

Effect of soil suction on slope stability at Notch Hill¹

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The side slopes of a railway embankment in central British Columbia, constructed with local lacustrine silt, on relatively flat ground, began to fail several years after construction. Shallow instability ultimately developed on both sides of the embankment over a distance of several kilometres. Initially, the soil had a significant apparent cohesive strength. With time, the strength appeared to diminish owing 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 stability.

Key words: soil suction, slope stability, nonsaturated soils, shear resistance, tensiometers.

Les pentes latérales d'un remblai de voie ferrée situé dans le centre de la Colombie-Britannique et qui a été construit sur un terrain relativement plat avec un limon lacustre local, ont commencé à être affectées par des ruptures plusieurs années après la construction. Éventuellement, une instabilité peu profonde s'est développée des deux côtés du remblai sur une distance de plusieurs kilomètres. En fonction du temps, la résistance semble diminuer par suite de la dissipation des pressions interstitielles négatives. La résistance en frottement résultante n'était pas suffisante pour maintenir la stabilité puisque les pentes avaient été construites à des angles se rapprochant de l'angle de frottement effectif de résistance de pic du sol. Cette étude de cas, complétée par des résultats d'essais en laboratoire de résistance saturée et non saturée et de mesures de suction sur le terrain, démontre l'effet important des pressions interstitielles négatives sur la stabilité de la couche superficielle du talus.

Mots clés : suction de sol, stabilité de talus, sols non saturés, résistance au cisaillement, tensiomètres.

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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 side slopes began to fail. Eventually, movements of varying amounts occurred on both sides of the embankment over a distance of several kilometres.

The gradual loss of stability resulted from a decrease in the strength of the soil with time, since the side 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 (i) describes the project and the slope instability, (ii) presents the details of the testing and analysis conducted,

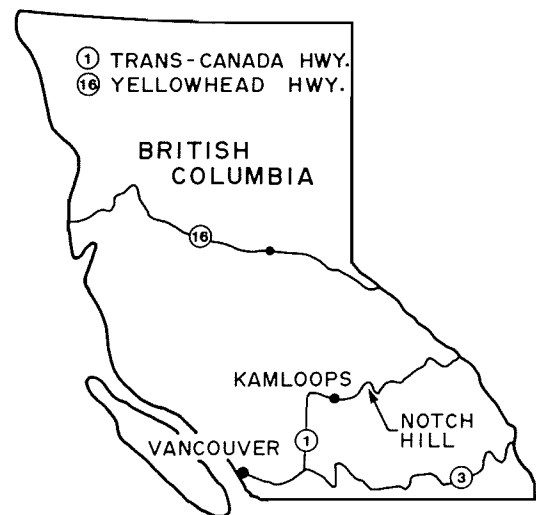


FIG. 1. Site location plan showing Notch Hill.

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and (iii) discusses the implications of the findings with respect to the design of slopes.

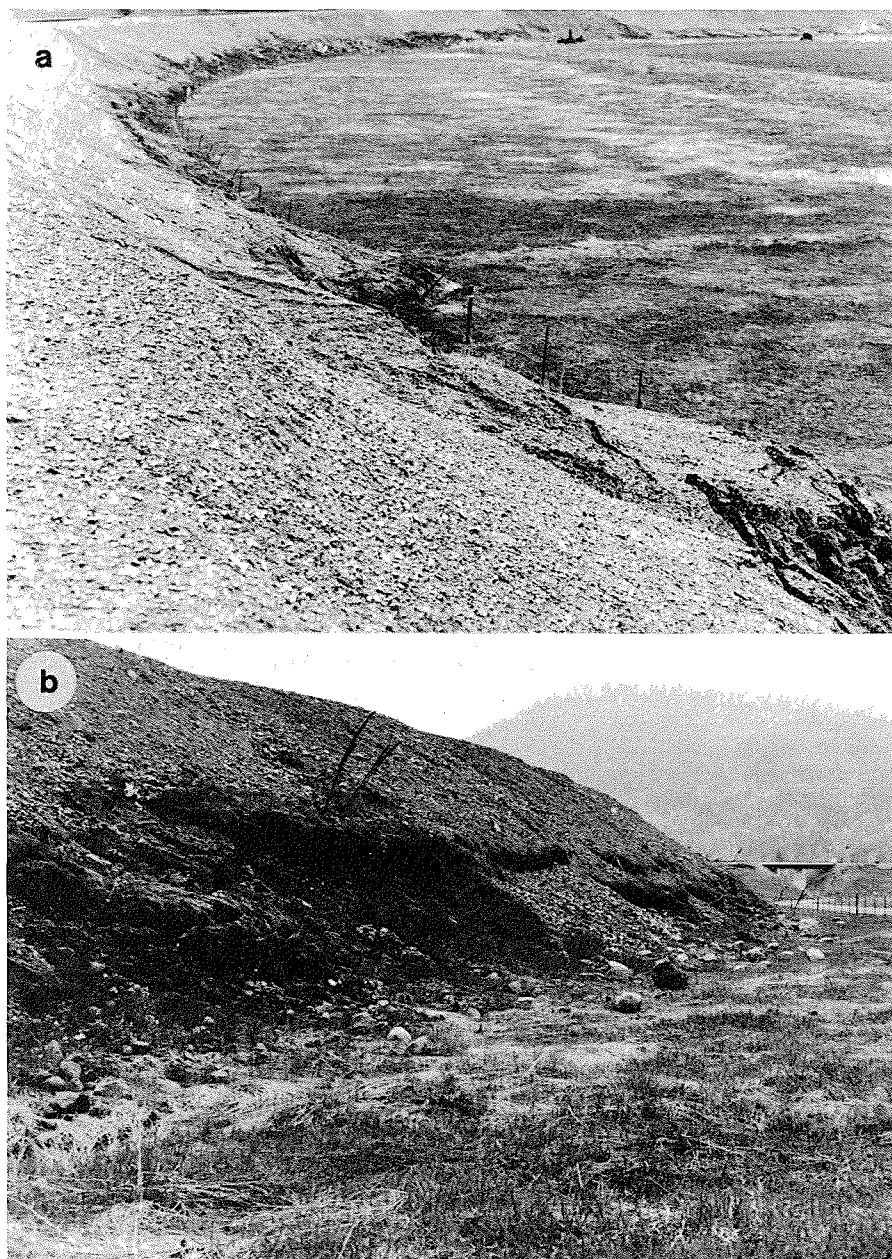


FIG. 2. Typical slope conditions inside and outside the loop: (a) Sliding mass moving beyond the toe; (b) sliding mass well above the toe of the slope.

Location

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

Site and instability description

A large loop was incorporated into the railway alignment in 1978 to reduce the grade on the rail line. This required building an embankment on relatively flat ground. The majority of the embankment was constructed with side slopes at about 1.5H:1.0V and with heights varying from about 2 m to more than 10 m.

Several years after completion of the project, the embankment side slopes began to fail. By 1982, the side slopes 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) to slips as deep as 1-2 m with movements in the order of metres. 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 Fig. 2 show some of the features of the instability. Figure 2a shows locations where the slide mass moved beyond the toe; Fig. 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.

By far the most common mode of instability was as illustrated in Fig. 3. The upper edge of the unstable portion of

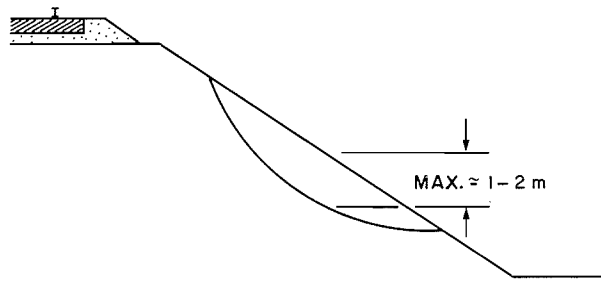


FIG. 3. Typical mode of failure.

the slope was some distance below the crest, the lower edge was above the toe, the maximum depth was about a metre, and the base was approximately circular. These features are further depicted by the photos in Fig. 4. The dashed line added to the photo delineates the base of the unstable portion of the slope.

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 being covered with a thick (i.e., about 5 m) layer of gravel, which was dumped from rail cars and then spread with a dozer.

Classification indices

The embankment was constructed with varved lacustrine soil from the area. According to observations in two test pits excavated in the slopes, the material consists predominantly of nonplastic silt with some small pockets of highly plastic clay. Grain size distribution tests revealed that the nonplastic samples consisted of approximately 90% silt, a small amount of clay, and a fraction of fine sand. The clay samples had 12–40% clay, 34–84% silt, and 2–20% fine sand. The grain size curve on the soil used for the laboratory testing program is shown in Fig. 5.

Liquid and plastic limits for the clay samples varied between 43 and 72, and between 17 and 34, respectively.

Effective strength parameters

Four triaxial tests were performed to measure the effective strength parameters of the predominant nonplastic 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. Laboratory compaction tests showed an optimum water content and dry density of 21.5% and 15.6 kN/m³ (99.3 pcf), respectively (Fig. 6). At the time of the field investigation, no *in situ* measurements of density were performed on the soil near the failure zone. At the time of construction, the embankment soil was compacted to approximately 100% of standard AASHTO conditions.

Following the completion of the index tests, water was added to the aggregate sample to bring its 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

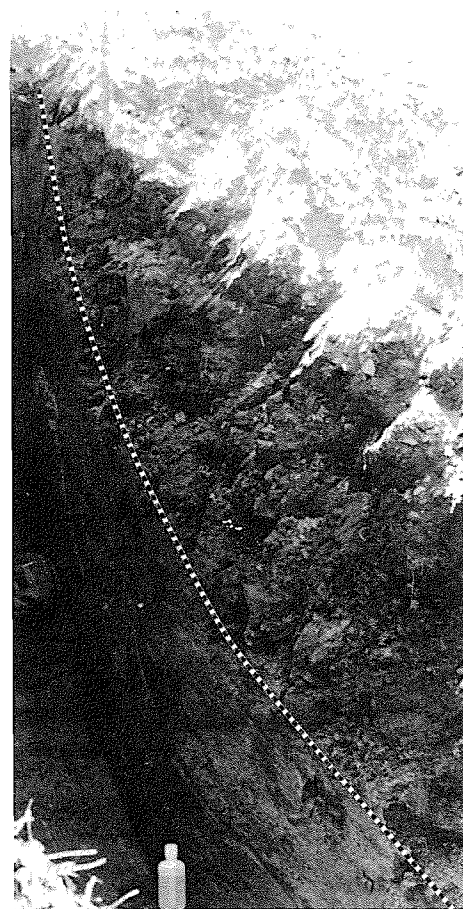


FIG. 4. Extent and depth of sliding mass.

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, 138, and 276 kPa and then tested undrained with measurements of the pore-water pressure. The specimens were back-pressured to ensure saturation, which was confirmed by the measurement of the *B* pore pressure parameter. Table 1 summarizes the test conditions, initial and final water contents, and initial and final densities for each specimen. The stress versus strain curves were similar in character for all specimens. Figure 7 shows the results from specimen 2.

The saturated triaxial test results are presented in Fig. 8 on a plot of shear strength versus mean principal effective stress. The strength envelope represents 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 developed.

The last two specimens (i.e., specimens 3 and 4) show slightly higher shear strengths, which is possibly related to their slightly lower initial water content and higher initial density.

The results presented in Fig. 8 for the Notch Hill silt show a peak effective stress angle of friction of 35° and an effective cohesion intercept equal to zero. Both the deviator stress and stress ratio failure criteria gave similar shear strength parameters. It is not anticipated that a modest variation in

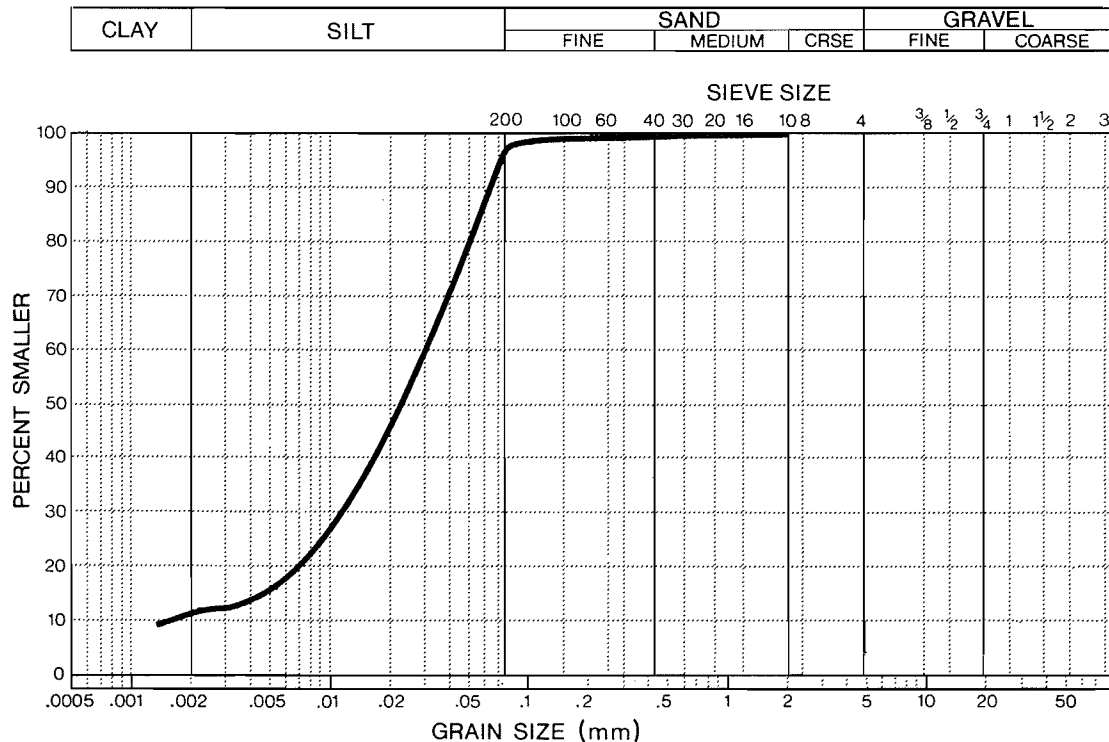


FIG. 5. Grain size distribution of soil used for the laboratory testing program.

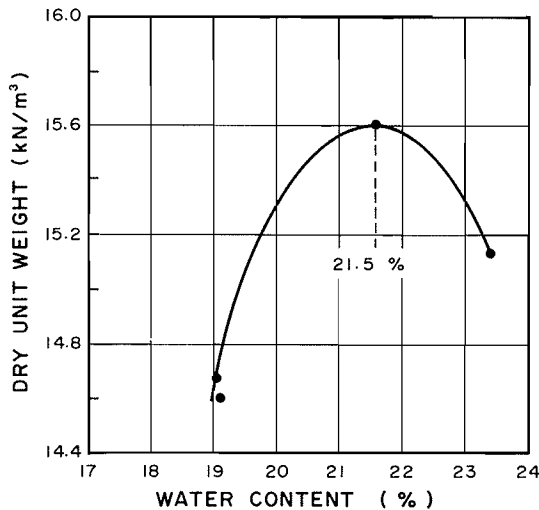


FIG. 6. Water content versus dry unit weight relationship for the soil used in the laboratory testing program.

density would produce a significant variation in the shear strength parameters, even at low confining pressures.

An attempt was made in the laboratory program to keep the initial effective confining pressure as low as possible while at the same time performing tests over a wide range of effective confining pressures. The lowest initial effective confining pressure was 34 kPa, which corresponds to a total stress approximately 2 m below ground surface. The specimens were tested in undrained loading with pore-water pressure measurements. Because of the dilatant nature of the soil at these confining pressures, the final stress states appear to be much higher than the overburden pressure. Possibly, testing along other stress paths should be given consideration when studying shallow landslides.

Unsaturated soil shear strength

The shear strength of a soil with negative pore-water pressures can play an important role in the stability of a slope, particularly when the slip surfaces are shallow. Fredlund *et al.* (1978) showed that the shear strength of an unsaturated soil can be represented by the equation

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

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

Equation [1] describes a planar surface on a three-dimensional plot of τ , $(\sigma - u_a)$, and $(u_a - u_w)$, as illustrated in Fig. 9. Any section parallel to the $\tau - (\sigma - u_a)$ plane appears as shown in Fig. 9b. 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

$$[2] \quad \tau = c + (\sigma - u_a) \tan \phi'$$

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 for expressing the shear strength of soils with matric suction, since it lends itself to conventional stability analysis. The strength contributed by matric suction can be accounted for by the value specified for cohesion.

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

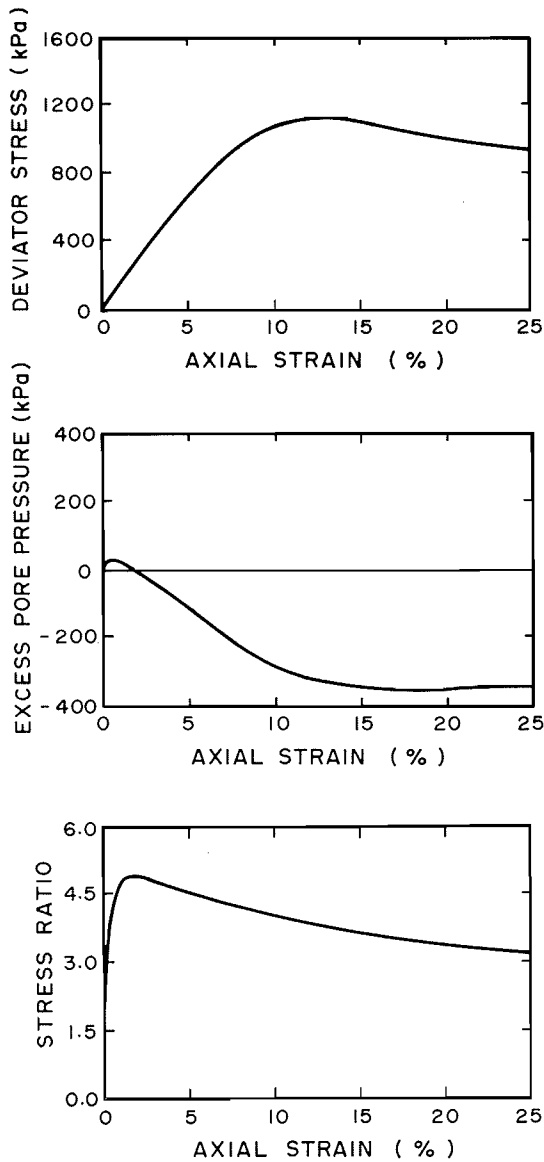


FIG. 7. Typical stress versus strain curves for specimen 2.

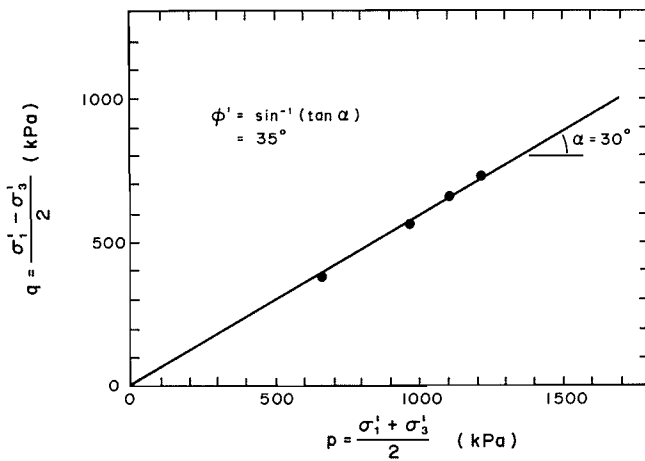


FIG. 8. Failure envelope for the saturated Notch Hill silt.

using the equipment and procedures developed by Ho and Fredlund (1982a, b).

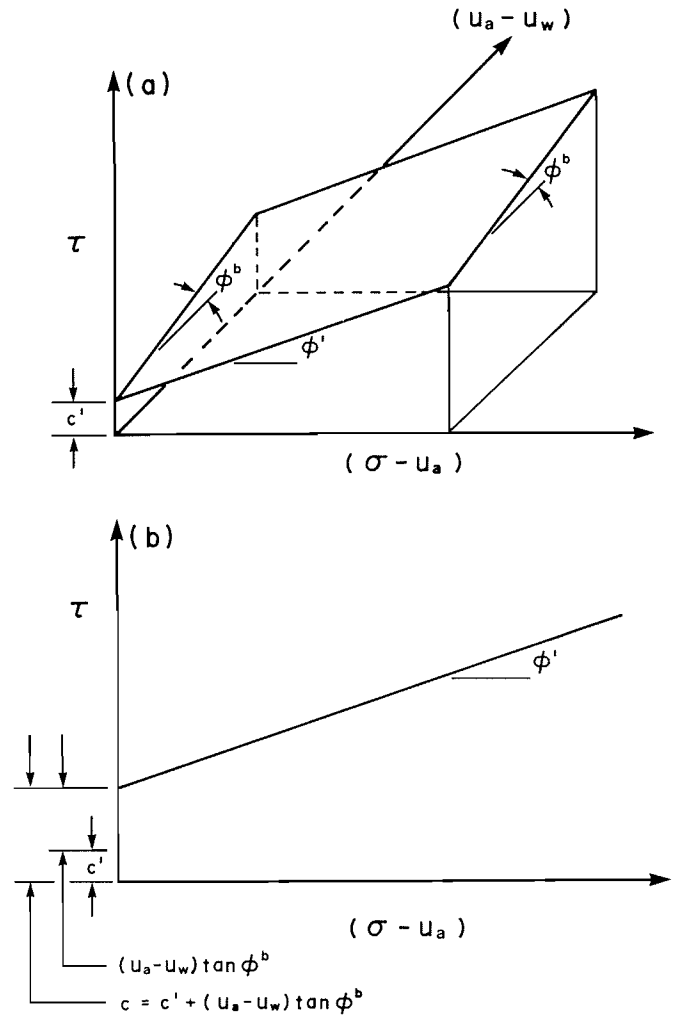


FIG. 9. Graphical representation of the shear strength relationship for unsaturated soils: (a) Extended Mohr-Coulomb failure surface; (b) increase in strength with respect to matric suction.

TABLE 1. 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	1530

NOTE: 1 pcf = 1/0.06243 kg/m^3 = 16.02 kg/m^3 .

Measuring the strength parameter, ϕ^b , involved conducting triaxial tests at several matric suction levels. Multistage testing procedures such as those used by Ho and Fredlund

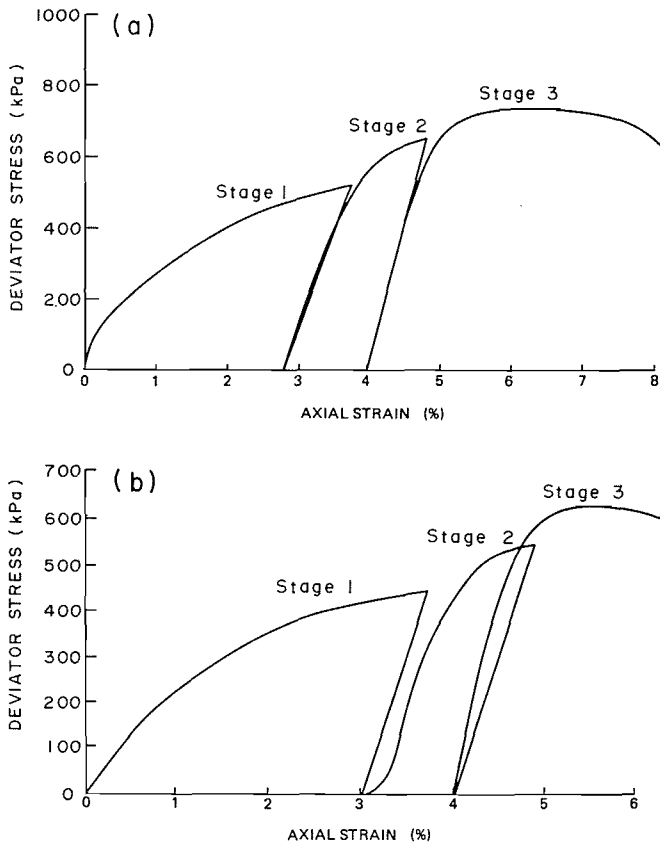


FIG. 10. Multistage stress versus strain curves for the unsaturated specimens: (a) Stages 1-3 for specimen E-2; (b) stages 1-3 for specimen E-3.

TABLE 2. Unsaturated triaxial test stress conditions

Specimen 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).

(1982a, b) make 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 variability.

The test procedure can be summarized as follows: A specimen is consolidated at a predetermined confining stress and at a low suction. Water is added to the specimen 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 a peak value. The procedure is repeated for the third suction level. Generally, the specimen becomes too distorted to perform a test procedure of 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

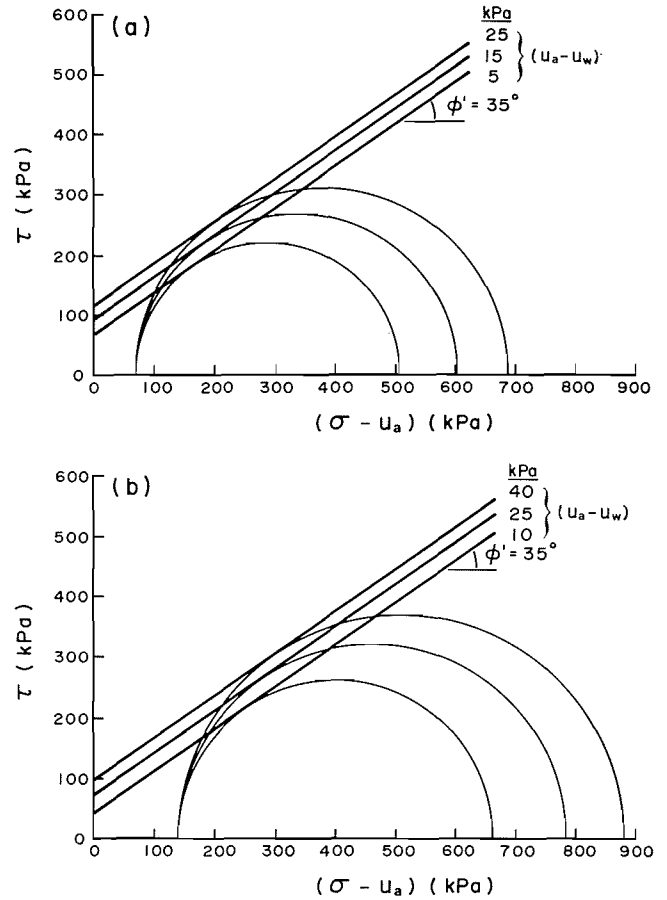


FIG. 11. Triaxial test results on the unsaturated Notch Hill silt: (a) Mohr circles for specimen E-2; (b) Mohr circles for specimen E-3.

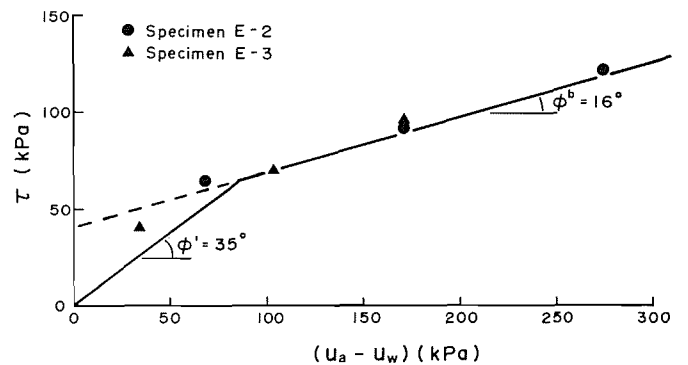


FIG. 12. Shear stress versus matric suction for the Notch Hill silt.

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 a manner similar to that described for the saturated soil testing. Table 2 presents the stress states used in the unsaturated tests.

The multistage stress-strain curves obtained for the two specimens are shown in Fig. 10, and the associated Mohr circles are presented in Fig. 11 together with lines parallel to the Mohr-Coulomb failure envelope at the various suction levels.

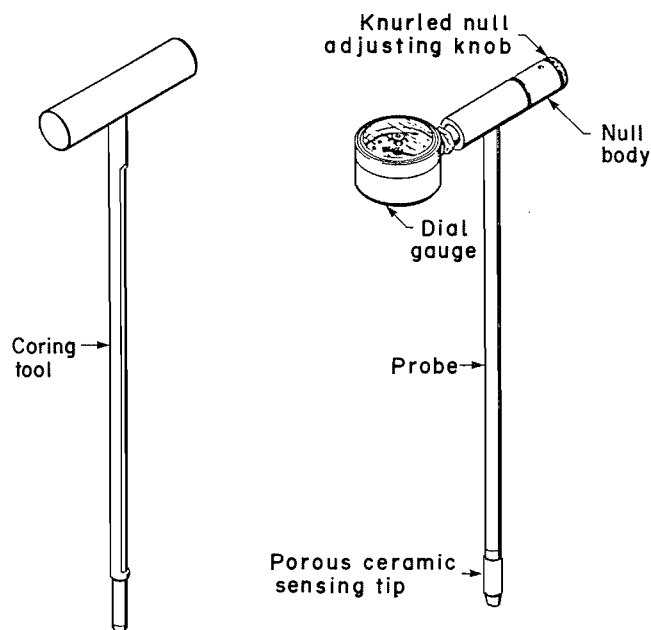


FIG. 13. Quickdraw tensiometer manufactured by Soilmoisture Incorporated.

Figure 12 presents the relationship obtained between shear strength and matric suction. The measured ϕ^b angle is 16° . This value falls within the range of typical values (i.e., 12.6 – 22.6°) published by Fredlund (1985). Extending the best-fit line at 16° back to zero matric suction results in a cohesion intercept. There are two possible explanations for this intercept. First, the unsaturated soil specimen may have had somewhat different physical properties than the soil tested for the saturated shear strength parameters. Second, it is possible that the strength versus matric suction relationship is nonlinear. Recent analyses by Fredlund *et al.* (1987) have shown that at low matric suctions, the ϕ^b approaches the effective angle of internal friction, ϕ' . The break in the envelope appears to occur at about 90 kPa, possibly close to the air-entry value for the silt.

Field suction measurements

Two test pits were excavated in the embankment slopes to sample the soils, to inspect the soil conditions, and to measure the *in situ* suctions. The initial fieldwork was done in May, 1982. Some additional suction measurements were obtained in July about 3 months later to obtain an indication of any changes in suction.

Measurements of the *in situ* suctions were made using a portable Quickdraw tensiometer³ capable of measuring suctions up to about 85 kPa (i.e., 0.85 atm) (Fig. 13). The instrument has a high-air-entry stone at one end of a metal tube 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 and 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 14 shows a photo of the instrument inserted into the sidewall of one of the test pits.

³The Quickdraw tensiometer is manufactured by Soilmoisture Incorporated, Santa Barbara, California.

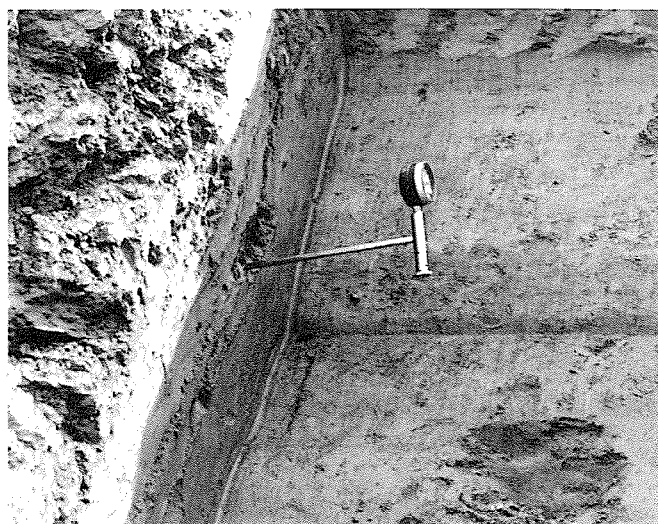


FIG. 14. The portable tensiometer being inserted in the sidewall of a trench.

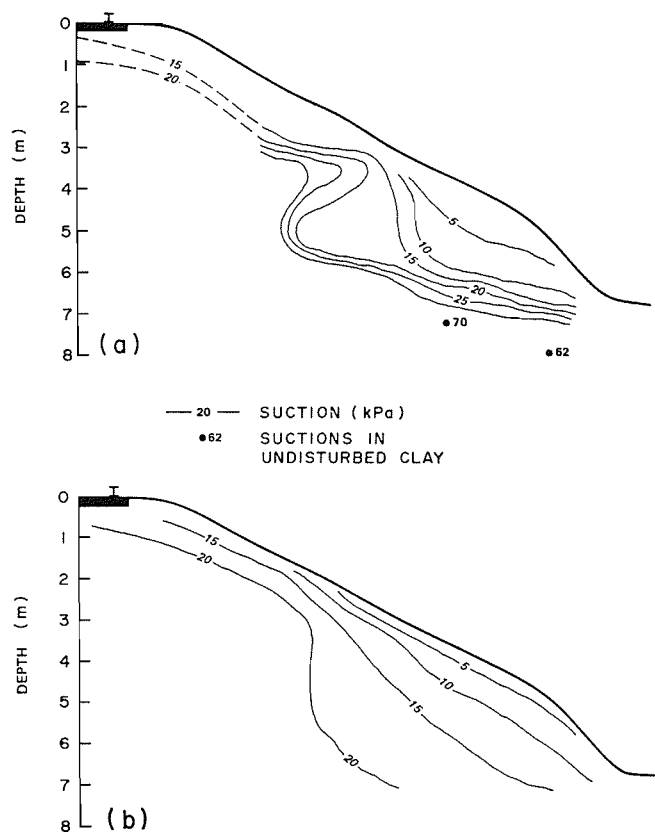


FIG. 15. Matric suction contours along the sidewalls of (a) test pit 1 and (b) test pit 2. NOTE: Suction values are in kPa.

Suction measurements were made at about 20 locations on the sidewalls of each test pit. Disturbed samples were collected at the majority of suction measurement locations for the purpose of measuring the natural water contents later.

The results of the suction measurements are summarized by the contours in Fig. 15. In both test pits there were one or two readings that were inconsistent with the general trends. These readings were consequently ignored in drawing the contours.

The suction trends and patterns of significance can be summarized as follows: The suctions were lowest near the

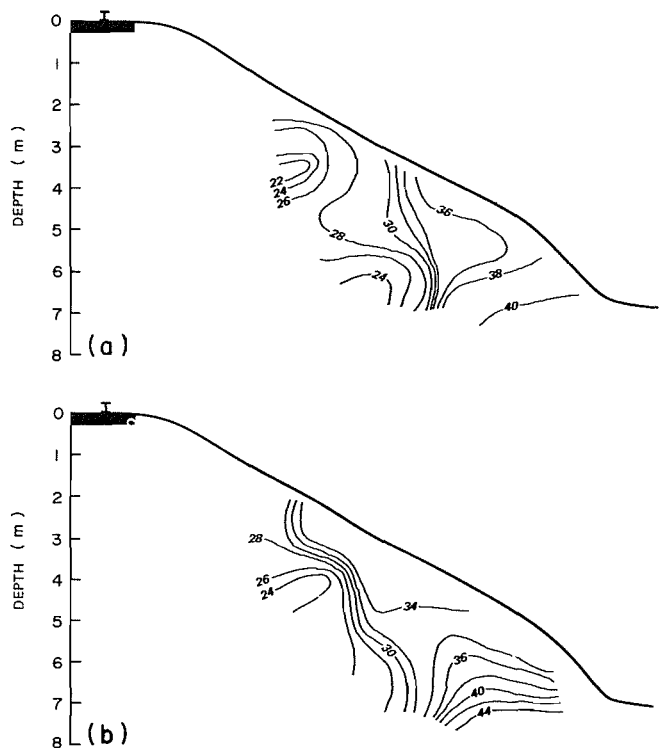


FIG. 16. Water content contours on the sidewalls of (a) test pit 1 and (b) test pit 2.

surface of the slope and increased with depth into the embankment. The 5, 10, and 15 kPa contours have a shape not unlike the shape of the observed slip surface.

The water contents (Fig. 16) were also plotted and contoured. No apparent meaningful trends or patterns emerge from the 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.

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

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 Fig. 18 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%. Also shown in Fig. 18 are the suctions that correspond to cohesion values when the ϕ^b angle is 16° . A suction of about 7 kPa becomes equivalent to a cohesion of 2 kPa. The associated change in the factor of safety is approximately 30%. 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

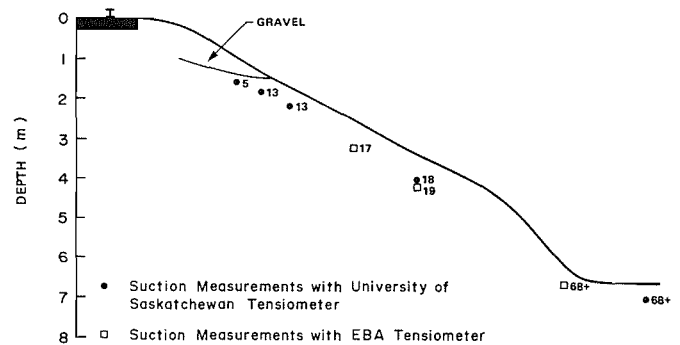


FIG. 17. Suction measurements taken in July, 1982. NOTE: Suction values are in kPa.

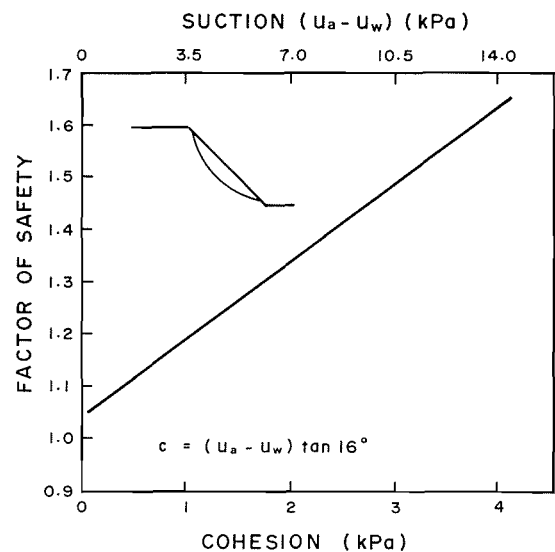
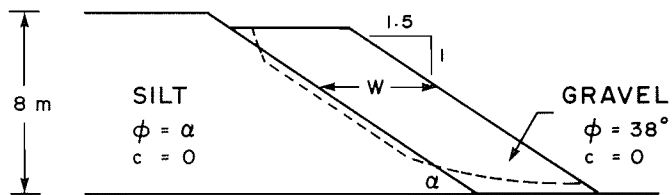


FIG. 18. Effect of cohesion and suction on factor of safety for a near-surface slip surface.

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 on the stability of a slope.

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 19 and 20 illustrate the effect of a gravel berm and the flattening of the slope with gravel. A blanket only 2 m thick can increase the factor of safety by 20% (i.e., from 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 was found for the case 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 logistical reasons rather than wholly on stability considerations. Both alternatives improve overall stability. The procedures used to place the gravel will likely vary from site to site and dictate, to some extent, 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 to spread it parallel to the slope to create a berm.



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

FIG. 19. Effect of gravel berm with slope angle the same as the embankment angle.

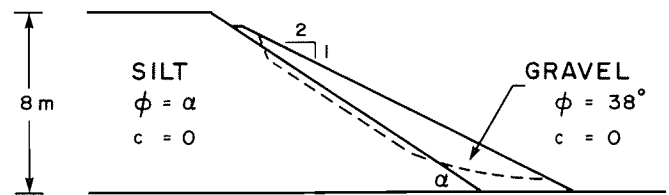
Discussion

The Notch Hill embankments were initially stable. Therefore, the soil 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 only the frictional strength, the factors of safety would at best have been near unity. As the slopes were initially stable but did not remain so, it would appear 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 those at depths within the embankment.

It is not certain what the magnitude of the suctions were at the onset of the instability. The suctions may have been lower than the values measured in May, 1982. At the time the fieldwork was conducted, 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 fieldwork. Furthermore, analytical considerations also suggest 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. There was likely some interplay between moisture infiltration, absorption, and moisture migration due to thermal gradients. Since the instability was 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



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

FIG. 20. Effect of flattening the gravel slope angle.

resulted in sufficient moisture concentrations near the slope surface after the ground had thawed to allow the suction to approach zero. The mechanism appears to be similar to that observed by Widger and Fredlund (1979) on a highway overpass built of expansive soils.

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 a steep fine-grained soil slope could conceivably be maintained by covering the slope 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 it was at the time of construction. For large and long embankments, this may not be practical but at isolated locations where rights-of-way restrict 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 criterion, the instability should likely have extended into the foundation soils where the water contents were the highest. This was not the case and indicates that water content alone, even in seemingly similar soils, cannot be considered as the factor controlling stability. As demonstrated by the suction measurements, it is the pore-water pressure that is the primary controlling factor in stability. In other words, the water content is not a direct measure of the pore-water state of stress. The water content versus suction relationship is sensitive to small changes in the composition of the soil. Hysteresis also makes it difficult to uniquely relate water content and suction.

Summary and conclusion

The railway embankment side slopes at Notch Hill were initially stable owing 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. The negative water pressure gives rise to an apparent cohesive strength, which has been demonstrated in the laboratory. The field measurements showed that there was a decrease in suction 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 emphasizes the importance of attempting to understand unsaturated soil behavior in terms of the negative pore-water pressures involved as opposed to using water contents.

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