PRELIMINARY SOIL MECHANICS ASPECTS

OF THE

RED RIVER FLOODWAY

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INTRODUCTION

Winnipeg, situated at the confluence of the Red and Assiniboine Rivers, has been plagued by floods of large magnitudes since 1826. Subsequent large floods occurred in 1852, 1861, 1916, 1948 and 1950 (1). The flood of 1950 caused such severe damage as to stimulate positive action towards alleviating the flood problem. As a result of the 1950 flood the Dominion Government set up the Red River Basin Investigation (2) whose purpose was to investigate the various possibilities of alleviating the flood problem. This organization studied many schemes and combinations thereof. Subsequently the Royal Commission on Flood Cost Benefit (3) was set up to select the most economically feasible combination of schemes submitted by the Red River Basin Investigation. The result was the selection of a diversion channel bypassing the Greater Winnipeg Area in combination with several other projects.

Additional studies regarding the location of the diversion channel were commenced in 1959 by the Water Control and Conservation Branch of the Province of Manitoba. This resulted in some relocations of the diversion channel from the alternatives recommended by the Red River Basin Investigation, primarily at the south end where it now includes the suburb of St. Norbert. This diversion channel, known as the Red River Floodway, is shown in fig. 1.

GENERAL DESIGN CONCEPTS

The Red River Floodway will be about 30 miles long with a top width of approximately 1000 feet, having a depth varying from 25 to 75 feet, with the average depth being approximately 30 feet. Typical channel cross sections are shown in fig. 2. The quantity of excavation is some 100 million cubic yards. Its design capacity of flow is 60,000 cfs.

The Floodway will have a fixed inlet elevation with a control dam located in the Red River immediately downstream of the inlet to control diversion of flood waters into the Floodway. Several Provincial Trunk Highway, municipal road, and railroad bridges will cross the Floodway in addition to under and overpasses by water supply mains and hydro-electric power lines which service Metropolitan Winnipeg. Near its re-entry into the Red River an outlet structure will control its hydraulic gradient and provide proper hydraulic re-entry conditions.

GENERAL SOIL CONDITIONS

The Floodway is located in the highly plastic lacustrine clays of glacial Lake Agassiz, underlain generally by glacial till. An exception occurs through the Birds Hill ridge, which is a granular deposition by a large, swift-flowing stream during the last glacial age (4). The soils south and north of Birds Hill are similar but vary in depth to the glacial till.

The general soil profile predominant along most of the Floodway and common to the Red River Valley has been previously well described by several investigators, references (5), (6) and (7).

INVESTIGATIONS

Design and construction of the Floodway required the solution of numerous soil mechanics problems that required investigation. The principal problem was the selection of slopes which would be stable almost indefinitely. It was known from earlier investigations and experience on river banks that rather flat slopes would be required. Experience elsewhere in highly plastic clays (8) indicated that this long term stability could be unreliable if based on conventional strength tests. The stability problem was further complicated by the fact that it was economically desirable to place excavated soils as close as possible to the sides of the

Floodway to save on haulage and land acquisition. Other problems involved the effects of rebound on excavated grades, the effects of erosion, the problem of embankment compaction, percent volume reduction due to varying degrees of compaction, the determination of best methods of excavation, the effects of ground water seepage during excavation, possible depletion of local ground water supplies, foundations of structures, and numerous other problems.

(1) Review of Earlier and Related Investigations

Much use was made of earlier investigations. However, when studies on the Floodway were resumed by the Water Control and Conservation Branch in 1959, the relocation of portions of the Floodway and the need for detailed information for final design, made several additional subsurface investigations necessary. For example, the very irregular conditions in the Birds Hill ridge required a very thorough investigation. Another investigation was conducted to establish more accurately the hardpan profile along, and transverse to the Floodway centreline north of the Birds Hill ridge where the hardpan was known to be above the Floodway bottom grade.

Soil samples obtained during these investigations were in general subjected to unconfined compression strength tests, undisturbed density, moisture content, and classification tests. In addition, other tests, such as hydrometer grain size analysis, standard and modified proctor compaction tests, consolidation (and rebound), and controlled triaxial compression tests with and without pore pressure measurements were performed.

(2) Detailed Tests

Upon recommendation by Professor A. Casagrande, (Soils Consultant on the Red River Floodway) and Engineers of the P.F.R.A., more detailed tests were run on very carefully obtained undisturbed samples. These samples were obtained from two test holes, 17C and 18C, bored at locations selected to be representative of that part of the Floodway south and north of the Birds Hill ridge, respectively. Almost continuous 3 and 5 inch diameter Shelby tube samples were obtained in both holes. The samples were subjected to extensive triaxial testing by the P.F.R.A. to supplement the Floodway slope stability and selection studies.

(3) Riverbank Survey in Metropolitan Winnipeg

In 1960 a survey was conducted of banks along the Red and Assiniboine Rivers, mostly within Metropolitan Winnipeg, to determine at what slopes natural banks were stable, the degree of stability where possible, at what slopes natural banks failed and at what slopes such failed banks again became stable.

All the banks were visually examined, classified as to stability, and cross-sections were taken of banks considered appropriate for the study. Factors affecting this stability, such as planar alignment of the banks, estimated degree of toe erosion, vegetation, existence of shrinkage and tension cracks, and depth to the rigid strata (hardpan) were noted. Consideration was given to these factors as to their existence and non-existence along the Floodway and therefore their effect on stability of Floodway slopes. The results of this survey are shown in fig. 4.

(4) Floodway Test Pits

The Floodway test pits are the most extensive part of the investigations. Upon recommendation by Professor A. Casagrande, an open test pit (no.1), with surface dimensions of 760 feet by 320 feet was excavated at a selected location (see fig.5) along the Floodway to obtain additional data for the slope stability and selection studies by estimating the in-situ shear strength of the clay after rapid excavation and after rapid drawdown conditions. The test pit, with extensive instrumentation, as shown in fig. 5 (designed by Engineers of the P.F.R.A. Soil Mechanics and Materials Division), was excavated with a slope of one horizontal to one vertical on the north side and four horizontal to one vertical on the south side.

Modified Casagrande piezometers were installed in clay and till in two lines across the pit and along the slopes. A Shannon and Wilson slope indicator was installed at the centres of the 1 to 1 and 4 to 1 slopes. Modified slide plane detectors were installed in each of the slopes. Alignment hubs were placed at mid-height and at the top of each slope. Nails were driven into the tops of the hubs forming a straight line to permit measurement of displacement from the straight line by movement of the slope. A transit was used to check for displacement. Resulting slope movements are graphically represented in fig. 6.

Another test pit (no. 2) was proposed to be excavated along the Floodway north of the Birds Hill ridge for the purpose of studying the efficiency of hardpan excavation by conventional equipment, to observe the effects of exposure to free water on hardpan, and to observe the quantity of seepage into the excavation.

RESULTS OF INVESTIGATIONS

(1) Earlier and Related Investigations

The results of classification and basic strength testing on samples obtained during the subsurface investigations conducted by the Water Control and Conservation Branch have been summarized in fig. 3. These results agree very closely with those obtained by P.F.R.A. during the 1951 investigation and Professor A. Baracos during the 1952 investigation.

(2) Riverbank Survey in Metropolitan Winnipeg

The following conclusions were drawn from the survey of riverbanks in Metropolitan Winnipeg.

1) Very few stable riverbanks were found in Winnipeg and adjoining areas.

2) "Convex riverbanks" cannot be used in slope stability studies for the Floodway because of the different soils involved. Floodway soils are of glacial lake origin and are primarily clays whereas "convex riverbanks" soils are of fluvial origin and are primarily silts.

3) Since the majority, (80%), of concave riverbanks, in the highly plastic clays, between 25 ft. to 40 ft. high which failed in a rotational manner became stable at slopes between $4\frac{1}{2}$ to 1 and 6 3/4 to 1, and therefore naturally failed at slopes steeper than the slopes at which they became stable, it may be assumed that the slopes of 6 to 1 as recommended for the Floodway by Professor A. Casagrande should be stable along <u>most</u> of the Floodway. This suggests that some failures might occur, however, it may be more economical to rectify a few failures in the 6 to 1 slopes than to construct the Floodway at a gentler slope which will not result in any failures.

(3) <u>Floodway Test Pits</u>

(a) <u>Preliminary Considerations</u>

Two test holes, 17C and 18C, were bored near the likely locations for each of the two proposed test pits (one south of Birds Hill and one north of Birds Hill) as recommended by Professor A. Casagrande. The results of testing of samples obtained from these holes have been summarized by the P.F.R.A. Soil Mechanics Laboratory in Table 1. The main purpose of the testing of samples obtained from test hole 17C was to determine the depth of excavation required to result in a failure in the 1 to 1 slope in the test pit.

TABLE 1 SUMMARY OF ALL TESTS ON HOLES 17C and 18C

Hole No.	Depth ft.	Lab.	Avg. w%	Dry Density p.c.f.	"R" TESTS Low Lateral Pressure c'p.s.i.	Ø'deg.	Ø'p*deg.	"S" TESTS Low Lateral Pressure c'p.s.i.	Ø'deg.	Ø' p* deg.
17C	18-20	UofM	53.4	69.4	10.5	7.5	12.5			
	20	PFRA	54.2	69.0	1.5	27.0	16.9			
	20	PFRA	55.3	68.3				0	16.9	16.9
	40-42	UofM	62.3	63.3	5.0	7.5				
	42	PFRA	60.2	64.7	3.0	13.0	14.0			
	42	PFRA	60.8	64.5				2.0	16.0	15.0
18C	12-13	UofM	46.3	74.5	4.0	11.4	12.5			
	12-13	UofM	45.4	75.4				3.0	11.5	13.5
	20	UofM	67.1	58.8	2.6	13.0	15.0			
	20-21	UofM	73.4	57.1				2.5	13.0	15.0

NOTE: Ø' p* = effective angle of internal friction beyond the effect of preconsolidation

SOIL PROPERTY	TEST HOLE 17C AVERAGE	TEST HOLE 17C RANGE	TEST HOLE 18C AVERAGE	TEST HOLE 18C RANGE
Water Content, %	51.1	27.9-64.3	50.7	36.6-81.0
Wet Density, pcf	106	100-115	106	95-113
Liquid Limit, %	89	52-116	77	56-88
Plastic Limit, %	22	18-25	19	15-21
Unconfined Compressive Strength, psi	15.4	9.2-25.0	10.2	3.5-19.1
w% from Unconfined Strength Tests	49.2	27.2-66.4	51.0	35.9-78.0
Sensitivity	2.2	0.9-4.3	2.0	0.4-3.7
Strain at Failure (Unconfined Test)	2.9	0.8-6.9	2.3	0.8-4.3

Stability analyses were performed on the basis of total stresses using rather high strength values to ensure that the slope would fail. From a total stress stability analysis it was evident that there was very little difference in stability between a failure arc tangent to the firm layer and an arc through the toe; in fact it appeared that a tension crack would likely result in a toe circle failure (9).

Preliminary effective stress analysis were also considered. However, for conditions immediately following excavation, it was found that the wide range of pore pressures, erratic values of c' and ϕ', and non-uniform actual slopes as the excavation proceeded, made such calculations quite erratic and inconclusive. For example the $\bar{R}$ test showed $c' = 1.5$ psi and $\phi' = 27^\circ$ whereas the S tests showed $c' = 1.5$ psi and $\phi' = 17^\circ$. Therefore, primarily on the basis of the total stress stability analysis it was decided to excavate the test pit to a depth of some 45 feet anticipating that failure would occur. Further, it was anticipated to construct a loading berm should failure not occur as a result of excavation.

(b) $\bar{Q}$, $\bar{R}$, S (10) and consolidation tests on samples obtained from the test pit.

During the excavation of the test pit large chunk samples were obtained from the 1 to 1 slope from three horizontal locations at five foot depth intervals. Several $\bar{Q}$, $\bar{R}$, and S tests were conducted by P.F.R.A. on the chunk samples with one sample also being tested by the National Research Council.

The $\bar{R}$ tests were conducted since in the correct range of normal pressures the $\bar{\bar{R}}$ tests should give the strength based on rapid excavation. Similarily, the S tests were conducted since the strengths determined from S tests should give the long term strengths if the S tests are carried out at a rate which is applicable to field conditions.

The results of these tests are summarized here as follows:

TABLE 2 RESULTS OF $\bar{\bar{Q}}$ TESTS ON SAMPLES FROM THE FLOODWAY TEST PIT

Depth ft.	Average Dry Density p.c.f.	Average Water Content w %	Average Compressive Strength p.s.i.	Range of Strain at Failure %
5	84.2	31.9	13.9	2 - 6
15	67.2	56.7	22.7	0.9 - 1.5
20	67.5	56.0	17.2	1.6 - 2.6
25	70.0	53.1	19.2	1.6 - 2.2
30	67.8	56.1	18.9	1.7 - 2.4
35	64.1	61.0	18.1	1.2 - 1.8
40	62.1	64.3	17.0	1.5
45	61.6	65.1	13.7	0.8 - 1.8

TABLE 3 RESULTS OF $\bar{R}$ TESTS ON SAMPLES FROM THE FLOODWAY TEST PIT

Location	Depth ft.	Average Dry Density p.c.f.	Average Water Content w %	Low Lateral Pressure c' psi.	Low Lateral Pressure Ø' degrees	Ø' p* degrees
75' W of ℄	20	68.3	54.6	6.0	13.0	13.9
75' W of ℄	20	66.5	57.7	2.5	32.0**	
At ℄	30	68.1	55.2	7.0	11.6	14.6
75' W of ℄	30	68.4	54.8	6.5	12.0	16.0
150' W of ℄	45	60.9	66.1	6.0	14.0	15.0

** Tests conducted at very low lateral pressures.

TABLE 4 RESULTS OF S TESTS ON SAMPLES FROM THE FLOODWAY TEST PIT

Location	Depth ft.	Average Dry Density p.c.f.	Average Water Content w %	Low Lateral Pressure c' psi.	Low Lateral Pressure Ø' degrees	Ø' p* degrees
75' W of ℄	20	68.3	54.9	6.0	12.1	15.0
at ℄	30	66.8	57.1			16.1
75' W of ℄	30	69.4	53.6	4.5	16.5	15.9

NOTE: Ø' p* = effective angle of internal friction beyond the effect of preconsolidation.

Curvature of the Mohr envelopes for these tests on clay samples made the selection of a representative strength difficult, therefore, initially two strengths were selected for the clay and one for the till. The results indicated that the average effective normal stress on the failure arc was approximately 7 psi. Therefore, a further set of $\bar{R}$ tests was conducted at very low lateral pressures and a strength selected to correspond with the very low effective lateral pressures. The following strength equations were used in the stability analyses:

s (clay) = 2.5 psi + $\bar{\sigma}$ tan 32°

s (till) = $\bar{\sigma}$ tan 40° (not shown in Table 3)

A Mohr's circles plot of S tests results from all depths, with effective lateral pressures below 40 psi, showed good agreement between S tests from the various depths and also showed a definite curvature of the failure envelope. Comparison of the envelope for $\bar{R}$ tests with effective lateral pressure less than 40 psi with the above S envelope revealed both to be of similar shape, however, the S envelope indicated a slightly higher strength.

Several consolidation tests were performed. The results indicated that the virgin compression branch of the pressure-void ratio plots has some curvature therefore making the selection of compression indices difficult and subject to error. The compression indices for these tests were found to range from 0.8 to 1.0, indicating that the soil is highly compressible when loaded in excess of the preconsolidation load. The preconsolidation load was found to decrease uniformly from approximately 5 tons/ft^2 at the 5 ft. depth, to 2.5 tons/ft^2 at the 45 ft. depth. Under these conditions a normally consolidated soil would be subject to an effective stress of approximately one ton/ft^2 at the 45 ft. depth.

Further, the effects of stratification on consolidation and permeability characteristics were investigated. The results indicated little difference in the permeability although this is rather unusual since it has been commonly accepted that the permeability in the horizontal direction through the stratifications is higher than in any other direction. These results are contradictory to the results of an investigation recently conducted at the University of Manitoba where it was found that the permeability in the horizontal direction was twice that in the vertical.

A chunk sample from the test pit taken at the 30 ft. depth, tested by the National Research Council, yielded moisture content and physical tests results in close agreement with those obtained by P.F.R.A., with the exception that the liquid and plastic limit results of 94 and 34, respectively, obtained by the National Research Council, are much higher than those obtained by the P.F.R.A. on similar samples.

The National Research Council conducted slow tests at rates of strain varying from ½% to 2% per hour, yielding an effective angle of internal friction above the effects of preconsolidation ranging from 10.5° to 11.5°. These results are lower than those obtained by the P.F.R.A. perhaps because of the more rapid rates of loading employed by the National Research Council.

(c) <u>Piezometers</u>

Prior to excavation the piezometric level of the clay and till was observed to be from six to nine feet below natural ground surface. As excavation progressed the piezometric level in the clay piezometers within the excavation area was observed to drop at a rate equivalent to the rate of excavation, that is, the piezometric level generally dropped one foot for one foot of excavation.

Eventually it became evident that some of the piezometers in the clay in the deeper part of the excavation would become dry with excavation because the highly plastic lacustrine clay expands with unloading and therefore these were replaced by high air pressure entry piezometers and read by Bourdon Gauges. Several piezometers in the clay at the 40 ft. depth became dry when the excavation reached a depth of 30 - 35 ft. That this phenomena occurs in the clays is further substantiated by the fact that when the clay was reloaded by water flowing into the test pit, the readings in the deep clay piezometers increased at a rate almost equivalent to the rate of water inflow into the pit, as shown in fig. 11. This does not apply after surface water suddenly filled the pit in April of 1962.

(d) Slope Movement Indicators

Alignment hubs and slope indicators were read daily as excavation progressed. No sign of movement was noticed until the excavation reached an average depth of 25 ft. (on September 15). At this time a movement of $1\frac{1}{4}$ in. toward the excavation was measured at the west end of the 1 to 1 slope on the south alignment line. A movement of 1 3/4 in. was measured at the centre of the south alignment. Only slight movement was measured at the east end of the alignment line. Though these movements had occurred, no openings or cracks were obvious at the surface.

The first definite indication of movement of the entire 1 to 1 slope was given by the slope indicator on September 29 when the average cut at the toe of the 1 to 1 slope was 34 ft. This movement occurred at the 48 ft. depth.

On October 2, when parts of the excavation were at a 36 ft. depth, a movement of $2\frac{1}{4}$ in. was measured at the west end of the south alignment line on the 1 to 1 slope. On October 7, a crack, 3/4 in. wide, occurred 50 ft. north of the edge of the 1 to 1 slope as shown on fig. 5. Further, the slope indicator instrument would not pass below the 48 ft. depth and the movement deflection was measured at 2.7 in.

Slide plane detectors 2,3,4 and 5 (fig.5) in the 1 to 1 slope, showed that the ground movements which had occurred confirmed that the failure plane was at the 48 ft. depth at the slope indicator.

On October 8 a failure occurred at the west end of the 1 to 1 slope. The failed area was approximately 135 ft. long and its surface had dropped vertically about 17 ft. At this time the average cut at the toe of the 1 to 1 slope was 34 ft. Subsequently two smaller failures occurred in the vicinity of the initial failure on October 31 and December 18, respectively. Also, the crack some 50 ft. north of the top edge of the 1 to 1 slope was observed to open and lengthen continuously. By February 27, this crack had opened 4 in. Although movements of the entire mass of the 1 to 1 slope have been observed and measured by several methods, an actual failure of the entire 1 to 1 slope has not yet occurred. However, to induce such failure the test pit was filled with water in the fall of 1961 to saturate the slopes. Currently the water is being rapidly pumped out to simulate rapid drawdown conditions with the anticipation that a failure will occur.

(e) <u>Stability Analysis</u>

Both total and effective stability analyses were conducted on the 1 to 1 slope of the test pit at two different locations. These were at the centre line and 75 feet west of the centre line (in the vicinity of the failure) of the 1 to 1 slope.

A total stress analysis conducted on the failed slope using May's modification of the Swedish slices method indicated that a shear strength of 4.31 psi was required for stability. Similarily, a shear strength of 4.24 psi was required for stability of the worst arc through the centreline of the 1 to 1 slope. Using the results of test hole 17C and adjusting for moisture content of 60% instead of 50% it was possible to obtain good correlation to the actual failure. The correlation is based on the results of fig. 10 which shows several samples of grey clay with moisture contents in the vicinity of 60%.

Three major difficulties were encountered in applying the effective stress method of analysis.

Initially, difficulty was experienced in the selection of a representative strength especially in the range of normal effective stresses expected. Further, difficulty was experienced in selecting the pore pressures at the time of failure and lastly, the actual failure plane was not the simple arc normally used in calculation.

Numerous failure arcs, some through the clay and some through the underlying till, were analyzed. Arcs through the clay yielded lowest factors of safety of 1.28 along the centreline of the excavation and 1.64 through the failure 75 ft. west of the centre line. Arcs assumed to pass through the noncemented till yielded lowest safety factors of 0.65 at the centre line and 0.86 seventy-five (75) feet west of the centre line. Although the possibility of failure through the noncemented till does not agree with the failure plane as established by test installations it is possible that the failure occurred through the clay as established by the test installations and then dipped into the noncemented till. This applies only to the movement of the entire 1 to 1 slope.

It is very unlikely that the failure 75 ft. west of the centreline of the pit extended as deep as the till.

(f) <u>General Observations</u>

Observations were made of the efficiency of excavation by various types of equipment. Rubber tired equipment was found to function satisfactorily to a depth of approximately 25 ft. where the clays became too wet to permit efficient loading. Below the 25 ft. depth and to approximately 36 ft., excavation was successfully carried out by means of steel track equipment and pushers for loading. Below the 36 ft. depth excavation with steel track equipment became impractical and drag line excavation had to be effected. By means of a drag line and trucks to haul the excavated soils, the excavation reached a depth of 45 ft. along the toe of the 4 to 1 slope.

SUMMARY

The most important aspect of the soil mechanics studies on the Floodway, that of slope selection, has received the most consideration by extensive investigations and analysis. The results of these investigations, which are not yet complete, indicate that a slope of six horizontal to one vertical for most of the Floodway should be stable. Though 6 to 1 slopes are not required for the entire length of the Floodway such slopes are necessary for maintenance of the slopes in the cutting of grass. Where vegetative growth is not anticipated, as through the Birds Hill ridge, slopes of 3 to 1 have been selected to reduce the quantity of excavation.

To maintain stability of the slopes an adequate berm had to be determined between the top of the 6 to 1 slope and bottom of the embankment slope. Stability analysis indicated that a berm of 150 ft. would be adequate for the most severe condition along the Floodway.

Further, to insure stability, slopes of 9 to 1 have been selected at bridges.

Studies of rebound along the Floodway bottom and slopes indicated a small magnitude of rebound and therefore the amount of additional excavation to achieve Floodway grade is insignificant.

It is known from experience that the highly plastic lacustrine clays are not subject to erosion at low velocities of the order of $3\frac{1}{2}$ to 5 f.p.s. which will occur in the Floodway and therefore very little erosion is anticipated in these soils. Erosion is anticipated in the clayey silts and silty clays which occur at the top of the channel slopes, however, at the low velocities this erosion is anticipated to be negligible. Erosion is also anticipated in the noncemented glacial till which will be encountered north of Birds Hill, however, on the basis of previous experience this erosion is not expected to be severe.

Numerous standard and modified Proctor compaction tests were performed on the various soil types encountered along the Floodway. On the basis of these results and other data, it was decided that a density of 95% of the modified Proctor compaction density should be specified for the dyke section of the embankment with maximum layer thickness being nine (9) inches. No density has been specified for the embankment other than the dyke since specific compaction of the embankment is not required. Appreciable compaction of the embankment is anticipated from equipment traversing the embankment in a routed manner and from the compactive effect of a tamping roller specified to make two passes over the embankment. The maximum lift thickness in the embankment has been specified to be 18 inches.

Volume reduction of excavated soils when placed in the embankment and compacted has been determined for the various types of soils and reaches along the Floodway along which the volume reduction of the soils is considered to be quite consistent.

Hardpan excavation has been established along the Floodway by numerous test holes within the area of such hardpan excavation. Several test holes bored transversely to the Floodway centre line indicate the variations in the hardpan surface in a direction transverse to the Floodway centre line. With such information the quantity of hardpan excavation has been determined more accurately.

Ground water seepage from the water bearing glacial till layer is a major problem in itself and extensive investigations have been and are being conducted to determine what quantities of seepage should be anticipated, what remedial measures may be taken and further, how such seepage will affect water supplies along the Floodway where seepage occurs. Though a lot of information has been obtained, this problem is, as indicated, a major study in itself and could not be justly dealt with in this paper.

Extensive field and laboratory investigations were conducted to locate a site suitable for the construction of a control dam at the inlet. Initially several sites were investigated and all but two were eliminated on the basis of the field investigations. Further investigations of the two possible sites indicated one to be somewhat better than the other. The selected site was then investigated with further extensive borings, sampling and laboratory testing.

The soil mechanics studies pertaining to the control structure are quite extensive and cannot be effectively related within the scope of this paper. It will suffice that soil conditions at the site of the proposed structure are very similar to the Floodway soils with the exception that one side of the river has been affected by a severe bank failure.

Further, investigations have also been conducted for the outlet structure, however, the problems at this site are not serious since the structure will be founded on hardpan.

CONCLUSIONS

The Floodway, although it has often been simply described as being a "big ditch" is largely dependent on the integrated applications of several sciences. In this case, the science under consideration, that of soil mechanics, plays a prominent role in the success of the Red River Floodway. Its major function in selecting slopes for the excavation has involved extensive theoretical studies based on practial observations and data. Other factors under study also, cannot be overlooked, however, although extensive investigations and studies have already been conducted and more investigations are planned and some currently being conducted, the success of these investigations and studies will be evident only in the completed project itself.

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NOMENCLATURE AND SYMBOLS USED

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Q test	A triaxial compression test at constant water content, in which the specimen is first subjected to a hydrostatic confining pressure and then the axial stress is increased to failure.
$\overline{Q}$ test	A triaxial compression test the same as the Q test, except that pore pressures are measured.
R test	A triaxial compression test in which the specimen is first consolidated under an effective hydrostatic consolidation pressure σ_c', and then, without permitting any further change in water content, is subjected to axial load increase to failure.
$\overline{R}$ test	A triaxial compression test the same as the R test except that pore pressures are measured.
$\overline{R}$ circle	The strength circle from an R test plotted in terms of the effective stresses.
$\overline{R}$ envelope	The envelope to a series of $\overline{R}$ circles.
S test	A triaxial compression test in which the specimen is first consolidated under a hydrostatic confining pressure, and is then subjected to axial load increase which is applied in small increments, allowing full consolidation under each increment, until failure is reached; also, a direct shear test in which the specimen is first consolidated under a normal stress, and is then subjected to shear stress which is applied in small increments, allowing full consolidation under each increment, until failure takes place.
S circle	The strength circle from an S test.
S envelope	The envelope to a series of S circles.
s	Shear strength.
B.	Before test.
A.	After test.
**	Tests conducted at very low lateral pressures.
w.%	Water content.
U	Pore pressure measured at the ends of a triaxial specimen.
Uo	Initial back pressure, i.e. the pore pressure applied at the ends of the specimen and maintained constant during consolidation under the hydrostatic confining pressure in an R or $\overline{R}$ test.

c'	Effective cohesion intercept.
ϕ'	Effective angle of shearing resistance or friction.

In terms of effective stress: $s = c' + \bar{\sigma} \tan. \phi'$

$\phi'p^*$	Effective angle of internal friction beyond the effect of preconsolidation.
σ	Total normal stress.
$\bar{\sigma}$	Effective normal stress.
σc	Hydrostatic confining pressure; the chamber pressure in a triaxial test.
$\sigma c'$	The effective consolidation pressure in an R or S test, i.e. the hydrostatic confining pressure σc minus the initial back pressure Uo.
$\sigma 1$	Total major principal stress.
$\sigma 3$	Total minor principal stress.
$\sigma 1 - \sigma 3$	Deviator stress; the difference between the major principal stress and the minor principal stress.
$\bar{\sigma} 1$	Effective major principal stress.
$\bar{\sigma} 3$	Effective minor principal stress.
$\frac{\bar{\sigma} 1}{\bar{\sigma} 3}$	Effective principal stress ratio; the ratio of the effective major principal stress to the effective minor principal stress.

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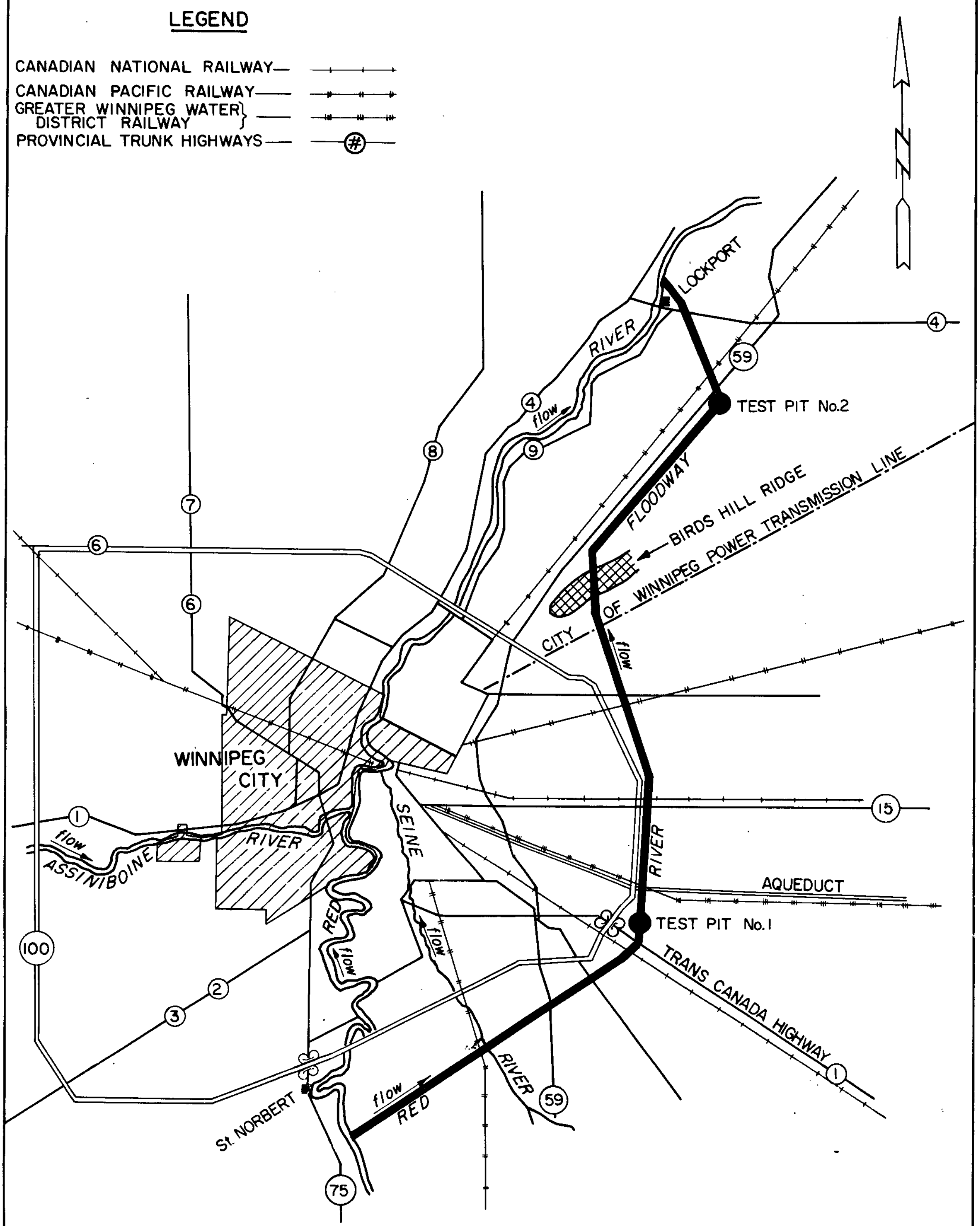


FIGURE 1 - RED RIVER FLOODWAY LOCATION PLAN

SCALE: 0 1 2 3 4 MILES

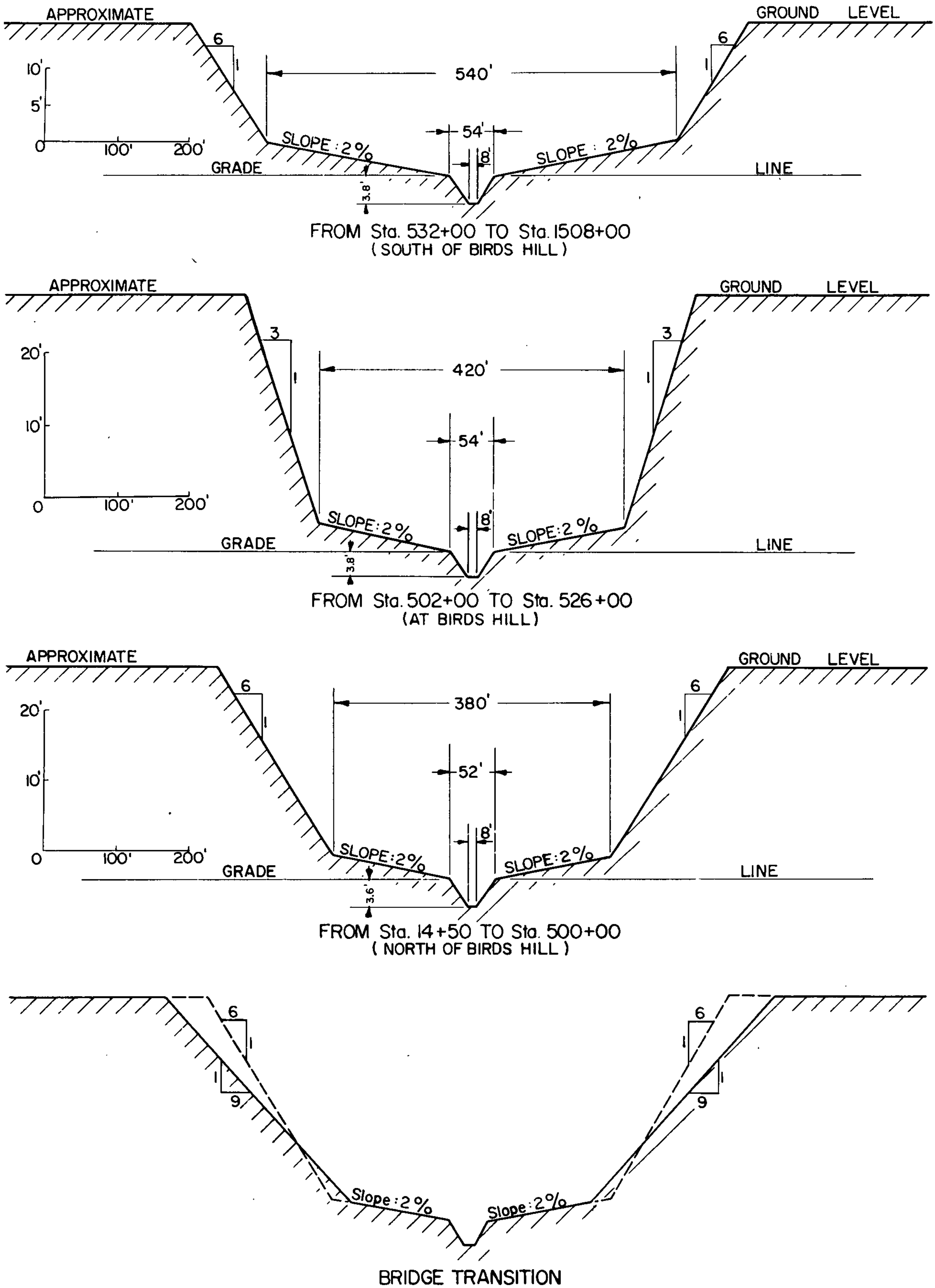


FIGURE 2 - RED RIVER FLOODWAY
TYPICAL CROSS-SECTIONS

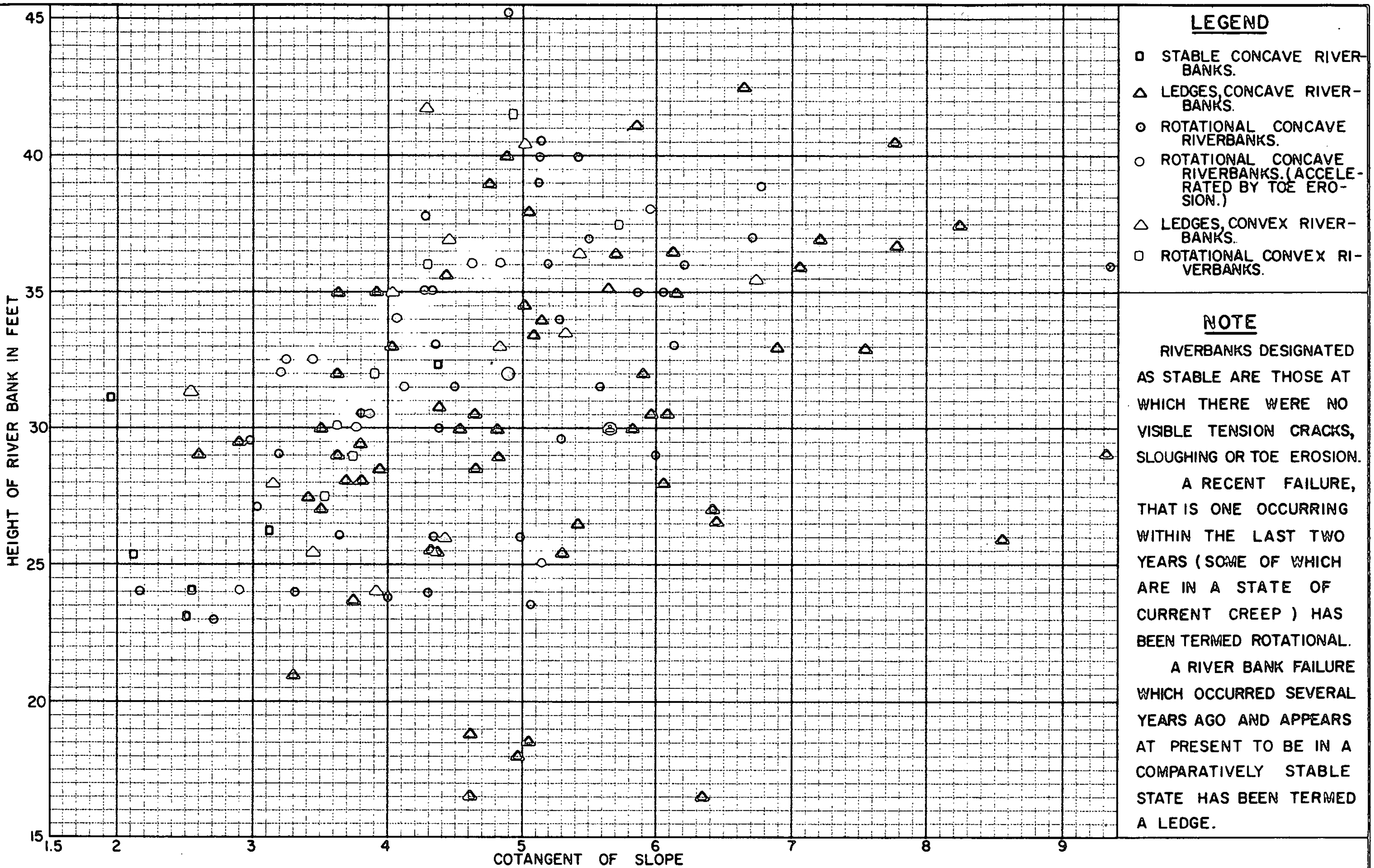
SCALE : AS SHOWN

SOIL TYPE Nº	WATER CONTENT %	VOID RATIO	DEGREE OF SATURATION %	DRY DENSITY P.C.F.	WET DENSITY P.C.F.	UNCONFINED COMPRESSION P.S.F.	SENSITIVITY	GRAIN SIZE DISTRIBUTION, %					ATTERBERG LIMITS		
								CLAY	SILT	SAND 200-40	SAND 40-10	STONE 10 +	LIQUID LIMIT	PLASTIC LIMIT	PLASTI-CITY INDEX
1	39.8	—	—	—	—	—	—	—	—	—	—	—	—	—	—
2	31.5	0.950	89.7	87.3	113.6	3161	—	57	39	4	0	0	69	24	45
3	42.9	1.171	93.8	77.0	113.0	2026	1.37	68	26	5	0	1	85	27	58
4	51.0	1.241	93.3	75.2	108.4	1696	2.59	65	25	6	1	3	78	25	53
5	28.6	—	—	98.0	124.6	1069	2.78	—	—	—	—	—	36	13	23
6	11.9	0.517	93.4	123.6	139.4	1882	—	20	39	17	8	16	19	12	7
7	10.4	0.411	92.5	118.4	142.6	1889	—	17	34	14	5	30	15	13	2
8	9.9	0.246	91.6	136.7	144.3	10050	—	—	—	—	—	—	16	12	4
9	9.3	—	—	132.5	144.2	11688	—	—	—	—	—	—	15	12	3
10	8.8	0.313	92.8	128.3	142.1	2280	—	—	—	—	—	—	15	12	3
11	8.4	0.181	100.0	142.7	153.5	6898	—	—	—	—	—	—	—	—	—

NOTE: ALL SHOWN VALUES ARE AVERAGE.

SOIL TYPE No.	GENERAL SOIL DESCRIPTIONS
1	TOP SOIL.
2	CLAYEY SILT OR SILTY CLAY.
3	BROWN CLAY.
4	GREY CLAY.
5	TRANSITION, GREY CLAY TO NON-CEMENTED GLACIAL TILL.
6	LIGHT GREY NON-CEMENTED GLACIAL TILL.
7	LIGHT BROWN NON-CEMENTED GLACIAL TILL.
8	HIGHLY CONSOLIDATED NON-CEMENTED GLACIAL TILL. (LIGHT GREY OR LIGHT BROWN.)
9	HIGHLY CONSOLIDATED PARTIALLY CEMENTED GLACIAL TILL. (LIGHT GREY OR LIGHT BROWN.)
	BIRD'S HILL SOILS
10	SILTY SAND
11	GLACIAL OUTWASH. (HIGHLY CONSOLIDATED PARTIALLY CEMENTED ROCK FLOUR, CLAY, SILT, SAND AND GRAVEL.)

FIGURE 3 - RED RIVER FLOODWAY SOIL PROPERTIES



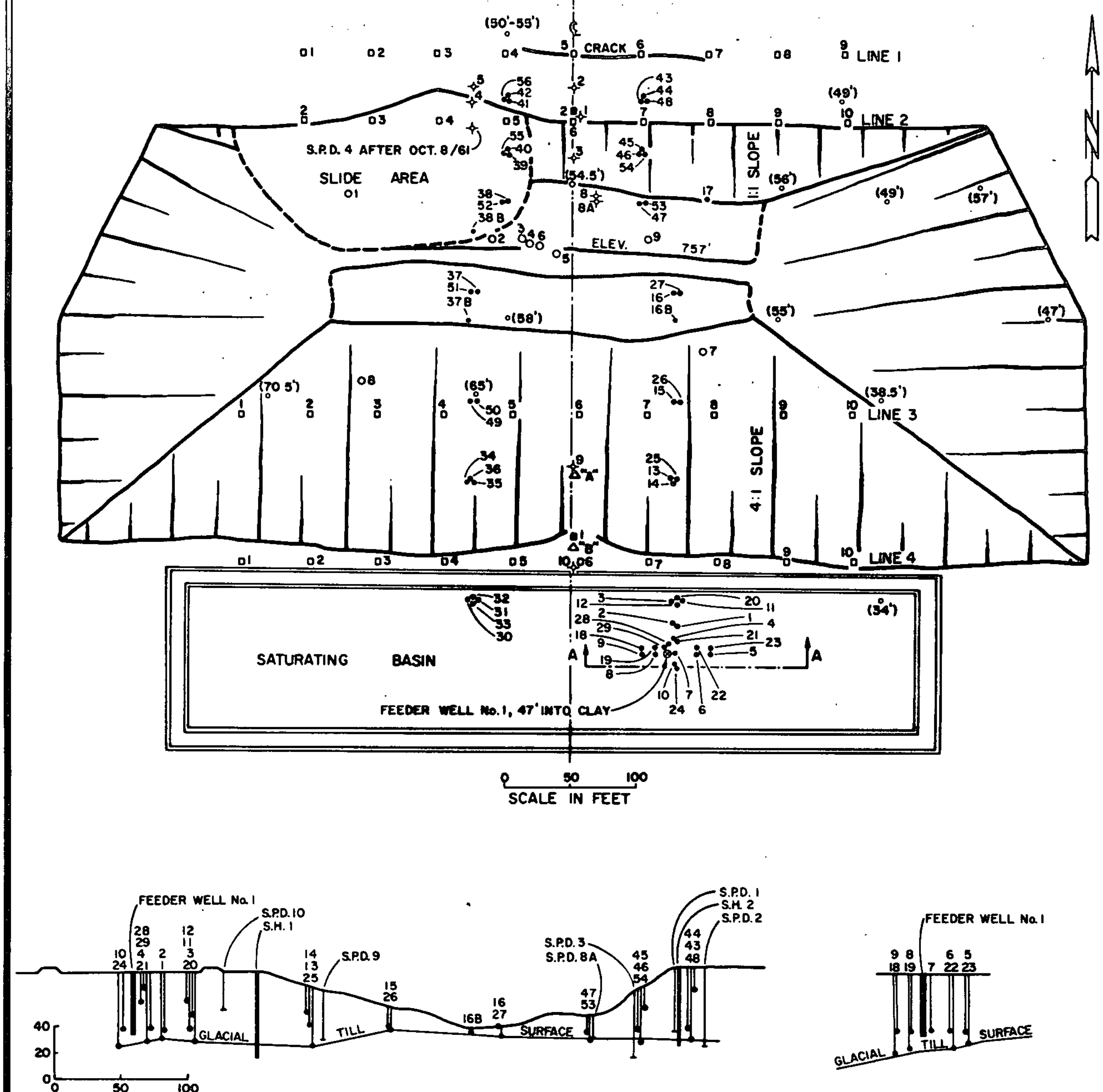
NOTE

RIVERBANKS DESIGNATED AS STABLE ARE THOSE AT WHICH THERE WERE NO VISIBLE TENSION CRACKS, SLOUGHING OR TOE EROSION.

A RECENT FAILURE, THAT IS ONE OCCURRING WITHIN THE LAST TWO YEARS (SOME OF WHICH ARE IN A STATE OF CURRENT CREEP) HAS BEEN TERMED ROTATIONAL.

A RIVER BANK FAILURE WHICH OCCURRED SEVERAL YEARS AGO AND APPEARS AT PRESENT TO BE IN A COMPARATIVELY STABLE STATE HAS BEEN TERMED A LEDGE.

FIGURE 4 - ASSINIBOINE AND RED RIVER BANKS IN METROPOLITAN WINNIPEG - HEIGHT vs. SLOPE



TYPICAL CROSS-SECTION
(TAKEN APPROXIMATELY ALONG CENTRE LINE)
NOTE: NUMBERS WRITTEN ABOVE GROUND LEVEL REPRESENT PIEZOMETER IDENTIFICATION AT THE SITE, BY DEPTH.

SECTION A-A

NOTES

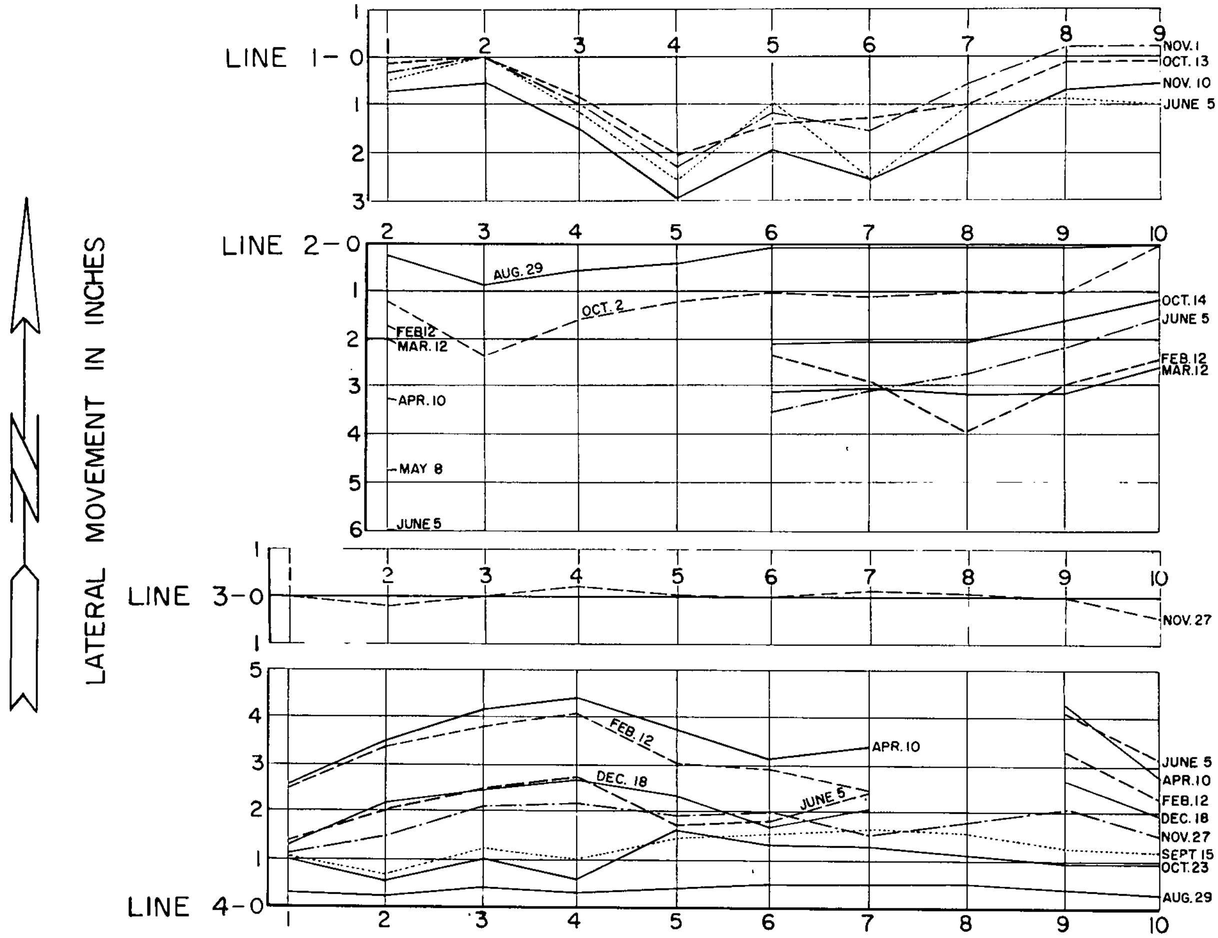
1- ALL INSTRUMENTS IN THE ZONE MARKED : "SLIDE AREA" WERE DESTROYED BY THE SLIDE OF OCTOBER 8, 1961.

2- NUMBERS SHOWN THUS : (54') ARE APPROXIMATE DEPTHS FROM ORIGINAL GROUND SURFACE TO TOP OF NONCEMENTED GLACIAL TILL.

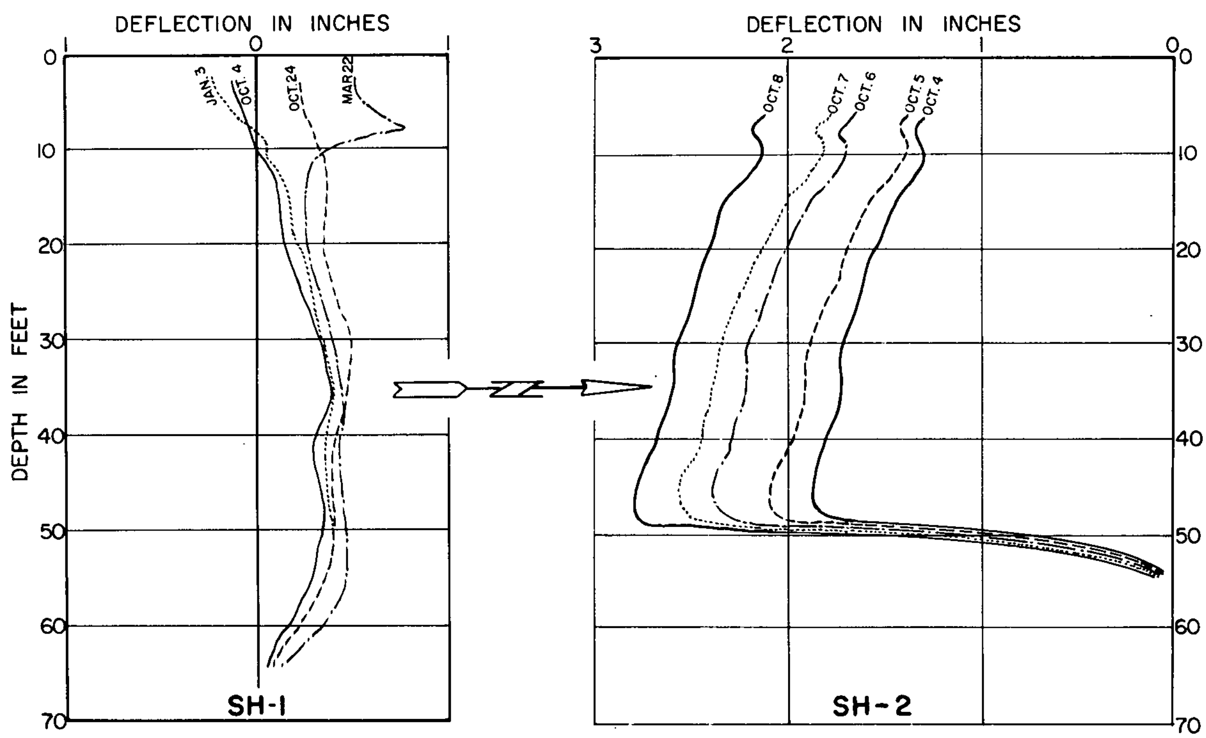
LEGEND

WELL TO DRAIN TILL ———— o
METHYLENE BLUE FROST DEPTH INDICATOR ———— △
SLOPE INDICATOR HOLE ———— ▪
PIEZOMETER ———— •
SIDE PLANE DETECTOR ———— +
TEST HOLE ———— •
ALIGNMENT HUB ———— □

FIGURE 5 - RED RIVER FLOODWAY TEST PIT
PLAN AND SECTIONS SHOWING INSTRUMENTATION



LATERAL MOVEMENT OF ALIGNMENT HUBS

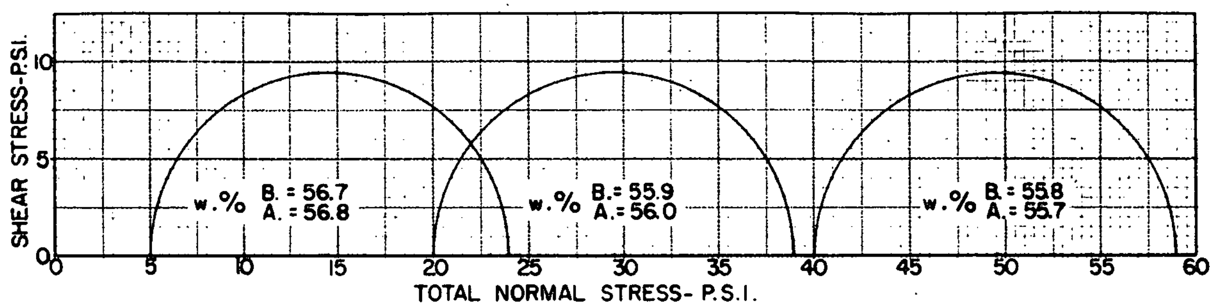


DEFLECTION OF SLOPE INDICATORS

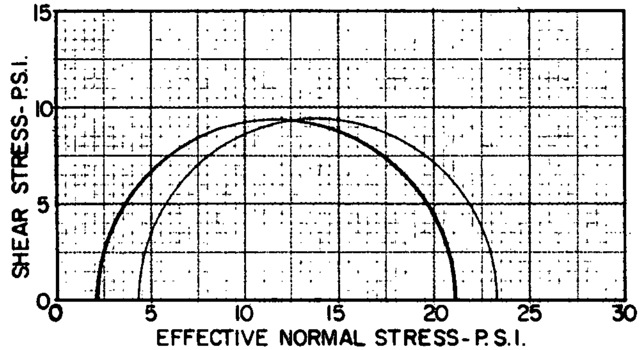
GENERAL NOTE: ALL DATES SHOWN ARE FROM AUGUST 29-1961 TO JUNE 5-1962.

FIGURE 6-RED RIVER FLOODWAY TEST PIT
SLOPE MOVEMENTS

$\overline{Q}$ TEST TOTAL STRESS



$\overline{Q}$ TEST EFFECTIVE STRESS

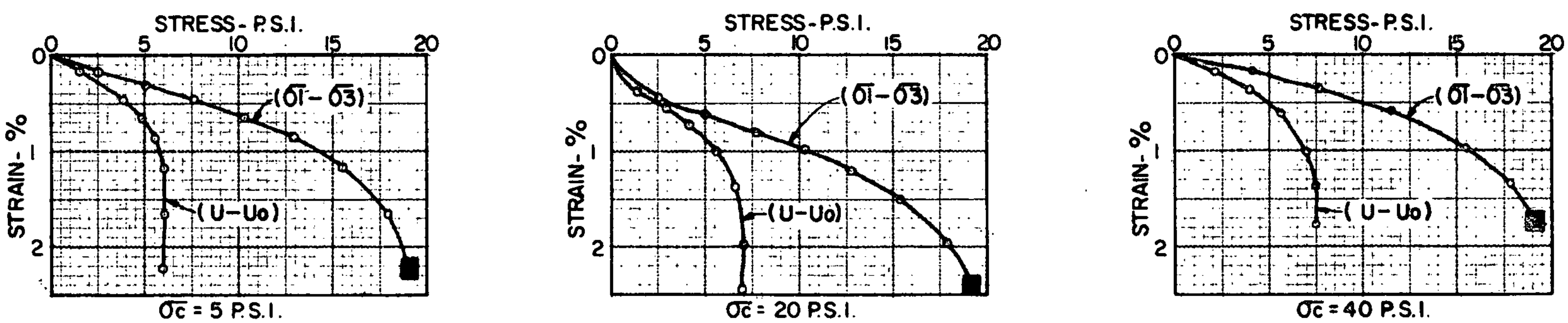


$\overline{Q}$ TESTS AT 30'

AVERAGE WATER CONTENT = 56.1 %
AVERAGE DRY DENSITY = 67.8 P.C.F.
LIQUID LIMIT = 95
PLASTIC LIMIT = 21
PLASTICITY INDEX = 74
INITIAL PORE PRESSURES = −3.1, 11.0, 28.2 P.S.I.

NOTE:
FAILURE CRITERION USED WAS: $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$ = MAXIMUM.

B. = WATER CONTENT BEFORE TEST.
A. = WATER CONTENT AFTER TEST.
$(\overline{\sigma_1}-\overline{\sigma_3})$ = DEVIATOR STRESS.
$(U-U_o)$ = PORE PRESSURE.
$\overline{\sigma_c}$ = HYDROSTATIC CONFINING PRESSURE.
w% = WATER CONTENT.
■ = STRESS AT WHICH : $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$ = MAXIMUM.

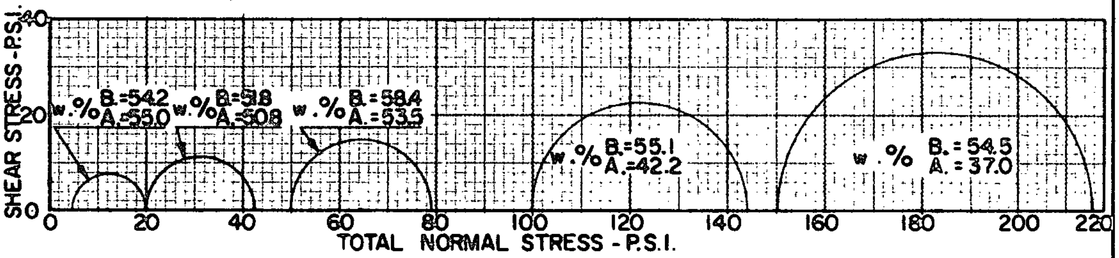


STRESS STRAIN RELATION FOR $\overline{Q}$ TESTS AT 30'

FIGURE 7 - RED RIVER FLOODWAY TEST PIT - $\overline{Q}$ TESTS RESULTS FROM 30' DEPTH

$\bar{R}$ TEST

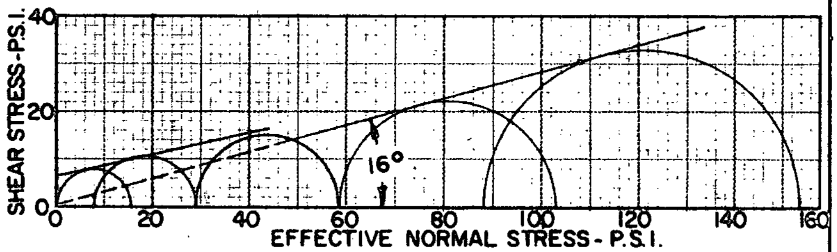
TOTAL STRESS



$\bar{R}$ TEST EFFECTIVE STRESS

C' = 6.5 P.S.I.

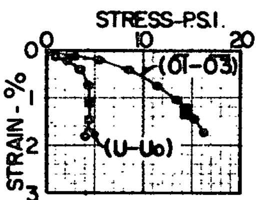
Ø = 12.0°



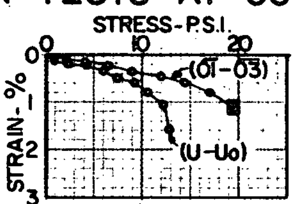
w% = WATER CONTENT.
B. = WATER CONTENT BEFORE TEST.
A. = WATER CONTENT AFTER TEST.
$(\sigma_1 - \sigma_3)$ = DEVIATOR STRESS.
U-Uo = PORE PRESSURE.
σ_c' = EFFECTIVE CONSOLIDATION PRESSURE.
C' = EFFECTIVE COHESION INTERCEPT.
Ø' = EFFECTIVE ANGLE OF SHEARING RESISTANCE OR FRICTION.

AVERAGE WATER CONTENT = 54.8 %
AVERAGE DRY DENSITY = 68.4 P.C.F.
LIQUID LIMIT = 95
PLASTIC LIMIT = 21
PLASTICITY INDEX = 74
NOTE: FAILURE CRITERION USED WAS $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$ = MAXIMUM

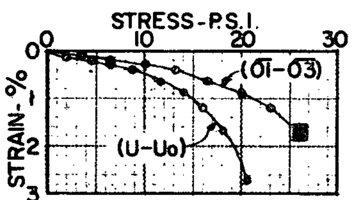
$\bar{R}$ TESTS AT 30'



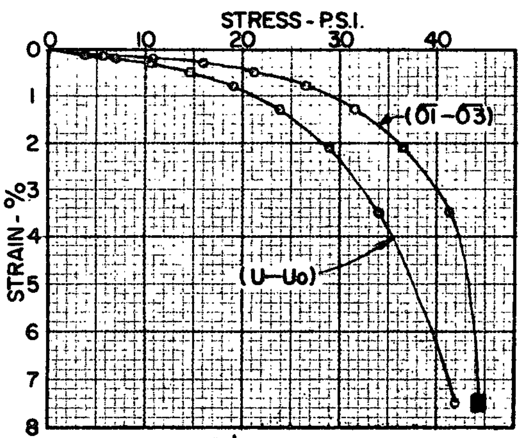
σ_c' = 5 P.S.I.



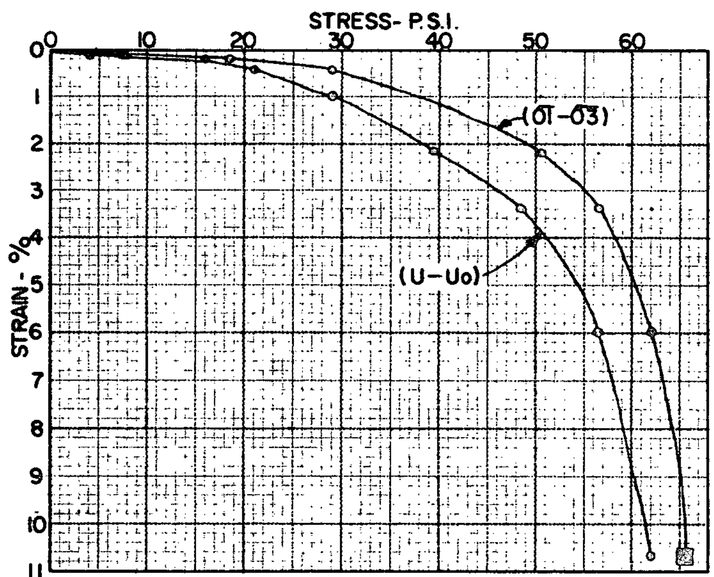
σ_c' = 20 P.S.I.



σ_c' = 50 P.S.I.



σ_c' = 100 P.S.I.

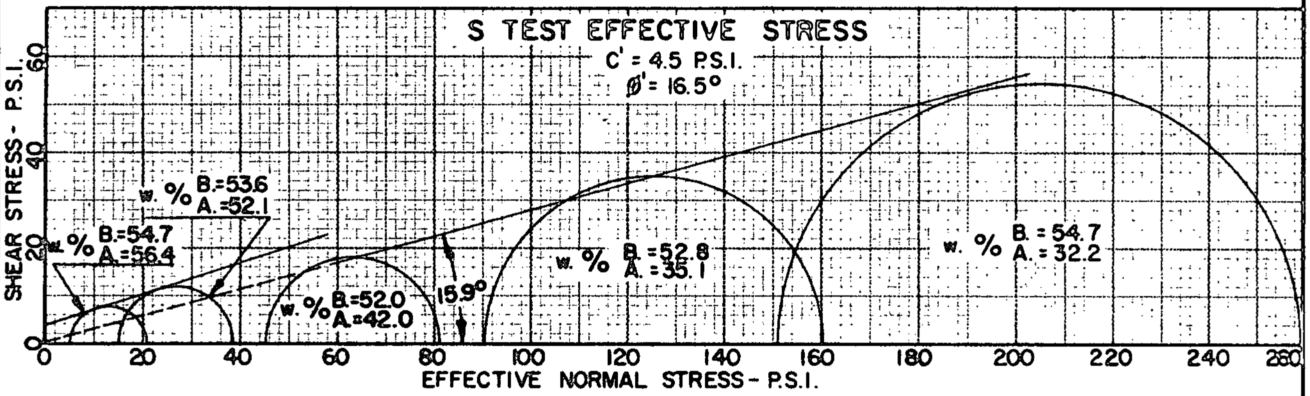


σ_c' = 150 P.S.I.

NOTE:
STRESS AT WHICH $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$ = MAXIMUM SHOWN THUS: ■

STRESS STRAIN RELATION FOR $\bar{R}$ TESTS AT 30'

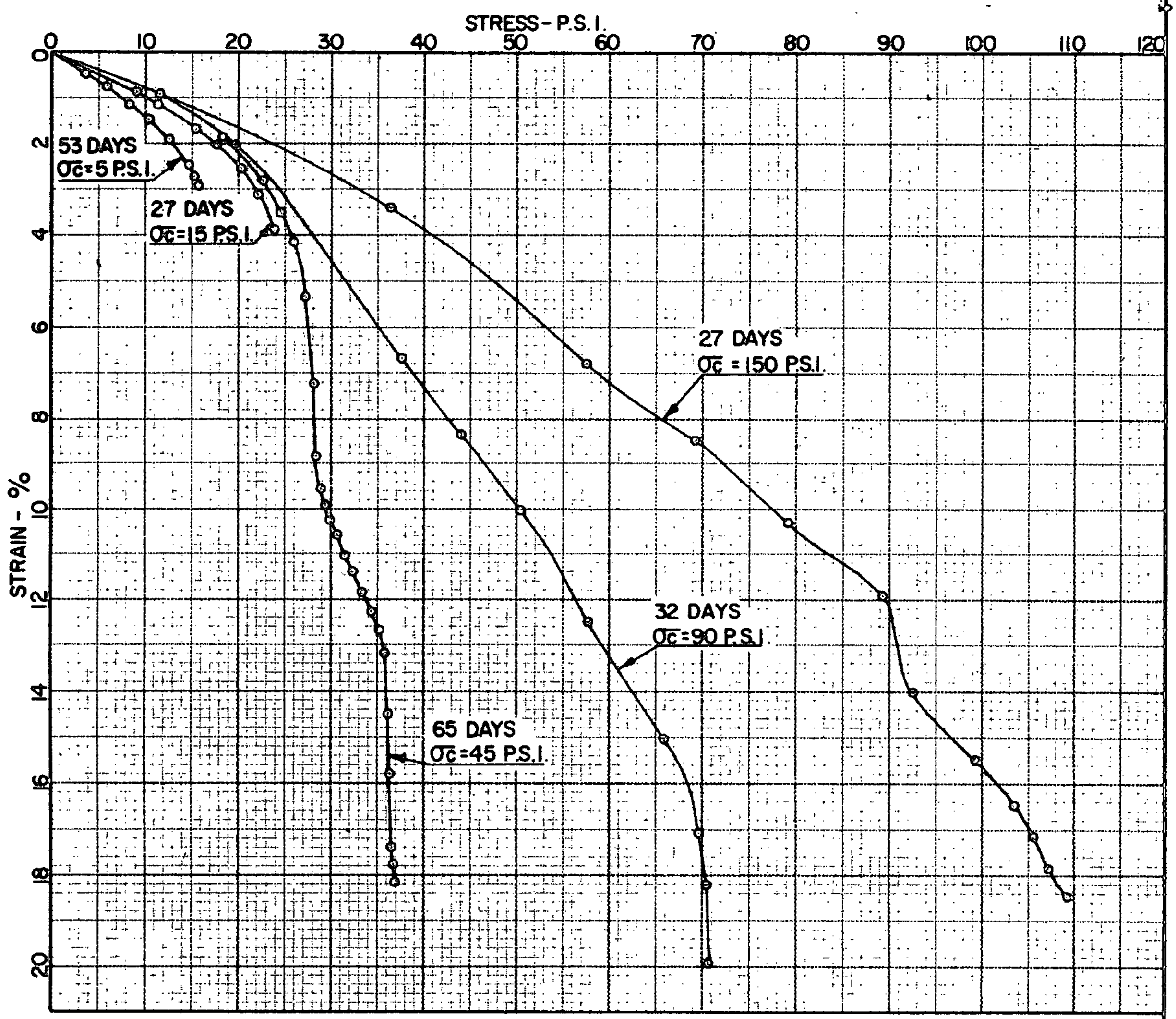
FIGURE 8 - RED RIVER FLOODWAY TEST PIT $\bar{R}$ TESTS RESULTS FROM 30' DEPTH



AVERAGE WATER CONTENT = 53.6%
AVERAGE DRY DENSITY = 69.4 P.C.F.
LIQUID LIMIT = 95
PLASTIC LIMIT = 21
PLASTICITY INDEX = 74
NOTE: FAILURE CRITERION USED WAS: $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$ = MAXIMUM

w% = WATER CONTENT.
B. = WATER CONTENT BEFORE TEST.
A. = WATER CONTENT AFTER TEST.
C' = EFFECTIVE COHESION INTERCEPT.
Ø' = EFFECTIVE ANGLE OF SHEARING RESISTANCE OR FRICTION.
σ_c = HYDROSTATIC CONFINING PRESSURE.

S TESTS AT 30'



STRESS STRAIN RELATION FOR S TESTS AT 30'

FIGURE 9 - RED RIVER FLOODWAY TEST PIT S TESTS RESULTS FROM 30' DEPTH

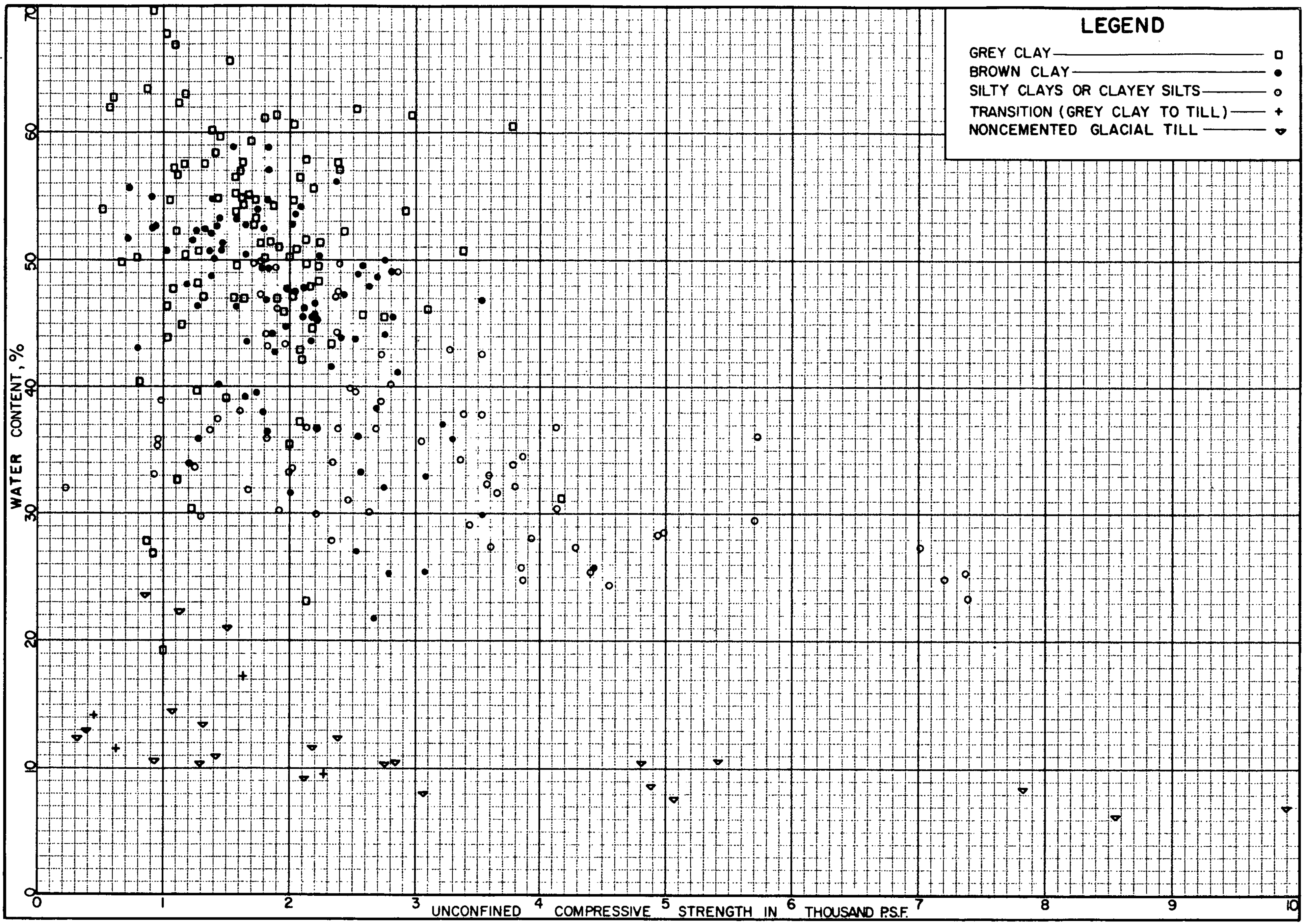


FIGURE 10-RED RIVER FLOODWAY- WATER CONTENT vs. UNCONFINED COMPRESSIVE STRENGTH

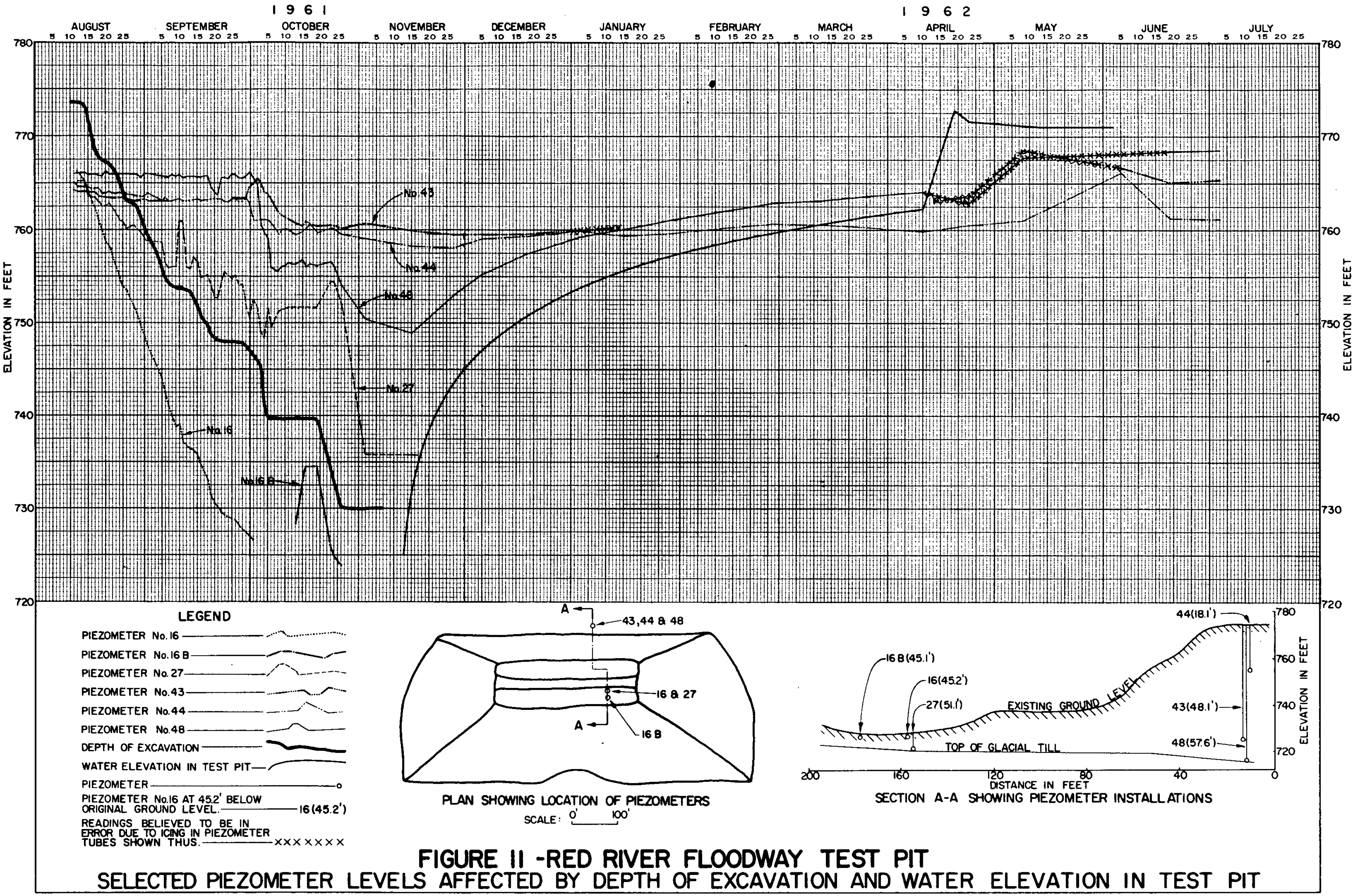


FIGURE 11 -RED RIVER FLOODWAY TEST PIT
SELECTED PIEZOMETER LEVELS AFFECTED BY DEPTH OF EXCAVATION AND WATER ELEVATION IN TEST PIT