil resistance of an unstable bank.

Before any form of drainage is considered a soils investigation is necessary to determine any water-bearing zones or pockets, their extent and permeabilities. The type of drains employed depends on a number of reasons such as mild or steep slopes, underlying water-bearing zones, artesian pressures, etc. The following types of drains are usually considered:

1. Horizontal Drains

Horizontal drains are applicable where zones of water-bearing soils are within the embankment itself and drainage is possible by gravity. The number of drains and their spacing is a function of the amount of water to be drained and the permeability of the soil.

Two types of horizontal drains are usually used. The first is a perforated pipe, either jacked into the embankment or placed into a pre-augered hole, with a slight slope down to the point of discharge. For some fine-grained soils it is usually adviseable to use a filter material around the perforated pipe to avoid a silting-in of the pipe. The filter is sometimes placed by forcing a granular material around the perforated pipe inside hollow stem augers by reversing the augers. Probably the best known example of the effectiveness of a horizontal drain is the stabilization of an embankment overlooking a highway in California as described by Smith and Stafford (13). Movement of the embankment was halted when a horizontal drain released a large volume of water from within the embankment. Another form of horizontal drain commonly employed is the trench drain. For embankments that have a relatively flat slope and are low (less than ten feet high) horizontal trench drains should be considered. There are a number of local examples of shallow trench drains that appear to be performing satisfactorily.

A number of small slides on a small creek on the west side of Winnipeg plagued the Metro Winnipeg Parks Board. A reshaping of the slopes stabilized major portions of the unstable bank. However, a small slide at one point along the bank persisted. Test holes in the bank indicated a pocket of granular material near the back of the failure. The water in a test hole in the granular material rose to within three feet of the top of the test hole. Test holes above and below the pocket were dry. A recommendation in a report by the Water Resources Branch of the provincial Department of Mines, Resources and Environmental Management (14) was to excavate and construct a horizontal trench drain back into the slope. Measurements taken over a period of about two years after remedial measures had been implemented detected little or no movement of the the former slide.

A similar problem existed further upstream on the same creek next to a roadway as shown in Photograph P22.1. Recommendations called for a reshaping of the bank and the installation of two trench drains plus riprap at the toe of the bank as shown in Photograph P22.2. To date the remedial measures have been successful.

2. Vertical Wells

Where drainage by gravity is not possible as in the case of a water-bearing stratum, with a high piezometric head, below the toe of the embankment, drainage by means of vertical wells extending down into the water-bearing zone may be utilized.

If artesian pressures exist the wells are allowed to flow free. However, if the top of the well is too high to lower the piezometric head sufficiently enough to increase the stability of the bank, pumping may be employed to lower the head.

The wells may consist of cased boreholes with a section of perforated pipe in the zone to be drained. A filter may be necessary to avoid the migration of fines into the casing and to avoid a reduction in its effectiveness.

3. Vertical Sand Drains

A lower piezometric head may be required in some instances to ensure future stability. If the flow is intermittent and pumping is not economical, augered holes backfilled with sand and gravel may be used. The vertical sand drains act as releases for the excess pore pressures as in the case of free flowing wells. Vertical sand drains may be drained by interconnecting horizontal drains and a horizontal drain emerging near the toe of the bank. Sand drains are most frequently used downstream of dams to pick up any seepage from beneath the dam.

D. Protection of Slope and Toe Against Scour

One of the frequent causes of instability of a riverbank is the erosion of material from the toe of the slope. This scouring away of material is usually most severe on the outside of a bend in the river. Means of protecting the toe against scour are available and have been frequently employed with both success and failure. Failures are usually the result of improper materials and placement. Some of the more common forms of scour protection include riprap, retards and revetments. Probably the most frequently employed method is riprapping. Of concern is the extent of the protection both into the river and up the slope. The bank should be stable before any form of protection is implemented.

The loss of material at the toe constitutes a reduction in the forces resisting sliding and results in a loss of strength along a potential failure plane. As the shear strength of the soil approaches a limiting value, creep may occur, eventually resulting in a failure of the bank. To reduce or stop this loss of strength, the soil at the toe should be protected.

1. Riprap

Riprap consists usually of granular materials and is used to protect the bank from being eroded. A procedure for determining the size of material required to resist displacement by the river current is outlined by the U.S. Bureau of Public Roads (9). The riprap must be well-graded and placed on a graded filter to prevent the movement of fines up through the rock.

The riprap should extend from below the toe of the slope to at least several feet above normal water level and in critical areas may extend above the flood level. However, before the riprap is placed, the stability of the bank should be investigated and remedial measures implemented to provide an adequate safety factor against sliding.

There are many examples of the use of riprap in the Winnipeg area. A few are shown in photographs P7, P9, P10 ans P11. Photograph P7 shows a section of riverbank along the Assiniboine River in Winnipeg where the riprap at the toe of the slope appears to be performing satisfactorily. P9 shows a section of the Assiniboine River where riprap was placed to above the flood level. A failure of the bank with the riprap is evident to the right in the photograph indicating that the bank did not have sufficient resistance to sliding prior to the placing of the riprap. Photograph P10 illustrates the use of a well-graded riprap on a proper bedding as protection against scour at the toe in Omands Creek in Winnipeg. Photograph P11 shows an example at another site along the Red River of riprap that does not meet design criteria. It is not well-graded. A closer inspection of the bank indicated evidence of recent movement.

2. Retards

Retards usually consist of piles driven into the ground along the toe of the bank to be protected. Usually round timber piles are driven in a single row or multiple rows. The piles may be spaced close together or far apart. Sometimes horizontal planking is placed between the piles when they are spaced several pile diameters apart. Steel or wooden sheet piling is also used as retards.

Retards may be used to pick up additional resistance to sliding of the bank provided that the piles are driven deep enough. Frequently the piles cannot be driven deep enough to develop sliding resistance because of shallow bedrock or other hard strata. The piles must also be designed against loss of lateral support due to scour.

There are many examples of the use of retards to protect the toe of a bank along both the Red and Assiniboine Rivers in Winnipeg. A check of many of these installations indicates a high failure rate. This may be attributed to the relatively shallow pile penetration which gave inadequate lateral support. The piles are frequently laterally displaced or overturned.

Photographs P3, P8 and P13 to P21 inclusive are examples of attempts to protect the toe of the banks. Photograph P3.1 shows a row of widely spaced timber piles at the toe of Tache Avenue riverbank. Photograph P3.2 shows the remains of the row of piles 12 years later. Only a few of the piles remain and these have moved appreciably. Photograph P14 and P15 show two other examples along the Red River of retards with widely spaced piles. The loss of soil from behind the piles is evident. Photographs P16, P17 and P18 show examples on the Red River of retards with closely spaced piles. Again the loss of material from behind the piles is evident. Photograph P18 is an example of piles being overturned by the failure of the bank.

Photographs P19, P20 and P21 show other local examples of vertical pile retards with horizontal planking between the piles. In these cases there appears to be little or no loss of soil from behind the retards and the banks appear stable.

Comparison to the retards shown in Photographs P14 to P17 would indicate that the horizontal planking minimizes the loss of soil.

Photograph P8 and P13 show local examples of steel and wooden sheet piling retards respectively. Failure is evident at one end of the steel sheet piling retard in Photograph P8. Photograph P13 shows a stable riverbank.

It would appear from the photographs that timber pile retards with horizontal planking or sheet pile retards can effectively be used to protect the toe of a bank. A filter is recommended placed back of the planking to reduce pore pressures in the soil behind the retards. Special attention is required to properly tie the ends of the retard into the bank.

Another factor that must be considered is that timber pile retards are susceptible to rot above the water level and need expensive maintenance. Rotted piles are shown in Photographs P16 and P17.

3. Revetments

Revetments are a form of continuous bank protection. They may be rigid, semi-rigid or flexible. An example of a rigid revetment is a reinforced concrete slab or rock grouted in-place. Woven willow mattresses, lumber mattresses and articulated concrete mattresses are examples of semi-rigid revetments. A riprapped slope may be considered to be a flexible revetment in that it adjusts readily to movement beneath the revetment. Details and installations are discussed by Winkley (10).

The material for revetments must be sufficiently durable to prevent destruction by the force of the current in the river and by impact of ice and debris. Revetments, especially of the articulated concrete mattress type, are relatively expensive to install and are susceptible to damage during installation. Filter cloth may be used to prevent the movement of fines through openings in the semi-rigid and flexible revetments. Rigid revetments are very susceptible to cracking and breaking-up if the revetment is undermined. Photograph P12 shows the use of rock grouted in-place as a means of protecting a bank along the Red River. The revetment is failing due to undermining.

E. Retaining Structures

A recognized method for stabilizing a slope is to increase the resisting forces of the bank by using structural methods such as retaining walls or anchored bulkheads. These structures obtain their strength outside of the slide area, either from beneath or behind the failure.

1. Retaining Walls

Retaining walls are usually in the form of rigid walls on massive footings. Typical examples are shown on Figure 10.1. Three points deserve consideration in the design of a proper retaining wall in clay:

a. the pressure exerted on the wall by the backfill,

b. the adequacy of the foundation soil to support the structure, and

c. the drainage system to minimize pore pressures in the backfill.

Numerous theoretical and empirical methods for computing the earth pressures have been developed. The allowable soil pressure to prevent overturning and settlement and the safety against sliding may be determined from strength tests of the soil. Where the adequate resistance cannot be obtained at shallow depths, piles may be used.

For stabilizing a failed bank, the footing of the retaining wall should be below the base of the failure. However, at the Tache Avenue site this would require excavation at the toe of the bank which would further endanger the bank.

2. Anchored Bulkheads

Anchored bulkheads usually consist of a flexible sheeting restrained by tie-backs and penetrating below the mass to be stabilized. A typical example is shown on Figure 10.2.

The design of anchored flexible bulkheads is available in soil mechanics literature. Wall pressures, wall movements, drainage and anchorage must be considered in the design. The wall pressures are determined on the basis of the type of backfill and the foundation conditions. The type of sheeting used depends on the wall moments to be resisted. Anchorage must be obtained from soil behind that to be stabilized, or from deeper firm strata. Precautions must be taken to prevent damage to the tiebacks from corrosion or settlement of the backfill.

Generally speaking it would appear that retaining walls are more suitable for already stable banks where additional stability is required for new loads on the bank. Where the bank has already failed or has a low safety factor, excavation for a retaining wall would only further endanger the bank. Anchored bulkheads may be more applicable in such a case since no excavation is required. Driving of the sheeting may, however, not be desireable in that it could trigger another slide.

No examples of retaining walls or anchored bulkheads were observed along the Red and Assiniboine Rivers in the Winnipeg area. The anchored bulkhead may have application where a vertical bank to prairie level is required. At the Tache Avenue site there is no need for a vertical bank at the water's edge and the anchors that would be required would possibly be very expensive.

Such retaining structures are not recommended for the Tache Avenue site, although they have other application in the Winnipeg area.

F. Chemical Stabilization

In the last few years considerable work has been done in the use of chemicals in stabilizing soils. The method of stabilizing road bases by mixing the chemical with the soil is well known. However, the use of chemicals in stabilizing insitu soils is still in the experimental stage. Results of the insitu methods are difficult to evaluate and little information is available. Another difficulty in implementing the use of chemicals to stabilize embankments appears to be the time required. Handy and Williams (20) indicated that the full effect of this stabilizing method is delayed by as much as three years. Cost factors are also relatively unknown.

A number of chemicals have been used in soil stabilization attempts. Only those considered applicable to Winnipeg soil are referred to in this thesis, and include: quick lime, hydrated lime and calcium chloride.

The chemicals may be applied by either using drilled auger holes or trenches filled with a chemical solution, or by pressure injection of a chemical slurry. In both cases electro-osmosis may be used to speed up the process.

The theory behind the reaction of the chemicals with soil is beyond the scope of this report. However, some general comments can be made.

Lime stabilization is used for stabilizing of embankments either in the form of quick lime (CaO) or hydrated lime ($Ca(OH)_2$). Lime stabilization takes place mainly by the process of base exchange and cementation. Base exchange is the process whereby the different cations in the solution in the pore water of the soil replace the adsorbed cations on the clay mineral. The ability to adsorb exchangeable cations is known as base exchange capacity. Different clay constituents have different base exchange capacities and of the three most common constituents of clay (montmorillonite, illite and kaolinite) montmorillonite has the largest capacity. Clays in the Winnipeg area are generally high in montmorillonite. The process, when taking place, results in the agglomeration of the fine-grained clay particles with a resulting reduction in plasticity index. The process is then followed by cementation or pozzolanic reaction, which probably represents the most important aspect of lime stabilization. The reactivity of lime with soil also increases with plasticity index. The cementation results in the gain of strength by the soil.

The use of calcium oxide or quick lime is probably the simplest form of lime stabilization. It requires the least complicated equipment. Auger holes are drilled at some predetermined spacing and filled with quick lime. Water is added to the quick lime and the hole sealed off with a clay plug. This method relies on the movement of the ground water to help the migration of the lime through the soil. Where a soil is relatively homogeneous and impermeable, such as is the case for a high plastic clay with some fissures, silt and sand lenses, etc., it may be more effective to use a hydrated lime slurry injected into the soil. The lime slurry is injected into the soil under high pressures by means of specially designed injection jets which are pushed into the ground to the desired depth and slowly withdrawn as the lime slurry is injected. To avoid loss of slurry alongside the injection pipes it may be neccessary to first seal the surface of the entire area by means of mixing lime with the surface soil much as for stabilizing a road base. This forms an impermeable mat and is also useful in providing a working surface for the machines.

The slurry is mixed in proportions which are best suited for maximum reaction with the type of soil being stabilized. A wetting agent is used to promote dispersion. The slurry can also be dumped into boreholes or trenches. However, the lime tends to settle out and only the water migrates through the soil unless the slurry is constantly agitated. Considering the time in years required for complete dispersion of the slurry, this method is not practical.

The use of injected hydrated lime appears to be a better means of stabilizing soils as demonstrated by experimental sites reported by Lundy and Greenfield (15) and others reported by the National Lime Association (16). The hydrated lime was found to create a moisture barrier by penetrating along seams and fissures in the soil. Penetration of post glacial soils (soft clayey silt) was found to be in the order of a five foot radius. Some problems were encountered, such as loss of slurry alongside injection pipes but these could be partially overcome by a surface seal as described previously. The high pressure method appeared moderately successful. Physical characteristics of the soil were not appreciably changed after one year except for shear strength which was found to be approximately doubled where lime penetration was good.

Another chemical that has been tried experimentally with some success in Sweden by Talme (17) is calcium chloride.

The salt is placed by injection or electro-injection. Injection is employed as described for lime stabilization. Electro-injection uses the process of electro-osmosis to speed up the movement of the ions of the salt in the soil pore water. Calcium chloride was found to increase the shear strength of all clays, the degree of increase depending on the type of clay being stabilized and how much salt was being used. The increase in shear strength also depended on the degree of penetration of the salt into the clay itself.

At another site described by Talme (17) use was made of electro-injection to move the fluid containing the salt ions through the soil. The clay content of the soil varied from 50 to 70 percent and had a sensitivity of between 25 and 50. After about a week it was found that the strength of the insitu soil had increased by 28 percent although the variation was great depending on where the samples were taken in relation to the injection hole. It was also found that after the injection, the salt continued to diffuse through the soil.

None of the experiments described above dealt with a high plastic clay of low sensitivity such as are encountered in the Winnipeg area. However, the method of stabilization does lend itself to areas of limited right-of-way. Since this is the case in many riverbanks where the banks have receded until the buildings are threatened, it may be desireable to use a method such as chemical stabilization. The chemical calcium chloride reacts very well with clays having a high montmorillonite content such as the Winnipeg clays. Because of the many unknowns in the use of chemicals for stabilizing a riverbank in clay, a test stabilization of a failed and fully instrumented site would be a prerequisite to any full-scale use of this method.

G. Eliminating Fall Drawdown

The previous methods dealt with the stabilization of relatively short reaches of riverbank and therefore would be of interest to individual property owners having similar problems. However, there is an encompassing remedial measure which would help to improve stability of practically all banks in the Winnipeg area.

This can be effected by maintaining the present summer water levels for the entire year except during spring runoff. Previous to the construction of the Red River Floodway around Winnipeg, it was necessary to lower the river in the fall in order to provide for additional storage during spring runoff. The gates of the St. Andrews locks at Lockport were not designed to withstand ice pressures and therefore had to be raised every fall to prevent ice damage. The only means of maintaining the higher water levels in summer and winter would be to construct another dam or fixed-crest weir upstream of the existing structures.

That a relationship between the numerous bank failures and the fall drawdown exists was suggested as early as 1953 when the "Report on Investigations into Measures for the Reduction of the Flood Hazard in the Greater Winnipeg

Area", Appendix E (18) stated that there is a "controversial question of whether the operation of the dam at Lockport is responsible for the bank slippage occurring in the fall of the year", and went on to suggest that lowering the water in two stages as is presently done actually reduces the number of bank failures that would occur if the water was lowered to winter level in one stage. In 1953 there was no alternative to lowering the water level in the fall.

In 1966 Sutherland (19) reported that the "lower the drawdown, the more unstable a bank becomes, and an unnecessarily rapid rate of autumn drawdown may have been the trigger that set off failures in the past". Sutherland did not mention maintaining the summer water level in the winter but rather stated that "control can be exercised over the autumn drawdown and it is strongly recommended that this drawdown should be carried out ...at the slowest rate possible".

In 1969, 1970 and 1971 measurements taken by means of slope indicators again showed a relationship between the fall drawdown and the bank movements both at Tache Avenue and St. Vital (Figure 9). Stability analyses by Janzen (4) indicate that there is an increase in safety factor of between 40 and 50 percent from winter levels to summer levels.

It is therefore recommended that consideration be given to not lowering the water levels in the Red River in the fall. If the gates at Lockport cannot be strengthened to resist the ice pressure, some other form of control may be necessary, such as a rock weir upstream of the gates or a completely new structure. A new dam would be a very expensive undertaking. However, if repairs to the existing structure are continually necessary and the riverbank property is becoming increasingly more valuable, the cost-benefit ratio of a new dam may become more acceptable.

A study would have to be undertaken to determine the problems to be anticipated with the construction of a new dam. An investigation would have to be carried out to determine the backwater effect of a weir crest fixed at some elevation such as the summer level, which would provide adequate capacity for spring flood flows. The flooding of low lying areas would also have to be investigated.

CHAPTER VIII

RECOMMENDATIONS AND DESIGN CONSIDERATIONS

All the remedial measures discussed in Chapter VII could be used to some degree to stabilize a clay riverbank such as the Tache Avenue bank. Counterbalancing with a rock berm at the toe would stabilize the bank by providing the necessary resistance to sliding. It can be implemented with readily available materials and equipment without further endangering the bank. Its effect is also easily evaluated. Flattening of the slope or reducing the bank height would reduce the loads on the bank, but these methods are limited by right-of-way and the need for relocating underground services. In addition erosion protection at the toe would still be required. Drainage by means of horizontal trench drains could be used to reduce the pore pressures. However, it is difficult to predict the degree of the effectiveness of the drains. They would have to be instrumented with piezometers so that if required, additional drains could be added. Therefore it may be desireable to use it with some other stabilizing measures. Erosion protection only eliminates the loss of soil at the toe and does not increase the stability of the bank. A failed bank would have to be stabilized first before toe protection would be effective. Riprap appears to be the most suitable type of toe protection in the Winnipeg area. Retaining structures do not appear to be very applicable to the Tache Avenue site as a means of stabilizing the failure.

Installation of a retaining wall would endanger the bank because of the necessary excavation. The effects of chemical stabilization are too difficult to evaluate at this time as is the cost. However, the method does lend itself to areas of limited right-of-way where conventional methods are not possible. Further investigation would have to be carried out to determine the feasibility of maintaining the summer water level throughout the summer and winter before the fall draw down could be eliminated.

On the basis of the above, it is recommended that counterbalancing with a rock berm be employed as a means of stabilizing a riverbank such as the Tache Avenue bank. It is also recommended that a form of toe drainage be provided to reduce the pore pressures at the toe as an additional safety factor. It is recommended that rock be used for the counterweight. The rock should be of sufficient size to resist displacement by the current in the river, ice and wave action. The toe drainage would be in the form of horizontal trench drains, initially spaced at 100 foot intervals.

The size of berm required to increase the safety factor by 20 percent is determined by trial and error. A typical cross-section of the bank was analyzed as follows: A failure plane was approximated from visual evidence of cracks and movement and from slope indicator and alignment pin data. The case of fall drawdown was simulated by assuming summer pore pressures with the river at winter ice level. An average residual value of 9.6 degrees, as determined by Janzen (4), was used for the effective angle of friction. A safety factor of 1.0 was obtained by introducing a small cohesion of 0.6 pounds per square inch.

A berm was super-imposed on the cross-section as shown on Figure 11 in order to obtain a safety factor of 1.2 under fall drawdown conditions. This represents an increase in safety factor of 20 percent, neglecting any increase in safety factor due to the horizontal drains and the shear strength through the rock fill.

The average summer water level in the river is approximately elevation 734.0. To protect against some fluctutions in water level and to protect against wave action, the rock should extend up to elevation 736.0. To protect against undermining, the rock should extend below the winter ice level of approximately elevation 727.0. A side slope of 3:1 was used for the rock berm for added stability and to help dissipate some of the wave energy. A typical section through the berm is shown on Figure 11.

The maximum velocity of the river is approximately six feet per second. The average riprap size required to resist displacement by this velocity along a relatively straight stretch of channel would be four inches, as recommended by the USS. Bureau of Public Roads (9). For direct impingement, it is recommended that the design velocity be doubled. This would result in a riprap size of fourteen inches. The Tache Avenue site is on the outside of a bend in the river and just downstream of where the Assiniboine

River flows into the Red River. Since this is somewhere between the cases of straight alignment and direct impingement, it is recommended that the riprap be designed for a velocity of 10 feet per second. The average size of riprap required to resist a velocity of 10 feet per second on a 3:1 channel side slope is 10 inches. The maximum size of riprap required is usually taken as 1.5 times the average size, or in this case 15 inches. The minimum size should be three inches. A typical gradation is as follows:

Riprap Size (inches)	Percent smaller than by weight
15	100
12	25 - 100
10	15 - 50
8	0 - 20
3	0

Where the riprap is placed directly on the bank, a bedding or filter is required for the rock. The bedding should meet the following criteria:

$$\frac{D_{15}\text{ (coarser material)}}{D_{85}\text{ (finer material)}} < 5 < \frac{D_{15}\text{ (coarser material)}}{D_{15}\text{ (finer material)}} < 40$$

Since the bank material is a highly plastic clay, the second criteria is not so critical and may be neglected. Therefore, the filter should be a clean (free-draining), well-graded, sandy gravel having about 85 percent passing the 2 inch sieve.

Since a thickness of riprap equal to the maximum size of rock required is sufficient for protection against displacement, the core of the rock berm may consist of any available granular material such as crushed rock or broken-up concrete as long as the material is free-draining and the larger sizes do not create voids in the berm.

The portion of the riverbank above the riprap should be graded and levelled to facilitate surface drainage. The street should be reconstructed so that all street drains discharge into existing storm sewers and not onto the slope through a pipe beneath the street. A curb should also be constructed on the river side of the street to prevent run-off from washing onto the bank.

The horizontal trench drains may be excavated by means of a tractor-mounted backhoe. An initial spacing of 100 foot intervals is suggested. The trench widths need be only about two feet at the bottom and as narrow as the excavation will permit at the top. The bottom of the trench should be at approximately the winter water level with a slight slope towards the river. The trenches should extend back into the bank a distance of 50 feet as shown on Figure 11.

The trenches should be backfilled with a 12 inch layer of sand, followed by crushed stone, one-half inch to six inch diameter, to within two feet of the ground surface. Over this should be placed another foot of sand followed by a layer of topsoil to grade.

The effect of the trench drains on the pore pressures in the bank will have to be monitored. Piezometers should be installed in a section of the bank similarly as for the two earlier sites. If the pore pressures are not sufficiently reduced with the drains at 100 foot intervals, it may be necessary to decrease the spacing to 50 feet. In addition to the piezometers, it would be desireable to install a number of slope hole indicator pipes in the "stabilized" bank to monitor the effectiveness of the remedial measures.

REFERENCES

1. Mishtak, J., 1960, "Soil Mechanics Aspects of the Red River Floodway", Canadian Geotechnical Journal, Vol.1, No, 3.

2. Baracos, A., 1961, "The Stability of Riverbanks in the Metropolitan Winnipeg Area", Proceedings, 14th Annual Conference on Soil Mechanics, Canada, A.C.S.M., TM 69.

3. Van Cauenberghe, R.A., 1971, "Slope Stability Considerations of a Riverbank in Metropolitan Winnipeg", Thesis pending submission to the Faculty of Graduate Studies, University of Manitoba, Winnipeg, Canada.

4. Janzen, P., 1971, "An Analysis of Field and Laboratory Data for Unstable River Banks in the Metropolitan Winnipeg Area", Thesis submitted to the Faculty of Graduate Studies in partial fulfilment of the requirements for the Degree of Master of Science, University of Manitoba, Winnipeg, Canada

5. Minutes of Meeting No. 8, January 26, 1965, Assiniboine River Advisory Board, re design safety factors for structures on the Portage Diversion.

6. Long, E and George, W., 1967, "Buttress Design for Earthquake-induced Slides", ASCE Journal of Soil Mechanics and Foundation Division, Vol. 93, SM4, July.

7. Presentation to the Winnipeg Geotechnical Section of the Winnipeg Branch of E.I.C., 1969, "Red River Valley Dyking".

8. Baracos, A., 1951, Consultants' report to the Red River Dyking Board.

9. "Use of Riprap for Bank Protection", Hydraulics Engineering Circular No.11, 1967, U.S. Bureau of Public Roads.

10. Winkley, B.R., 1970, "Practical Aspects of River Regulation and Control", Institute of River Mechanics, Colorado State University, Fort Collins, Colorado, June.

11. Terzaghi, K., and Peck, R.B., 1967, Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, Inc., New York.

12. Turnbull, W.J., and Hvorslev, M.J., 1967, "Special Problems in Slope Stability", ASCE Journal of Soil Mechanics and Foundation Division, Vol. 93, SM4, July.

13. Smith and Stafford, 1957, "Horizontal Drains on California Highways", ASCE Journal of Soil Mechanics and Foundations Division, July.

14. "Stabilization of Omands Creek Bank, South of Potage Avenue", Internal Report, 1969, Planning Division, Water Resources Branch, Department of Mines Resources and Environmental Management, Province of Manitoba.

15. Lundy and Greenfield,"Evaluation of Deep Insitu Soil Stabilization by High Pressure Lime Slurry Injection" Highway Research Board.

16. National Lime Association, 1969, Reprinted from Construction Methods and Equipment, April.

17. Talme, O.A., 1969, "Clay Sensitivity and Chemical Stabilization", Rapp. Fran Byggforskningen (Reports from Swedish Building Research), 56/68 Stockholm 1969, Thesis.

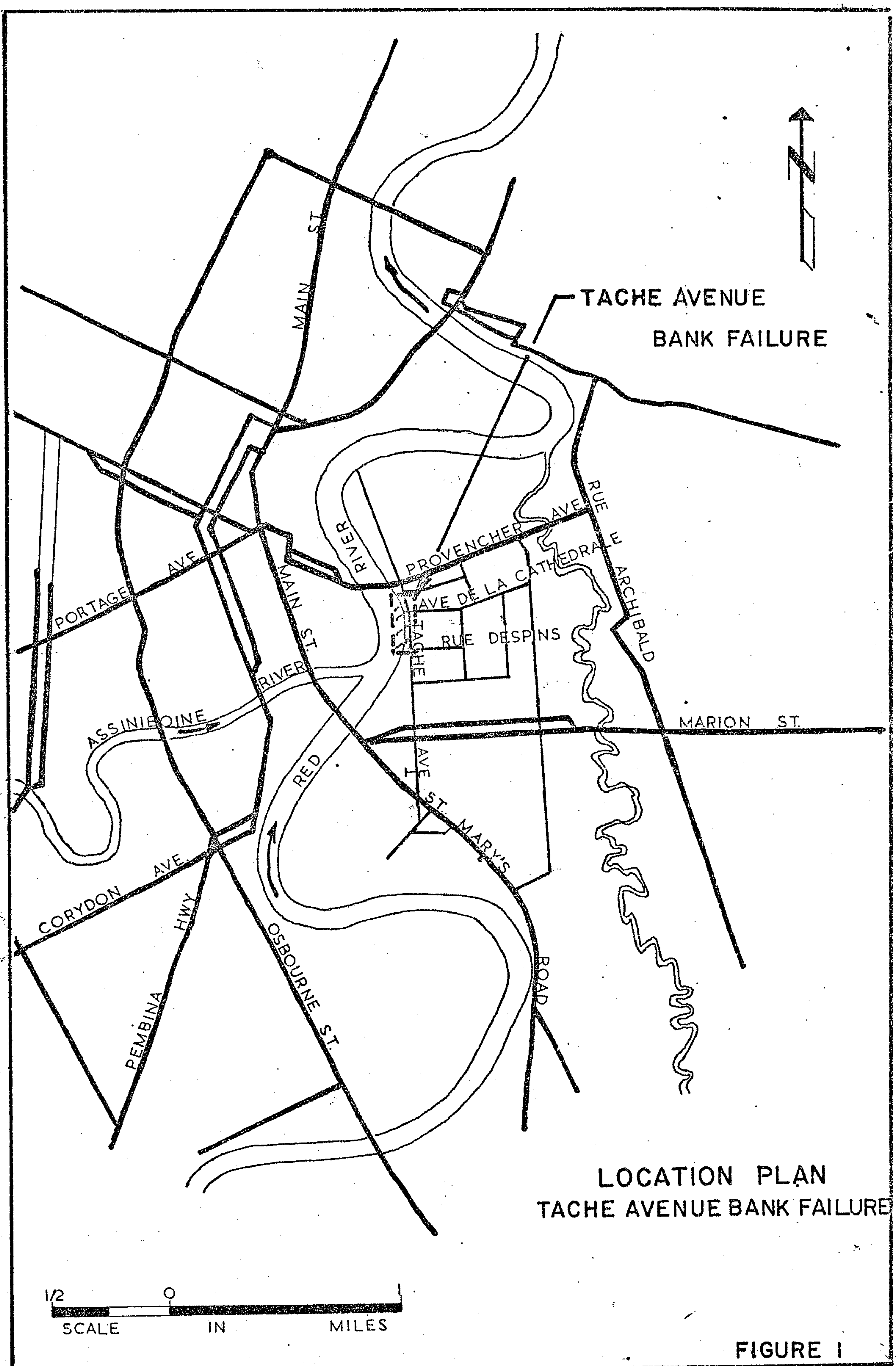
18. "Report on Investigation into Measures for the Reduction of Flood Hazard in the Greater Winnipeg Area", Appendix E, 1950

19. Sutherland, H.B., 1966, "The Stability of the Riverbanks in the Winnipeg Area", Report to Rivers and Streams Authority No. 1, Winnipeg, Manitoba, June.

20. Handy, R.L.,and Williams, W.W., 1967, "Chemical Stabilization of an Active Landslide", ASCE Journal of Civil Engineerig.

21. Design Manual: Soil Mechanics, Foundations and Earth Structures, Navdocks DM-7, Department of the Navy, Bureau of Yards and Docks, Washington 25, D.C., 1962.

FIGURES



LOCATION PLAN
TACHE AVENUE BANK FAILURE

FIGURE 1

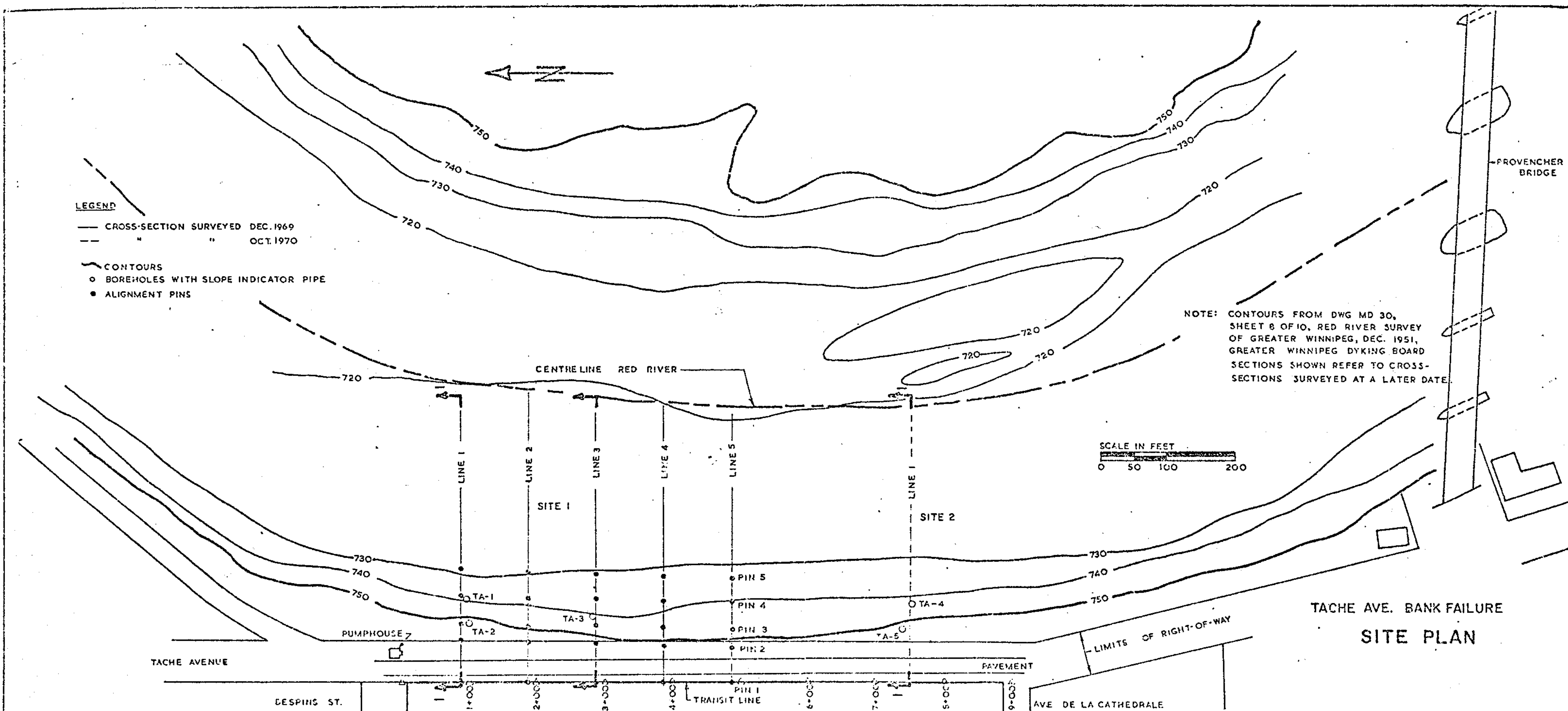

LEGEND
CROSS-SECTION SURVEYED DEC. 1969
" " OCT. 1970
CONTOURS
BOREHOLES WITH SLOPE INDICATOR PIPE
ALIGNMENT PINS
NOTE: CONTOURS FROM DWG MD 30, SHEET 8 OF 10, RED RIVER SURVEY OF GREATER WINNIPEG, DEC. 1951, GREATER WINNIPEG DYKING BOARD
SECTIONS SHOWN REFER TO CROSS-SECTIONS SURVEYED AT A LATER DATE.
CENTRE LINE RED RIVER
PROVENCHER BRIDGE
SCALE IN FEET
0 50 100 200
750
740
730
720
LINE 1
LINE 2
LINE 3
LINE 4
LINE 5
SITE 1
SITE 2
TA-1
TA-2
TA-3
TA-4
TA-5
PIN 5
PIN 4
PIN 3
PIN 2
PIN 1
PUMPHOUSE
TACHE AVENUE
PAVEMENT
TRANSIT LINE
LIMITS OF RIGHT-OF-WAY
DESPINS ST.
AVE DE LA CATHEDRALE
1+00
2+00
3+00
4+00
6+00
7+00
9+00
TACHE AVE. BANK FAILURE
SITE PLAN

FIGURE 2

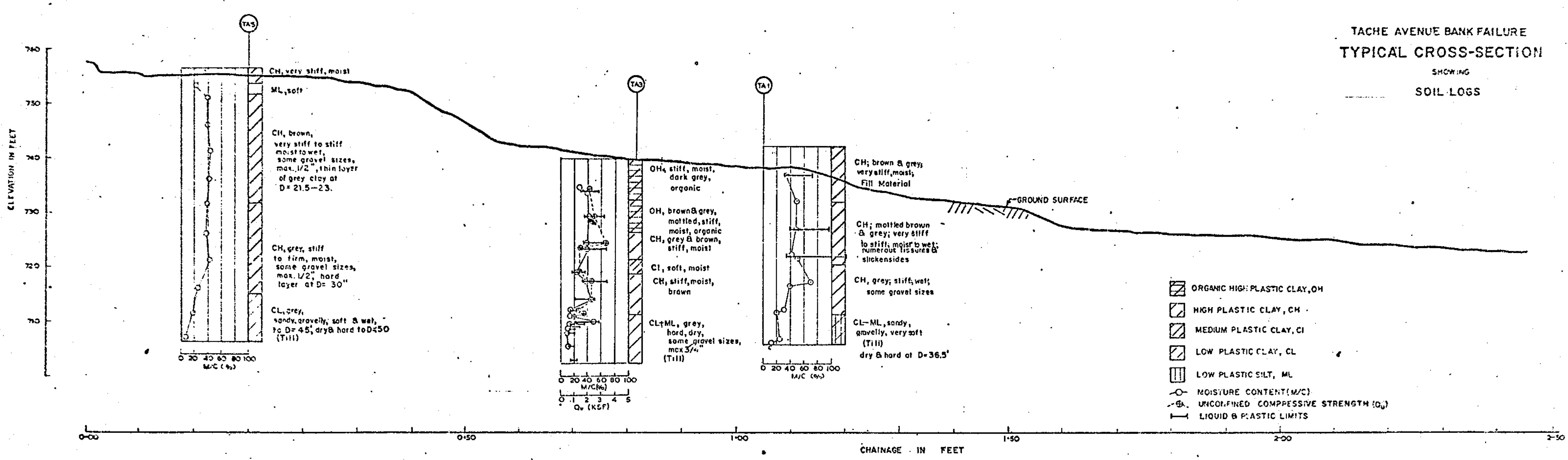
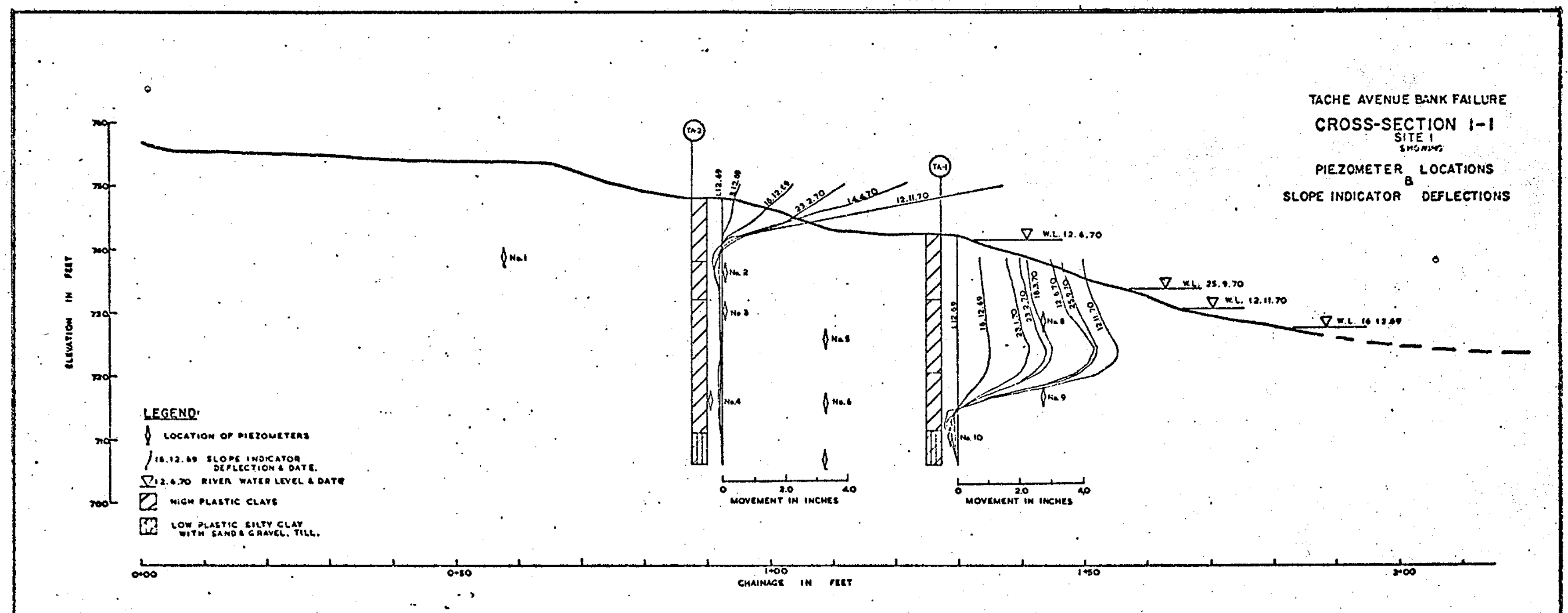

TACHE AVENUE BANK FAILURE
TYPICAL CROSS-SECTION
SHOWING
SOIL LOGS
ELEVATION IN FEET
760
750
740
730
720
710
TA5
CH, very stiff, moist
ML, soft
CH, brown, very stiff to stiff moist to wet, some gravel sizes, max. 1/2", thin layer of grey clay at D= 21.5—23.
CH, grey, stiff to firm, moist, some gravel sizes, max. 1/2", hard layer at D= 30"
CL, grey, sandy, gravelly, soft & wet, to D= 45', dry & hard to D<50 (Till)
0 20 40 60 80 100
M/C (%)
TA3
OH, stiff, moist, dark grey, organic
OH, brown & grey, mottled, stiff, moist, organic
CH, grey & brown, stiff, moist
CI, soft, moist
CH, stiff, moist, brown
CL+ML, grey, hard, dry, some gravel sizes, max 3/4" (Till)
0 20 40 60 80 100
M/C (%)
0 1 2 3 4 5
Qu (KSF)
TA1
CH; brown & grey; very stiff, moist; Fill Material
CH; mottled brown & grey; very stiff to stiff; moist to wet; numerous fissures & slickensides
CH, grey; stiff; wet; some gravel sizes
CL—ML, sandy, gravelly, very soft (Till) dry & hard at D=36.5'
0 20 40 60 80 100
M/C (%)
GROUND SURFACE
ORGANIC HIGH PLASTIC CLAY, OH
HIGH PLASTIC CLAY, CH
MEDIUM PLASTIC CLAY, CI
LOW PLASTIC CLAY, CL
LOW PLASTIC SILT, ML
MOISTURE CONTENT (M/C)
UNCONFINED COMPRESSIVE STRENGTH (Qu)
LIQUID & PLASTIC LIMITS
0+00
0+50
1+00
1+50
2+00
2+50
CHAINAGE IN FEET

FIGURE 3



SECTION I-I

FIGURE 4.1

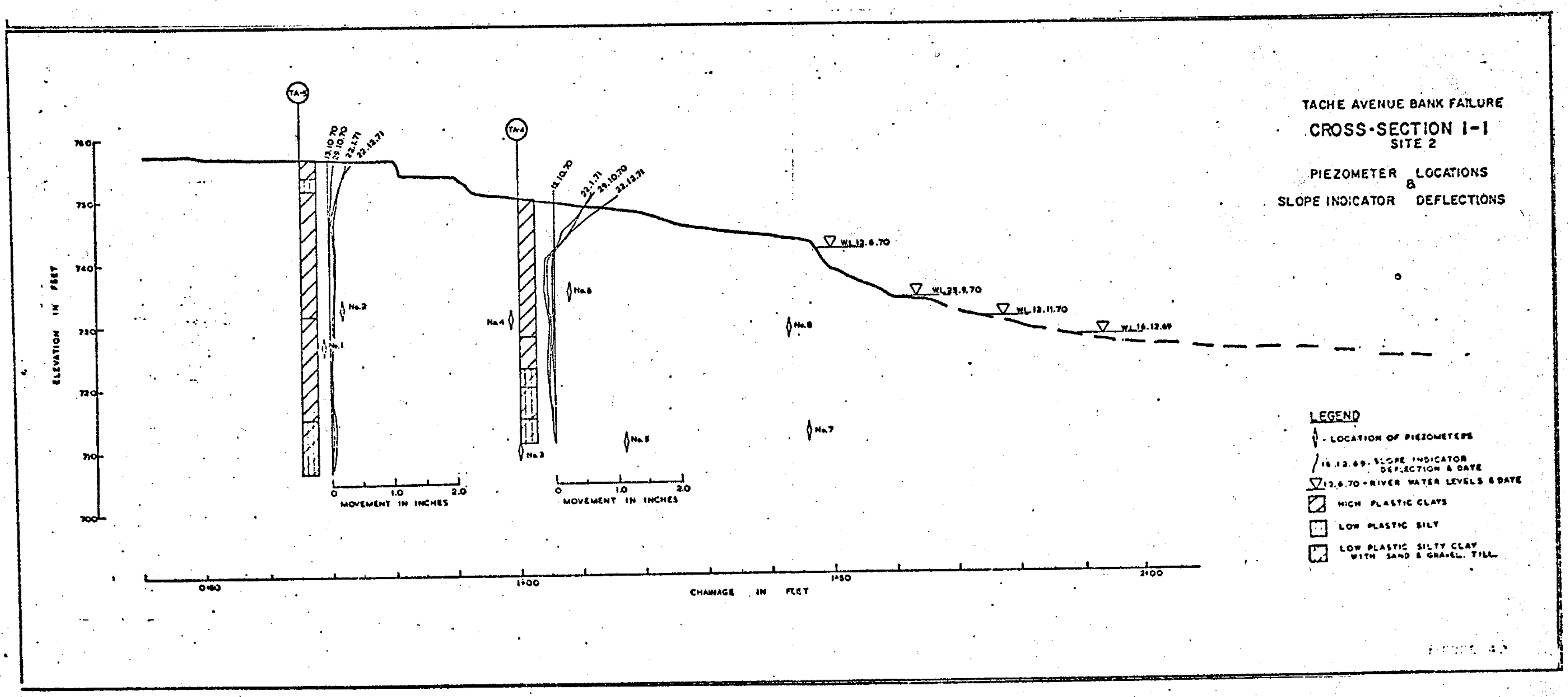
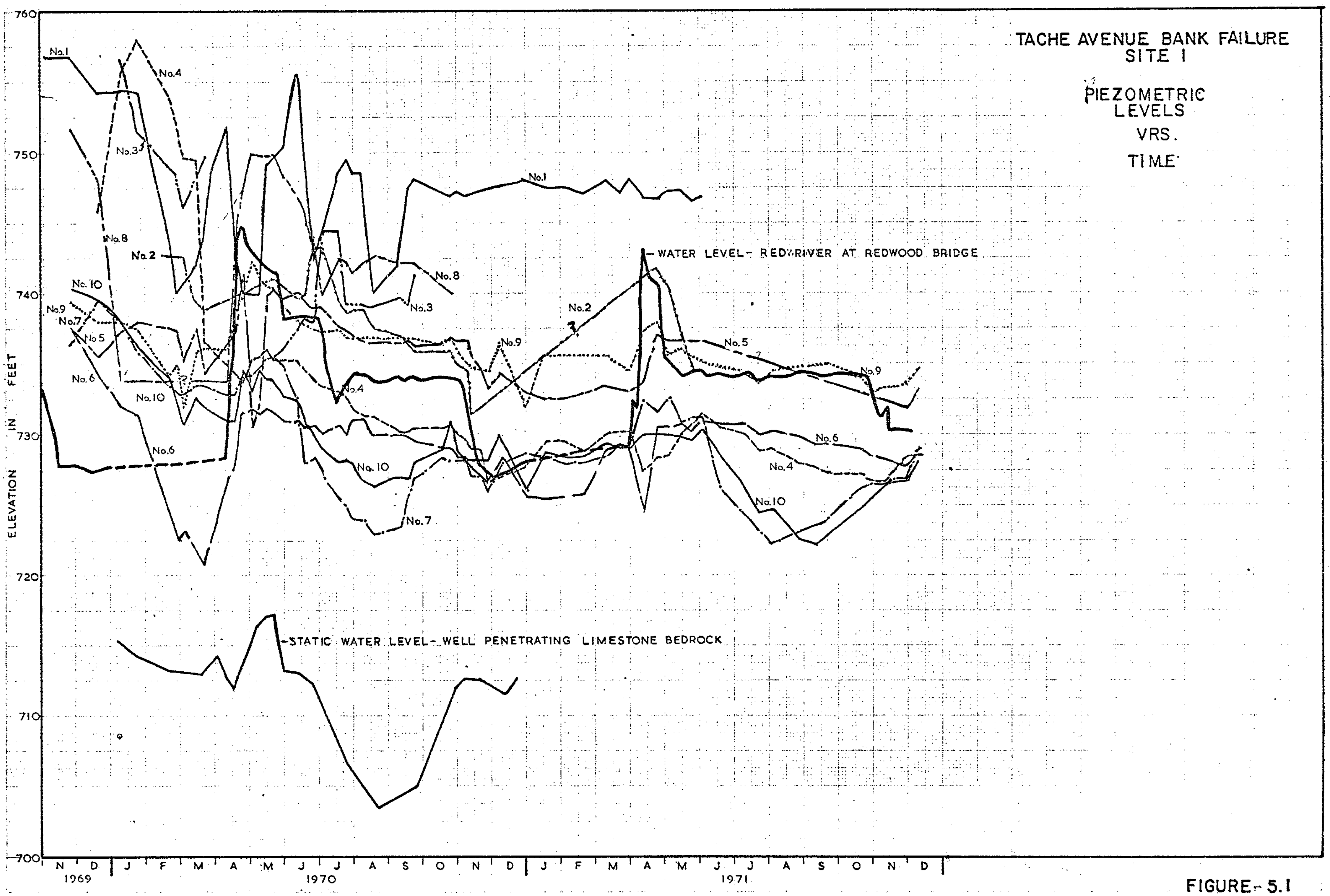

TACHE AVENUE BANK FAILURE
CROSS-SECTION I-I
SITE 2
PIEZOMETER & LOCATIONS
SLOPE INDICATOR DEFLECTIONS
LEGEND
- LOCATION OF PIEZOMETERS
16.12.69 - SLOPE INDICATOR DEFLECTION & DATE
12.6.70 - RIVER WATER LEVELS & DATE
HIGH PLASTIC CLAYS
LOW PLASTIC SILT
LOW PLASTIC SILTY CLAY WITH SAND & GRAVEL TILL
TA-5
TA-4
ELEVATION IN FEET
MOVEMENT IN INCHES
CHAINAGE IN FEET
WL 12.6.70
WL 25.9.70
WL 13.11.70
WL 16.12.69
No.1
No.2
No.3
No.4
No.5
No.6
No.7
No.8

FIGURE 4.2



TACHE AVENUE BANK FAILURE
SITE I
PIEZOMETRIC
LEVELS
VRS.
TIME
WATER LEVEL- RED RIVER AT REDWOOD BRIDGE
STATIC WATER LEVEL- WELL PENETRATING LIMESTONE BEDROCK
ELEVATION IN FEET
760
750
740
730
720
710
700
N D J F M A M J J A S O N D J F M A M J J A S O N D
1969
1970
1971
No.1
No.2
No.3
No.4
No.5
No.6
No.7
No.8
No.9
No.10

FIGURE- 5.1

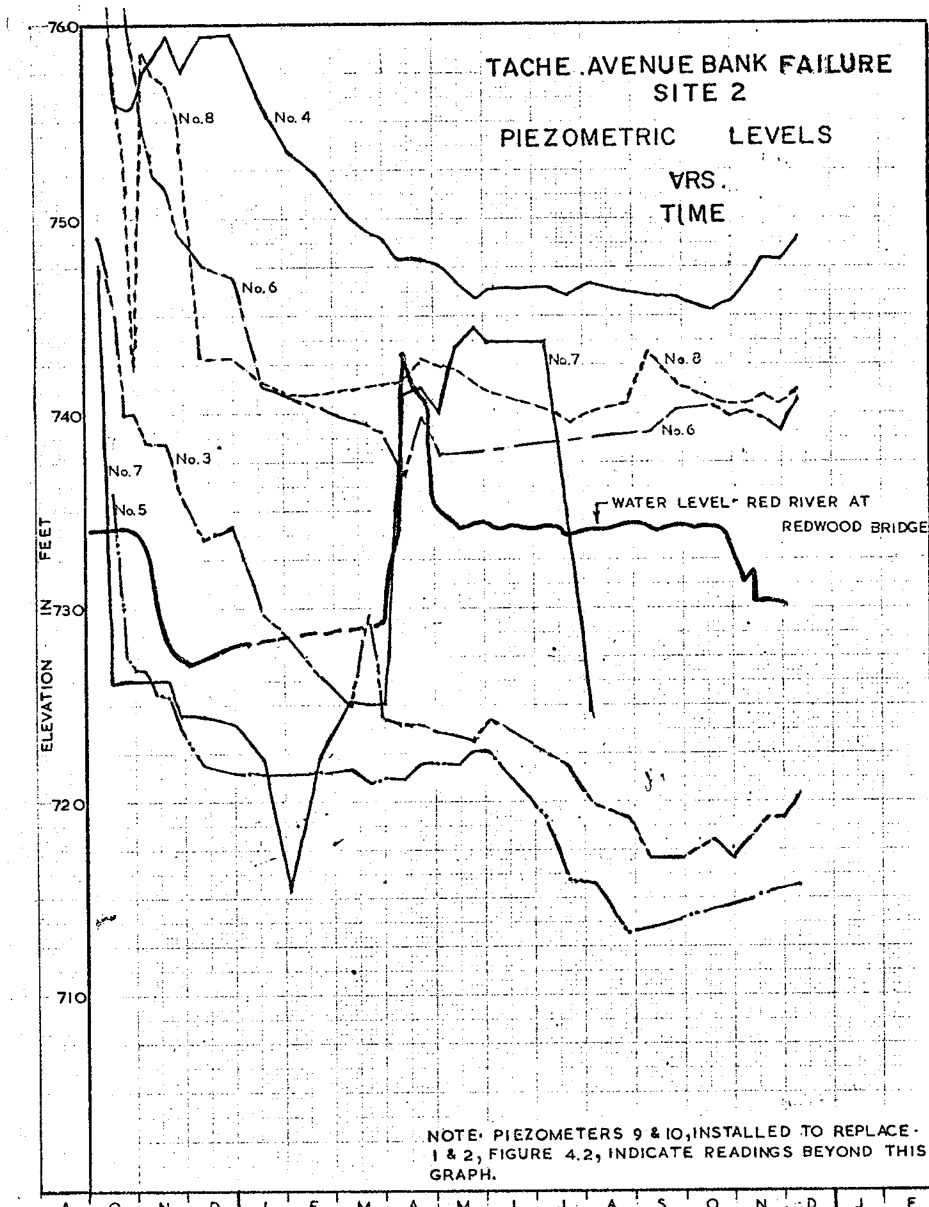

TACHE AVENUE BANK FAILURE
SITE 2
PIEZOMETRIC LEVELS
VRS.
TIME
ELEVATION IN FEET
760
750
740
730
720
710
No. 4
No. 8
No. 6
No. 7
No. 3
No. 5
WATER LEVEL- RED RIVER AT REDWOOD BRIDGE
NOTE: PIEZOMETERS 9 & 10, INSTALLED TO REPLACE 1 & 2, FIGURE 4.2, INDICATE READINGS BEYOND THIS GRAPH.
A
O
N
D
J
F
M
A
M
J
J
A
S
O
N
D
J
F
1970
1971

FIGURE 5.2

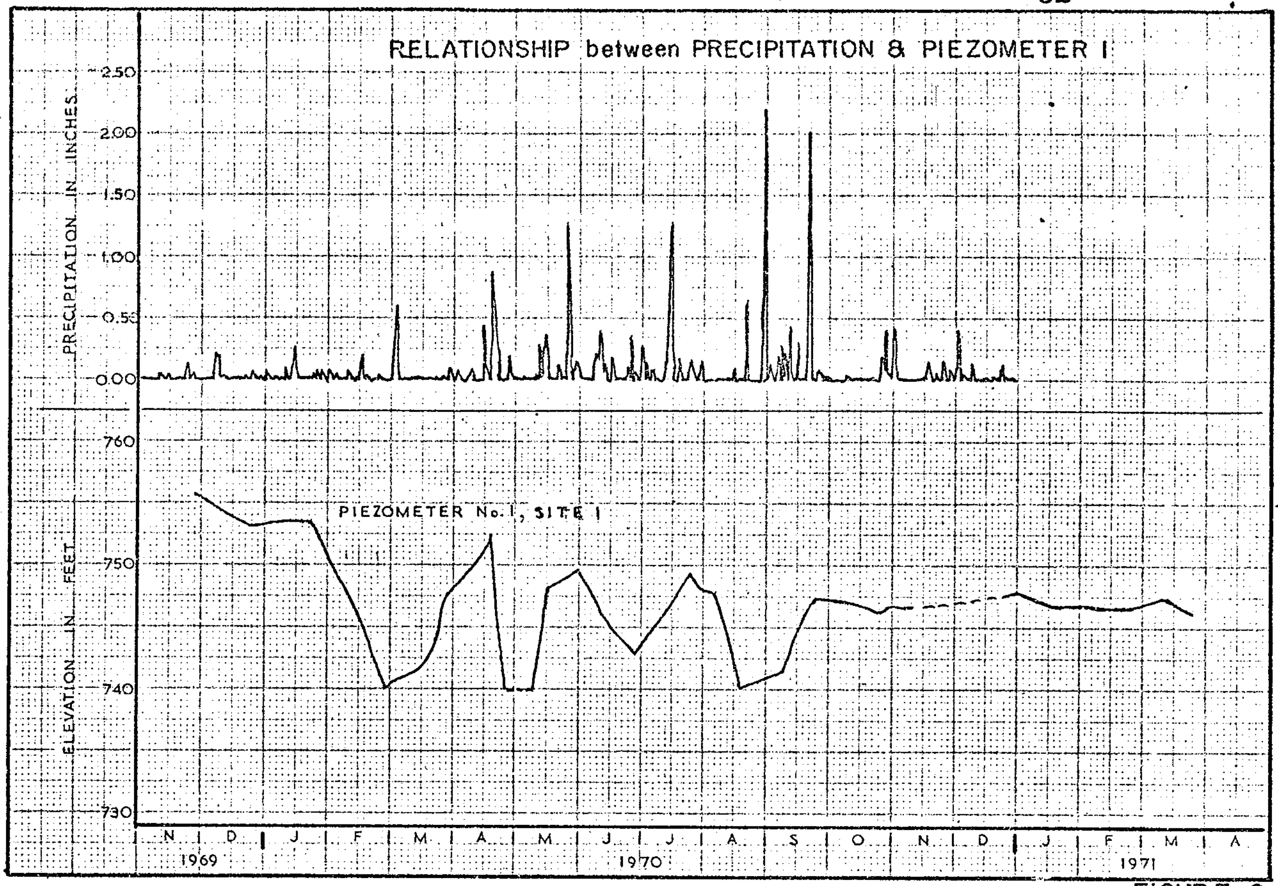
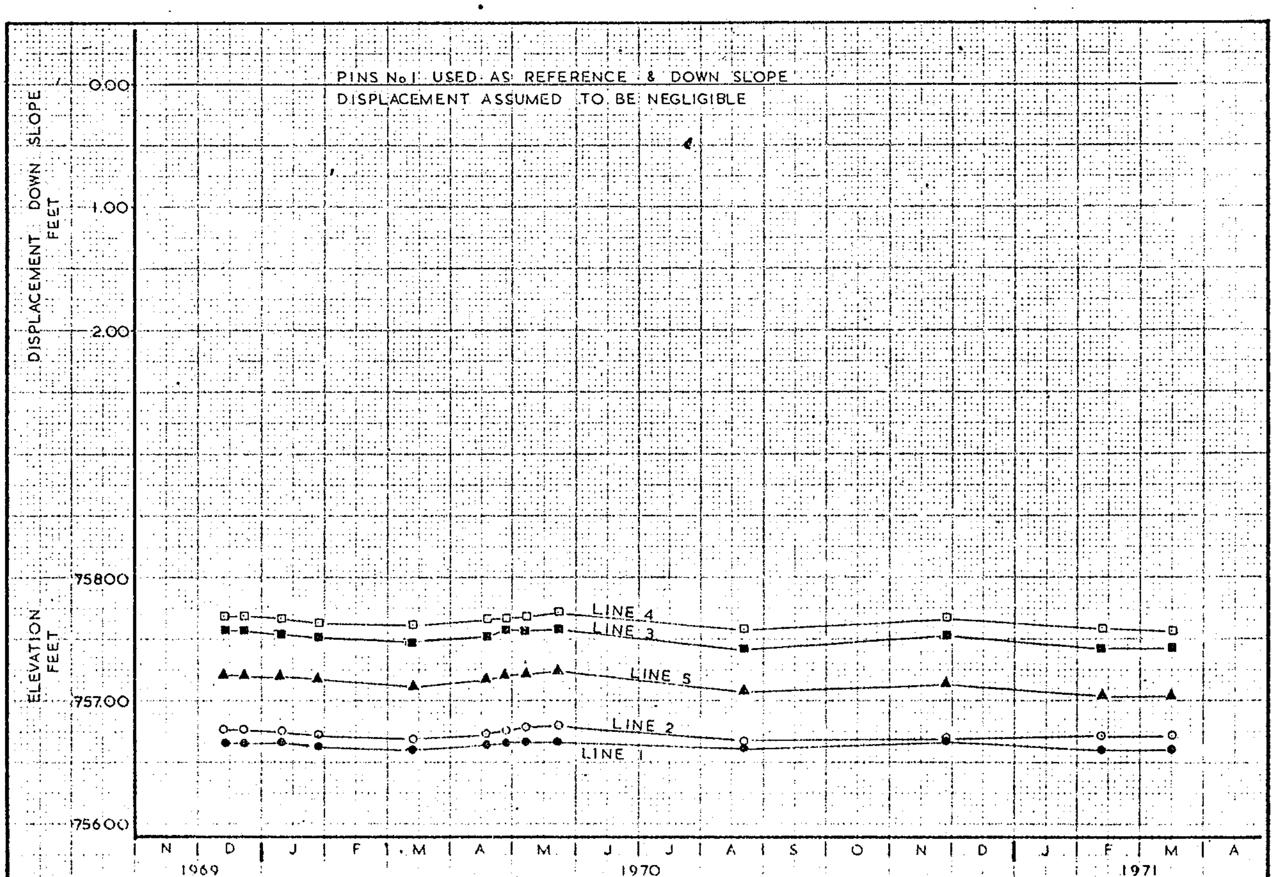
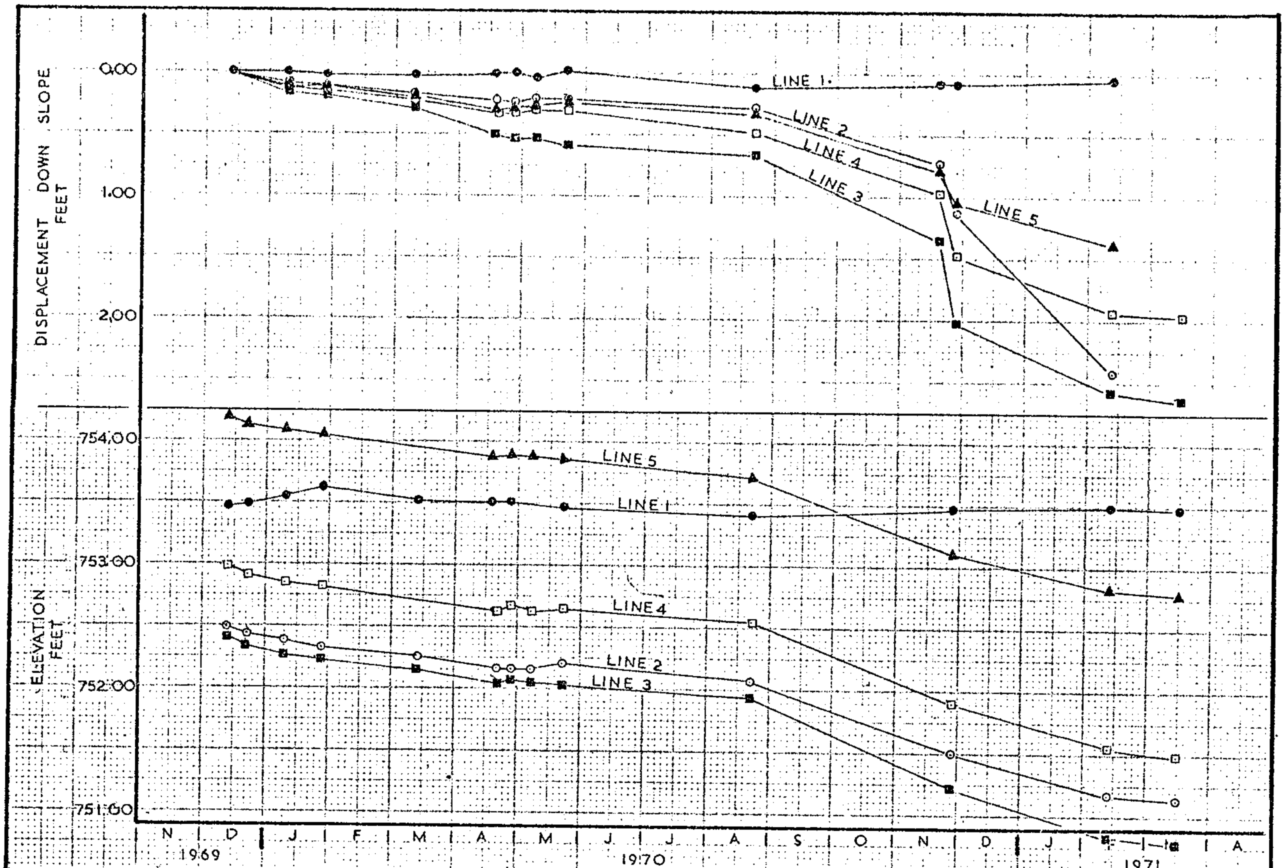


FIGURE 6

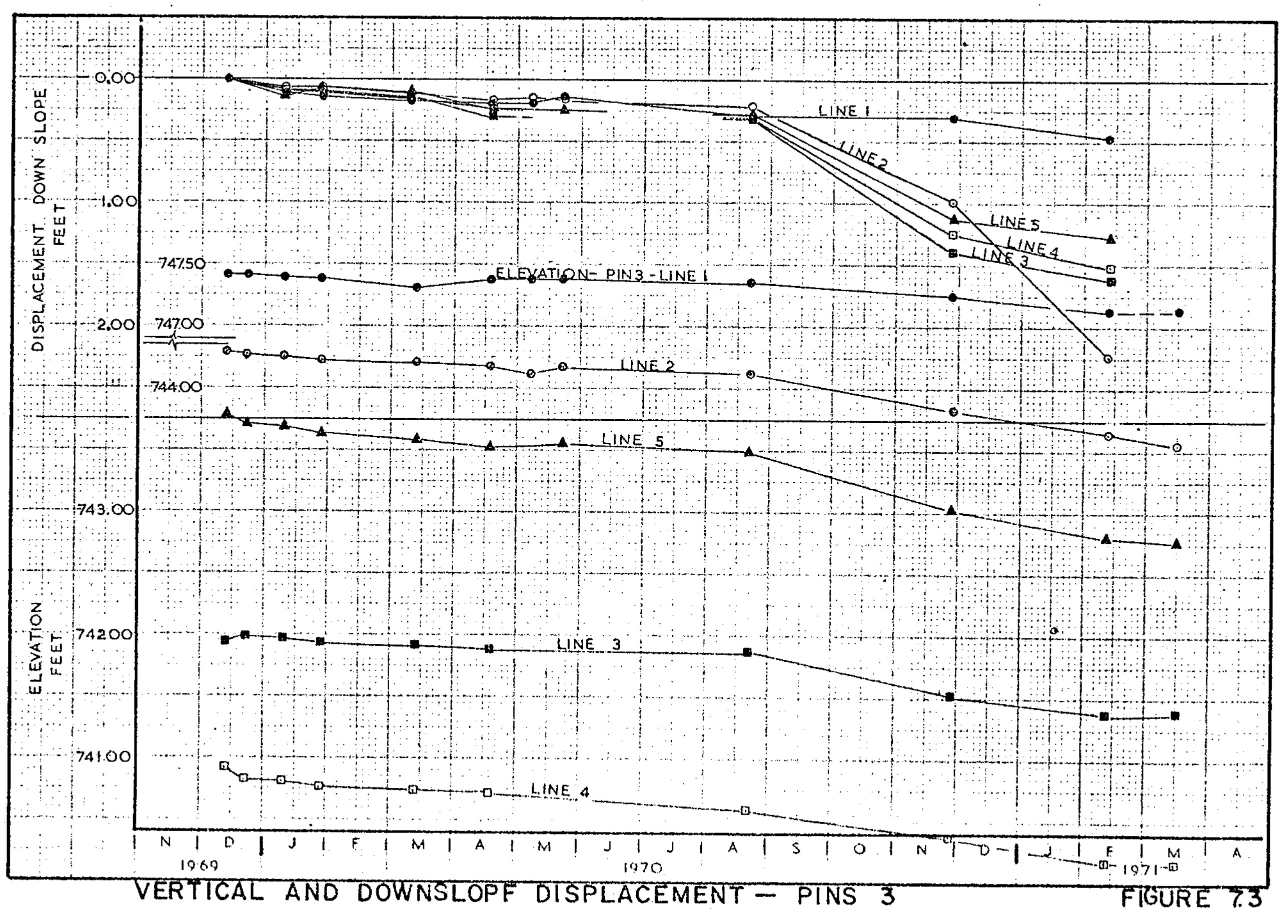


VERTICAL AND DOWNSLOPE DISPLACEMENT — PINS I

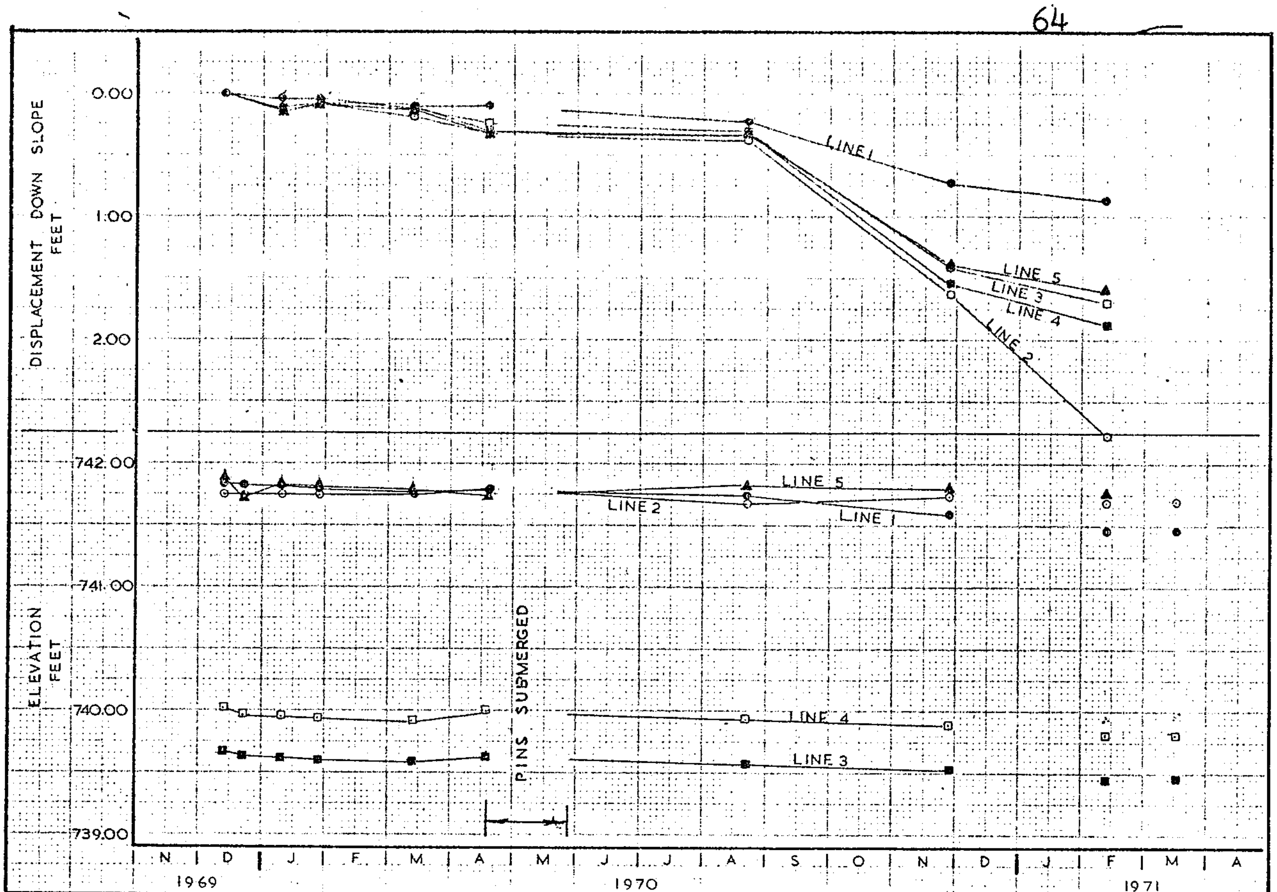
FIGURE 7.I



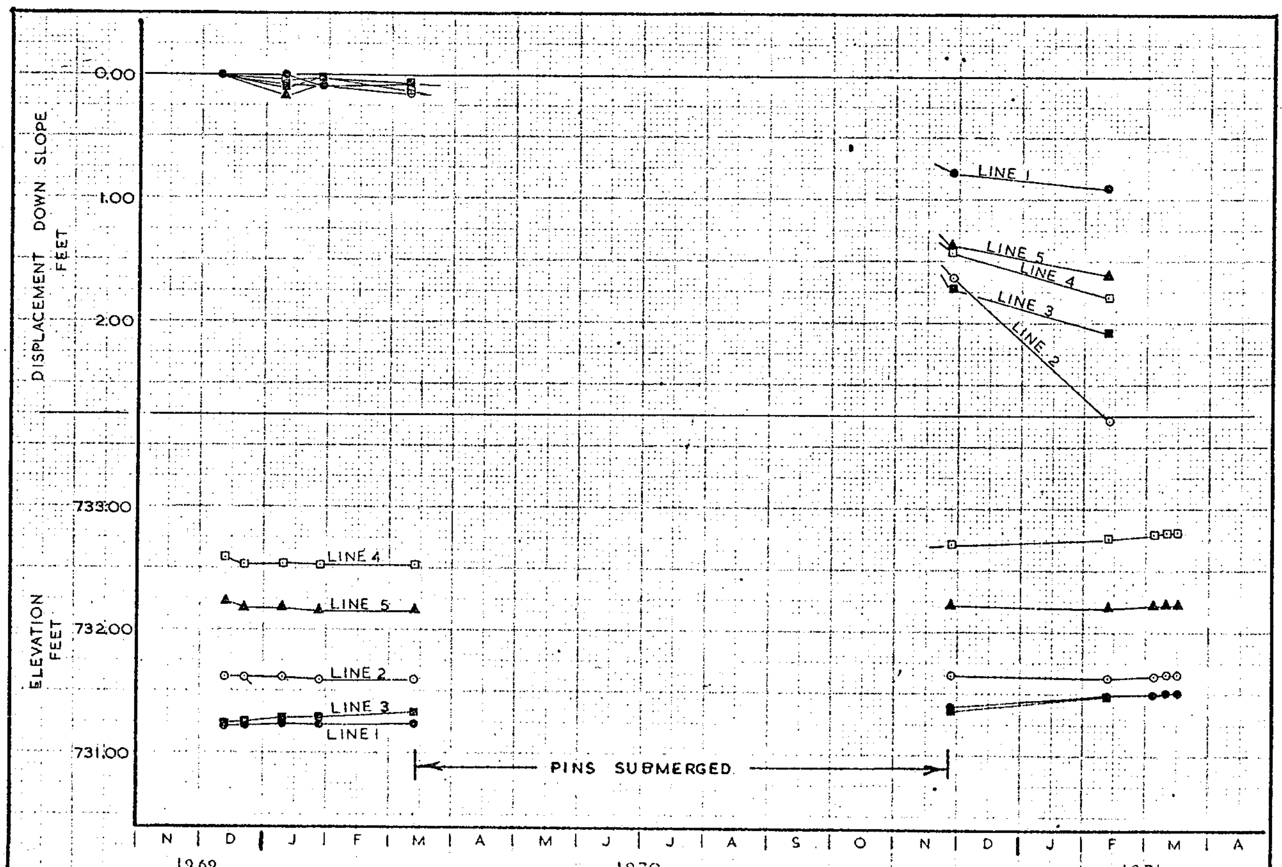
VERTICAL AND DOWNSLOPE DISPLACEMENT — PINS 2 FIGURE 7.2



VERTICAL AND DOWNSLOPF DISPLACEMENT — PINS 3 FIGURE 7.3



VERTICAL AND DOWNSLOPE DISPLACEMENT — PINS 4 FIGURE 7.4



VERTICAL AND DOWNSLOPE DISPLACEMENT — PINS 5 FIGURE 7.5

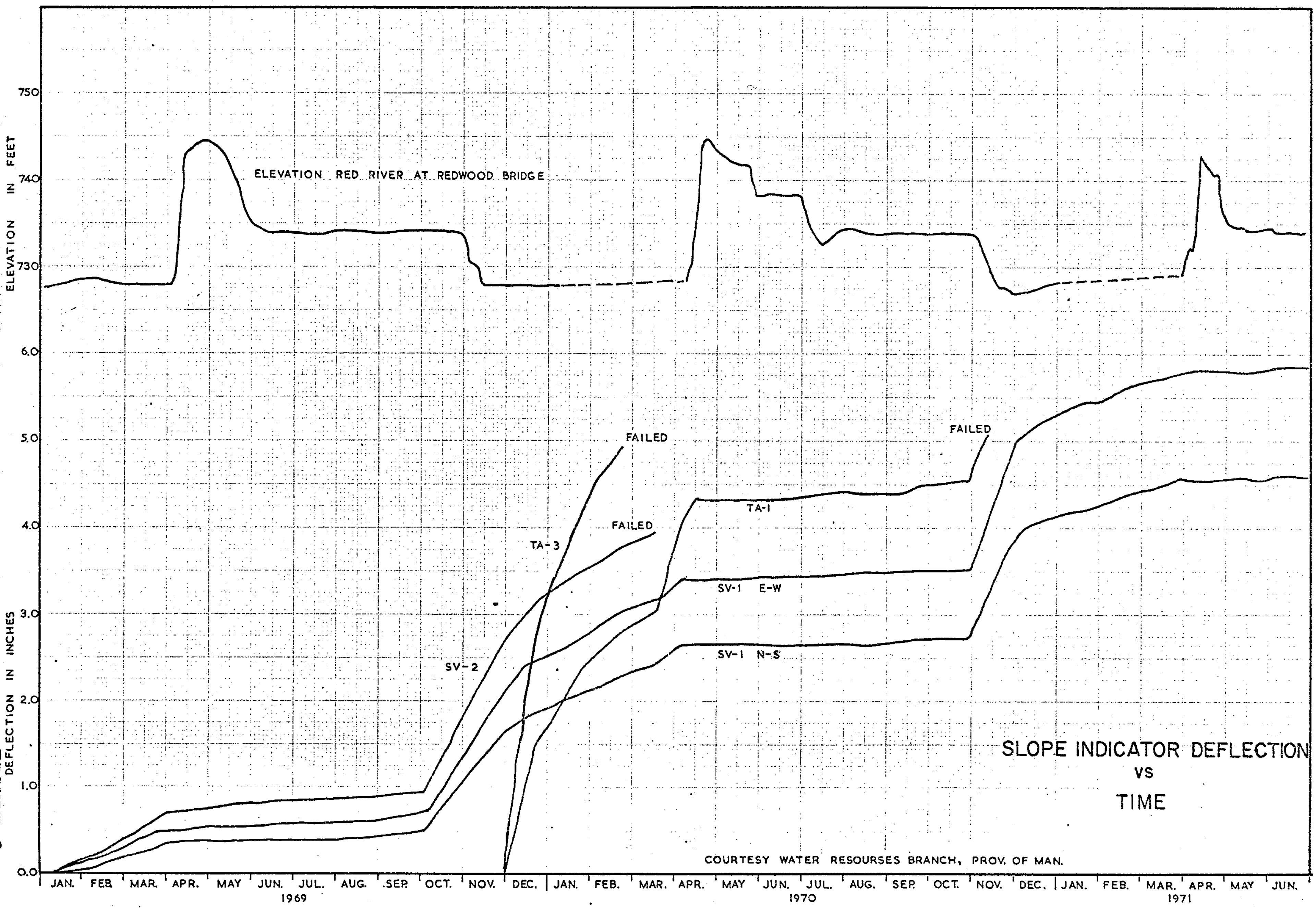

ELEVATION RED RIVER AT REDWOOD BRIDGE
ELEVATION IN FEET
750
740
730
6.0
5.0
4.0
3.0
2.0
1.0
0.0
DEFLECTION IN INCHES
FAILED
FAILED
FAILED
TA-3
TA-1
SV-1 E-W
SV-1 N-S
SV-2
SLOPE INDICATOR DEFLECTION
VS
TIME
COURTESY WATER RESOURSES BRANCH, PROV. OF MAN.
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEP. OCT. NOV. DEC.
1969
JAN. FEB. MAR. APR. MAY JUN. JUL. AUG. SEP. OCT. NOV. DEC.
1970
JAN. FEB. MAR. APR. MAY JUN.
1971

FIGURE 8

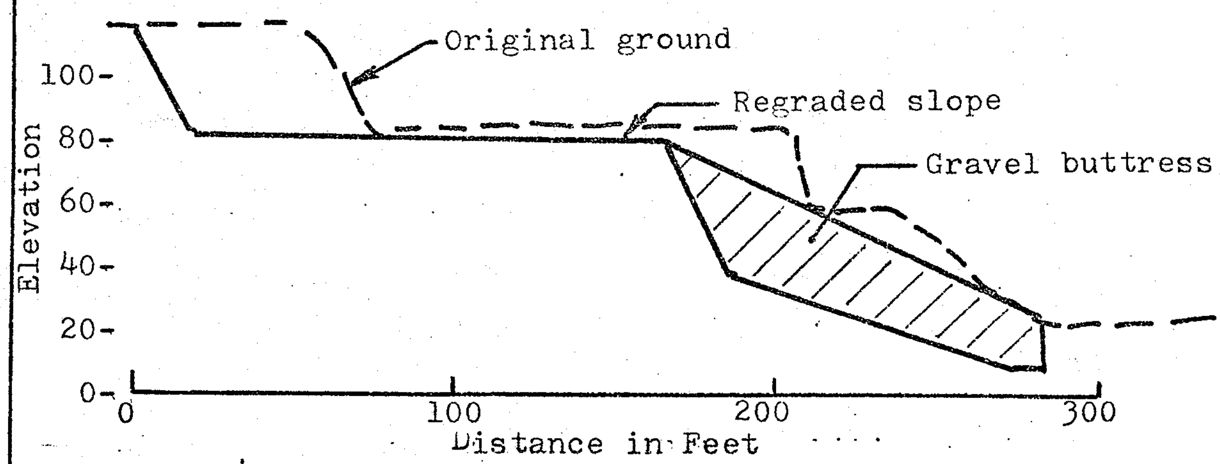


Figure 9.1. Gravel buttress used to stabilize a slope.

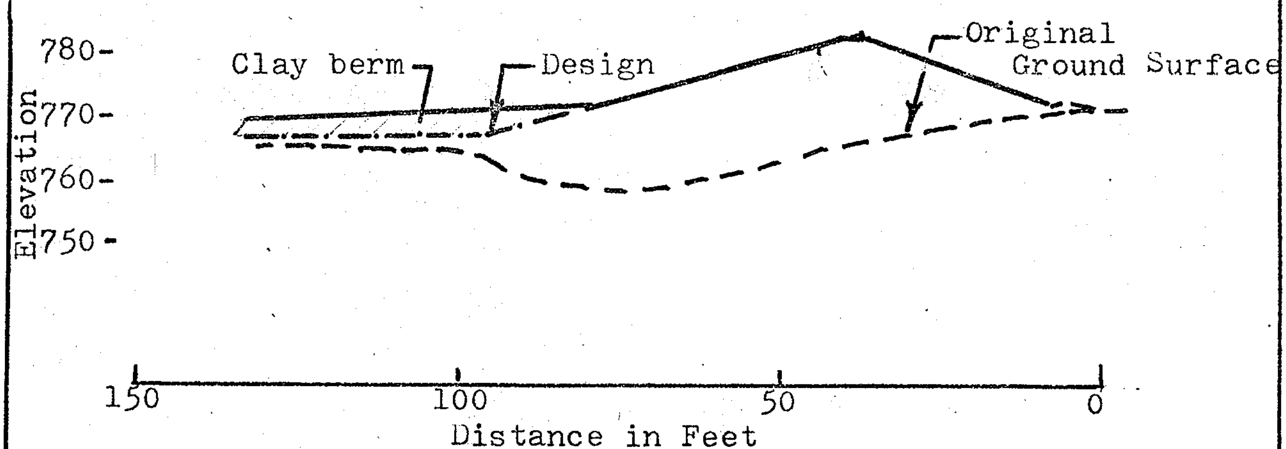


Figure 9.2. Clay berm used to stabilize a dyke, Morris, Manitoba.

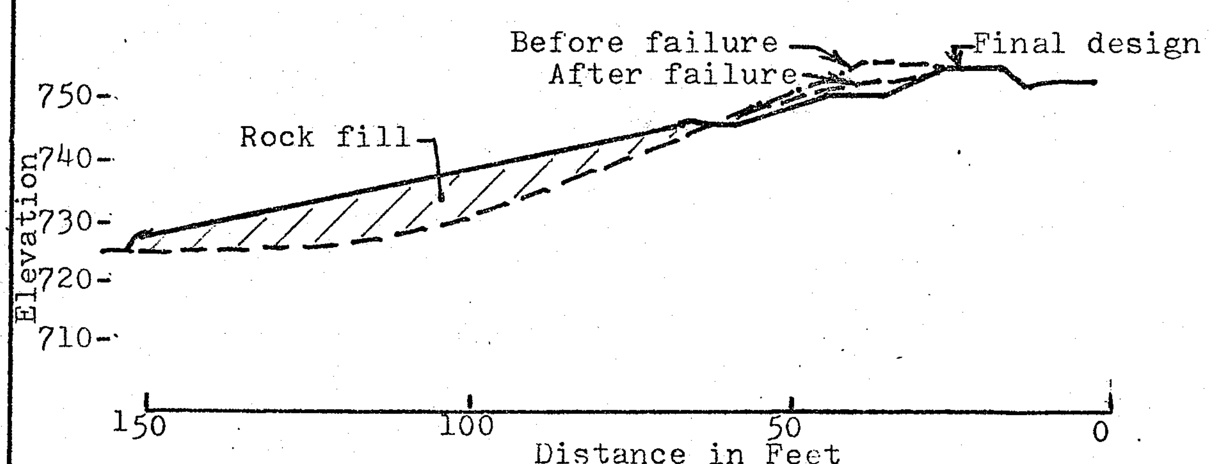


Figure 9.3. Rock fill used to stabilize a failure along the Red River, Winnipeg

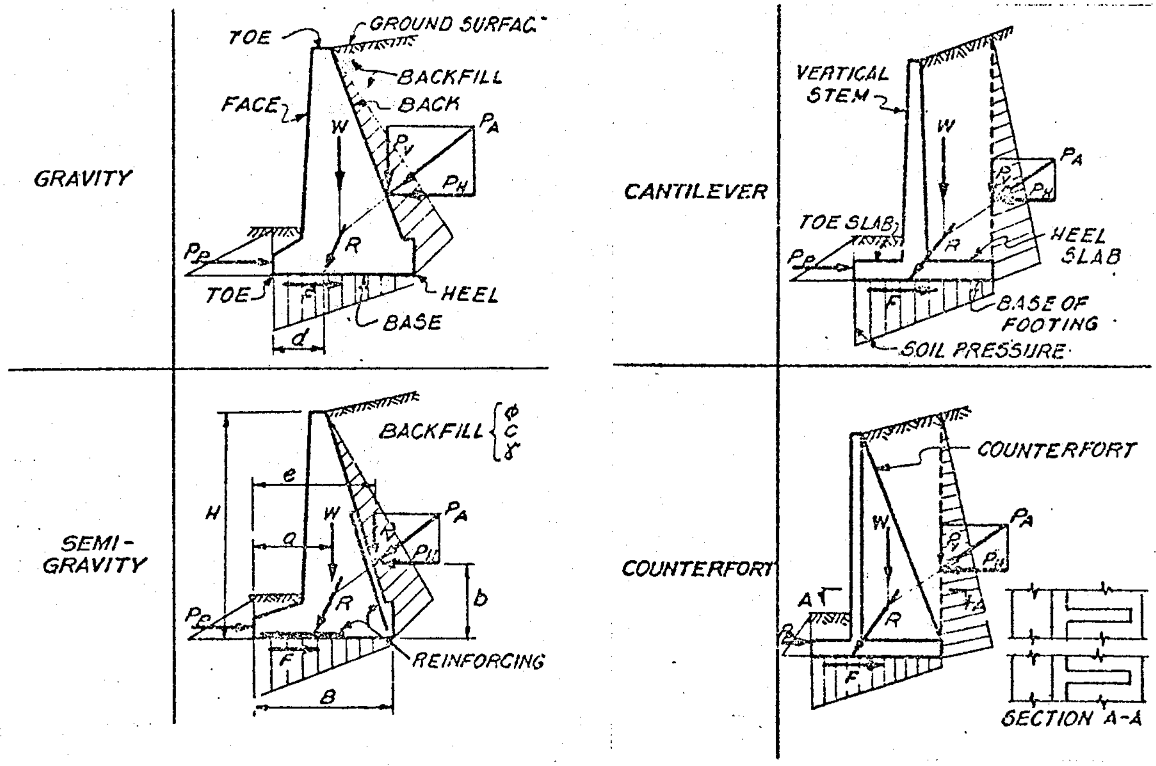


Figure 10.1. Examples of Retaining Walls (21)

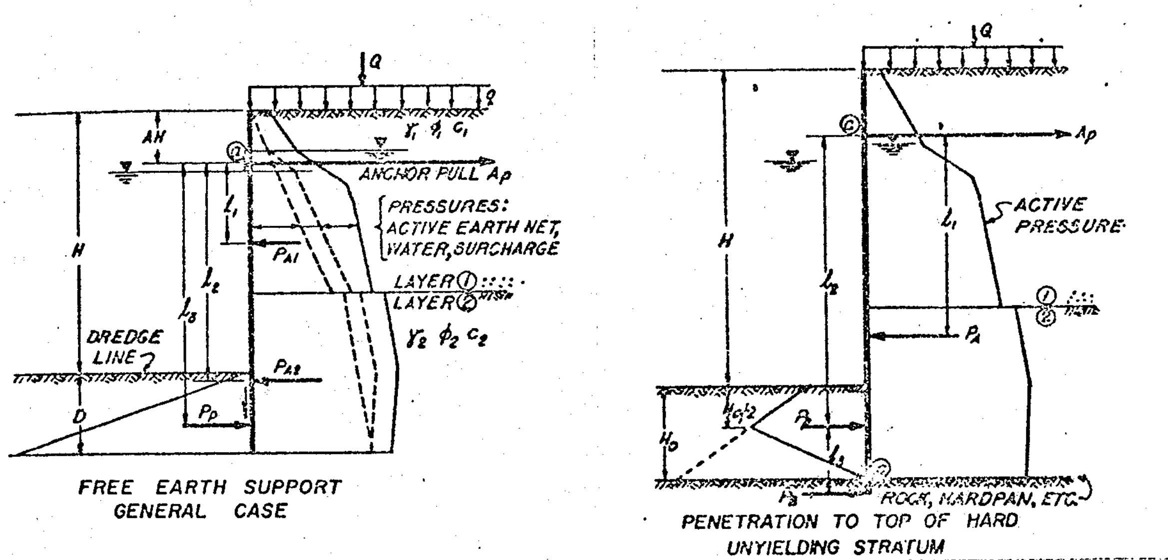


Figure 10.2. Examples of Anchored Bulkheads (21)

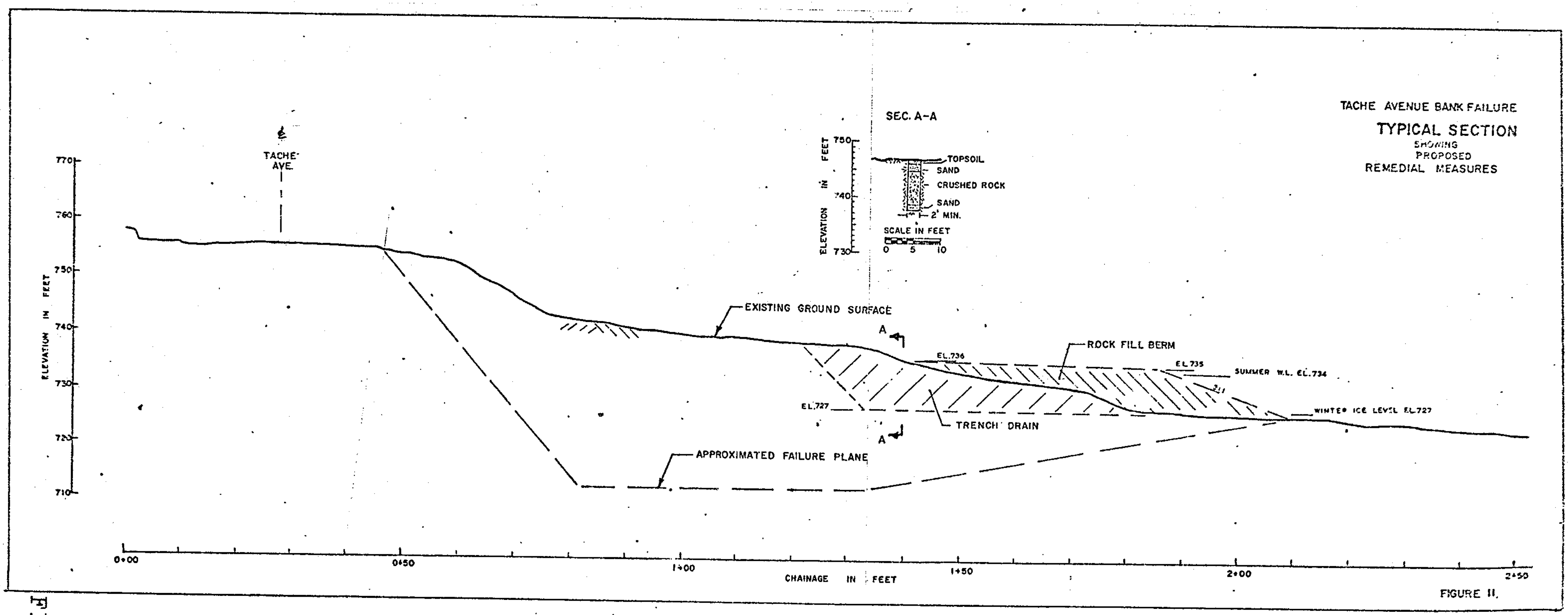


FIGURE 11.

FIGURE 11

TABLES

a.) Mottled brown and grey, highly plastic clay			
Test	Average	Range	Number
Moisture Content (%)	44.9	39.8 - 49.8	23
Liquid Limit (%)	81.3	62.6 - 110.5	5
Plastic Limit (%0	33.3	28.5 - 37.7	5
Unconfined Compressive Strength (PSF)	2420	1500 - 3280	3
Wet Density (PCF)	112	109 - 114	16
Direct Shear (residual values)*			
Effective Angle of Internal Friction (degrees)	9.9	8.3 - 13.0	4
Effective Cohesion(PSF)	72	0.0 - 115	4

* Data from a St. Vital site also included as per Janzen (4).

b.) Grey, highly plastic clay			
Test	Average	Range	Number
Moisture Content (%)	48.2	24.8 - 69.0	22
Liquid Limit (%)	59.8	--	1
Plastic Limit (%)	25.5	--	1
Unconfined Compressive Strength (PSF)	2340	--	1
Wet Density (PCF)	111	105 - 117	17
Direct Shear (residual values):			
Effective Angle of Internal Friction (degrees)	9.5	8.0- 11.5	3
Effective Cohesion(PSF)	130	72 - 216	3

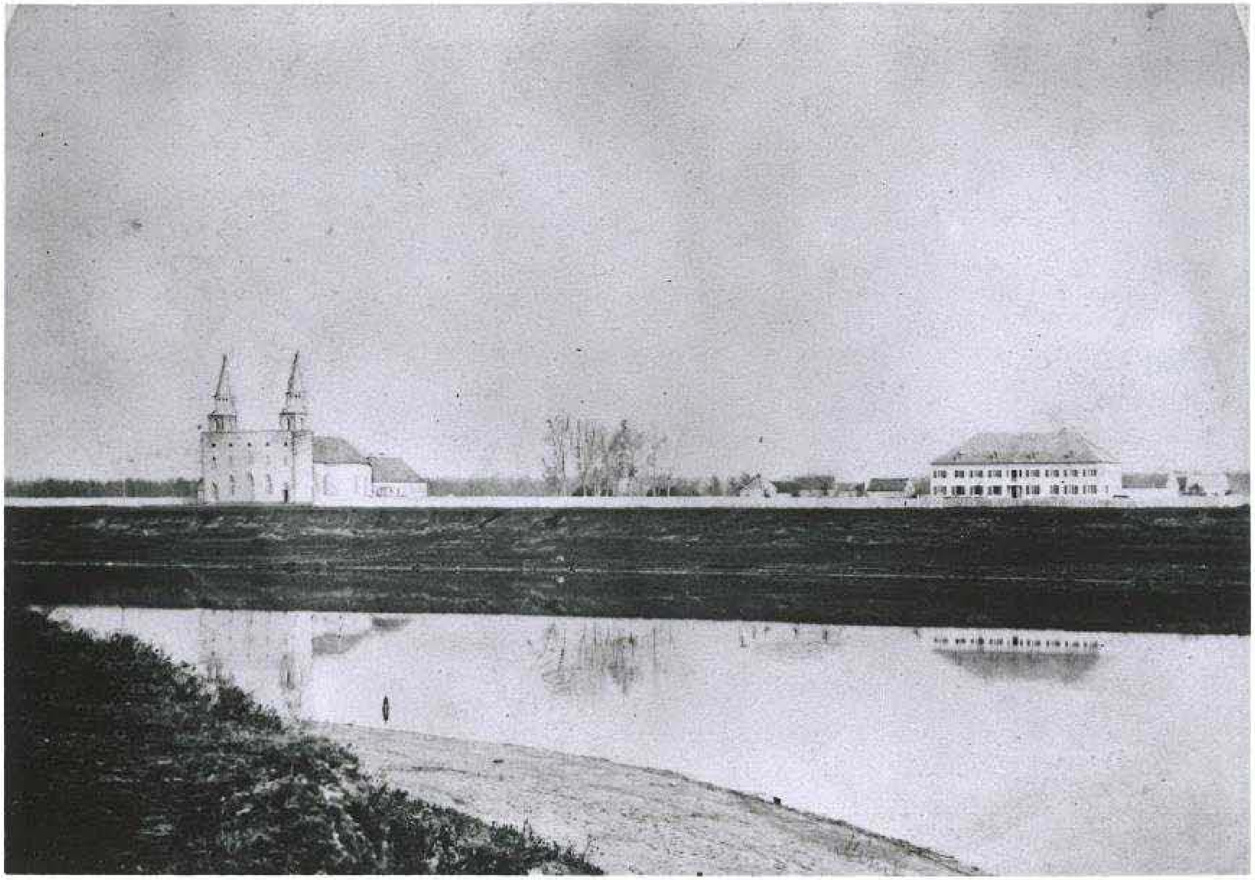
c.) Low plastic, silty, sandy, gravelly clay (Till)			
Test	Average	Range	Number
Moisture Content (%)	16.8	7.5 - 30.8	23
Liquid Limit (%)	24.2	17.9 - 36.6	7
Plastic Limit (%)	13.2	12.3 - 13.8	7
Wet Density (PCF)	128	108 - 142	11
Unconfined Compressive Strength (PSF)	1000	900 - 1800	3
Direct Shear (residual values):			
Effective Angle of Internal Friction (degrees)	31.5	--	1
Effective Cohesion(PSF)	0.0	--	1
Consolidated Undrained Triaxial:			
(with pore pressurements)			
Effective Angle of Internal Friction (degrees)	29.0	25.4, 32.5	2
Effective Cohesion(PSF)	288		2

TABLE I

Summary of Laboratory Tests

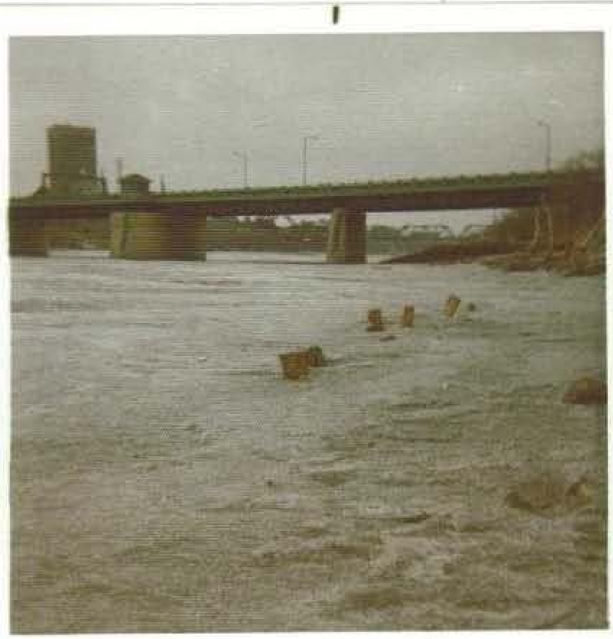
PHOTOGRAPHS

Pl. Tache Avenue riverbank, August 1971.



P2. Tache Avenue riverbank site, 1858. Note what appears to be a fresh failure scarp. Courtesy Manitoba Provincial Archives.

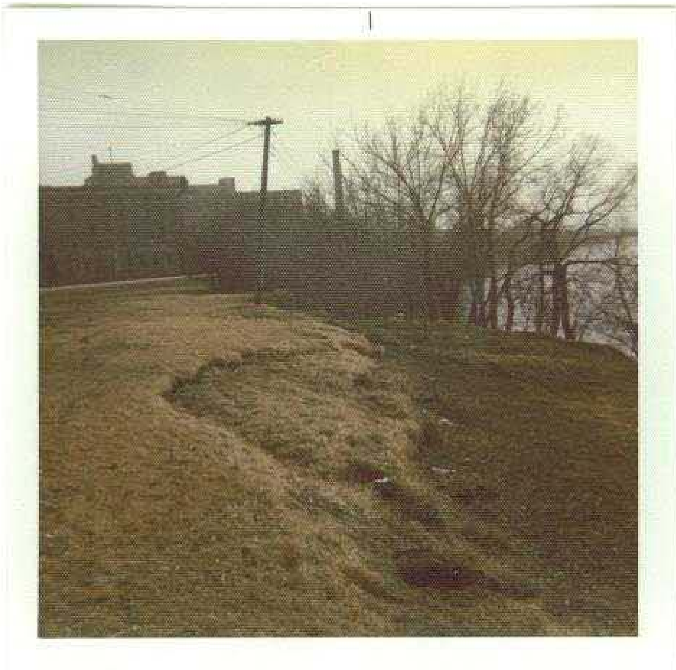
P3.1 Timber pile retard at toe of slope. Tache Avenue riverbank, 1959.



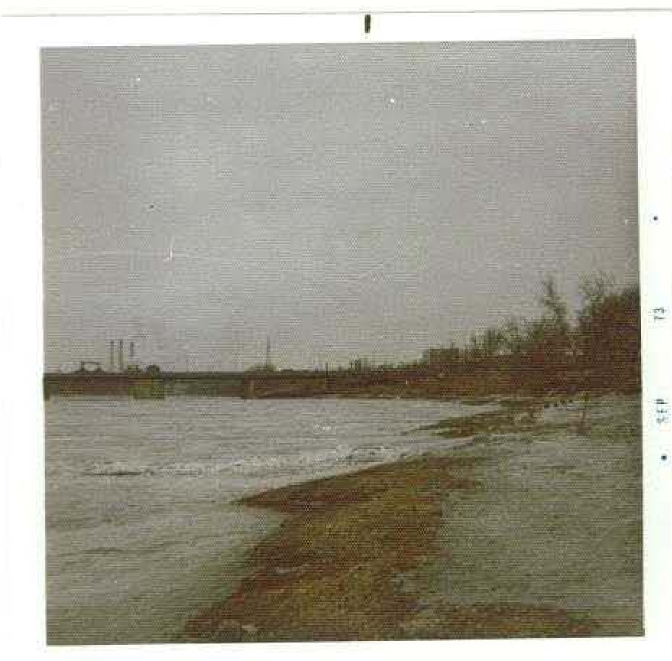
P3.2 Timber pile retard in P3.1 displaced by bank movement, Tache Avenue, 1971.

P4.1 New failure scarp, Tache Avenue site, 1959.

P4.2 Failure scarp of P4.1 with grass cover and small trees, Tache Avenue, 1971.



P5. Example of an older failure with fresh cracks and slumps appearing, Tache Avenue, 1971.



P6. Example of an overthrust of soil at the toe of the failure, Tache Avenue, 1971.

P7. Slope flattening to improve stability with riprap at the toe for erosion protection, Assiniboine River, 1972.

P8. Slope flattening to improve stability with a sheet piling retard for protection against erosion, Red River, 1972.

P9. Riprap protection of slope to above flood stage. Note failure of bank through riprap at right, Assiniboine River, 1972.

P10. Riprap protection to just above summer water level, Omands Creek, 1972.

P11. Riprap protection to just above summer water level. Note size and gradation of riprap. Red River, 1972.

P12 Rigid revetment for slope protection. Note failure of revetment on the right. Red River, 1972.

P13. Combination of timber piling retard and riprap slope protection, Red River, 1972.

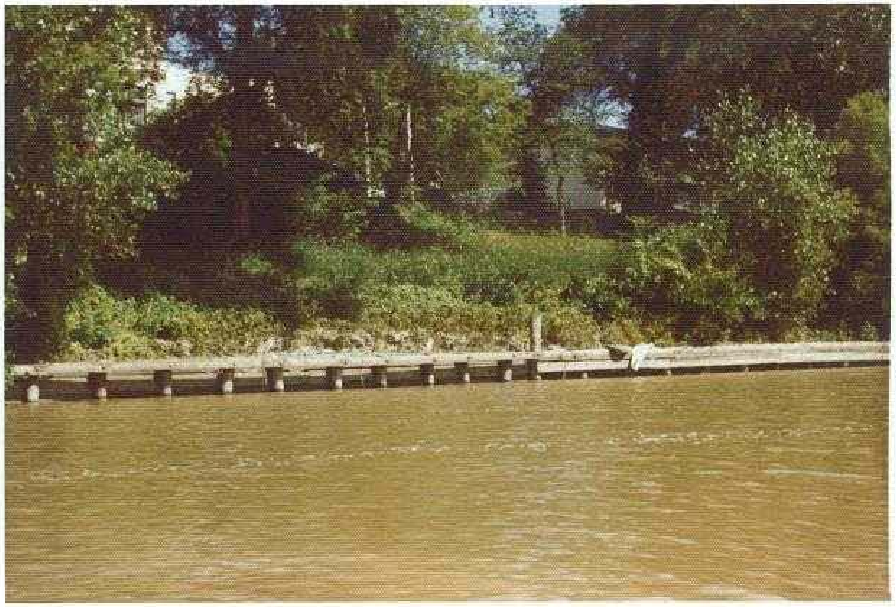
P14. Timber pile retard. Note spacing of piles. Red River, 1972.

P15. Timber pile retard, single pile spacing. Note condition of bank behind retard. Red River, 1972.

P16. Timber pile retard, piles closely spaced. Red River, 1972.

P17. Timber pile retard, piles closely spaced. Note condition of piles above summer water level and failure of bank behind damaged portion of retard. Red River, 1972.

P18. Timber pile retard, closely spaced piles. Note overturning of piles due to bank failure. Red River, 1972.



P19. Timber pile retard with horizontal planking. Assiniboine River, 1972.

P20. Timber pile retard with horizontal planking. Note stone retaining wall well up the slope. Red River, 1972.

P21. Timber pile retard with horizontal planking. Note established growth of shrubs and small trees behind retard. Red River, 1972.

P22.1 Bank failure in backfilled portion of an old channel. Omands Creek, 1970.

P22.2 Bank in P22.1 reshaped with a clay berm and riprap for erosion protection at the toe. Two horizontal trench drains were also installed.

PIEZOMETER DATA -----SITE 1

DATE	1	2	3	4	5	6	7	8	9	10
Nov. 26/69	56.8*	0	0	0	37.6	37.6	36.3	51.7	39.5	40.3
Dec. 20/69	54.3	0	0	45.8	35.5	34.1	39.5	48.0	37.9	39.6
Jan. 10/70	54.5	0	56.4	55.7	37.3	32.1	37.9	33.8	37.9	38.0
24/70	54.3	-	51.4	58.0	38.0	31.4	35.8	0	37.4	36.2
Feb. 21/70	43.9	42.7	49.1	54.1	37.6	24.5	33.5	0	34.2	33.9
28/70	0	42.7	48.4	52.1	37.3	22.6	32.8	0	34.9	33.0
Mar. 4/70	40.7	42.5	46.1	49.7	35.7	23.1	32.8	0	31.9	30.7
14/70	41.9	39.5	-	49.5	37.6	0	33.5	0	35.8	32.7
21/70	43.7	0	49.4	36.9	34.3	20.8	33.5	0	36.1	32.1
28/70	48.6	0	-	-	-	-	-	-	-	31.6
Apr. 11/70	51.8	0	-	23.5	36.6	26.8	32.8	0	36.1	30.9
18/70	42.8	0	0	27.4	37.3	27.9	32.8	0	38.6	30.9
25/70	0	0	0	33.7	41.5	31.4	34.4	47.3	40.0	34.6
May 2/70	0	0	0	35.3	30.6	31.8	34.2	50.0	42.3	33.2
9/70	0	0	0	35.5	32.2	31.4	34.7	49.8	40.7	34.6
16/70	49.2	0	0	36.0	39.9	31.8	35.4	49.8	40.4	33.9
22/70	49.7	41.1	35.5	35.3	40.3	31.6	35.4	49.8	40.4	33.7
29/70	50.4	40.6	36.0	35.3	39.6	30.9	34.9	48.5	39.1	32.7
Jun. 12/70	55.6	39.7	37.4	35.3	40.1	31.1	32.2	46.8	37.9	32.3
19/70	-	39.9	38.3	24.4	39.4	30.5	28.0	45.7	37.7	30.7
26/70	42.3	44.1	37.9	33.7	38.9	30.7	28.4	42.9	37.9	30.0
Jul. 3/70	44.6	39.9	44.5	33.4	38.9	30.0	27.3	43.4	37.2	29.1
17/70	48.6	42.5	44.5	33.0	37.6	30.9	25.5	39.5	37.4	28.1
24/70	49.5	42.2	39.2	32.5	37.3	30.0	25.0	38.6	36.8	28.3
31/70	48.6	41.5	39.2	31.8	37.0	31.1	24.1	38.8	36.8	27.9
Aug. 7/70	48.6	42.0	39.2	31.3	36.6	31.1	23.9	38.8	36.8	26.8
17/70	0	42.7	39.0	31.3	36.6	30.0	22.9	37.4	36.8	26.3
Sep. 8/70	41.9	42.2	39.7	30.2	36.4	30.0	23.6	36.9	36.8	27.0
18/70	47.4	42.2	39.2	30.2	36.6	29.8	25.7	-	36.8	27.0
23/70	48.1	42.2	41.3	30.4	36.2	29.5	26.9	35.8	36.8	28.1
Oct. 17/70	47.2	-	-	30.4	36.4	29.1	28.4	35.8	36.3	29.1
25/70	46.9	40.0	-	30.7	36.9	30.9	28.2	35.8	36.8	29.1
31/70	47.2	-	-	30.2	36.6	28.8	28.2	35.1	36.1	28.4
Nov. 7/70	46.9	-	-	29.5	36.6	27.9	28.2	34.4	35.2	28.4
13/70	-	-	-	29.0	35.5	27.5	27.1	31.4	34.7	28.2
20/70	-	-	-	29.0	34.6	27.0	27.1	-	34.7	28.2
27/70	-	-	-	26.7	33.2	26.5	25.9	-	34.5	28.2
Dec. 7/70	-	-	-	-	34.3	28.4	28.0	-	36.5	30.0
31/70	48.1	-	-	27.7	33.0	25.6	28.7	-	31.9	26.1
Jan. 18/71	47.5	-	-	29.5	32.5	25.4	28.5	-	35.6	28.8
Feb. 3/71	47.5	-	-	29.5	32.5	25.4	28.0	-	35.6	28.4
20/71	47.1	-	-	28.8	32.9	25.8	28.0	-	35.6	28.4
Mar. 10/71	48.0	-	-	30.0	33.4	28.9	28.5	-	35.6	29.3
21/71	47.1	-	-	30.2	33.2	29.3	29.2	-	29.8	28.8
31/71	48.1	-	-	30.2	33.0	28.9	29.2	-	34.5	28.8
Apr. 12/71	46.8	-	-	27.4	33.7	24.5	32.4	-	37.3	30.0
23/71	46.8	-	-	29.3	37.1	30.5	31.5	41.8	37.9	30.0
May 4/71	47.3	-	-	29.5	36.6	30.5	32.6	40.4	35.6	30.0
14/71	47.3	-	-	30.0	36.6	30.7	30.8	36.9	36.1	29.8
24/71	46.6	-	-	30.7	36.6	30.2	31.2	34.6	35.4	29.6

* Datum 700.0

PIEZOMETER DATA ----- SITE 1 (cont'd)

DATE		1	2	3	4	5	6	7	8	9	10
Jun.	3/71	46.8	-	-	31.4	36.6	30.9	31.2	-	35.1	30.3
	18/71	-	-	-	30.4	-	30.7	26.2	-	34.7	28.6
Jul.	7/71	-	-	-	30.0	-	30.7	-	-	34.5	26.6
	22/71	-	-	-	28.8	-	30.0	-	-	33.6	24.5
Aug.	3/71	-	-	-	29.0	-	30.2	22.2	-	34.5	24.7
	26/71	-	-	-	27.9	-	29.8	-	-	28.9	22.6
Sep.	10/71	-	-	-	27.7	-	29.3	-	-	29.2	22.2
	20/71	-	-	-	27.2	-	29.3	23.9	-	34.9	22.9
Oct.	19/71	-	-	-	27.2	-	28.9	26.2	-	34.2	24.7
	29/71	-	-	-	26.7	-	28.4	26.4	-	32.8	25.5
Nov.	9/71	-	-	-	26.7	-	28.2	26.4	-	33.3	26.3
	19/71	-	-	-	27.4	-	27.9	26.9	-	33.3	26.5
	29/71	-	-	-	28.1	31.8	27.7	26.9	-	33.5	26.7
Dec.	10/71	-	-	-	28.6	33.2	28.9	29.2	-	34.7	28.1

PIEZOMETER DATA ----- SITE 2

DATE	1	2	3	4	5	6	7	8	9	10
Oct. 9/70	60.8*	54.9	49.1	64.6	49.6	73.0	47.7	65.3		
17/70	45.4	54.6	35.8	55.9	26.1	67.8	-	57.7		
25/70	40.6	55.5	39.9	55.7	27.5	63.2	-	52.7		
31/70	36.0	49.8	40.1	55.7	26.8	58.2	-	42.3		
Nov. 7/70		60.3	38.5	57.3	26.8	54.5	26.3	58.6		
13/70		59.6	38.5	58.2	25.6	52.2	26.3	57.3		
20/70		57.1	38.5	59.4	25.4	51.5	26.3	56.8		
27/70		50.0	36.2	57.5	23.8	49.3	24.5	55.0		
Dec. 11/70		-	33.6	59.4	21.9	47.6	24.5	42.9		
31/70		-	34.1	59.6	21.5	46.9	24.0	42.8		
Jan. 18/71			29.7	55.9	21.5	41.4	22.0	41.7		
Feb. 3/71			28.4	53.6	21.5	-	15.3	41.0		
20/71			26.6	52.3	21.5	-	22.0	41.0		
Mar. 10/71			25.0	50.2	21.7	-	25.2	35.2		
21/71			29.5	49.3	21.0	-	25.0	39.5		
31/71			24.2	49.0	21.2	39.1	25.0	36.2		
Apr. 12/71			24.0	47.9	21.2	36.8	41.1	41.7		
23/71			24.0	47.9	22.0	39.8	41.3	42.8		
May 4/71			23.6	47.6	21.9	37.9	34.9	42.4		
14/71			23.3	46.7	21.9	-	43.6	42.2		
24/71			23.1	45.9	22.6		44.4	41.7		
Jun. 3/71			24.2	46.3	22.6		43.7	41.2		
Jul. 7/71			22.6	46.5	19.2		43.7	40.3		
22/71			20.8	46.1	15.9			34.6		
Aug. 6 /71			19.9	46.7	15.7		24.3	40.1		
26/71			19.2	46.3	13.2			40.5		
Sep. 10/71			17.1	46.1		39.1		43.3		
28/71			16.9	45.9		40.2		41.5		
Oct. 19/71			18.1	45.3		40.4		40.8		
29/71			17.1	45.8		39.8		40.5	47.7	65.2
Nov. 9/71			18.1	46.7		40.2		40.5	37.9	68.4
19/71			19.2	47.9		39.8		41.0	51.0	75.8
29/71			19.2	47.9		39.1		40.5	71.5	86.1
Dec.10 /71			20.4	49.1	15.7	40.9		41.2	76.2	93.7

* Datum 700.0

APPENDIX B. DAILY PRECIPITATION DATA FOR THE WINNIPEG AREA

ANNUAL METEOROLOGICAL SUMMARY
for
WINNIPEG, MANITOBA

DAILY PRECIPITATION - 1969

DATE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1	Tr.	.03	Tr.	Tr.	.07	.39	Tr.			.24	Tr.	
2	.06	Tr.	Tr.						.01			
3	Tr.	Tr.		Tr.		.22	.18	.04	.01	.41		
4	Tr.	Tr.	.01	Tr.	.02		.37		1.05	Tr.		
5	.08		Tr.		.57			.08	.06			
6	.05	Tr.	Tr.		.15	Tr.	.09	.23	.02	Tr.		.08
7	.05	Tr.	.18		.01		.08	.35	.08	.15		.21
8	.04	Tr.	.10	Tr.	.04		.11				Tr.	.19
9	Tr.	Tr.	.05	Tr.	Tr.		.11				Tr.	.03
10	Tr.	.04			Tr.	.03	.07		.01	Tr.	Tr.	.02
11	Tr.	.01				Tr.		Tr.		Tr.	.05	.02
12	.03	Tr.		Tr.	.14	.03		.12			.02	.02
13	.03	Tr.			.10		.08	.57		Tr.	Tr.	Tr.
14	Tr.	.05			Tr.		.06			.02	.04	Tr.
15	.35	Tr.		Tr.	Tr.	Tr.	.01		.04	.06	Tr.	Tr.
16	.24	Tr.			Tr.	Tr.			Tr.		Tr.	Tr.
17	Tr.	Tr.				Tr.		.18				
18	.02	.05			.17	.07	.33	Tr.			Tr.	Tr.
19	.06	Tr.			Tr.	Tr.	.10				Tr.	Tr.
20	.01				Tr.				Tr.	Tr.		Tr.
21	.14	.04					Tr.		.53	.10		.02
22	.30	.04	Tr.		Tr.		Tr.		Tr.		.05	Tr.
23	.03	.04			Tr.		.04		Tr.	Tr.	.13	.01
24	.05	.01							.38	.01	Tr.	.06
25	Tr.			.22		.59			.01	.07	.02	.02
26	Tr.		Tr.	.48	.48	.80	1.92	.08	Tr.		.06	Tr.
27	.20	Tr.	.04		Tr.	1.26	.03		Tr.		Tr.	Tr.
28	Tr.	.03				.16			.03			.02
29	.13					.58		.59	Tr.	Tr.	Tr.	.01
30	Tr.			.30	.09		.41		.17	.01		Tr.
31	Tr.		.01		1.18			.01		Tr.		.03
Total	1.87	.34	.39	1.00	3.02	4.13	3.99	2.25	2.40	1.07	.37	.74

ANNUAL METEOROLOGICAL SUMMARY
for
WINNIPEG, MANITOBA.

DAILY PRECIPITATION - 1970.

DATE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1	.07	.08			Tr.				Tr.		.42	.41
2	.03	Tr.	.34	.08	Tr.	.01	.17		.16	Tr.	.08	
3	.01	.02	.60	.04			.02		Tr.			.05
4	.01	.04			.02							.04
5	Tr.	Tr.	Tr.				.08		.20			.01
6	.01						.09			Tr.	Tr.	Tr.
7	.01	Tr.	Tr.	.01			Tr.		.28	Tr.	Tr.	Tr.
8	Tr.	Tr.	Tr.	.07	Tr.	.21			.06			.12
9	Tr.				.05	.17	Tr.		.22	.03		
10	.09	.06			.03	.40			Tr.	.04		Tr.
11	Tr.	Tr.	Tr.		.29	.07	.04		.44			
12		.01	Tr.		.01	.11	.21		.08	Tr.	Tr.	.02
13	.05		Tr.		.02	Tr.	.01		Tr.			.01
14	.27	.01			.37		1.28			Tr.		
15	.12			.44	.21	Tr.		.08	.31			Tr.
16	Tr.	.11		.12		.18					.02	Tr.
17		.22				Tr.	Tr.	.02			.15	.04
18	Tr.	Tr.					.18	Tr.			.02	Tr.
19		.01	Tr.	.88		Tr.					Tr.	.04
20	Tr.	.02		.53	.13				2.01		Tr.	
21	Tr.			.15	Tr.			.65	.53		.06	.04
22	Tr.		.01	Tr.			.06		.01		.02	.08
23	.03			.02		.12	.15		Tr.		Tr.	Tr.
24	Tr.	Tr.			1.27		.17		Tr.	Tr.	.16	.03
25	.07	.01	Tr.		.54	.36	.12		.06	.17	Tr.	Tr.
26	Tr.	Tr.	.04		Tr.		Tr.	Tr.	.05	.08		.03
27	.06			.02	.03	.08				.42		
28	.05			.20	.15	Tr.	Tr.			.05	.09	Tr.
29	.02		.09	.01	.10		.16	2.20		.01		
30			Tr.	Tr.	.13	.28				Tr.	.03	Tr.
31	Tr.		Tr.		.06					.25		Tr.
Total	.90	.59	1.08	2.57	3.41	1.99	2.74	2.95	4.41	1.05	1.05	.92

APPENDIX C: RED RIVER WATER LEVELS AT WINNIPEG (REDWOOD BRIDGE)

RED RIVER AT WINNIPEG - STATION NO. 05OJ001

DAILY WATER LEVEL IN FEET FOR 1970

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
727.61	---	---	728.24	743.35	738.14	737.72	---	733.71	733.75	---	727.01	1
727.61	728.37	728.20	728.24	---	738.19	736.72	---	733.71	733.65	733.58	727.21	2
---	728.35	728.20	728.24	---	738.49	736.22	733.31	733.69	---	733.35	727.42	3
---	728.35	728.22	---	742.09	738.89	---	733.52	733.68	---	733.01	727.41	4
727.60	728.35	728.22	---	742.11	739.13	---	733.42	---	734.17	732.75	---	5
727.87	728.35	728.22	728.26	742.15	---	734.72	733.35	---	734.44	732.48	---	6
727.93	---	---	728.27	742.18	---	734.29	733.74	733.88	734.05	---	727.39	7
737.93	---	---	728.30	742.89	739.14	734.01	---	733.89	733.89	---	727.39	8
727.93	728.34	728.22	728.30	---	739.31	733.89	---	734.01	733.65	731.26	727.38	9
---	728.33	728.22	728.30	---	739.77	733.71	734.15	734.01	---	731.05	727.38	10
---	728.33	728.22	---	742.44	739.14	---	734.02	734.05	---	---	727.35	11
727.93	728.33	728.20	---	742.05	738.43	---	734.01	---	733.49	729.95	---	12
727.97	728.33	728.20	730.06	741.85	---	733.35	733.88	---	733.58	729.19	---	13
727.98	---	---	732.18	741.65	---	733.05	733.54	733.95	733.69	---	727.35	14
727.93	---	---	734.74	741.89	738.16	731.95	---	733.98	733.85	---	727.36	15
727.99	728.33	728.24	736.56	---	738.05	732.89	---	733.54	733.86	728.89	727.36	16
---	728.33	728.24	738.40	---	737.94	732.88	733.68	733.65	---	728.14	727.37	17
---	728.34	728.24	---	741.88	738.06	---	733.75	733.58	---	727.89	727.36	18
727.99	728.34	728.23	---	741.88	738.08	---	733.85	---	733.95	727.75	---	19
728.01	728.34	728.23	744.04	741.89	---	732.89	733.89	---	734.01	727.55	---	20
728.17	---	---	743.82	741.87	---	733.18	734.05	734.15	734.09	---	727.38	21
728.17	---	---	743.82	741.42	738.88	733.41	---	734.11	734.11	---	727.89	22
728.20	728.20	728.23	744.06	---	738.75	733.65	---	733.91	734.01	727.55	728.02	23
---	728.20	728.24	744.85	---	738.82	733.35	734.01	733.53	---	727.49	728.02	24
---	728.20	728.24	---	741.05	738.82	---	733.88	733.55	---	727.11	---	25
728.21	728.20	728.23	---	740.89	738.95	---	733.75	---	733.95	726.71	---	26
728.23	728.20	728.23	743.25	739.75	---	733.25	733.78	---	733.88	726.71	---	27
728.27	---	---	743.31	739.55	---	733.81	733.79	734.25	733.66	---	728.05	28
728.33		---	743.31	738.15	738.41	733.91	---	733.89	733.55	---	728.05	29
728.37		728.24	743.32	---	738.07	734.05	---	733.65	733.51	726.89	728.06	30
---		728.24		---		734.11	733.74		---		728.06	31

TYPE OF GAUGE - MANUAL
LOCATION - LAT 49 55 00 N
LONG 97 07 30 W

REGULATED

ER LEVELS ARE REFERRED TO GEODETIC SURVEY OF CANADA DATUM.

RED RIVER AT WINNIPEG - STATION NO. 05OJ001

DAILY WATER LEVEL IN FEET FOR 1971

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	---	728.19	728.24	731.31	---	734.22	733.12	---	733.85	734.06	732.16	730.38	1
2	---	728.19	728.24	732.13	---	734.26	733.62	733.79	733.85	---	732.59	730.38	2
3	---	728.19	728.23	---	734.40	733.95	---	733.78	733.86	---	732.15	730.38	3
4	728.07	728.20	728.24	---	734.39	733.63	---	733.72	---	733.63	731.24	---	4
5	728.07	728.20	728.24	731.46	734.40	---	733.64	733.72	---	733.64	731.21	---	5
6	728.08	---	---	731.70	734.37	---	733.66	733.72	733.76	733.64	---	730.39	6
7	728.08	---	---	731.80	734.85	733.45	733.62	---	733.79	733.68	---	730.39	7
8	728.09	728.22	728.24	732.56	---	733.58	733.52	---	733.81	733:70	731.25	730.39	8
9	---	728.26	728.24	734.90	---	733.69	733.54	733.70	733.70	---	731.34	730.39	9
10	---	728.24	728.21	---	734.70	733.72	---	733.72	733.71	---	731.68	730.39	10
11	728.09	728.21	728.21	---	734.59	733.76	---	733.73	---	733.66	731.75	---	11
12	728.10	728.26	728.24	742.90	734.36	---	734.02	733.74	---	733.63	731.95	---	12
13	728.12	---	---	742.56	733.94	---	734.02	733.75	733.77	733.61	---	730.36	13
14	728.13	---	---	742.10	733.64	733.80	733.95	---	733.78	733.58	---	730.36	14
15	728.16	728.24	728.35	741.76	---	733.87	734.04	---	733.78	733.57	730.26	730.35	15
16	---	728.21	728.35	741.36	---	733.99	734.01	733.82	733.78	---	730.25	730.35	16
17	---	728.22	728.11	---	733.86	734.09	---	733.99	733.79	---	730.22	730.35	17
18	728.17	728.24	728.08	---	733.66	734.10	---	734.09	---	733.64	730.22	---	18
19	728.17	728.41	728.11	740.59	733.60	---	733.65	734.10	---	733.72	730.21	---	19
20	728.18	---	---	740.56	733.48	---	732.31	734.09	733.78	733.72	---	730.38	20
21	728.18	---	---	740.46	733.94	734.84	733.39	---	733.73	733.82	---	730.38	21
22	728.18	728.35	728.11	740.32	---	734.06	733.41	---	733.71	733.81	730.29	730.38	22
23	---	728.31	727.79	740.30	---	733.50	733.45	734.00	733.63	---	730.35	730.39	23
24	---	728.24	727.79	---	734.00	733.65	---	733.96	733.56	---	730.36	730.39	24
25	728.18	728.31	728.11	---	734.02	733.80	---	733.90	---	734.98	730.38	---	25
26	728.18	728.31	728.13	737.02	734.13	---	733.59	733.88	---	734.68	730.38	---	26
27	728.19	---	---	736.26	734.16	---	733.68	733.85	733.91	734.18	---	730.39	27
28	728.19	---	---	735.89	734.20	733.79	733.75	---	734.00	733.99	---	730.39	28
29	728.19		728.90	735.19	---	733.79	733.75	---	734.01	733.48	730.38	730.38	29
30	---		729.66	735.07	---	733.72	733.81	733.85	734.06	---	730.38	730.38	30
31	---		729.69		734.22		---	733.85		---		730.38	31

TYPE OF GAUGE - MANUAL
LOCATION - LAT 49 55 00 N
LONG 97 07 30 W

REGULATED

WATER LEVELS ARE REFERRED TO GEODETIC SURVEY OF CANADA DATUM.

APPENDIX D: SLOPE INDICATOR DEFLECTION DATA, SITES 1 AND 2.

ANALYSIS OF SLOPE INDICATOR DATA

LOPE INDICATOR DATA IN BORINGS

Slope Indicator measures the inclination of the ing in an observation well at frequent intervals depth. One set of readings is taken in one ove, and an additional set is taken in the groove the opposite wall of the casing. The instrument djusted so that the inclination (i) of the casing m the vertical may be computed by the follow- equation:

$$\tan i = \frac{D_N - D_S}{2k}$$

ere

D_N = Dial Reading in North Groove

D_S = Dial Reading in South Groove

k = 2000 for Series 200-B Instrument

y change in inclination at a given depth pro- es a change in dial differences.

$$\Delta \text{Dial} = [D_N - D_S] \text{ new reading} - [D_N - D_S] \text{ initial reading}$$

change in inclination may be expressed as:

$$\Delta \tan i = \frac{\Delta \text{Dial}}{2k}$$

compute the lateral movements at each interval depth

$$\Delta M = \ell\,(\Delta \tan i)$$
$$= c\,(\Delta \text{ dial})$$

where:

ΔM = lateral movement at each interval of depth

ℓ = vertical distance between successive readings

$c = \ell/2k$

If the vertical distance between successive readings is constant, then the total lateral movement (M) of any point with respect to the bottom is:

$$M = \Sigma\, \Delta M$$
$$= c\, \Sigma\, \Delta \text{Dial}$$

For an instrument constant of k=2000, an average distance between readings of ℓ=20 inches (three readings per five-foot length),

$$c = \frac{20}{2\,(2000)} = 0.005$$

and $$M = 0.005\ \Sigma\ \Delta \text{Dial}$$

Thus the movements are computed very easily by summing up from the bottom the changes in dial differences and multiplying each sum by 0.005 (answer is in inches).

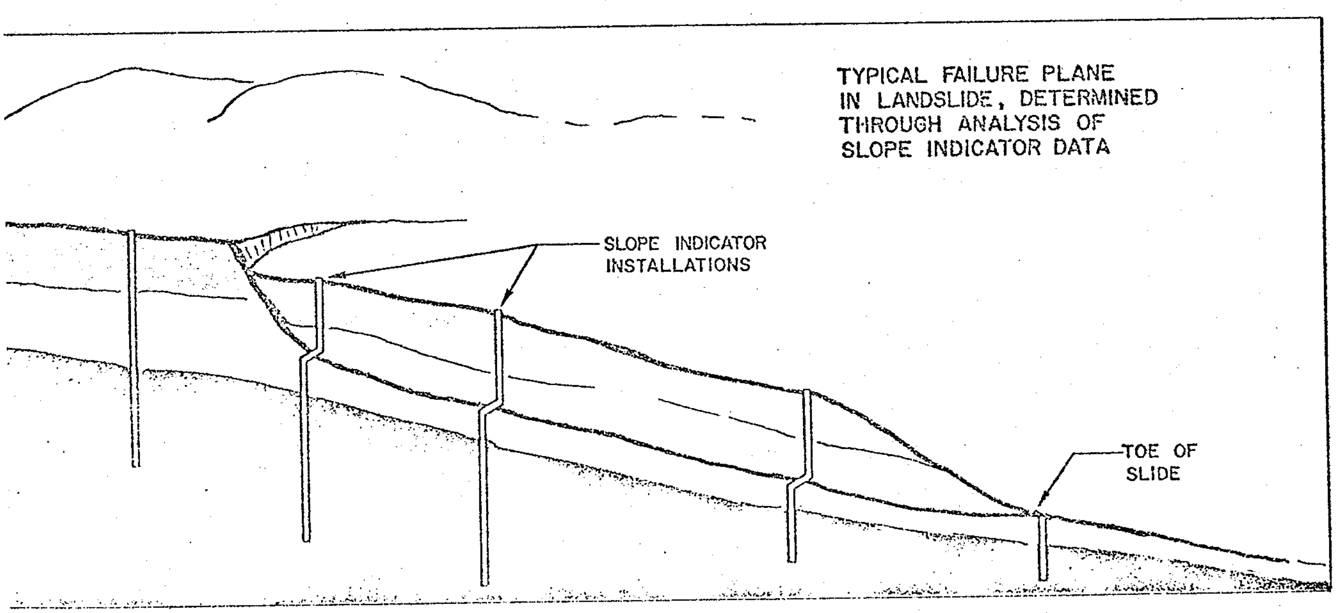


FIG. V-A TYPICAL FAILURE PLANE IN LANDSLIDE

THE SLOPE INDICATOR CO. FIELD SHEET SLOPE INDICATOR DATA SLOPE INDICATOR INSTALLATION						OBSERVATION WELL NO. SI-I				
DATE 2-1-65		INSTR 200-B				READ TGB	CALC. TGB		CHKD. CWH	SHEET 1 OF 1
DEPTH	DIFF	AZ. N+	S-	DIFF.	CHANGE	DIFF	AZ. E+	W-	DIFF.	CHANGE
1		522	546^{68}	-24			523	547^{70}	-24	
2.5		523	546^{69}	-23			522	545^{67}	-23	
4		523	547^{70}	-24			522	545^{67}	-23	
6		525	545^{70}	-20			528	540^{68}	-12	
7.5		518	549^{67}	-31			528	540^{68}	-12	
9		524	546^{70}	-22			520	547^{67}	-27	
11		524	546^{70}	-22			533	535^{68}	- 2	
12.5		526	545^{71}	-19			536	533^{69}	+ 3	
14		528	542^{70}	-14			537	530^{67}	+ 7	
16		528	542^{70}	-14			537	531^{69}	+ 6	
17.5		527	542^{69}	-15			536	533^{69}	+ 3	
19		527	543^{70}	-16			536	533^{69}	+ 3	
21		542	526^{68}	+16			532	534^{66}	- 2	
22.5		543	527^{70}	+16			534	533^{67}	+ 1	
24		544	524^{69}	+20			536	531^{67}	+ 5	
26		542	528^{70}	+14			534	532^{66}	+ 2	
27.5		541	527^{69}	+14			534	532^{66}	+ 2	
29		543	527^{70}	+16			538	531^{69}	+ 7	
31		536	531^{67}	+ 5			538	528^{66}	+10	
32.5		534	535^{69}	- 1			539	528^{67}	+11	
34		532	536^{68}	- 4			539	528^{67}	+11	
36		529	541^{70}	-12			538	529^{67}	+ 9	
37.5		528	542^{70}	-14			537	531^{68}	+ 6	
39		529	540^{69}	-11			537	530^{67}	+ 7	
41		527	540^{67}	-13			528	537^{65}	- 9	
42.5		527	541^{68}	-14			528	538^{66}	-10	
44		526	543^{69}	-17			524	542^{66}	-18	
46		523	545^{68}	-22			524	544^{68}	-20	
47.5		521	546^{67}	-25			523	545^{68}	-22	
49		521	547^{69}	-26			522	545^{67}	-23	

FIG. V-B TYPICAL FIELD DATA SHEET FOR BORINGS

The movement is usually calculated only once per five-foot section, except in zones of active movement.

The dial differences taken from the Field Data Sheet (Fig. V-B) are transcribed directly onto the Summary Sheet (Fig. V-C). As additional sets are obtained, the new dial differences are likewise transcribed onto the Summary Sheet. If the dial changes are positive, the upper portion of the casing is being displaced to the North (or positive groove) with respect to the bottom.

Plots of dial changes and of deflections are prepared as shown in (Fig. V-D). The deflection curve should be drawn as a smooth, free hand curve. It should not be inferred from this method of plotting that the initial casing installation is vertical. For the purpose of measuring subsequent movement the initial casing inclination is not plotted.

If a zone of movement is determined from the analysis of the data, it is recommended that field readings be obtained at six-inch intervals within the zone. This will better define the limits of the zone and the maximum movement.

THE SLOPE INDICATOR CO.
SEATTLE, WASHINGTON

SLOPE INDICATOR DATA

PROJECT SLOPE INDICATOR

JOB NO. S-I Sht. 1 of 1
OBSERVATION WELL NO. SI-I
SLOT BEARING N-S

DEPTH	INITIAL 2-1-65	DATE 3-1-65 INST 200-B K= 2000 C= 0.005				DATE 4-1-65 INST 200-B K= 2000 C= 0.005				DATE INST. K= C=				DATE INST. K= C=				DATE INST. K= C=				DATE INST. K= C=			
		DIFF	CHANGE	ΣCHANGE	DEFL	DIFF.	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF	CHANGE	ΣCHANGE	DEFL	DIFF.	CHANGE	ΣCHANGE	DEFL.	DIFF.	CHANGE	ΣCHANGE	DEFL.
1	-24	-23	+1	+200		-22	+2	260																	
2.5	-23	-19	+4	+199	1.00	-22	+1	258	1.29																
4	-24	-21	+3	+195		-21	+3	257																	
6	-20	-18	+2	+192		-17	+3	254																	
7.5	-31	-25	+6	+190	0.95	-25	+6	251	1.25																
9	-22	-15	+7	+184		-10	+12	245																	
11	-22	-8	+14	+177	0.88	-2	+20	233	1.16																
2.5	-19	-2	+17	+163	0.81	+11	+30	213	1.07																
14	-14	-4	+10	+146	0.75	+3	+17	183	0.92																
16	-14	-10	+4	+136		-4	+10	166																	
.5	-15	-13	+2	+132	0.66	-19	-4	156	0.78																
19	-16	-14	+2	+130		-19	-3	160																	
21	+16	+15	-1	+128		+15	-1	163																	
.5	+16	+19	+3	+129	0.64	+15	-1	164	0.82																
24	+20	+18	-2	+126		+21	+1	165																	
26	+14	+16	+2	+128		+15	+1	164																	
.5	+14	+15	+1	+126	0.63	+16	+2	163	0.82																
9	+16	+13	-3	+125		+15	-1	161																	
1	+5	+7	+2	+123		+8	+3	162																	
.5	-1	+6	+7	+125	0.63	+3	+4	159	0.80																
4	-4	+12	+16	+119		+18	+22	155																	
6	-12	+14	+26	+103	0.52	+24	+36	133	0.67																
5	-14	+22	+36	+77	0.39	+40	+54	97	0.49																
9	-11	+10	+21	+41	0.20	+15	+26	43	0.22																
1	-13	+1	+14	+20		+3	+16	17																	
3	-14	-12	+2	+6	0.03	-13	+1	1	0																
	-17	-16	+1	+4		-17	+0	0																	
	-22	-19	+3	+3		-21	+1	0																	
	-25	-26	-1	0	0	-26	-1	-1	0																
	-26	-25	+1	+1		-26	+0	0																	

FIG. V-C TYPICAL SUMMARY SHEET FOR BORINGS

SLOPE INDICATOR DATA -- TA-1

DATE	8.12.69	10.12.69	24.12.69	31.12.69	23. 1.70	6. 2.70	23. 2.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
2.2	-0.342	-0.732	-1.056	-1.128	-1.584	-1.752	-1.992
4.2	-0.360	-0.774	-1.134	-1.194	-1.674	-1.842	-2.070
6.2	-0.384	-0.816	-1.212	-1.266	-1.770	-1.938	-2.154
8.2	-0.414	-0.852	-1.260	-1.320	-1.830	-2.004	-2.220
10.2	-0.438	-0.888	-1.308	-1.380	-1.914	-2.088	-2.316
12.2	-0.462	-0.930	-1.362	-1.452	-2.004	-2.202	-2.436
14.2	-0.480	-0.984	-1.440	-1.542	-2.124	-2.346	-2.592
16.2	-0.504	-1.044	-1.536	-1.638	-2.304	-2.538	-2.802
18.2	-0.498	-1.032	-1.518	-1.620	-2.256	-2.502	-2.760
20.2	-0.456	-0.942	-1.380	-1.458	-2.016	-2.250	-2.490
22.2	-0.318	-0.660	-0.984	-1.008	-1.392	-1.542	-1.716
24.2	-0.132	-0.288	-0.456	-0.420	-0.600	-0.660	-0.738
26.2	0.018	0.036	0.0	0.078	0.060	0.072	0.072
28.2	0.048	0.078	0.060	0.138	0.138	0.150	0.162
30.2	0.036	0.066	0.048	0.108	0.108	0.114	0.132
32.2	0.024	0.042	0.030	0.066	0.066	0.060	0.078
34.2	0.006	0.012	0.006	0.018	0.012	0.018	0.024
DATE	18. 3.70	9. 4.70	16. 4.70	31. 7.70	14. 8.70	28. 8.70	14. 9.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
2.2	-2.172	-2.952	-3.096	-3.120	-3.072	-3.054	-3.048
4.2	-2.250	-3.060	-3.228	-3.270	-3.246	-3.240	-3.234
6.2	-2.322	-3.168	-3.360	-3.426	-3.426	-3.438	-3.426
8.2	-2.394	-3.276	-3.474	-3.564	-3.564	-3.570	-3.570
10.2	-2.490	-3.402	-3.606	-3.708	-3.708	-3.714	-3.720
12.2	-2.616	-3.558	-3.774	-3.882	-3.876	-3.882	-3.894
14.2	-2.802	-3.780	-4.014	-4.122	-4.116	-4.110	-4.128
16.2	-3.024	-4.068	-4.320	-4.422	-4.404	-4.404	-4.416
18.2	-2.982	-4.044	-4.296	-4.350	-4.326	-4.326	-4.338
20.2	-2.688	-3.654	-3.876	-3.882	-3.864	-3.870	-3.882
22.2	-1.836	-2.472	-2.610	-2.634	-2.610	-2.628	-2.640
24.2	-0.780	-1.026	-1.056	-1.092	-1.074	-1.092	-1.122
26.2	0.096	0.156	0.222	0.234	0.234	0.222	0.186
28.2	0.198	0.264	0.324	0.342	0.318	0.324	0.288
30.2	0.156	0.192	0.240	0.276	0.246	0.252	0.234
32.2	0.090	0.108	0.138	0.180	0.156	0.162	0.150
34.2	0.030	0.036	0.048	0.078	0.060	0.066	0.060

D4

DATE	25. 9.70	13.10.70	29.10.70	12.11.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
2.2	-3.372	-3.390	-3.396	-3.936
4.2	-3.474	-3.510	-3.510	-4.050
6.2	-3.624	-3.624	-3.624	-4.170
8.2	-3.714	-3.744	-3.738	-4.290
10.2	-3.846	-3.876	-3.864	-4.428
12.2	-4.008	-4.038	-4.020	-4.602
14.2	-4.224	-4.254	-4.242	-4.830
16.2	-4.494	-4.518	-4.524	-5.118
18.2	-4.416	-4.434	-4.434	-5.046
20.2	-3.942	-3.954	-3.966	-4.530
22.2	-2.640	-2.682	-2.682	-3.066
24.2	-1.098	-1.134	-1.110	-1.284
26.2	0.234	0.198	0.234	0.216
28.2	0.336	0.312	0.336	0.330
30.2	0.270	0.240	0.270	0.264
32.2	0.174	0.144	0.174	0.162
34.2	0.072	0.066	0.072	0.072

SLOPE INDICATOR DATA -- TA-2

DATE	8.12.69	16.12.69	24.12.69	31.12.69	31. 1.70	6. 2.70	23. 2.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-0.516	-1.368	-2.064	-2.292	-3.144	-3.390	-3.822
3.0	-0.402	-1.116	-1.680	-1.866	-2.804	-2.802	-3.240
5.0	-0.300	-0.870	-1.308	-1.452	-2.064	-2.220	-2.634
7.0	-0.204	-0.588	-0.900	-0.996	-1.448	-1.554	-1.932
9.0	-0.043	-0.174	-0.258	-0.276	-0.444	-0.444	-0.696
11.0	0.036	0.048	0.096	0.120	0.114	0.150	-0.024
13.0	0.060	0.114	0.210	0.240	0.282	0.330	0.180
15.0	0.048	0.090	0.186	0.216	0.248	0.288	0.138
17.0	0.024	0.042	0.120	0.150	0.150	0.180	0.024
19.0	0.018	0.012	0.084	0.114	0.096	0.108	-0.042
21.0	0.018	-0.012	0.060	0.090	0.060	0.072	-0.072
23.0	0.012	-0.012	0.042	0.072	0.042	0.054	-0.078
25.0	0.012	-0.018	0.018	0.054	0.030	0.030	-0.078
27.0	0.024	-0.018	0.012	0.054	0.036	0.036	-0.060
29.0	0.036	-0.006	0.036	0.078	0.054	0.060	-0.036
31.0	0.042	0.006	0.054	0.096	0.078	0.078	-0.012
33.0	0.048	0.012	0.054	0.102	0.084	0.084	0.0
35.0	0.048	0.012	0.048	0.102	0.078	0.078	0.0
37.0	0.048	0.0	0.042	0.090	0.060	0.066	-0.006
39.0	0.030	-0.006	0.024	0.060	0.042	0.036	-0.018
41.0	0.018	-0.006	0.012	0.042	0.012	0.024	-0.024
43.0	0.0	-0.006	0.0	0.012	-0.006	0.006	-0.024
45.0	-0.006	-0.006	-0.006	-0.006	-0.006	-0.006	-0.018

DATE	18. 3.70	7. 4.70	16. 4.70	31. 7.70	14. 8.70	28. 8.70	14. 9.70
DEPTH (ft)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-3.818	-5.190	-5.946	-7.218	-7.098	-7.128	-7.470
3.0	-3.280	-4.206	-5.946	-5.478	-5.370	-5.394	-5.598
5.0	-2.712	-3.216	-3.372	-3.762	-3.648	-3.702	-3.738
7.0	-1.986	-2.172	-2.064	-2.046	-1.932	-1.980	-1.872
9.0	-0.665	-0.804	-0.720	-0.888	-0.774	-0.828	-0.606
11.0	0.048	-0.042	0.048	-0.210	-0.084	-0.150	-0.102
13.0	0.246	0.198	0.306	0.060	0.180	0.120	0.354
15.0	0.192	0.162	0.264	0.084	0.198	0.138	0.354
17.0	0.066	0.024	0.126	0.024	0.120	0.060	0.270
19.0	0.006	-0.048	0.066	-0.048	0.030	-0.030	0.168
21.0	-0.012	-0.090	0.024	-0.108	-0.048	-0.078	0.114
23.0	-0.018	-0.102	0.006	-0.120	-0.066	-0.096	0.078
25.0	-0.030	-0.108	0.0	-0.114	-0.078	-0.102	0.060
27.0	-0.018	-0.096	0.018	-0.096	-0.066	-0.084	0.066
29.0	-0.006	-0.066	0.048	-0.060	-0.048	-0.060	0.084
31.0	0.012	-0.042	0.084	-0.036	-0.030	-0/036	0.090
33.0	0.018	-0.036	0.102	-0.024	-0.012	-0.024	0.096
35.0	0.018	-0.042	0.102	-0.018	-0.012	-0.024	0.084
37.0	0.0	-0.054	0.090	-0.030	-0.024	-0.048	0.054
39.0	-0.012	-0.054	0.078	-0.048	-0.042	-0.060	0.018
41.0	-0.018	-0.048	0.054	-0.054	-0.030	-0.060	0.0
43.0	-0.024	-0.042	0.024	-0.047	-0.024	-0.048	-0.006
45.0	-0.030	-0.018	0.0	-0.024	-0.030	-0.024	-0.006

SLOPE INDICATOR DATA -- TA-2(cont'd)

DATE	25. 9.70	13.10.70	29.10.70	12.11.70	19.11.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-7.980	-8.058	-8.004	-8.880	-11.100
3.0	-6.060	-6.102	-6.054	-6.744	-8.874
5.0	-4.152	-4.152	-4.116	-4.620	-6.594
7.0	-2.244	-2.190	-2.172	-2.494	-4.170
9.0	-0.930	-0.906	-0.864	-0.946	-1.374
11.0	-0.186	-0.174	-0.138	-0.120	-0.030
13.0	0.084	0.090	0.144	0.132	0.276
15.0	0.096	0.096	0.150	0.114	0.180
17.0	0.018	0.018	0.072	0.024	0.012
19.0	-0.054	-0.054	-0.012	-0.060	-0.060
21.0	-0.090	-0.096	-0.060	-0.108	-0.102
23.0	-0.108	-0.114	-0.078	-0.126	-0.120
25.0	-0.114	-0.120	-0.072	-0.126	-0.126
27.0	-0.096	-0.114	-0.060	-0.108	-0.114
29.0	-0.066	-0.078	-0.036	-0.066	-0.066
31.0	-0.036	-0.042	-0.012	-0.030	-0.030
33.0	-0.012	-0.030	-0.006	-0.012	-0.006
35.0	-0.012	-0.024	0.0	-0.004	0.0
37.0	-0.024	-0.024	-0.018	-0.012	-0.006
39.0	-0.030	-0.036	-0.036	-0.030	-0.024
41.0	-0.018	-0.048	-0.042	-0.030	-0.024
43.0	-0.012	-0.042	-0.036	-0.024	-0.024
45.0	-0.012	-0.030	-0.024	-0.018	-0.018

SLOPE INDICATOR DATA --TA-4

DATE	29.10.70	12.11.70	19.11.70	26.11.70	3.12.70	10.12.70	30.12.70
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-0.096	-0.126	-0.228	-0.168	-0.174	-0.150	-0.438
3.0	-0.078	-0.072	-0.174	-0.120	-0.126	-0.196	-0.336
5.0	-0.048	-0.024	-0.120	-0.072	-0.078	-0.036	-0.222
7.0	-0.024	0.018	-0.072	-0.036	-0.024	0.030	-0.096
9.0	-0.006	0.048	-0.030	0.0	0.018	0.084	0.012
11.0	-0.006	0.066	-0.006	0.024	0.042	0.108	0.078
13.0	0.006	0.072	0.0	0.030	0.048	0.114	0.084
15.0	0.012	0.066	0.0	0.030	0.048	0.108	0.078
17.0	0.006	0.066	0.006	0.036	0.042	0.096	0.078
19.0	0.006	0.066	0.012	0.042	-0.048	0.090	0.078
21.0	-0.006	0.042	0.012	0.030	0.036	0.066	0.066
23.0	-0.006	0.030	0.006	0.030	0.030	0.066	0.066
25.0	-0.012	0.030	0.006	0.030	0.030	0.072	0.072
27.0	0.006	0.042	0.024	0.042	0.042	0.078	0.084
29.0	-0.006	0.036	0.030	0.048	0.048	0.078	0.090
31.0	-0.012	0.024	0.024	0.036	0.036	0.060	0.072
33.0	-0.006	0.012	0.012	0.018	0.024	0.048	0.060
35.0	0.0	0.006	0.006	0.006	0.018	0.042	0.054
37.0	-0.006	0.0	0.0	-0.006	0.006	0.030	0.036
39.0	-0.006	0.0	0.0	-0.006	0.006	0.018	0.024

DATE	22. 1.71	11. 2.71	24. 2.71	12. 3.71	26. 3.71	8. 4.71	26. 4.71
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in,)	DEFL. (in.)	DEFL. (in.)
1.0	-0.594	-0.684	-1.068	-1.134	-1.164	-1.062	-1.290
3.0	-0.462	-0.522	-0.882	-0.960	-0.990	-0.876	-1.020
5.0	-0.306	-0.354	-0.690	-0.768	-0.804	-0.702	-0.750
7.0	-0.108	-0.108	-0.408	-0.486	-0.510	-0.420	-0.480
9.0	0.054	0.108	-0.138	-0.186	-0.198	-0.114	-0.198
11.0	0.150	0.228	0.048	0.036	0.054	0.132	0.048
13.0	0.156	0.222	0.048	0.036	0.054	0.132	0.054
15.0	0.156	0.204	0.042	0.030	0.048	0.126	0.048
17.0	0.144	0.186	0.042	0.024	0.036	0.114	0.036
19.0	0.126	0.168	0.042	0.018	0.036	0.108	0.030
21.0	0.096	0.138	0.024	0.0	0.012	0.084	0.012
23.0	0.096	0.144	0.036	0.012	0.024	0.090	0.012
25.0	0.102	0.138	0.042	0.024	0.036	0.096	0.012
27.0	0.114	0.144	0.060	0.048	0.060	0.108	0.030
29.0	0.120	0.150	0.066	0.060	0.072	0.114	0.042
31.0	0.102	0.126	0.060	0.054	0.066	0.102	0.036
33.0	0.072	0.096	0.042	0.036	0.048	0.072	0.024
35.0	0.048	0.072	0.030	0.024	0.036	0.054	0.012
37.0	0.036	0.048	0.024	0.012	0.030	0.036	0.006
39.0	0.024	0.030	0.018	0.012	0.030	0.030	0.012

SLOPE INDICATOR DATA --TA-4 (cont'd)

DATE	10. 5.71	25. 5.71	11. 6.71	13. 7.71	25. 8.71	22. 9.71	26.10.71
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-1.248	-0.936	-0.774	-0.744	-0.762	-0.900	-0.804
3.0	-1.032	-0.726	-0.594	-0.5888	-0.576	-0.678	-0.612
5.0	-0.810	-0.510	-0.420	-0.432	-0.408	-0.468	-0.432
7.0	-0.552	-0.312	-0.258	-0.276	-0.240	-0.276	-0.252
9.0	-0.282	-0.114	-0.102	-0.126	-0.096	-0.096	-0.078
11.0	-0.030	-0.060	0.042	0.012	-0.042	0.054	0.084
13.0	-0.018	0.066	0.048	0.030	0.042	0.060	0.090
15.0	-0.042	0.060	0.048	0.036	0.036	0.060	0.090
17.0	-0.030	0.060	0.048	0.036	0.036	0.054	0.090
19.0	-0.036	0.060	0.042	0.036	0.024	0.054	0.084
21.0	-0.042	0.048	0.030	0.024	0.018	0.048	0.072
23.0	-0.030	0.054	0.042	0.024	0.018	0.048	0.072
25.0	-0.012	0.060	0.054	0.030	0.024	0.054	0.034
27.0	0.018	0.078	0.072	0.048	0.042	0.072	0.102
29.0	0.042	0.096	0.078	0.060	0.054	0.078	0.120
31.0	0.036	0.084	0.066	0.048	0.042	0.060	0.102
33.0	0.024	0.066	0.048	0.036	0.024	0.042	0.072
35.0	0.012	0.042	0.030	0.024	0.036	0.030	0.060
37.0	0.012	0.030	0.024	0.024	0.030	0.024	0.042
39.0	0.018	0.018	0.024	0.018	0.030	0.030	0.030

DATE	23.11.71	22.12.71
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)
1.0	-0	
1.0	-0.852	-1.008
3.0	-0.654	-0.750
5.0	-0.450	-0.486
7.0	-0.258	-0.252
9.0	-0.072	-0.054
11.0	0.084	0.102
13.0	0.084	0.102
15.0	0.078	0.096
17.0	0.078	0.090
19.0	0.072	0.084
21.0	0.060	0.066
23.0	0.060	0.060
25.0	0.066	0.073
27.0	0.090	0.090
29.0	0.102	0.102
31.0	0.090	0.084
33.0	0.072	0.060
35.0	0.048	0.036
37.0	0.036	0.030
39.0	0.030	0.030

SLOPE INDICATOR DATA -- TA-5

DATE	29.10.70	12.11.70	19.11.70	26.11.70	3.12.70	10.12.70	30.12.7
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-0.114	-0.204	-0.132	0.018	-0.132	-0.126	-0.240
3.0	-0.102	-0.180	-0.120	0.006	-0.132	-0.108	-0.192
5.0	-0.084	-0.156	-0.102	0.0	-0.126	-0.084	-0.126
7.0	-0.066	-0/132	-0.090	0.0	-0.114	-0.066	-0.096
9.0	-0.060	-0.114	-0.-90	0.0	-0.120	-0.066	-0.096
11.0	-0.048	-0.090	-0.072	0.012	-0.114	-0.060	-0.084
13.0	-0.048	-0.090	-0.072	0.018	-0.114	-0.054	-0.084
15.0	-0.060	-0.102	-0.078	0.006	-0.120	-0.066	-0.090
17.0	-0.060	-0.090	-0.072	0.0	-0.114	-0.060	-0.084
19.0	-0.054	-0.084	-0.060	0.006	-0.102	-0/054	-0.072
21.0	-0.048	-0.072	-0.048	0.012	-0.096	-0.042	-0.060
23.0	-0.048	-0.066	-0.030	0.024	-0.084	-0.030	-0.048
25.0	-0.036	-0.054	-0.018	0.036	-0.072	-0.072	-0.018
27.0	-0.030	-0.048	-0.012	0.042	-0.066	-0.012	-0.018
29.0	-0.024	-0.030	-0.006	0.054	-0.054	0.006	-0.006
31.0	-0.012	-0.024	0.0	0.060	-0.042	0.024	0.0
33.0	0.0	-0.018	0.012	0.060	-0.024	0.030	0.006
35.0	0.0	-0.018	0.006	0.054	-0.018	0.024	0.006
37.0	-0.006	-0.012	0.0	0.048	-0.018	0.018	0.0
39.0	-0.018	-0.024	-0.012	0.030	-0.036	0.006	-0.018
41.0	-0.036	-0.042	-0.030	0.006	-0.054	-0.012	-0.030
43.0	-0.036	-0.048	-0.036	-0.006	-0.054	-0.024	-0.042
45.0	-0.030	-0.048	-0.030	-0.018	-0.048	-0.030	-0.042
47.0	-0.024	-0.036	-0.024	-0.018	-0.036	-0.024	-0.030
49.0	-0.018	-0.018	-0.012	-0.012	-0.018	-0.012	-0.012

DATE	22. 1.71	11. 2.71	24. 2.71	12. 3.71	26. 3.71	8. 4.71	26. 4.71
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-0.582	-0.438	-0.276	-0.306	-0.348	-0.522	-0.414
3.0	-0.474	-0.342	-0.180	-0.204	-0.252	-0.402	-0.282
5.0	-0.360	-0.248	-0.084	-0.108	-0.150	-0.270	-0.144
7.0	-0.270	-0.162	0.006	-0.012	-0.054	-0.162	-0.030
9.0	-0.228	-0.120	0.054	0.042	0.006	-0.096	0.030
11.0	-0.186	-0.084	0.090	0.084	0.054	-0.042	0.084
13.0	-0.198	-0.108	0.054	0.048	0.018	-0.072	0.054
15.0	-0.210	-0.132	0.018	0.012	-0.018	-0.102	0.024
17.0	-0.210	-0.138	0.0	0.0	-0.036	-0.108	0.012
19.0	-0.198	-0.126	0.0	0.0	-0.036	-0.102	0.012
21.0	-0.180	-0.102	0.006	0.012	-0.018	-0.084	0.018
23.0	-0.162	-0.084	0.018	0.018	0.0	-0.066	0.030
25.0	-0.128	-0.060	0.024	0.030	0.012	-0.048	0.036
27.0	-0.120	-0.042	0.030	0.042	0.024	-0.036	0.036
29.0	-0.090	-0.018	0.036	0.048	0.036	-0.018	0.042
31.0	-0.078	-0.0	0.042	0.054	0.048	-0.006	0.048
33.0	-0.072	0.006	0.048	0.060	0.054	0.0	0.054
35.0	-0.066	0.012	0.048	0.048	0.048	-0.006	0.042
37.0	-0.060	0.006	0.036	0.030	0.042	-0.012	0.030
39.0	-0.066	-0.018	0.012	0.006	0.024	-0.024	0.012
41.0	-0.072	-0.035	-0.012	-0.012	0.0	-0.036	-0.006
43.0	-0.072	-0.048	-0.030	-0.030	-0.018	-0.042	-0.024
45.0	-0.066	-0.054	-0.036	-0.042	-0.030	-0.042	-0.036
47.0	-0.048	-0.042	-0.030	-0.036	-0.024	-0.036	-0.035
49.0	-0.024	-0.024	-0.018	-0.024	-0.012	-0.024	-0.02[illegible]

SLOPE INDICATOR DATA -- TA-5 (cont'd)

DATE	10. 5.71	25. 5.71	11. 6 71	13. 7.71	25. 8.71	22. 9.71	26.10.71
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)	DEFL. (in.)
1.0	-0.376	-0.222	-0.246	-0.378	-0.174	-0.228	-0.234
3.0	-0.264	-0.162	-0.204	-0.300	-0.144	-0.180	-0.204
5.0	-0.138	-0.090	-0.150	-0.222	-0.096	-0.132	-0.162
7.0	-0.042	-0.036	-0.108	-0.162	-0.036	-0.096	-0.132
9.0	0.018	0.0	-0.078	-0.132	-0.006	-0.072	-0.102
11.0	0.072	0.042	-0.036	-0.096	0.036	-0.030	-0.060
13.0	0.030	0.012	-0.054	-0.108	0.018	-0.048	-0.078
15.0	-0.006	-0.030	-0.078	-0.130	-0.012	-0.084	-0.102
17.0	-0.024	-0.042	-0.090	-0.144	-0.018	-0.102	-0.108
19.0	-0.024	-0.042	-0.084	-0.132	-0.018	-0.108	-0.108
21.0	-0.018	-0.030	-0.066	-0.102	-0.018	-0.090	-0.090
23.0	-0.006	-0.018	-0.054	-0.078	-0.006	-0.072	-0.066
25.0	0.012	-0.006	-0.036	-0.048	0.0	-0.048	-0.042
27.0	0.030m	0.006	-0.024	-0.030	0.006	-0.030	-0.024
29.0	0.042	0.030	-0.012	-0.012	0.018	-0.012	-0.006
31.0	0.042	0.036	0.0	0.006	0.024	0.0	0.0
33.0	0.048	0.042	0.006	0.018	0.036	0.012	0.006
35.0	0.042	0.036	0.006	0.012	0.018	0.006	0.0
37.0	0.030	0.024	0.012	0.006	0.012	0.0	-0.012
39.0	0.012	0.006	-0.006	-0.006	-0.012	-0.024	-0.030
41.0	-0.006	-0.012	-0.024	-0.018	-0.018	-0.042	-0.042
43.0	-0.024	-0.030	-0.036	-0.036	-0.030	-0.054	-0.054
45.0	-0.030	-0.042	-0.042	-0.042	-0.030	-0.054	-0.054
47.0	-0.024	-0.036	-0.036	-0.036	-0.024	-0.048	-0.048
49.0	-0.012	-0.024	-0.012	-0.024	-0.012	-0.030	-0.030

DATE	23.11.71	22.12.71					
DEPTH (ft.)	DEFL. (in.)	DEFL. (in.)					
1.0	-0.198	-0.306					
3.0	-0.162	-0.264					
5.0	-0.120	-0.180					
7.0	-0.090	-0.114					
9.0	-0.060	-0.078					
11.0	-0.024	-0.048					
13.0	-0.030	-0.066					
15.0	-0.048	-0.090					
17.0	-0.054	-0.102					
19.0	-0.042	-0.102					
21.0	-0.024	-0.090					
23.0	-0.006	-0.072					
25.0	0.006	-0.060					
27.0	0.018	-0.054					
29.0	0.024	-0.036					
31.0	0.030	-0.024					
33.0	0.042	-0.006					
35.0	0.036	-0.006					
37.0	0.018	-0.012					
39.0	0.0	-0.030					
41.0	-0.018	-0.054					
43.0	-0.042	-0.072					
45.0	-0.054	-0.078					
47.0	-0.048	-0.060					
49.0	-0.030	-0.036					

APPENDIX B. DAILY PRECIPITATION DATA FOR THE WINNIPEG AREA

ANNUAL METEOROLOGICAL SUMMARY
for
WINNIPEG, MANITOBA

DAILY PRECIPITATION - 1969

DATE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1	Tr.	.03	Tr.	Tr.	.07	.39	Tr.			.24	Tr.	
2	.06	Tr.	Tr.						.01			
3	Tr.	Tr.		Tr.		.22	.18	.04	.01	.41		
4	Tr.	Tr.	.01	Tr.	.02		.37		1.05	Tr.		
5	.08		Tr.		.57			.08	.06			
6	.05	Tr.	Tr.		.15	Tr.	.09	.23	.02	Tr.		.08
7	.05	Tr.	.18		.01		.08	.35	.08	.15		.21
8	.04	Tr.	.10	Tr.	.04		.11				Tr.	.19
9	Tr.	Tr.	.05	Tr.	Tr.		.11				Tr.	.03
10	Tr.	.04			Tr.	.03	.07		.01	Tr.	Tr.	.02
11	Tr.	.01				Tr.		Tr.		Tr.	.05	.02
12	.03	Tr.		Tr.	.14	.03		.12			.02	.02
13	.03	Tr.			.10		.08	.57		Tr.	Tr.	Tr.
14	Tr.	.05			Tr.		.06			.02	.04	Tr.
15	.35	Tr.		Tr.	Tr.	Tr.	.01		.04	.06	Tr.	Tr.
16	.24	Tr.			Tr.	Tr.			Tr.		Tr.	Tr.
17	Tr.	Tr.				Tr.		.18				
18	.02	.05			.17	.07	.33	Tr.			Tr.	Tr.
19	.06	Tr.			Tr.	Tr.	.10				Tr.	Tr.
20	.01				Tr.				Tr.	Tr.		Tr.
21	.14	.04					Tr.		.53	.10		.02
22	.30	.04	Tr.		Tr.		Tr.		Tr.		.05	Tr.
23	.03	.04			Tr.		.04		Tr.	Tr.	.13	.01
24	.05	.01							.38	.01	Tr.	.06
5	Tr.			.22		.59			.01	.07	.02	.02
6	Tr.		Tr.	.48	.48	.80	1.92	.08	Tr.		.06	Tr.
7	.20	Tr.	.04		Tr.	1.26	.03		Tr.		Tr.	Tr.
8	Tr.	.03				.16			.03			.02
9	.13					.58		.59	Tr.	Tr.	Tr.	.01
0	Tr.			.30	.09		.41		.17	.01		Tr.
1	Tr.		.01		1.18			.01		Tr.		.03
al	1.87	.34	.39	1.00	3.02	4.13	3.99	2.25	2.40	1.07	.37	.74

ANNUAL METEOROLOGICAL SUMMARY
for
WINNIPEG, MANITOBA.

DAILY PRECIPITATION - 1970.

DATE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1	.07	.08			Tr.				Tr.		.42	.41
2	.03	Tr.	.34	.08	Tr.	.01	.17		.16	Tr.	.08	
3	.01	.02	.60	.04			.02		Tr.			.05
4	.01	.04			.02							.04
5	Tr.	Tr.	Tr.				.08		.20			.01
6	.01						.09			Tr.	Tr.	Tr.
7	.01	Tr.	Tr.	.01			Tr.		.28	Tr.	Tr.	Tr.
8	Tr.	Tr.	Tr.	.07	Tr.	.21			.06			.12
9	Tr.				.05	.17	Tr.		.22	.03		
10	.09	.06			.03	.40			Tr.	.04		Tr.
11	Tr.	Tr.	Tr.		.29	.07	.04		.44			
12		.01	Tr.		.01	.11	.21		.08	Tr.	Tr.	.02
13	.05		Tr.		.02	Tr.	.01		Tr.			.01
14	.27	.01			.37		1.28			Tr.		
15	.12			.44	.21	Tr.		.08	.31			Tr.
16	Tr.	.11		.12		.18					.02	Tr.
17		.22				Tr.	Tr.	.02			.15	.04
18	Tr.	Tr.					.18	Tr.			.02	Tr.
19		.01	Tr.	.88		Tr.					Tr.	.04
20	Tr.	.02		.53	.13				2.01		Tr.	
21	Tr.			.15	Tr.			.65	.53		.06	.04
22	Tr.		.01	Tr.			.06		.01		.02	.08
23	.03			.02		.12	.15		Tr.		Tr.	Tr.
24	Tr.	Tr.			1.27		.17		Tr.	Tr.	.16	.03
25	.07	.01	Tr.		.54	.36	.12		.06	.17	Tr.	Tr.
26	Tr.	Tr.	.04		Tr.		Tr.	Tr.	.05	.08		.03
27	.06			.02	.03	.08				.42		
28	.05			.20	.15	Tr.	Tr.			.05	.09	Tr.
29	.02		.09	.01	.10		.16	2.20		.01		
30			Tr.	Tr.	.13	.28				Tr.	.03	Tr.
31	Tr.		Tr.		.06					.25		Tr.
Total	.90	.59	1.08	2.57	3.41	1.99	2.74	2.95	4.41	1.05	1.05	.92

APPENDIX C: RED RIVER WATER LEVELS AT WINNIPEG (REDWOOD BRIDGE)

APPENDIX E: BANK MOVEMENT DATA - ALIGNMENT PINS

TACHE AVE. ALIGNMENT PIN DATA

DATE	PIN ELEVATIONS 1	2	3	4	5	SLOPE DISTANCE 1&2	2&3	3&4	4&5
LINE 1									
12/12/69	56.66*	53.47	47.42	41.84	31.21		24.41	42.36	41.45
13/12/69	56.70	53.47	47.35	41.87	31.23	64.07	24.13	42.33	41.44
22/12/69	56.67	53.43	47.42	41.83	31.22				
9/ 1/70	56.67	53.55	47.40	41.81	31.24	64.07	24.22	42.30	41.45
27/ 1/70	56.63	53.63	47.39	41.79	31.23	64.09	24.24	42.30	41.44
12/ 3/70	56.60	53.51	47.31	41.77	31.24	64.09	24.27	42.28	41.43
17/4/70	56.65	53.52	47.38	41.75	---	64.07	24.30	42.28	---
27/ 4/70	56.66	53.51	---	---	---	64.07	---	---	---
1/ 5/70	56.65	53.50	---	---	---	64.11	24.26	---	---
7/ 5/70	56.67	---	47.38	---	---	64.10	24.28	---	---
22/ 5/70	56.67	53.46	47.38	---	---	64.05	24.27	---	---
21/ 8/70	56.61	53.41	47.35	41.73	---	64.19	24.29	42.31	---
27/11/70	56.66	53.48	47.24	41.59	31.40	64.15	24.84	42.29	41.51
11/ 2/71						64.12	25.04	42.25	41.51
12/ 2/71	56.68	53.49	47.13	41.45	31.49				
15/ 3/71	56.66	53.48	47.13	41.46	31.51				
29/ 3/71	56.60	53.49	47.13	41.45	31.51		25.05	42.22	41.53
LINE 2									
12/12/69	56.76	52.49	49.28	41.75	31.62		25.49	41.83	40.32
13/12/69	56.81	52.47	44.58	41.79	31.66	60.82	25.49	41.84	40.35
22/12/69		52.44	44.27	41.75	31.62				
9/ 1/70	56.75	52.39	44.25	41.75	31.62	60.90	25.47	41.87	40.32
27/ 1/70	56.72	52.34	44.23	41.74	31.59	60.92	25.45	41.87	40.32
12/ 3/70	56.70	52.26	44.20	41.75	31.60	60.98	25.44	41.90	40.32
17/ 4/70	56.73	52.17	44.19	41.79	---	61.05	25.43	41.95	---
27/ 4/70	56.76	52.17	---	---	---	61.06	---	---	---
1/ 5/70	56.77	52.15	---	---	---	61.05	---	---	---
7/5 /70	56.79	52.16	44.12	---	---	61.03	25.42	---	---
22/ 5/70	56.80	52.20	44.17	---	---	61.03	25.42	---	---
21/ 8/70	56.68	52.57	44.12	41.67	---	61.14	25.41	41.96	---
27/11/70	56.79	51.50	43.83	41.74	31.65	61.95	25.35	42.36	40.44
11/ 2/71						62.24	25.31	42.36	40.35
12/ 2/71	56.72	51.17	43.64	41.69	31.63				
15/ 3/71	56.72	51.13	43.70	41.55	31.66				
29/ 3/71	56.72	51.13	43.66	41.74	31.66	62.39	25.28	42.33	40.30
LINE 3									
12/12/69	57.57	52.41	41.94	39.67	31.24		27.55	41.18	38.83
13/12/69	57.57	52.38	42.03	39.67	31.28	57.40	27.56	41.15	38.81
22/12/69	57.57	52.34	41.98	39.63	31.26				
7/ 1/70	57.54	52.27	41.96	39.62	31.29	57.57	27.50	41.17	38.81
27/ 1/70	57.50	52.23	41.93	39.60	31.29	57.59	27.45	41.14	38.79
12/ 3/70	57.48	52.15	41.91	39.59	31.34	57.69	27.42	41.13	38.81
17/ 4/70	57.52	52.05	41.88	39.63	---	57.90	27.34	41.20	---
27/ 4/70	57.58	52.08	---	---	---	57.93	---	---	---
1/ 5/70	57.52	52.02	---	---	---	57.95	---	---	---
7/ 5/70	57.54	52.03	---	---	---	57.93	---	---	---
22/ 5/70	57.58	52.04	---	---	---	57.99	---	---	---
21/ 8/70	57.42	51.94	41.87	39.57	---	58.06	27.22	41.17	---

TACHE AVE. ALIGNMENT PIN DATA (cont'd)

	PIN ELEVATION					SLOPE DISTANCE			
DATE	1	2	3	4	5	1&2	2&3	3&4	4&5
LINE 3 (cont'd)									
27/11/70	57.53	51.22	41.52	39.53	31.35	59.42	26.93	41.33	39.00
11/ 2/71						59.99	26.58	41.35	39.01
12/ 2/71	57.43	50.83	41.38	39.44	31.48				
15/ 3/71	57.43	50.78	41.39	39.46	31.56				
29/ 3/71	57.44	50.77	41.42	39.44	31.53	60.09	26.53	41.33	39.01
LINE 4									
12/12/69	57.69	52.98	40.91	40.02	32.57		29.40	40.30	37.47
13/12/69	57.69	52.95	40.88	40.03	32.56	53.94	29.39	40.24	37.39
22/12/69	57.69	52.91	40.82	39.96	32.52				
9/ 1/70	57.66	52.85	40.80	39.95	32.54	54.07	29.35	40,25	37.45
27/ 1/70	57.64	52.82	40.76	39.94	32.53	54.09	29.32	40.27	37.47
12/ 3/70	57.61	?	40.74	39.92	32.53	54.16	29.32	40.28	37.49
17/ 4/70	57.66	52.63	40.71	39.99	---	54.28	29.29	40.32	---
27/ 4/70	57.67	52.68	---	---	---	54.27	---	---	---
1/ 5/70	57.67	52.62	---	---	---	54.27	---	----	---
7/ 5/70	57.69	52.63	---	---	---	54.24	----	---	---
22/ 5/70	57.73	52.65	---	---	---	54.24	---	---	---
21/ 8/70	57.55	52.55	40.69	39.93	---	54.42	29.23	40.31	---
27/11/70	57.67	51.90	40.37	39.89	32.71	55.42	29.17	40.45	37.51
11/ 2/71						55.88	28.97	40.49	37.55
12/ 2/71	57.59	51.55	40.18	39.80	32.76				
15/ 3/71	57.57	51.49	40.17	39.81	32.81	55.90	28.97		
29/ 3/71	57.60	51.48	40.19	39.83	32.82	55.90	28.96	40.49	37.50
LINE 5									
12/12/69	57.20	54.19	43.79	41.89	32.22		30.02	41.24	35.75
13/12/69	57.20	54.16	43.76	41.76	32.21	50.65	30.03	41.24	35.75
22/12/69	57.20	54.13	43.70	41.73	32.17				
9/ 1/70	57.19	54.09	43.68	41.84	32.13	50.82	29.98	41.25	35.78
27/ 1/70	57.16	54.05	43.64	41.82	32.15	50.76	29.97	41.26	35.74
12/ 3/70	57.12		43.58	41.79	32.15	50.82	29.94	41.28	35.74
17/ 4/70	57.16	53.88	43.52	41.74	---	50.95	29.95	41.32	---
27/ 4/70	57.20	53.89	---	---	---	50.93	---	---	---
1/ 5/70	57.19	53.85	---	---	---	50.92	---	---	---
7/ 5/70	57.21	53.88	---	---	---	50.92	---	---	---
22/ 5/70	57.24	53.86	43.55	---	---	50.88	30.03	---	---
21/ 8/70	57.07	53.71	43.49	41.82	---	50.98	29.97	41.29	---
27/11/70	57.14	53.11	43.04	41.80	32.22	51.70	30.09	41.51	35.78
11/ 2/71						52.04	29.91	41.56	35.76
12/ 2/71	57.04	52.82	42.80	41.75	32.20				
15/ 3/71	57.04	52.78	42.77	41.76	32.23				
29/ 3/71	57.07	52.79	42.79	41.80	32.25	52.02	29.86	41.55	35.75

* Datum 700.00