The Effect of Soil Suction on Slope Stability at Notch Hill

J. KRAHN

EBA Engineering Consultants, Calgary, Alberta, Canada

D.G. FREDLUND

University of Saskatchewan, Saskatoon, Saskatchewan, Canada

M.J. KLASSEN

CP Rail, Calgary, Alberta, Canada

SYNOPSIS

The sideslopes of a railway embankment in central British Columbia, constructed on relatively flat ground with local lacustrine silt, began to fail several years after construction. Shallow instability ultimately developed on both sides of the embankment over a distance of several kilometers. Initially the soil had a significant apparent cohesive strength. With time, the strength appeared to diminish due to the dissipation of negative pore-water pressures. The remaining frictional strength was not sufficient to maintain stability since the slopes were constructed at angles close to the peak effective friction angle of the soil. This case history, together with the laboratory saturated and unsaturated strength test results and field suction measurements, demonstrates the dramatic effect of negative pore-water pressures on near surface slope stabilty.

INTRODUCTION

In the late 1970's a section of new railway embankment was constructed in central British Columbia using the local lacustrine silt. Several years after completion, large segments of the sideslopes began to fail. Eventually movements of varying amounts occurred on both sides of the embankment over a distance of several kilometers.

The gradual loss of stability obviously resulted from a decrease in the strength of the soil since the slopes were initially stable. The reason for the adequate initial strength was not fully understood at first, but it became obvious that the strength loss was due to changes in negative pore-water pressure. Since negative pore-water pressures can produce an apparent cohesive strength component, it would appear that it was the loss of this portion of the strength that led to instability. Field and laboratory tests later confirmed that indeed this was the case.

This paper (1) describes the project and the slope instability, (2) presents the details of the testing and analysis conducted, and (3) discusses the implications of the findings with respect to the design of slopes.

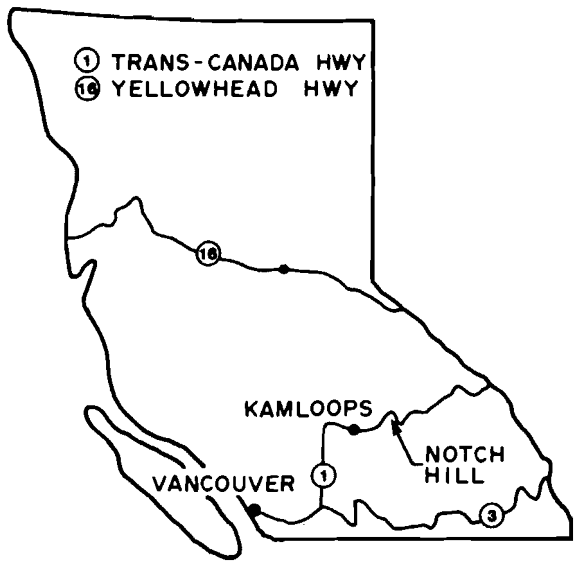


Figure 1 - Site Location

LOCATION

The subject embankment is located approximately 20 kilometers east of Kamloops, British Columbia near the small community of Notch Hill (see Figure 1). The points along the embankment investigated in detail were only a few hundred meters from the Trans-Canada Highway.

SITE AND INSTABILITY DESCRIPTION

A large loop was incorporated into the railway alignment in 1978 to reduce the grade. This required building an embankment on relatively flat ground. The majority of the embankment was constructed with sideslopes at about 1.5H:1.0V and with heights varying from a couple of meters to more then 10 meters.

Several years after completion of the project, the embankment sideslopes began to fail. By 1982, the sideslopes were failing on both sides along most of the embankment. The extent of instability and movement varied from shallow minor movements (i.e., less than 20 cm) up to slips as deep as one to two meters with movements in the order of meters. At some locations the slide mass flowed like a viscous fluid beyond the embankment toe. The upper edge of the unstable area was generally at or below the embankment crest and the lower edge was above the toe. At no location did the instability extend into the native foundation soil.

The two photographs in Figure 2 show some of the features of the instability. Photo (2a) shows locations where the slide mass moved beyond the toe; Photo (2b) shows an area where the lower edge of the slide mass was well above the embankment toe. The gravel on the slopes visible on the photos was placed as a temporary remedial measure to keep the track in operation.

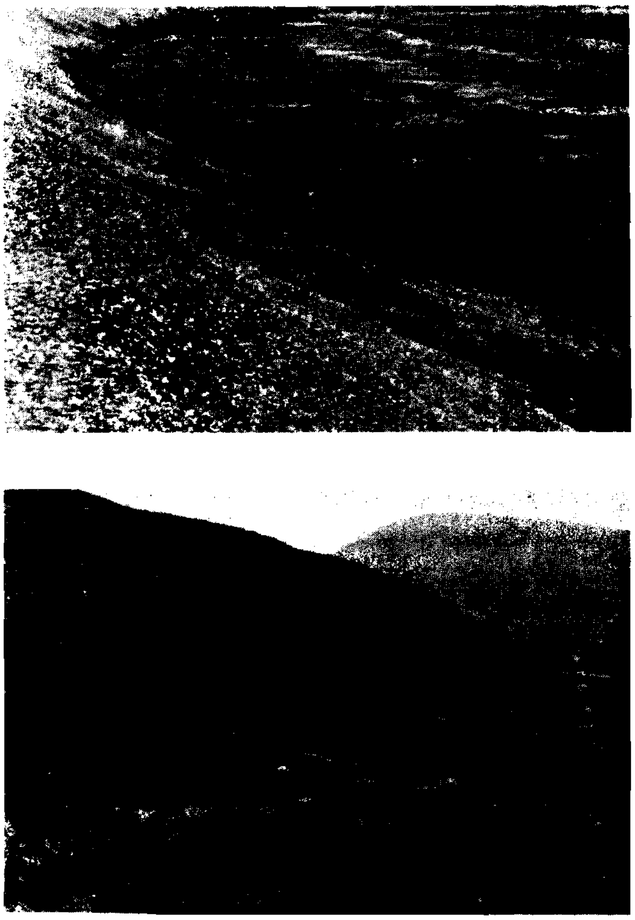


Figure 2 - Typical Slope Conditions
Inside and Outside the Loop

By far the most common mode of instability was as illustrated in Figure 3. The upper edge of the unstable portion of the slope was some distance below the crest, the lower edge was above the toe, the maximum depth was about a meter, and the base was approximately circular. These features are further depicted by the photos in Figure 4. The dashed line added to the photos delineates the base of the unstable portion of the slope.

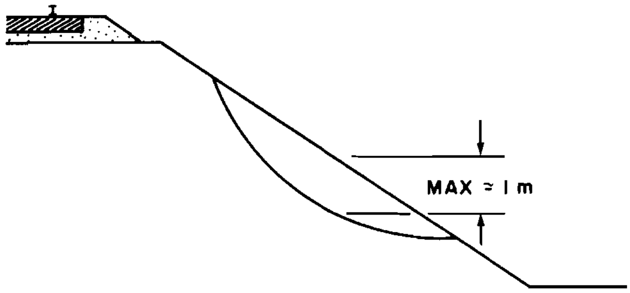


Figure 3 - Typical Mode of Failure

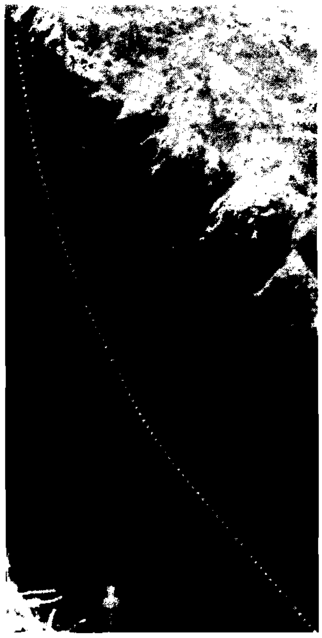


Figure 4 - Extent and depth of sliding mass

The instability was most active in the spring after the ground had thawed. Once the slopes began to dry, in the late spring and early summer, the movement generally came to a halt, or at least reduced to the point where maintenance work was not required to keep the track operational.

Ultimately the slopes were stabilized by covering them with a thick (i.e., about 5 m) layer of gravel by dumping from rail cars and then spreading the material with a dozer.

CLASSIFICATION INDICES

The embankment was constructed with the varved lacustrine soil present in the area. According to observations in two testpits excavated in the slopes, the material consists predominantly of non-plastic silt with some small pockets of highly plastic clay. Grain size distribution tests revealed that the non-plastic samples consisted of approximately 90% silt, a small amount of clay, and a fraction of fine sand; the clay samples had 12 to 40% clay, 34 to 84% silt and 2 to 20% fine sand. Liquid and plastic limits for the clay samples varied between 43 and 72, and between 17 and 34.

EFFECTIVE STRENGTH PARAMETERS

Four triaxial tests were performed to measure the effective strength parameters of the predominant non-plastic soil. Several bags of the material were mixed together to form a sample of sufficient size for all the strength testing. The aggregate sample was dried and then put through a grinder to break up lumps and produce a uniform sample.

The aggregate sample consisted of 10% clay, 85% silt, and 5% fine sand; and the optimum water content and dry density were 21.5% and 15.6 kN/m^3 (99.3 pcf), respectively.

Following the completion of the index tests, water was added to the aggregate sample to bring the water content up to approximately optimum conditions. The soil was then compacted in a Standard Proctor mold using the conventional compactive effort. Specimens for the triaxial testing were then trimmed from the sample extruded from the compaction molds. The height and diameter of the triaxial specimens were about 75 and 38 mm, respectively.

The triaxial specimens were consolidated at effective confining stresses of about 34, 68, 139, and 278 kPa and then tested undrained with measurements of the pore-water pressure. Table I summarizes the test conditions; initial and final water contents, and initial and final densities for each specimen.

Table I

Triaxial Test Conditions and Soil Properties

CONDITION OR PROPERTY	SPECIMEN NUMBER			
	1	2	3	4
Cell Pressure (kPa)	448	483	552	690
Back Pressure (kPa)	414	414	414	414
Initial σ_3' (kPa)	34	69	138	276
"B" Parameter	1.0	1.0	1.0	1.0
Water Content (%)				
- initial	20.9	20.8	19.7	19.7
- final	25.0	25.6	25.1	23.4
Dry Density (kg/m^3)				
- initial	1630	1640	1650	1660
- final	1530	1530	1530	1580

$pcf = \frac{1}{0.06243}\ kg/m^3 = 16.02\ kg/m^3$

The results of the saturated triaxial tests are presented in Figure 5 on a plot of shear stress versus normal stress. These results represent the conditions at the peak deviator stress which occurred at approximately 10% strain with all four specimens. The pore-water pressure response during application of the axial load was slightly positive up to about 3% strain and then went negative as the strain increased.

The results presented for the Notch Hill silt in Figure 5 show a peak effective stress angle of friction of 35 degrees and an effective cohesion intercept equal to zero.

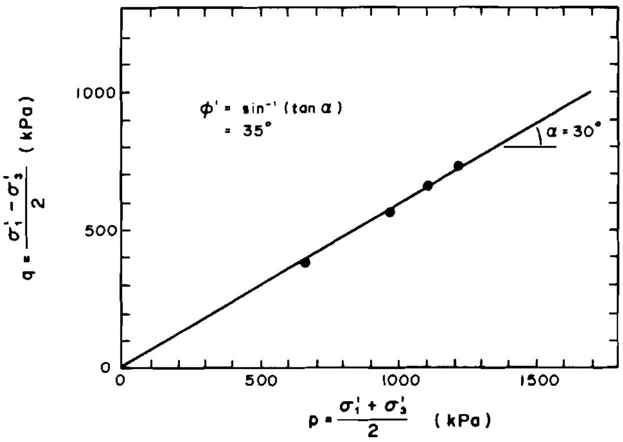


Figure 5 - Failure Envelope for the Notch Hill Silt

UNSATURATED SHEAR STRENGTH

In order to discuss the overall embankment stability, it is necessary to first discuss the shear strength of an unsaturated soil. Fredlund (1979), showed that the shear strength of an unsaturated soil can be represented by the equation,

$$\tau = c' + (u_a - u_w) \tan\phi^b$$
$$(\sigma - u_a) \tan\phi' \qquad [1]$$

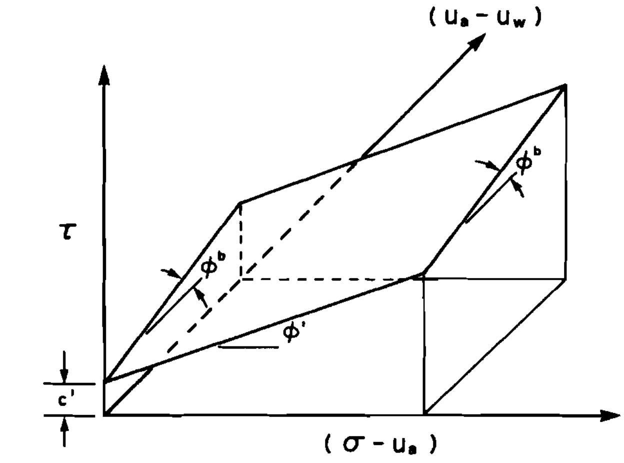
where c' is the cohesion intercept when the two stress variables (i.e., $(u_a - u_w)$ and $(\sigma - u_a)$) are zero, ϕ' is the friction angle with respect to changes in $(\sigma - u_a)$ and ϕ^b is the increase in shear strength with respect to changes in $(u_a - u_w)$.

This equation describes a surface on a three-dimensional plot of τ, $(\sigma - u_a)$ and $(u_a - u_w)$ as illustrated in Figure 6. Any section through this surface that is parallel to the τ and $(\sigma - u_a)$ axis, perpendicular to the $(u_a - u_w)$ axis and at some $(u_a - u_w)$ value greater than zero, appears as shown in Figure 6b. Therefore, the $(u_a - u_w) \tan\phi^b$ term can be considered to be a part of the cohesion of the soil. The shear strength of an unsaturated soil can therefore be written as,

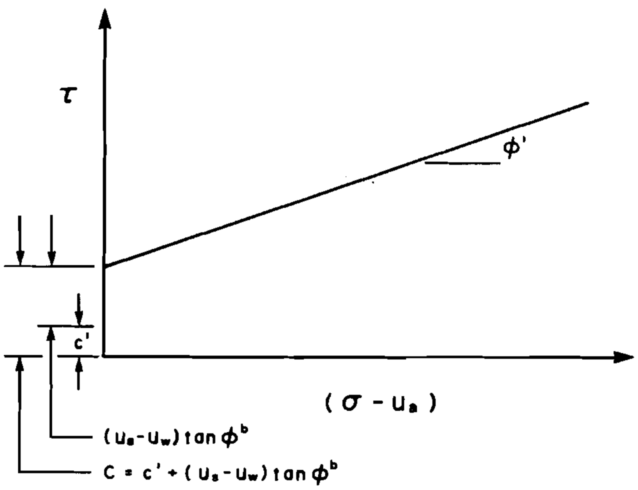
$$\tau = C + (\sigma - u_a) \tan\phi^b \qquad [2]$$

where C is the sum of the two components making up cohesion (i.e., c' and $(u_a - u_w) \tan\phi^b$.

This is a convenient manner of expressing the shear strength of unsaturated soils for conventional stability analyses. The strength contributed by the suction can be accounted for by the value specified for cohesion.



(a) Extended Mohr-Coulomb Failure Surface



(b) Increase in Strength with Respect to Matric Suction

Figure 6 Shear Strength Relationship for Unsaturated Soils

UNSATURATED SHEAR STRENGTH TESTS

Triaxial tests were performed on unsaturated samples of the Notch Hill silt to measure the strength parameter, ϕ^b. The tests were performed at the University of Saskatchewan using the equipment and procedures developed by Ho and Fredlund (1982a; 1982b).

Measuring the strength parameter, ϕ^b, requires conducting triaxial tests at several matric suction levels. Multistage testing procedures such as those used by Ho and Fredlund (loc. cit.) makes it possible to obtain the necessary data from one specimen tested at three different suctions. This maximizes the information obtainable from a limited number of specimens and reduces scatter in the data due to soil variabliity.

The test procedure can be summarized as follows: A specimen is consolidated at a predetermined confining stress and at a fairly low suction. Water was added to the specimen in order to reduce its matric suction. Load is then applied at a constant strain rate until the deviator stress approaches the peak strength. The axial load is then removed, the matric suction increased and the specimen is again loaded until the deviator stress approaches the strength. The procedure is repeated for a third and final suction level. Generally the specimen becomes too distorted to perform more than three stages.

Mohr circles are drawn for each test stage and lines at an angle of ϕ' with respect to the horizontal axis are drawn tangent to the circle and projected to intersect the shear strength axis. The intersection points are plotted as a function of the matric suction, and the slope of the line depicting the relationship between suction and shear strength is equal to the angle, ϕ^b.

Two samples of the Notch Hill silt were prepared for unsaturated testing, in the same manner as for the saturated testing described above. Table II presents the stress states used in the unsaturated tests.

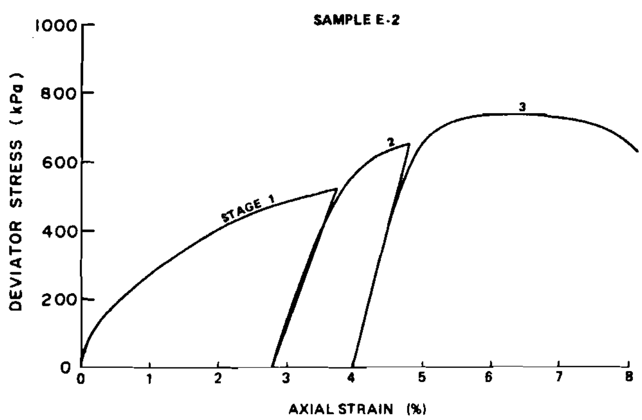
Table II

Unsaturated Triaxial Test Stress Conditions

SAMPLE NO.	STAGE	σ_3	u_a	u_w	$\sigma_3 - u_a$	$u_a - u_w$
E - 2	1	241	103	69	138	35
	2	310	172	69	138	103
	3	379	241	69	138	172
E - 3	1	207	138	69	69	69
	2	310	241	69	69	172
	3	414	345	69	69	276

Note: σ_3 - confining stress or cell pressure (kPa)
u_a - air pressure (kPa)
u_w - water pressure (kPa)
$(u_a - u_w)$ - suction (kPa)

The multistage stress-strain curves obtained for the two specimens are shown in Figure 7, and the resulting Mohr circles are presented in Figure 8 together with lines parallel to the Mohr-Coulomb failure envelope at the various suction levels.



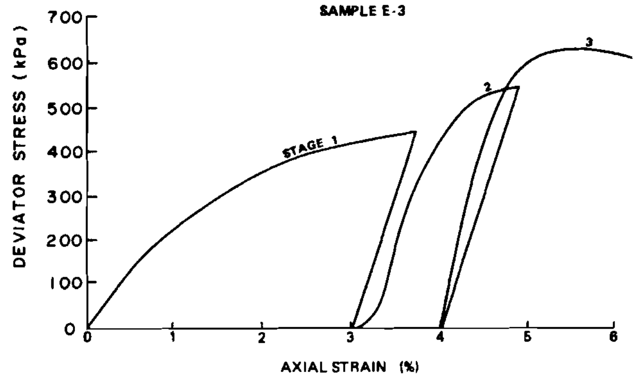


Figure 7 - Multistage Stress - Strain Curves

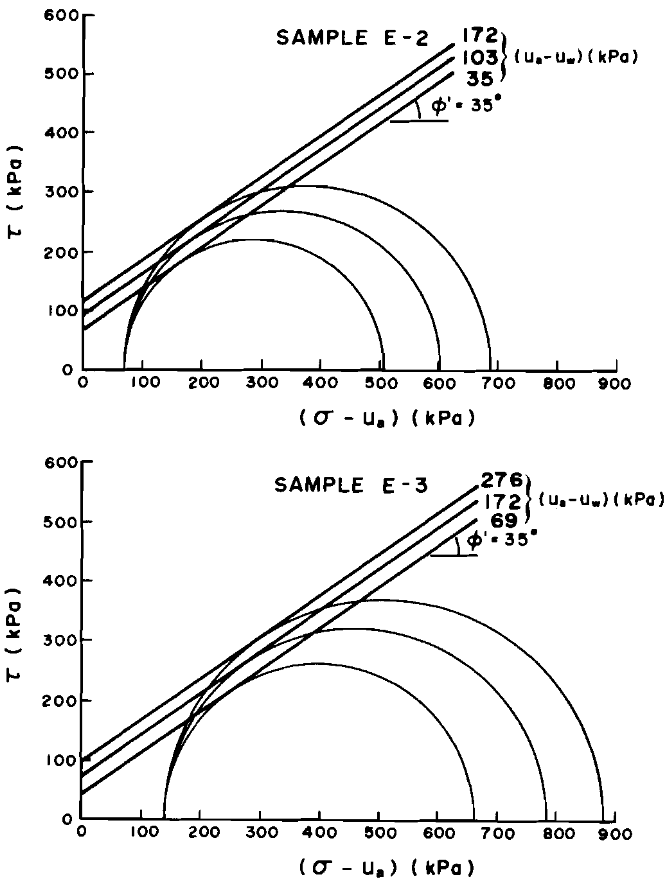


Figure 8 - Triaxial Test Results on Unsaturated Tappen - Notch Hill Silt

Figure 9 presents the relationship obtained between shear strength and matric suction. The measured, ϕ^b, angle is 16 degrees. This value falls within the range of typical values (i.e., 12.6 to 22.6 degrees) published by Fredlund (1985). The ϕ^b angle always appears to be less than the peak effective friction angle.

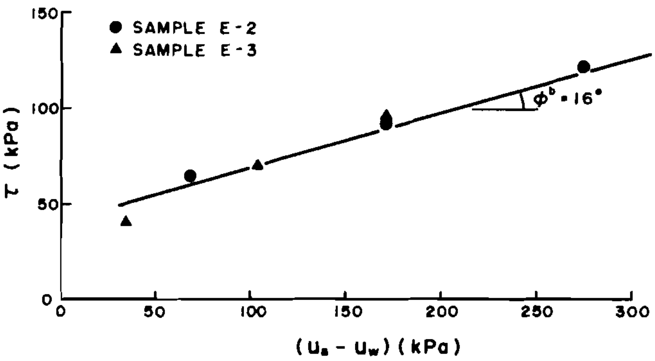


Figure 9 - Shear Stress Versus Matric Suction for the Tappen - Notch Hill Silt

FIELD SUCTION MEASUREMENTS

Two testpits were excavated in the embankment slopes to sample the soils, to inspect the soil conditions, and to measure the in-situ suctions. The initial field work was done in May, 1982. Some additional suction measurements were obtained in July about three months later to obtain an indication of any changes in suction.

Measurements of the in-situ suctions were made using a portable tensiometer capable of measuring suctions up to about 85 kPa (i.e., 0.85 of an atmosphere). The instrument has a high-air entry stone at one end of a metal tube that is filled with water. A vacuum gauge is placed at the other end of the metal tube. When the porous tip is inserted into a small diameter hole and comes in contact with the soil, there is a tendency for water to be drawn out of the tube into the soil. The potential for water to be drawn out of the instrument is a measure of the matric suction. The matric suction is registered on the vacuum gauge. Figure 10 shows a photo of the instrument inserted into the side wall of one of the testpits.

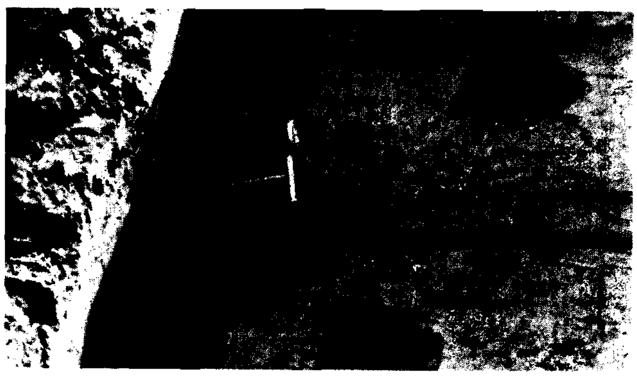


Figure 10 - The Portable Tensiometer

Suction measurements were made at about 20 locations on the side walls of each testpit. Disturbed samples were collected at the majority of suction measurement locations of purpose of later measuring the natural water contents.

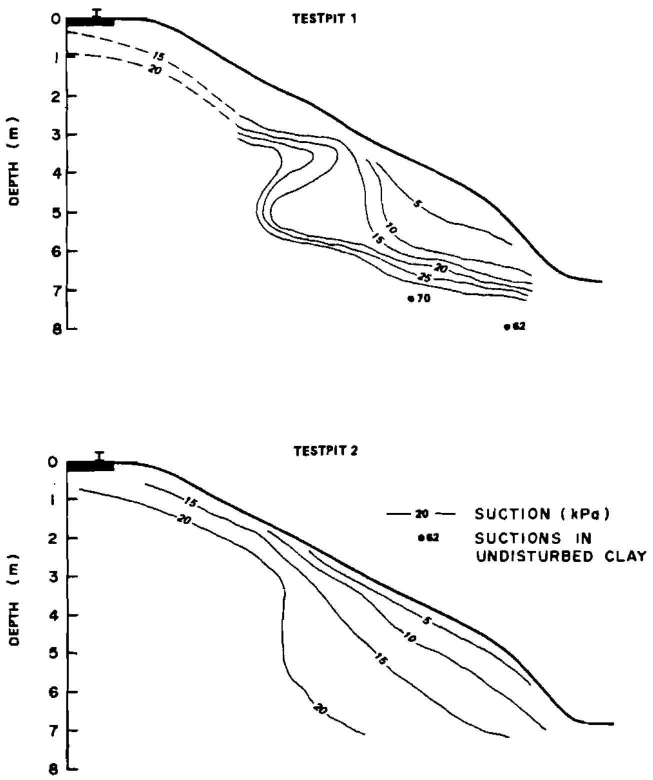
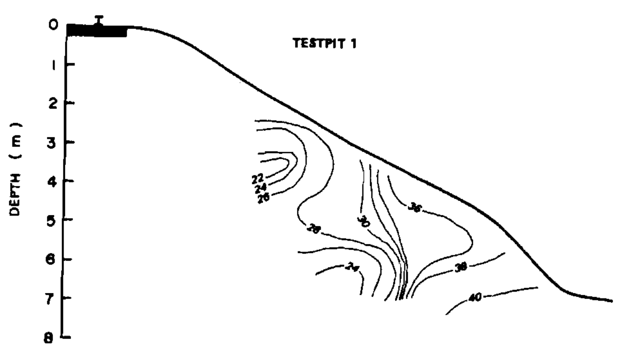


Figure 11 - Suction Contours in Two Testpits

The results of the suction measurements are summarized by the contours in Figure 11. In both testpits there were one or two readings that were inconsistent with the general trends. These readings were consequently ignored in drawing the contours.

Suction trends and patterns of significance are as follows: The suctions were lowest near the surface of the slope and they increased with depth into the embankment. The 5-, 10-, and 15-centibar contours have a shape not unlike the shape of the observed slip surface.

The water contents (Figure 12) were also plotted and contoured. No apparent meaningful trends or patterns emerge from these water content contours except that the highest water contents are in the underlying foundation soils, the lowest water contents are at depth within the embankment, and the water contents towards the slope surface are higher than at depth within the embankment.



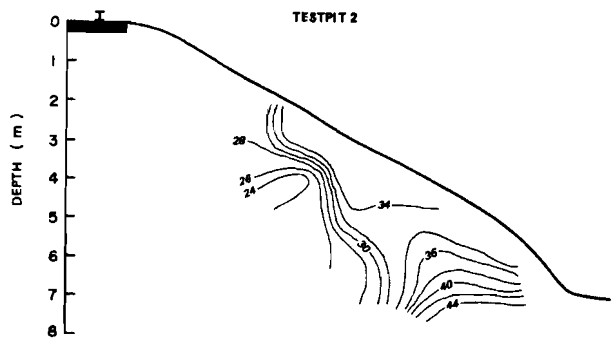


Figure 12 - Water content contours in Two Testpits

The July suction measurements were made by inserting the instrument about 300 mm into the slope. The results of these readings are presented in Figure 13. The July measurements are generally higher than the initial ones. In May the readings were generally less than 10 centibars near the surface; in July they were, except for one, all above 13 centibars. At similar locations on the lower portions of the slopes near the surface, the suctions increased from less than 10 to more than 17 centibars.

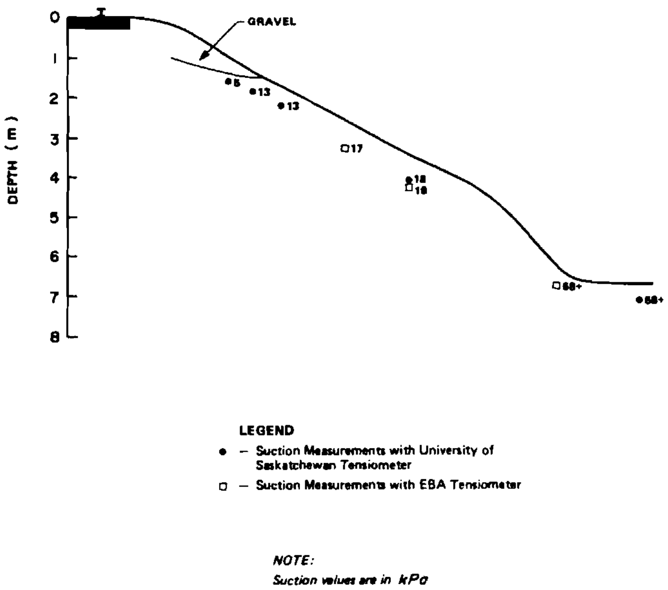


Figure 13 - Suction measurements Taken in July, 1982

EFFECT OF COHESION AND SUCTION ON STABILITY

The stability of the near surface portion of a slope is extremely sensitive to changes in cohesion. Consider, for example, a slope at an angle equal to the friction angle of the material. With no cohesion, the factor of safety against failure along a slip surface such as shown in Figure 14 is close to unity. Adding a mere 2 kPa of cohesion increases the factor of safety from 1.05 to 1.35, an increase of about 30 percent. Also shown in Figure 14 are the suctions that correspond to cohesion values when the ϕ^b angle is 16 degrees. A suction of about 7 kPa becomes equivalent to a cohesion of 2 kPa. Therefore, the change in the factor of safety is approximately 30 percent. If it is assumed that the Notch Hill embankment initially had a suction of 50 kPa, a value readily obtainable when compacting fine-grained soils near the optimum water content, the factor of safety against near surface instability would have been in excess of 3.0. This illustrates the dramatic influence of negative pore-water pressures.

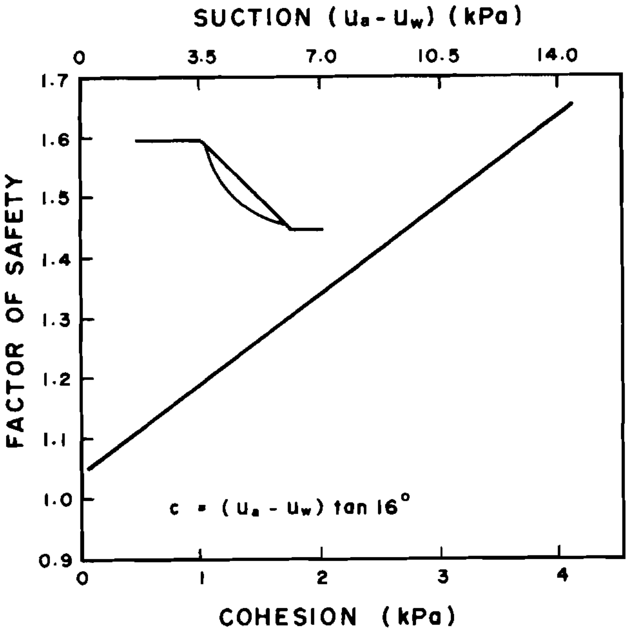
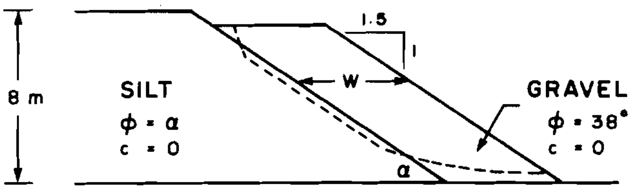
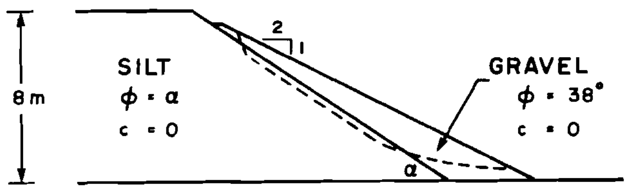


Figure 14 - Effect at Cohesion and Suction on Factor of Safety for a Near Surface Slip Surface



W (m)	Factor of Safety
0	1.01
2	1.21
3	1.34
5	1.53
Shallow slip Surface in Gravel	1.18

Figure 15 - Effect of Gravel Berm with Slope Angle the same as the Embankment Angle



Case	Factor of Safety
no gravel	1.01
Slip surface in the silt	1.56
Shallow slip surface in the gravel	1.55

Figure 16 - Effect of Flattening the Gravel Slope Angle

EFFECT OF GRANULAR COVER

Placing a clean granular blanket on a slope, such as was done at Notch Hill, can improve the stability substantially even if the suction in the underlying silt approaches zero. Figures 15 and 16 illustrate the effect of a gravel berm and the flattening of the slope with gravel. A blanket of only two meters thickness can increase the factor of safety by 20 percent (i.e., 1.01 to 1.21). Flattening the slope with gravel results in a more substantial increase in the factor of safety against instability in the gravel than with the berm at the same angle as the original embankment. In this sense, the flattening of the slope is a better alternative. Selecting one or the other of the alternatives, however, may be governed by practical and logistic reasons rather than stability considerations. Both alternatives improve overall stability. The procedures used to place the gravel will likely vary from site to site and dictate the details of the design. At the Notch Hill site, it was decided that the best technique would be to haul the material to the site with side dump rail cars and spread it parallel to the slope to create a berm.

DISCUSSION

The Notch Hill embankments were initially stable. The soil therefore must have had some cohesive strength. The frictional component of the strength alone would not have been sufficient to maintain the stability since the slopes were constructed at angles close to the peak effective friction angle. With just the frictional strength, the factors of safety would at best have been near unity. As the slopes were initially stable, but did not remain stable, it follows that the cohesive component of the strength must have decreased with time. When the cohesion portion of the strength essentially disappeared, the slopes became unstable.

The loss of the cohesive component of strength was likely due to the dissipation of negative pore-water stresses or suctions which gave the soil an apparent cohesion. The decrease in the suction is supported by the field measurements. It is reasonable to assume that the suctions were initially fairly uniform throughout the embankment. In May 1982, the suctions near the slope surface were considerably less than at depth within the embankment.

It is not certain what the magnitude of the suctions were at onset of the instability. The suctions may have been lower than the values measured in May, 1982. At the time the field work was carried out, there were no visible signs of ongoing movement. There was, however, clear evidence of prior movement. By inference, the suctions must have been lower at some point than at the time of the field work. Furthermore, analytical consideration also suggests that the suctions must have been at or close to zero when the slopes first reached the point of limiting equilibrium. The fact that some of the material flowed like a highly viscous liquid beyond the slope toe also suggests that the material must have been near saturation.

The exact physical process whereby the soil lost its initial suction is in part a matter of speculation. It was likely some interplay between moisture infiltration, absorption, and moisture migration due to thermal gradients. Since the instability was the most active in spring, immediately after the ground was thawed, thermal effects may have played a role. As moisture will migrate toward a freezing front, it is possible that moisture that infiltrated the ground during the year was drawn to the surface during the winter. This may have resulted in sufficient moisture concentrations near the slope surface after the ground had thawed to allow the suction to approach zero.

The reason for the instability being concentrated some distance below the crest but above the toe is also open to speculation. One factor may have been the variability in compaction. The zone where the majority of the instability occurred is also the part of embankment that is the most difficult to compact. It is possible that the middle outer portions of the slopes were not as well-compacted as the remainder of the embankment, which made it easier for moisture infiltration and migration to take place.

The stability of steep fine-grained soil slopes could conceivably be maintained by covering them with an impermeable membrane. This would prevent the infiltration and absorption of moisture and prevent the consequential loss of suction. The slope would, therefore, have the strength to remain as stable as at the time of construction. For large and long embankments, this may not be practical but in isolated locations where rights-of-way restricts flattening the slope, it may be a feasible solution.

A point of significance is that water contents alone do not provide a rational basis for understanding and analyzing the instability. If water content was the controlling criteria, the instability should likely have extended into the foundation soils where water contents were the highest. This was not the case and indicates that water content alone is not the controlling factor. As demonstrated by the suction measurements, it is the pore-water pressure that is a primary controlling factor. The reason that water content is not a direct measure of the pore-water pressure state is that the relationship between water content and suction is not unique. The water content suction relationship is a highly sensitive function of the soil. Hysteresis also contributes to non-uniqueness.

SUMMARY AND CONCLUSION

The railway embankment sideslopes at Notch Hill were initially stable due to the apparent cohesive strength of the soil, in addition to its frictional strength. With time, the soil lost its cohesive strength and the remaining frictional strength was not sufficient to maintain stability.

The loss of the cohesive strength would appear to be the result of the dissipation of the negative water pressure that was present for a time after construction. That negative water pressure give rise to an apparent cohesive strength has been demonstrated in the laboratory. The field measurements showed that there was a decrease in suctions near the slope surface. There is evidence that the suction may have been at or near zero at the point of instability.

This case history illustrates the important role that negative pore-water pressures play in the stability of slopes, particularly in the near surface stability. It also highlights the importance of working in terms negative pore-water pressures as opposed water contents.

REFERENCES

Fredlund, D.G. 1979. Appropriate Concepts and Technology for Unsaturated Soils, Second Canadian Geotechnical Colloquium, Canadian Geotechnical Journal, Vol. 16, No. 1, pp. 121-139.

Fredlund, D.G., Morgenstern, N.R. and Widger, R.A. 1978. Shear Strength of Unsaturated Soils, Canadian Geotechnical Journal, 15, pp. 313-321.

Fredlund, D.G. 1985. Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, ASCE, San Francisco, Vol. 1, pp. 465-472.

Ho, D.Y.F. and Fredlund, D.G. 1982. Increase in Strength Due to Suction for Two Hong Kong Soils, Proceedings of the Conference on Engineering and Construction in Tropical and Residual Soils, ASCE, Honolulu, Hawaii, January 11-15.