BIOGRAPHY

R.A. Widger obtained his Bachelor of Science degree in civil engineering from the University of Saskatchewan, Saskatoon in 1973. He spent the summer of 1973 working at the University of Saskatchewan, Saskatoon doing research and developing a computer program for the Slope Stability Analysis. He continued research at the university and obtained his Master of Science degree in 1976. He spent one year as a geotechnical engineer with the Saskatchewan Department of Highways in Regina and in May, 1976 moved to Prince Albert as District Materials Engineer for the Department of Highways.

BIOGRAPHY

D.G. Fredlund obtained his bachelor of science degree in civil engineering from the University of Saskatchewan, Saskatoon and his master of science degree in 1964 from the University of Alberta, Edmonton. During the summers 1962 and 1963, he worked with the Division of Building Research, National Research Council of Canada, Saskatoon. From 1964 to 1966 he was employed by R.M. Hardy and Associates Limited, Edmonton. In 1966 he accepted a teaching position in civil engineering at the University of Saskatchewan where he remains to the present. From 1970 to 1972 he completed his doctor of philosophy degree at the University of Alberta. In addition to teaching and research, he is a staff consultant to Ground Engineering Limited, Regina. His research has been primarily directed towards the areas of slope stability and the behavior of unsaturated soils.

STABILITY OF SWELLING CLAY EMBANKMENTS

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ABSTRACT

A common occurrence in cuts or fills of swelling soils is their reduction in strength with time. At the time of compaction, the clay generally has a high matrix suction. Correspondingly, it has a high strength and will stand at relatively steep side slopes. With time, the soil generally tends toward saturation and the matrix suction reduces towards zero. There is a reduction in strength and if the gravitational forces are too large, the slope fails.

During the past several years, numerous cut and fill slopes have been observed in the Regina area of Saskatchewan. Many of these slopes have remained stable for four to six years and then failed. There has been a long history of observations (approximately 20 years) on the Belle Plaine overpass west of Regina. A detailed field and laboratory investigation has been conducted on the Belle Plaine site.

Undisturbed block and tube samples from the failure planes shortly after failure have been tested using direct shear and triaxial tests with pore pressure measurements. Triaxial tests with pore pressure measurements have also been performed on the 'as-placed' compacted fill. Some matrix suction measurements were made on Regina Clay samples.

With a knowledge of the geometry of the slope and failure plane, the simplified Bishop method of stability analysis was used to back-in on suitable shear strength parameters. The shear strength parameters from the laboratory program are compared with those obtained from the stability analyses. The analyses indicate that the peak shear strength parameters from triaxial tests on the softened Regina Clay (i.e. c' = 0.75 psi (5.2 kN/m^2) and ϕ' = 17.5¼), along with the appropriate pore water pressures give a factor of safety of one for the failed surfaces. The effect of spring thawing appears to produce the condition of most serious pore water pressures.

STABILITE DES REMBLAIS ARGILEUX RENFLES

par: R.A. Widger et D.G. Fredlund

SOMMAIRE DU CONTENU (traduction professionnelle)

Une action fréquente au niveau des fissures et des remplissages des sols renflés, semble être leur perte de résistance au cours des années. Lors du tassement, l'argile possède habituellement un degré d'absorption élevé. La résistance est donc forte et gardera sa forme même dans des pentes relativement escarpées. Avec les années le sol s'imprègne et l'absorption devient nille. Il en résulte une perte de résistance de l'argile et si la force de gravité est tròp puissante, la pente se désintègre.

Au cours des dernières années, plusieurs pentes fissurées et remplies ont été observées dans la région de Régina en Saskatchewan. Plusieurs de ces pentes sont demeurées stables pendant quatre ou six ans pour ensuite s'écrouler. Des observations ont été compilées pendant vingt ans environ sur la voie élevée de Belle Plaine à l'ouest de Régina. Une étude détaillée aété entreprise sur place et en laboratoire relativement à ce site. Des échantillons de roches qui ne s'étaient pas déplacées ont été analysés par cisaillement et examen triaxial incluant des calculs sur la pression des cavités. Ces mêmes tests ont été effectués sur les remplissages qui ont conservé leur aspect original après la désintégration. Quelques calculs ont été effectués relativement au degré d'absorption sur des échantillons d'argile pris à Régina.

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Puisque nous connaissions la géométrie de la pente et de la surface de rupture, la méthode Bishop simplifiée de stabilité a été utilisée afin de corroborer les paramètres de résistance au cisaillement. Les paramètres de résistance obtenus en laboratoire sont comparés aux paramètres obtenus par l'analyse de stabilité. Les analyses démontrent que les paramètres de pointe de résistance au cisaillement tirés des tests triaxiaux sur l'argile ramolie à Régina , c-à-d ($c' = 0.75$ psi (5.2 kN/m^2) et ($\phi' = 17.5°$), ainsi que la pression de l'eau sur les cavités, permettent d'établir un facteur de sécurité de "1" pour la surface rupturée. Le dégel du printemps semble produire un effet considérable sur la pression de l'eau sur les cavités.

STABILITY OF SWELLING CLAY EMBANKMENTS

BY

R.A. Widger and D.G. Fredlund

Introduction

The use of overconsolidated, swelling clays as a construction material commonly presents stability problems (Holtz, 1959; Kassiff and Alpan, 1973). The clays exhibit a relatively high strength at the time of construction; however, their strength generally decreases with time. Although the process behind the loss of strength is quite well understood, there has been limited quantitative procedures advanced to render the problem amenable to solution.

There has been a long history of slope instability in cuts and fills constructed of a swelling clay in the Regina area of Saskatchewan, Canada. Cuts and fills commonly fail four to six years after their construction. The failure is generally a shallow circular slide day-lighting at the toe of the slope.

The long history of slope instability in Regina clay has prompted an in-depth study of the embankments at Belle Plaine, Saskatchewan. This paper presents the history, material characterization and analysis of the landslides at Belle Plaine.

Description of Study Area and Material

The study area involves two overpass embankments located at Belle Plaine, Saskatchewan on the Trans Canada Highway about 27 miles (43.4 Km) west of Regina (Figure 1). Belle Plaine is situated in the Assiniboine River Plaine physiographic region and in the proglacial Lake Regina Basin. The climate of the area is classified as dry subhumid (Ellis et al, 1965). The annual precipitation ranges from 6.3 inches (160 mm) to 23.7 inches (602 mm) with a mean of 14.7 inches (373 mm). The study area at Belle Plaine has a normal mean daily temperature in January of $-15^{o}C$ and in July of $18^{o}C$, the temperature ranges from $-50^{o}C$ to $43^{o}C$. The average depth of frost penetration is about 4 to 8 feet (1.2 to 2.4 m).

The surficial material in the study area is a highly plastic montmorilonite clay known as Regina Clay (Christiansen, 1961). The Regina Clay is highly overconsolidated by desiccation and has slickensided fissures caused by desiccation and frost action. Standard physical tests performed on samples obtained at Belle Plaine showed liquid limits ranging from 68% to 71% and plastic limits ranging from 22% to 24%. Hydrometer analyses showed 92% to 95% of the material smaller than 0.01 millimeters with 63% to 65% clay sizes. The specific gravity was 2.74. A standard AASHO compaction test showed an optimum water content of 28.4% and a maximum dry density of 90.5 pounds per cubic foot (1449 kg/m^3) (Figure 2).

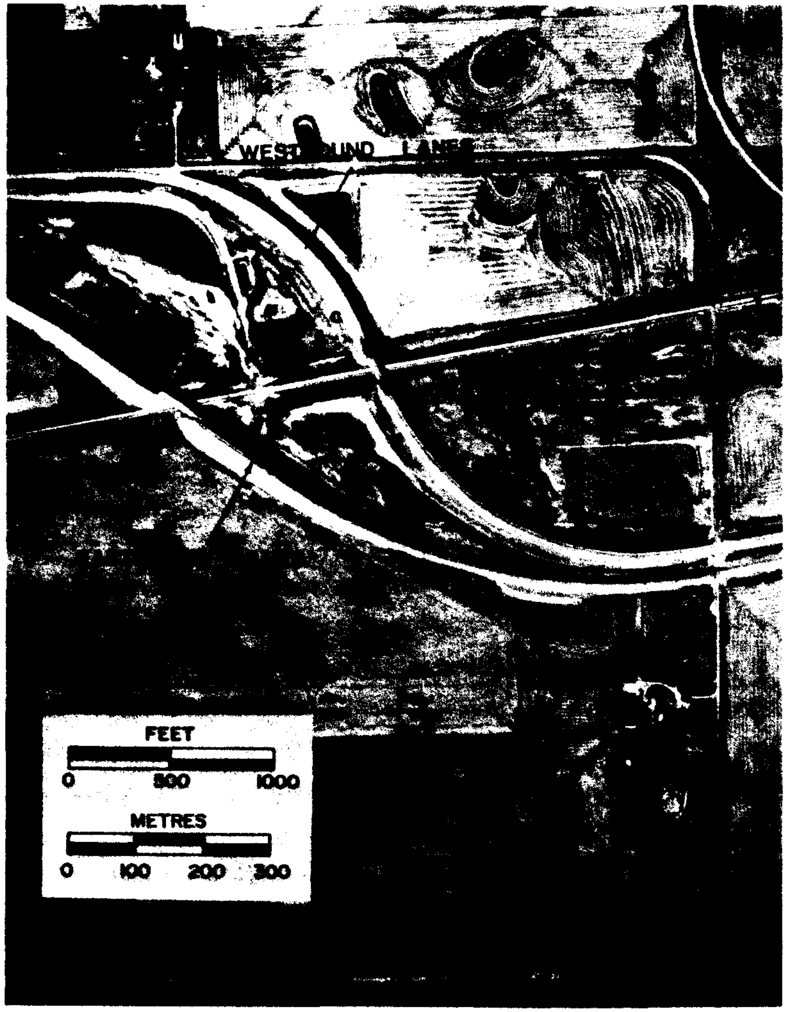

FIG. I OVERPASS LAYOUT AT BELLE PLAINE, SASKATCHEWAN

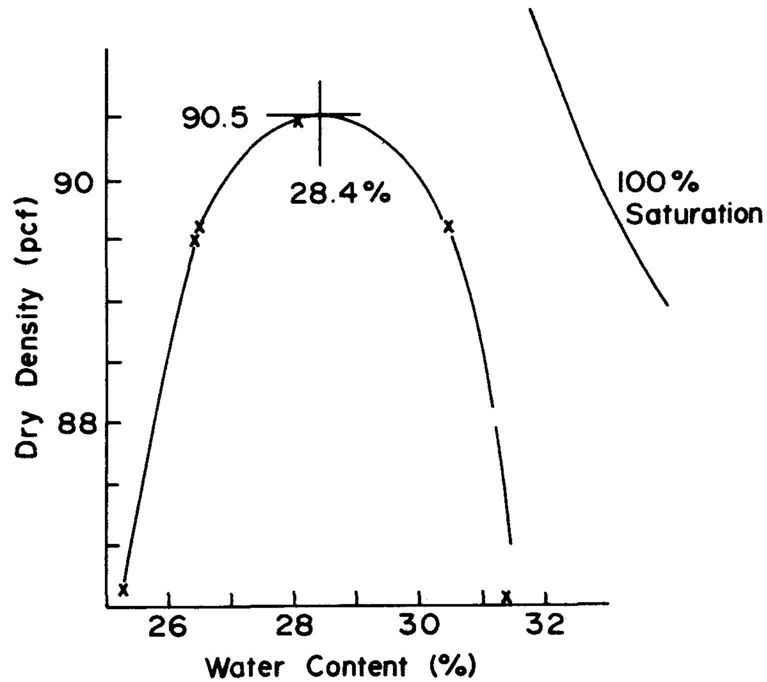

FIG. 2 COMPACTION CURVE FOR REGINA CLAY

History and Field Investigation of the Belle Plaine Overpass

The original highway overpass at Belle Plaine was built in 1954 and carried traffic in both directions. The embankments were constructed of Regina Clay with 2:1 side slopes and a maximum fill height of about 35 feet (10.7 m). Slides occurred on the embankments in 1959, but few details are available on the extent of sliding and on the repair work performed, other than that the new slopes were approximately 2:1.

In 1965 construction was started on a new embankment for an eastbound lane. During the construction of the eastbound overpass, a deep borrow pit was excavated at the toe of the old fill (Point 'a' in Figure 1). This increased the effective height of the old fill to 50 feet (15.2 m). In 1966 the eastbound overpass was completed with 3:1 side slopes and a maximum fill height of about 30 feet (9.1 m). It was paved the same year and overlaid in 1968.

In the fall of 1968, two landslides occurred on the northeast side of the westbound lanes (Point 'b') and a minor slide occurred west of the overpass on the westbound lanes (Point 'c'). Cracking accelerated in the fill directly above the borrow pit (Point 'd'). In the spring of 1969, a landslide about 300 feet (91 m) long that involved about half the height of the slope occurred above the borrow

pit (Point 'd'). The slide was accompanied by new cracking along the shoulder and in the driving surface. Cracks from 3 to 4 inches (76 to 102 mm) wide opened on the shoulder and cracks 1 inch (25 mm) wide opened in the driving mat.

In the fall of 1969, movements started in the east backslope region under the overpass on the eastbound lanes. The bolts in the expansion joints at the east abutment were bent and the concrete was cracked. The backslopes under the structure were 1.5:1 or steeper and constructed from relatively loose, uncompacted, dumped material. Some of the water from the deck drained onto this steep, poorly drained slope. The drainage conditions were subseqeuntly improved.

During the summer of 1970, the slide above the borrow pit (Point 'd') was repaired and part of the borrow pit (Point 'a') was filled. With the repair fill less than 10 feet (3 m) above the natural ground, tension cracks started to appear near the base of the approximately 4:1 slope. Fill was halted, but movements continued on a slide block 200 to 300 feet (61 to 91 m) long. The slide extended about 100 feet (30 m) inward from the toe of the fill and comprised a volume of about 15,000 cubic years (11,500 m^3). The vertical displacement was 3 to 4 feet (0.9 to 1.2 m) at the scarp and the horizontal displacement appeared to be similar. The toe of the slide pushed out approximately 15 feet (4.6 m) below natural ground level or 2 feet (0.6 m) above the water level in the old borrow pit. The rear scarp roughly followed the top edge of the old borrow pit sides (Point 'a') that were at a 2:1 slope. The slide plane appeared to be between the old borrow pit (Point 'a') slope and the clay fill (Point 'd').

On June 23, 1970, the Saskatchewan Department of Highways used a rotary drill to advance a testhole through the fill on the east abutment of the westbound lanes (Testhole #3-70, Figure 1). The upper 35 feet (10.7 m) was a highly plastic clay fill. The average liquid limit was 80 percent, the average plastic limit 42 percent and the average shrinkage limit 12 percent. The average water content in the upper 15 feet (4.6 m) was 34.5 percent, decreasing with depth to 29 percent. The average unconfined compressive strength of the fill material was 15.2 psi (105 kN/m^2). Insitu Regina Clay was encountered from 35 feet (10.7 m) to a depth of approximately 55 feet (16.8 m). The natural material had an average liquid limit of 86 percent and an average plastic limit of 35 percent. The average natural water content was 24.4 percent and the average unconfined compressive strength was 70 psi (480 kN/m^2). A dark grey brown, sandy clay till was encountered from 55 feet (16.8 m) to the bottom of the testhole (i.e. 76 feet or 23.2m).

In the spring of 1973, two landslides occurred on the northwest face of the eastbound lane (Point 'e'). The Authors visited the site for the first time in the fall of

1973. The landslides were surveyed and their locations are shown in Figure 3. Figure 4 is a cross-section along line A showing the scarp and the toe of the slope failure. Line B ran through the centre of the larger slide which appeared to be a small rotational slide at the toe of the fill and a series of sloughs above the original slide. Figure 5 shows the landslides along lines A and B, viewed perpendicular to the embankment.

During the spring of 1974, numerous trips were made to Belle Plaine to obtain samples and note further changes. No movements had occurred since the previous fall, although some new tension cracks had appeared at the top of the slope (Point 'e'). Hand auger holes were drilled on the slope to obtain undisturbed tube samples and water contents at various depths on May 15, 1974. The locations of testholes 1A, 2D, 3E, 4D, 5A, and 6A are shown on Figure 3. Figure 6 shows the variation in water content with depth below ground surface. The average of 48 water contents on the clay fill was 37.6 percent. All water contents are above optimum with the maximum values as high as 45 percent. The average of 5 unconfined compression tests in the upper 6 feet (1.8 m) was 14 psi (97 kN/m^2). Frost was encountered at depths ranging from 2.5 feet (0.8 m) at the top of the slope to 6.5 feet (2.0 m) in the middle and 3.5 feet (1.1 m) at the toe of the slope. The holes were left open and monitored until they were inundated during a heavy rainfall two days later. Very little water

5a Slides along Lines A and B at Belle Plaine.

5b Slide along Line A at Belle Plaine.

FIG. 5 FRONTAL VIEW ALONG LINE A AND B.

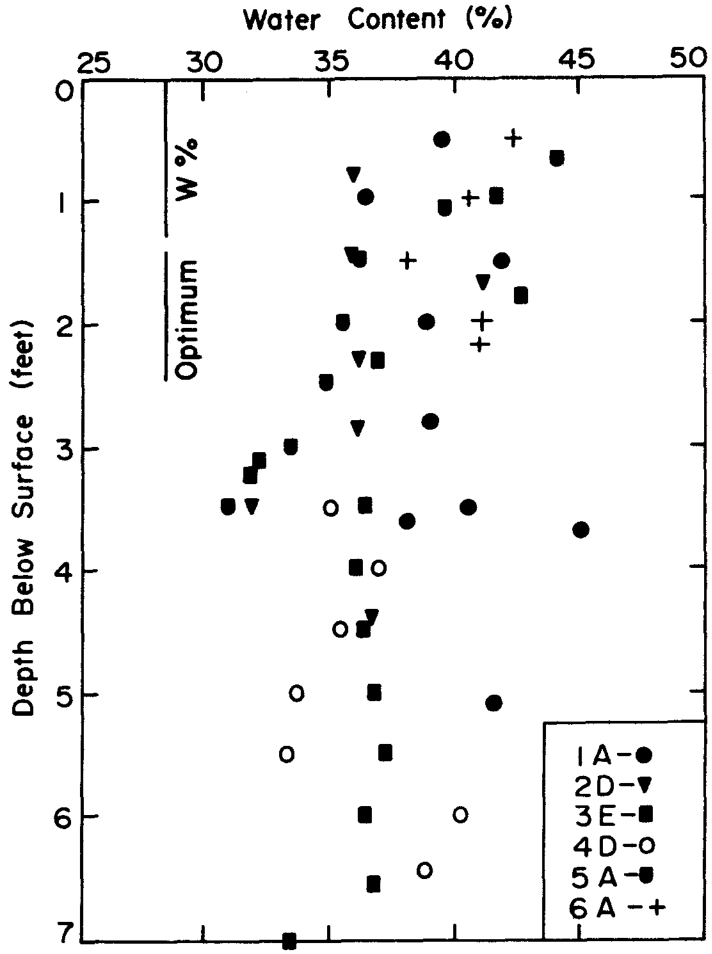

FIG. 6 WATER CONTENT VERSUS DEPTH BELOW SURFACE FOR HAND AUGER HOLES

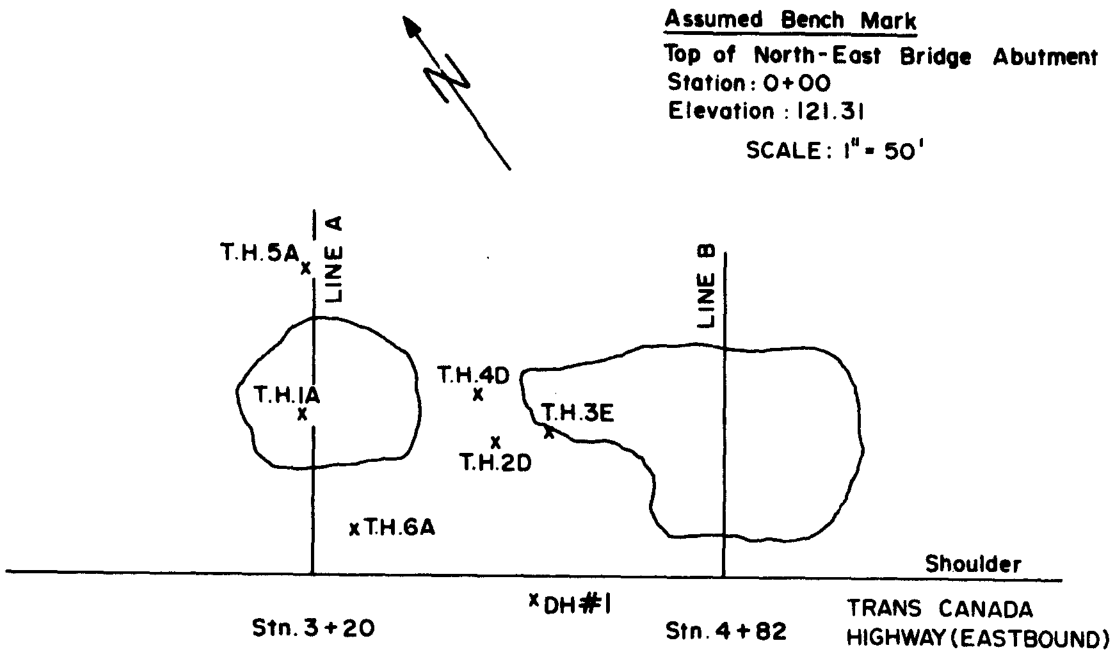

FIG. 3 SLIDE AREAS AND TEST HOLE LOCATIONS, BELLE PLAINE

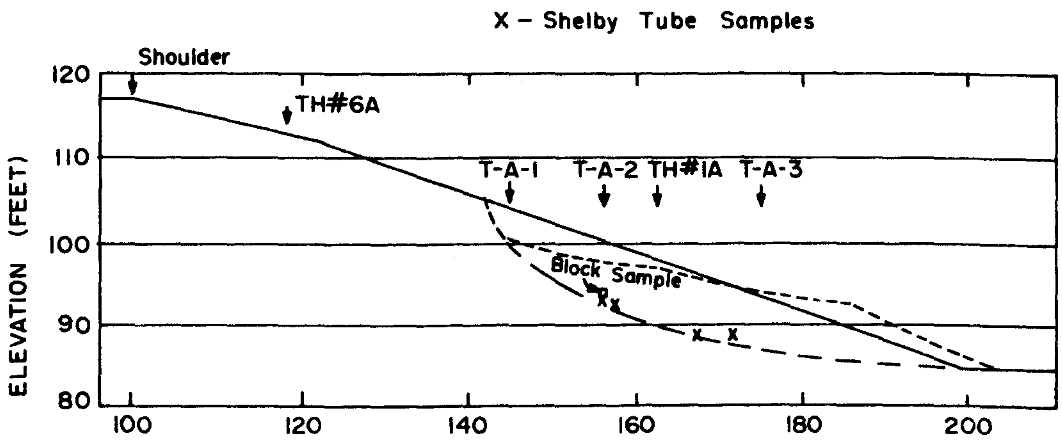

FIG. 4 CROSS SECTION ALONG LINE A

accumulated in testholes at the top and below the slope, while the testholes on the slope had as much as 2 feet (0.6 m) of water collect in a 24-hour period and were within 3 feet (0.9 m) of the surface of the slope. Details of the water levels are presented in Table 1.

A backhoe was used on May 16, 1974 to make a trench approximately 8 feet (2.4 m) deep through the slide along line A. The scarp could be readily observed for the first 4 feet (1.2 m) into the fill. Due to the wet, fissured nature of the clay, it was difficult to accurately determine the remainder of the slip surface. The highly disturbed zone indicated that the base of the slide was approximately 7 to 8 feet (2.1 to 2.4 m) below the surface. The failure surface was essentially circular with the exception of the base portion which was slightly composite due to the thaw front. The toe slid out horizontally over the natural ground. Ice lens were encountered in some of the fissures and the backhoe dragged out large, frozen chunks of clay. A definite change in the soil structure from "nuggetty" to "massive" was noted between the top and bottom of the trench. Water content samples were taken at 6 inch (152 mm) depth intervals along the side of the trench. From three profiles (T-A-1, T-A-2, T-A-3), along the trench (Figure 4), the average of 34 water contents was 36.6 percent (Figure 7).

TABLE 1

Summary of Water Levels in Hand Auger Holes

		1A	2D	3E	4D	5A	6A
Drilled	Date	14/5/74	14/5/74	15/5/74	15/5/74	15/5/74	15/5/74
	Time	12:00	4:00	11:30	12:00	2:00	2:30
Date	Time	Water Level Below Ground Surface (ft.)					
Depth of Hole		5.8	4.4	7.0	6.7	3.5	2.2
15/5/74							
	9:00	4.9	3.9				
	1:00		3.8	6.6	trace		
	2:15				6.7		
	3:15				6.7		
	4:30	4.8	3.6	6.4	6.5	None	None
16/5/74							
	8:30	4.1	3.3	5.3	5.5	None	Trace
	4:00		3.1	5.0	4.9	None	2.1

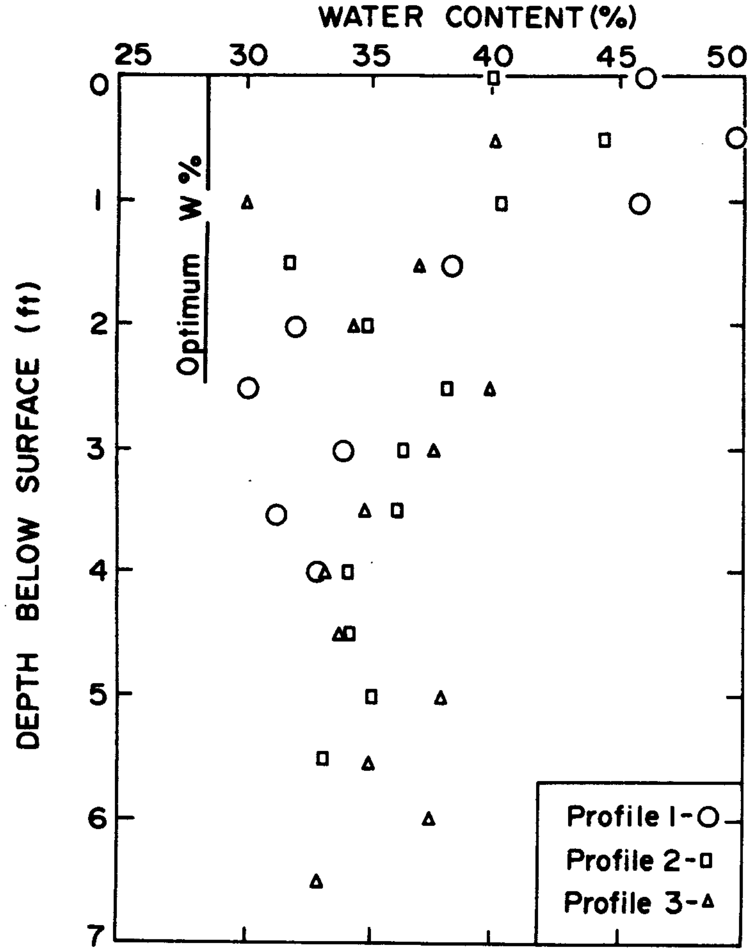

FIG. 7 WATER CONTENT VERSUS DEPTH BELOW SURFACE FROM TRENCH

A block sample was hand trimmed from the side of the trench at approximately the shear surface. The fissured structure of the clay caused small pieces of the block to break away during the trimming process. The block was trimmed to about 10 inches (254 mm) square, wrapped in cheese cloth, waxed and fitted into a wooden box which was then filled with wax. Thin-walled spoon samples were pushed into the bottom of the trench using the backhoe.

On May 21, 1974, 2 inches (51 mm) of rain fell on the area and three new slides occurred along the eastbound lane of the study area. The slide along line B now extended almost up to the shoulder and the slide on line A had an additional portion slide into the old scarp. The area between the two existing slides also experienced local failure.

By the late summer of 1974, no new landslides had occurred, but more local failures had taken place above the original slides. The debris in the slide area had become saturated and was difficult to walk across. A crawler tractor attempted to start repair work, but was unable to climb the embankment due to the very soft conditions.

On August 22, 1974, testhole D.H. #1 was drilled from the shoulder of the eastbound lanes on the overpass (Figure 3). The hole was dry augered for the first 10 feet (3 m) with continuous sampling, then wet drilled down to 50 feet (15 m). Thin-walled tube samples were taken at five-foot (1.5m)

intervals. The average water content in the upper 12 feet (3.7 m) was 30 percent. The drill truck sank 6 to 8 inches (150 to 200 mm) into the soft shoulder when it backed off the edge of the pavement.
In the late fall of 1974, the slide debris was removed and replaced with material from the edge of the borrow pit. The side slopes were maintained at about 3.5:1. The new fill material showed some cracking in the spring of 1975. In the early summer of 1975 another large slide occurred. The new slide was a reflection of the previous landslides.

The slide area was left untouched for the summer of 1975 and levelled off and repaired in the late fall with side slopes of approximately 3.5:1.

Testing Program

The soil testing program was aimed at evaluating the stress and strength changes that have occurred between the time of construction and the time of failure. The strength parameters are used to analyse the change in factor of safety with time and in particular, attempt to ascertain the strength parameters best describing the failure conditions.

The water content profiles indicate that although the embankment material was placed near optimum water content conditions (i.e. 28.4 percent), it has increased by approximately 9 percent at the time of failure. The increase in

water content results in a decrease in the matrix suction of the soil and correspondingly, a reduction in strength. Figure 8 shows a collection of matrix suction measurements plotted versus water content for Regina Clay. The tests were performed on a pressure plate apparatus (Pufahl, 1970). Some samples tested were from the Belle Plaine overpass (Widger, 1976) while others were on Regina Clay samples with similar classification properties. Also shown is one matrix suction estimation from a one-dimensional consolidation test performed on a sample compacted near optimum water content. Figure 9 shows the construction to evaluate the state of stress (Fredlund, 1975).

Using mean values, the matrix suction near optimum water content is 39 psi (270 kN/m^2). The average water content at failure was 37.6 percent which corresponds to a matrix suction of approximately 8 psi (55 kN/m^2). The reduction in matrix suction is believed to be first caused by a freeze-thaw cycle that establishes a nuggetty or fissured macro-structure (Mickleborough, 1970). Subsequent ingress of water into the macro-structure causes a further reduction in matrix suction.

Table II summarizes the shear strength testing program. The undrained shear strength of Regina Clay was evaluated from unconfined compression tests on laboratory and field compacted samples at various water contents (Figure 10). The scatter is believed primarily due to the effects of secondary structure in the samples. Using the best-fit curve, the undrained shear

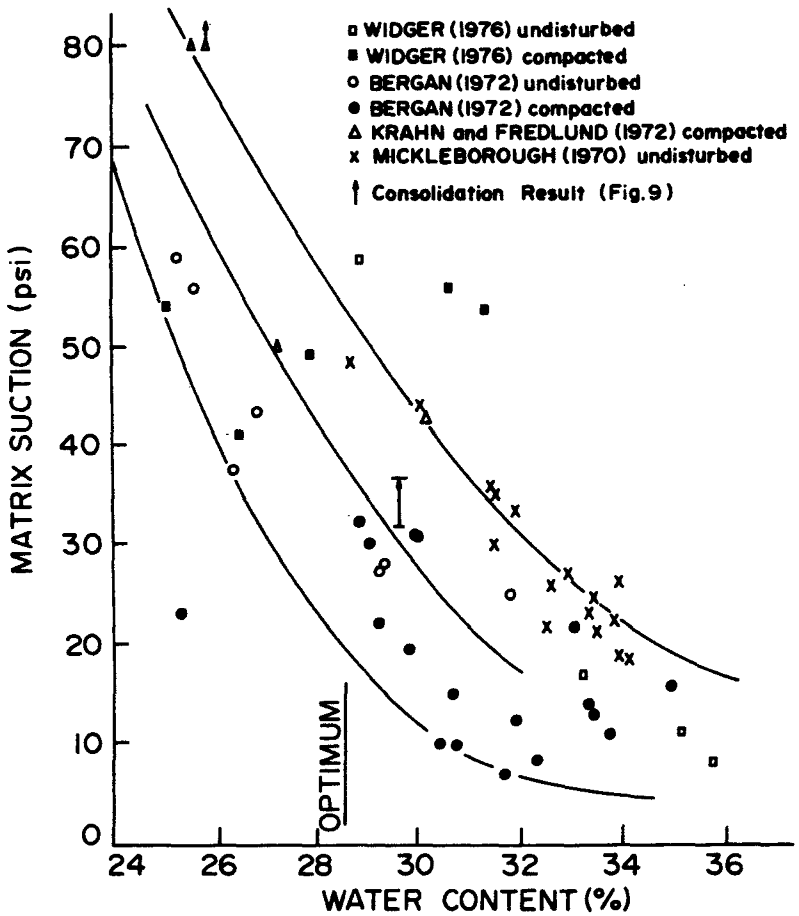

FIG. 8 MATRIX SUCTION VERSUS WATER CONTENT FOR REGINA CLAY

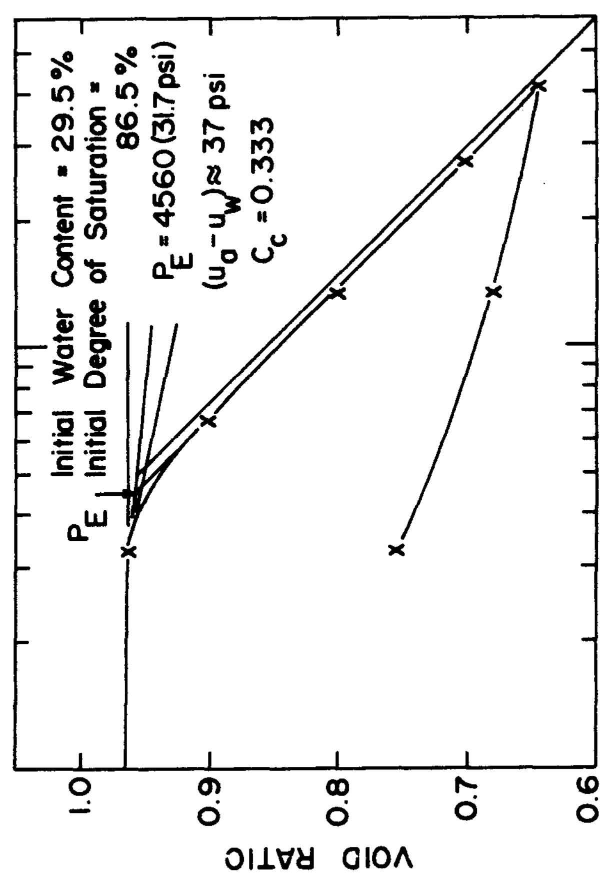

FIG. 9 CONSOLIDATION CURVE FOR COMPACTED REGINA CLAY

TABLE II

Shear Strength Testing Program

SHEAR STRENGTH PARAMETER EVALUATION

Undrained Shear Strength S_u		Effective Strength c' ϕ' (Peak)		Residual Strength* c, ϕ
(Unconfined Compression)		(Triaxial Compression)		(Direct Shear)
End of Construction	Time of Failure	End of Construction	Time of Failure	

* Strength parameters corresponding to high strains (approximately 12%) in the triaxial test are also presented.

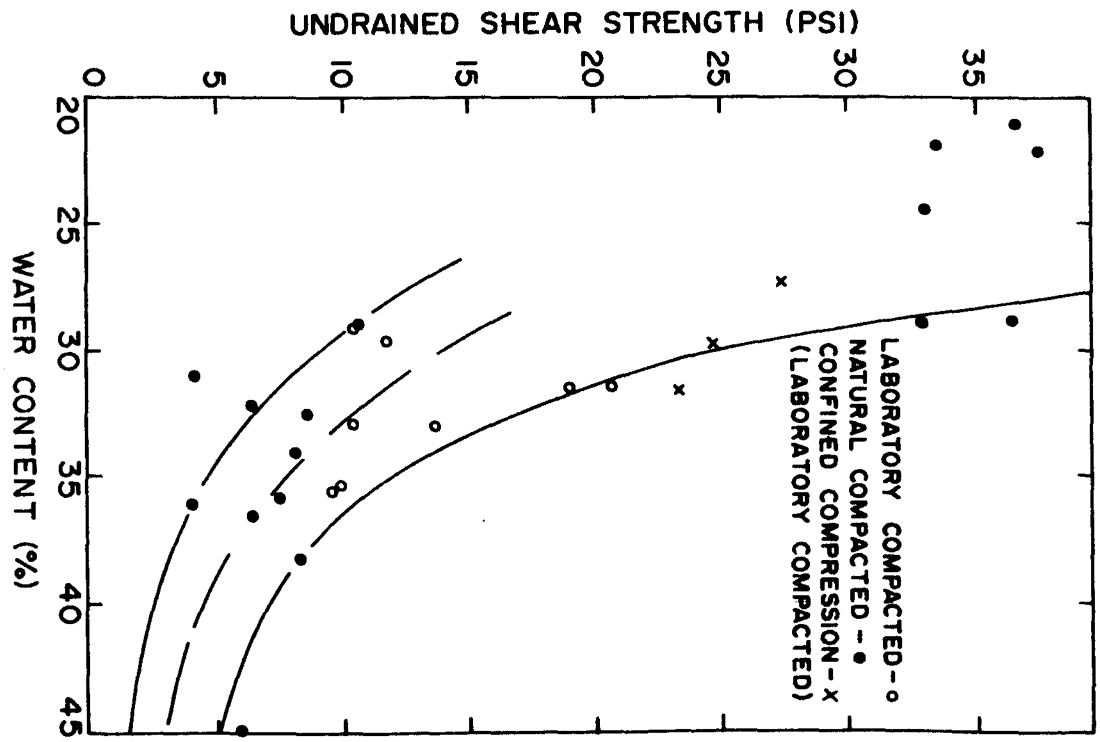

FIG. 10 WATER CONTENT VERSUS UNDRAINED SHEAR STRENGTH.

strength at the time of construction was approximately 17 psi (117 kN/m^2). At the time of failure, the undrained shear strength would be approximately 6 psi (41 kN/m^2). The lower limit curve for all values gives a shear strength of approximately 3.5 psi (24 kN/m^2) at the time of failure.

All consolidated, undrained triaxial tests with porewater pressure measurements were performed on 1½ inch (38 mm) diameter specimens. The rate of strain was approximately 0.04 inches (1 mm) per hour. Table III summarizes the triaxial data from tests performed on natural, undisturbed Regina Clay and laboratory compacted samples. The testing was carried out in 1970 in conjunction with an overpass on the Trans Canada Highway bypass south of Regina. The classification properties indicate that the Regina Clay was similar to that at the Belle Plaine site. The R-23 series of samples were from a depth of 9 feet (2.7 m) (Figure 11), and the R-26 series were from a depth of 16.5 feet (5.0 m) (Figure 12). The third series were backpressured to produce saturation. A summary of total and effective strength parameters corresponding to the peak and large strain conditions is shown in Table IV. The compacted samples corresponding to the end of construction situation indicate a peak effective angle of internal friction of 17.5 degrees and an effective cohesion of 5.5 psi (38 kN/m^2).

TABLE III

Data for Natural, Undisturbed and Compacted Regina Clay

Trial No.	Consolidation Pressure (psi)	Initial Water Content (%)	Final Degree of Sat. (%)	Response Up (%)	Peak Strain (%)	Peak σ_d (psi)	Peak u_w (psi)	High Strain Strain (%)	High Strain σ_d (psi)	High Strain u_d (psi)
R-23										
1	8.0	30.1	114.7	98.5	2.5	24.0	3.4	12.0	14.2	-3.3
2	30.0	30.2	96.1	92.9	4.0	41.0	11.9	12.0	25.1	3.9
3	45.0	30.0	97.3	91.0	5.0	50.9	17.2	12.0	28.7	9.2
4	60.0	30.0	96.7	74.2	6.0	59.3	22.1	12.0	38.2	15.4
R-26										
1	15.0	35.0	99.2	94.5	6.0	21.0	6.0	12.0	20.2	3.5
2	30.0	34.9	98.9	92.2	3.0	34.4	14.4	12.0	20.9	5.9
3	45.0	34.6	100.4	85.9	1.8	38.5	15.7	12.0	20.4	6.1
4	60.0	31.4	97.3	82.9	3.0	54.8	22.2	12.0	99.1	12.0
Compacted 100% Standard AASHO										
1	15.0	27.2	102.6	83.4	12.4	28.9	-1.3	15.9	20.9	-4.7
2	30.0	26.4	96.7	67.0	10.5	35.6	5.5	16.0	34.5	2.2
3	45.0	26.6	95.1	77.6	12.0	40.7	12.0	16.0	39.9	9.2
4	60.0	56.6	95.3	51.5	12.5	52.0	16.8	15.5	51.8	14.7

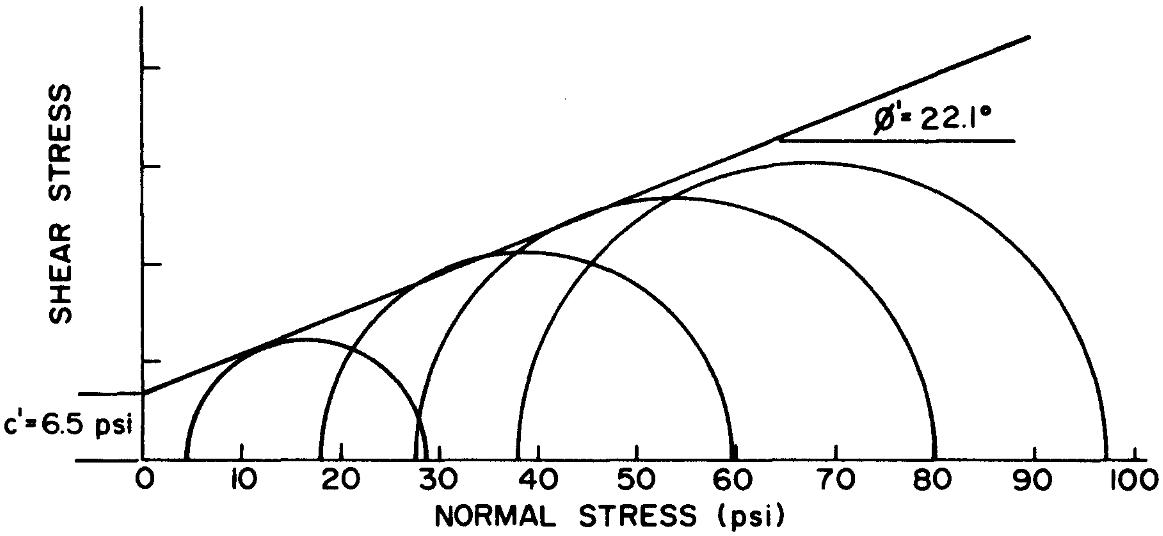

FIG. 11 PEAK EFFECTIVE STRENGTH ENVELOPE FOR R-23

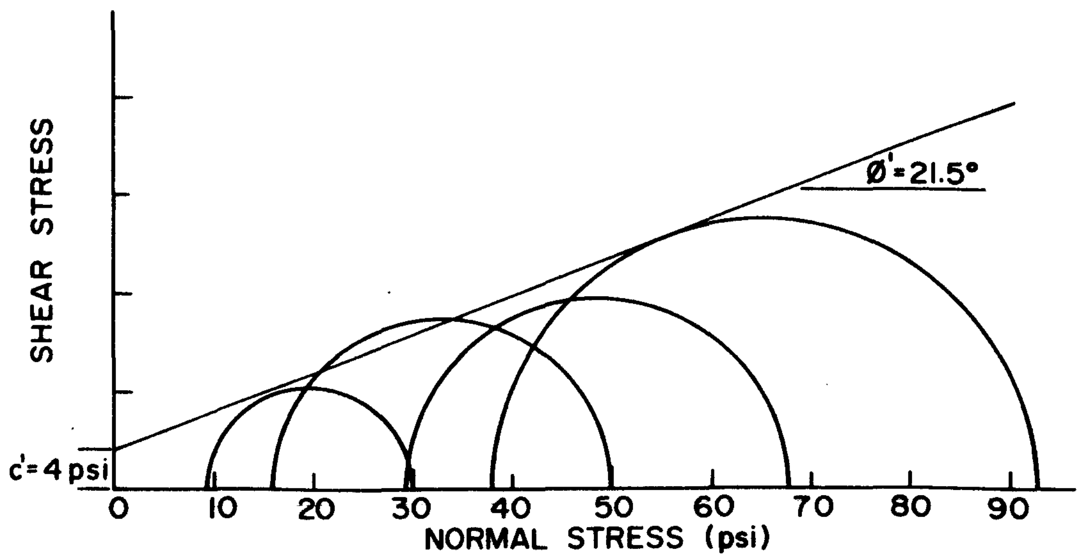

FIG. 12 PEAK EFFECTIVE STRENGTH ENVELOPE FOR R-26

TABLE IV

Summary of Total and Effective
Shear Strength Parameters

Series	Peak				Large Strain Conditions			
	c (psi)	ø (deg)	c' (psi)	ø' (deg)	c (psi)	ø (deg)	c' (psi)	ø' (deg)
R-23	7.0	15.8	6.5	22.1	4.5	10.8	2.5	15.0
R-26	4.0	16.0	4.0	21.5	6.0	6.5	6.0	8.0
100% Std.	8.5	12.0	5.5	17.5				

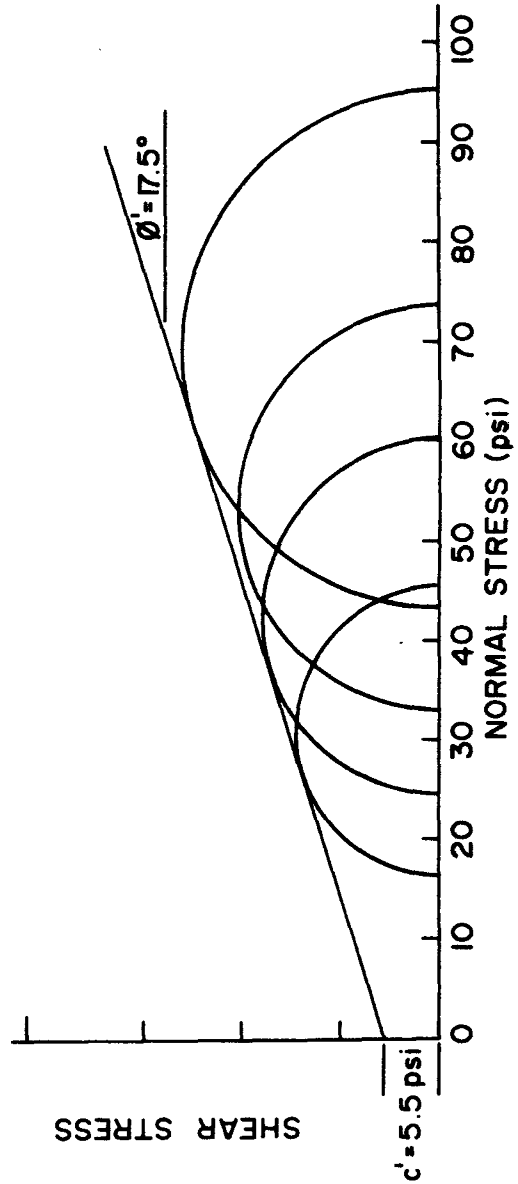

FIG. 13 PEAK EFFECTIVE STRENGTH ENVELOPE FOR STANDARD AASHO COMPACTION

The W series of triaxial tests were trimmed from the undisturbed block sample taken from the failure plane at Belle Plaine. All specimens exhibited a brittle type of failure, forming a slicked failure surface. Table V shows the triaxial data at peak and large strain conditions. Figure 14 shows the peak effective strength envelope and Figure 15 shows the large strain (i.e. approximately 12 percent) effective strength envelope. Under peak conditions, the effective angle of internal friction varies from 15.5 to 20 degrees with the effective cohesion varying from 1 to zero psi (7 to 0 kN/m^2) respectively.

A series of direct shear tests were performed on 2 inch (51 mm) square specimens cut from thin-walled tube samples taken approximately at the shear surface. Seven direct shear tests were performed at unit normal pressures ranging from 5 to 35 psi (34 to 241 kN/m^2). All failed specimens had slickenslided and somewhat wavey surfaces. The samples were subjected to six cycles of 0.2 inch (5 mm) displacements at a rate of strain of 0.1 inches (2.5 mm) per hour. Figure 16 shows the residual shear strength envelope. The best-fit envelope gives a residual friction angle of 7.5 degrees and a cohesion of 0.75 psi (5.2 kN/m^2).

TABLE V

Data for Softened, Compacted Regina Clay from Embankment

Trial No.	Consolidation Pressure (psi)	Initial Water Content (%)	Final Degree of Sat. (%)	Response Up (%)	Peak Strain (%)	Peak σ_d (psi)	Peak u_w (psi)	High Strain Strain (%)	High Strain σ_d (psi)	High Strain u_w (psi)
Series W										
1	30.0	35.7	100.1	118.2	5.6	13.7	12.5	12.3	11.0	12.2
2	50.0	35.5	99.5	102.1	5.5	33.5	19.6	12.9	16.5	12.3
3	90.0	34.2	95.1	72.7	7.7	47.9	29.6	12.5	23.8	25.8
4	30.0	33.8	98.1	85.2	7.6	21.9	9.9	12.5	14.5	6.2
5	15.0	33.4	98.7	68.6	3.2	9.3	4.9	12.4	7.4	3.3
6	50.0	34.3	99.3	75.6	5.3	28.1	14.3	12.2	15.6	9.3
7	30.0	33.2	98.3	72.7	4.7	22.5	8.1	12.0	14.1	5.0

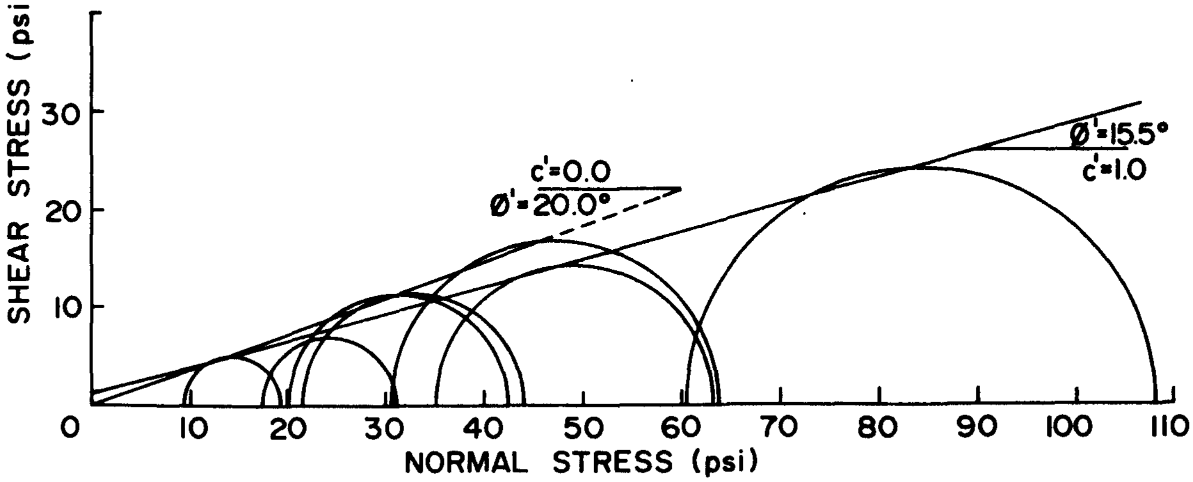

FIG. 14 PEAK EFFECTIVE STRENGTH ENVELOPE FOR SERIES W

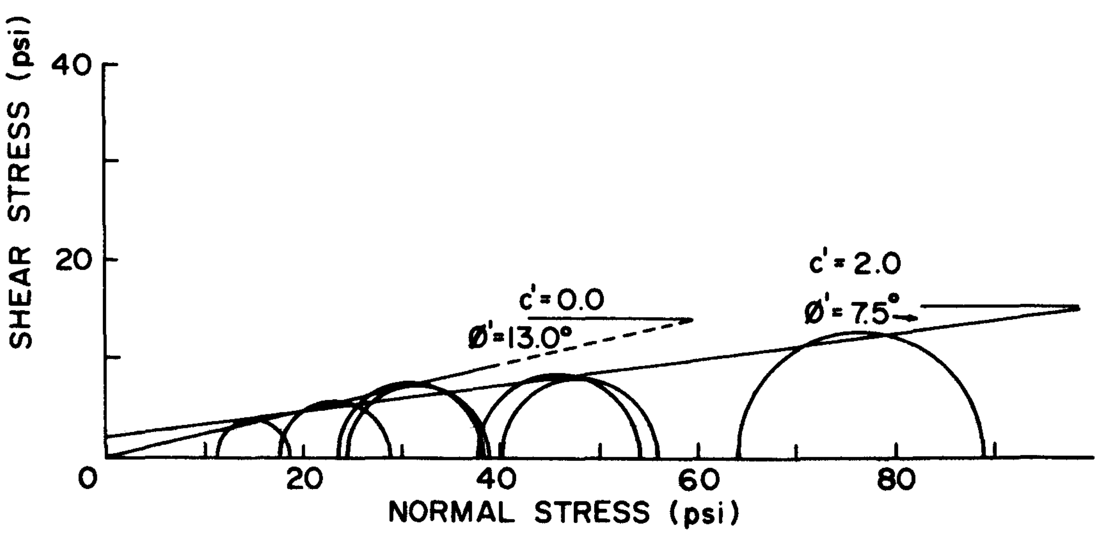

FIG. 15 EFFECTIVE STRENGTH ENVELOPE AT LARGE STRAINS FOR SERIES W

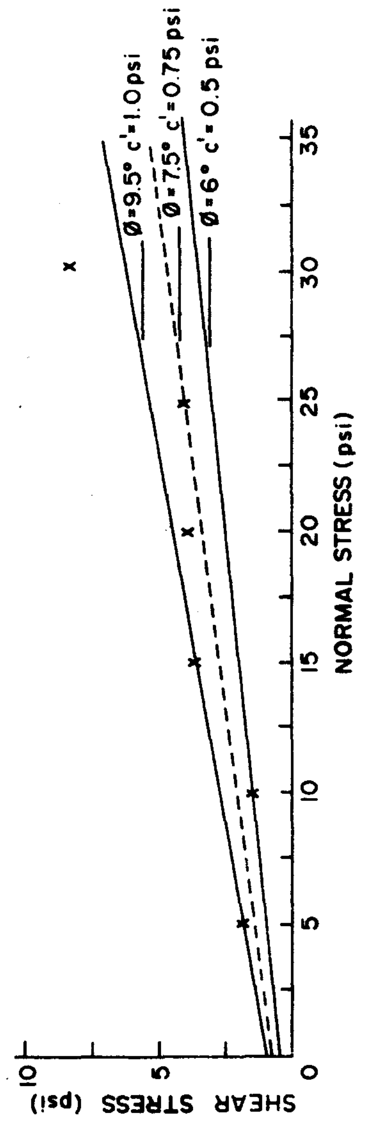

FIG. 16 RESIDUAL SHEAR STRENGTH PARAMETERS FROM DIRECT SHEAR TESTS

Stability Analyses

The factor of safety of the Belle Plaine overpass was evaluated for the end of construction situation and at the time of failure. The stability analyses were performed on the University of Saskatchewan Slope Stability program (Fredlund, 1974), using the Simplified Bishop method. Calculations were based on a total unit weight for the soil of 115 PCF (1840 kg/m^3). The rotational slide along line A (Figure 4) was used to analyse stability. The upper scarp and toe thrust were points that could be definitely observed in the field. In the central part of the slide, the failure zone appeared to be approximately 8 feet (2.4 m) below the embankment surface.

The computer analysis indicated the most critically stressed surface was 14 feet (4.3 m) below ground surface when the slip surface was controlled by the scarp and toe points in the field. This indicates that the slide would desire to go somewhat deeper if the soil properties and water pressure conditions were completely homogeneous.

The factors of safety for the end of construction case are presented in Table VI. All factors of safety are high and simply indicate the stable condition at the end of construction. The negative water pressure of 31.7 psi (219 kN/m^2) corresponds to the matrix suction at optimum water content conditions. If this value is used as a negative water pressure in the effective shear strength equation for a saturated soil, the factor of safety is approximately 9. However, this assumes that the water pressure is as effective as a total normal stress in producing shear strength. This is known to not be the case (Bishop et al, 1960) and it would be more realistic to use approximately 85 percent of the negative water pressure. In this case the factor of safety is approximately 8 and corresponds to results from the undrained analysis.

TABLE VI

Factor of Safety at the End of Construction *

Trial No.	Shear Strength	Factor of Safety	Comments
1	S_u = 17 psi	8.	Mean Undrained Strength
2	c' = 5.5 psi ø' = 17.5^{o}	4.3	u_w = 0
		9.2	u_w = 31.7 psi

* Factors of safety are on the critically stressed surface.

The computed factors of safety corresponding to the time of failure are shown in Table VII. Both the mean and lower limit undrained shear strength (Figure 10) produce factors of safety that are too high. Two sets of effective strength parameters are used to check factor of safety since two envelopes could feasibly fit the data. The results also show the effect of a small amount of cohesion. Three piezometric line conditions are considered. The first condition corresponds to the average piezometric line measured at the time

TABLE VII

Factor of Safety at Time of Failure

Trial No.	Shear Strength	Factor of Safety	Comments
1*	S_u = 6 psi	3.2	Mean Shear Strength.
	S_u = 3.5 psi	1.9	Lower Limit of Strength
2*	c' = 0 ø' = 20^{o}	0.94	Piezometric line 4 feet below slope surface.
		0.64	Piezometric line at ground surface**
		0.3	Piezometric line 4 feet below ground surface plus R = 0.9
3*	c' = 1.0 ø' = 15.5^{o}	1.23	Piezometric line 4 feet below slope surface.
		1.01	Piezometric line at ground surface**
		0.7	Piezometric line 4 feet below ground surface plus R = 0.9
4	c' = 0 ø' = 20^{o}	0.57**	Bottom of slip surface at 8 feet Piezometric line at ground surface
5	c' = 1.0 ø' = 15.5^{o}	1.21***	Bottom of slip surface at 8 feet. Piezometric line at ground surface.

* Critically Stressed Surface
** Corresponds to an r_u = 0.54
*** The failure surface is composite

of failure (i.e., 4 feet (1.2 m) below the slope surface). The second water pressure conditions corresponds to either the highest possible seepage pressure condition or the measured piezometric condition plus the effect of a thaw-consolidatation ratio (R) of 0.44. This thaw consolidation ratio is a lower limit value computed from estimated Regina Clay soil properties and thermal conditions at the site (Morgenstern and Nixon, 1971). The third water pressure condition corresponds to a thaw-consolidation ratio of 0.9, computed from average Regina Clay properties imposed on the measured piezometric line. The thaw consolidation pore pressure was considered since the history of field investigations indicated that instability was generally associated with the melting of the lowermost portion of the frozen soil and excessive spring rainfalls. Trials 2 and 3 show the results when the most critically stressed circular surface is analyzed whereas trials 4 and 5 correspond to a composite failure surface at a depth of 8 feet (2.4 m) below the slope surface.

The analyses indicates that realistic estimates of the water pressure conditions and the effective strength parameters produce factors of safety close to 1.0. The two strength envelopes considered show that a small amount of cohesion (i.e., less than 1 psi or 7 kN/m^2) has a significant effect on the factor of safety. Taking into account all measurements of the effective strength parameters (i.e., end of construction and time of failure conditions), the strength parameters best describing the failure conditions are an effective angle of internal friction of 17.5 degrees and an effective

cohesion intercept of 0.75 psi (5.2 kN/m^2). The water pressure considered most realistic corresponds to a piezometric line at the slope surface. It must be emphasized that the water pressures at the time of failure are difficult to accurately evaluate and plan a significant part in the factor of safety calculation.

Table VIII summarizes factor of safety calculations computed using the residual shear strength parameters. Since there does not appear to be any mechanism whereby the stresses on the unfailed geometry can exceed peak strength conditions, it is not realistic to apply the residual parameters to the unfailed slope (i.e. Trial #1). However, using the geometry of the failed slope and the measured field piezeometric conditions, the factor of safety using residual strength parameters is approximately 1.0.

The effect of a 3 foot (0.9 m) deep tension crack at the scarp had a negligible effect on the factors of safety.

TABLE VIII

Factor of Safety at Time of Failure
Considering Residual Strength Parameters

Trial No.	Shear Strength	Factor of Safety	Comments
1	$c' = 0.75$ $\phi' = 7.5^o$	0.95	Unfailed slope geometry No water pressure
		0.64	Piezometric line at ground surface
2	$c' = 0.75$ $\phi' = 7.5^o$	1.30	Failed slope geometry No water pressure
		1.03	Piezometric line 4 feet below ground surface
		0.89	Piezometric line at ground surface

CONCLUSIONS

(1) The failure surface in the swelling Regina Clay embankment at Belle Plaine, Saskatchewan was essentially circular with the exception of the base of the slide which was slightly composite due to the effect of the thawing front.

(2) The embankment had a high factor of safety at the time of construction. With time, the effect of freezing and thawing and softening due to the ingress of water produced a reduction in matrix suction and in turn, a reduction in the shear strength. The effective cohesion appears to be the main strength parameter reduced.

(3) At failure, the most realistic strength parameters for the softened Regina Clay appear to be an effective angle of internal friction of 17.5 degrees and an effective cohesion of 0.75 psi (5.2 kN/m^2). The water pressure conditions are difficult to predict but a piezometric line along the slope surface appears to be most realistic.

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