TECHNICAL ARTICLES

Slope Behaviour in Otumoetai, Tauranga

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Introduction

The purpose of this article is to put on record some general observations on the slips that occurred in the Otumoetai area of Tauranga as a result of the May 2005 storm, and the results of some specific investigations at one particular slip. Shortly after the slips occurred, Bernard Hegan and I were asked by Tauranga City Council to inspect all the main slips and to attend a public meeting (to answer possible questions from the floor), and also to provide a report on our findings to the Council. I want to emphasise that this is an article rather than a technical paper. I am not an expert on soil conditions or geology in Tauranga and until May 2005, I had not had any major involvement with slope stability or slips in the Tauranga area. I had, however, gained the impression that Tauranga volcanic ash soils were rather different to the "brown ash" soils I had some knowledge of in Taranaki and the central plateau of the North Island, as well as in Indonesia. To some extent I undertook the assignment out of curiosity. Bernard had had considerable previous involvement in the Tauranga scene, and did his best to educate me (hopefully not without some success) about the local geology, especially the sequence and nature of the various ash layers. The main points of our observations, together with the results of some limited laboratory tests from one particular site, are set out in the following sections.

General Observations on the Slips

Our main observations can be summarised as follows:

1. Very few of the slips inspected could be considered surprising or unusual given the general steepness of the topography in which they occurred. They also appeared to occur in places likely to be vulnerable to intense rainfall, that is, places where the local topography concentrated surface run-off at the top of vulnerable slopes.
2. Most of the slips had the form of relatively shallow circular arc failures. They did not generally occur over the full height of the slopes, being restricted to the upper half (or two thirds) of the slope. This was natural because of the buttressing effect of previous slip debris at the base of the slopes. The most common form of failure is illustrated in Figure 1(a) below. The slip material generally disintegrated as it moved, and became mixed with water to form minor "mud flows". The distance travelled depended on the shape of the slope, the sensitivity of the soil, and the quantity of water available.
3. One slip that was rather unique was the large slip in Vale St that pushed a house below it a considerable distance towards the road in front. This slip extended over almost the full height of the slope and had the form of a "block" slide, with movement occurring on a preferred layer with a gentle dip towards the road. The block of soil moved semi-horizontally taking the house with it. The block partly disintegrated as it moved. The form of this slide is shown in Figure 1(b).
4. At some of the slips vertical cracks or "fissures" were observed in the Hamilton brown ash layer. Such cracks would provide natural channels for water to enter the slope and could have the effect of providing an external water force on any potential failure block.
5. Figure 1(c) attempts to illustrate diagrammatically the way in which water induces slips in slopes. The mechanism is not one whereby previously "dry" soil becomes wet. All soils, especially those in Tauranga, which consist of fine-grained clayey materials, contain a large (and essentially constant) quantity of water all year round, and apart from a shallow zone near the surface, are fully saturated. The influence of periods of intense rainfall is to "pressurise" this water, and consequently to reduce the shear strength of the soil. The longer the time over which the site is subjected to abundant supplies of surface water, the greater will be the increase in pressure and the consequent weakening effect. *(Sorry if this is all very elementary).*

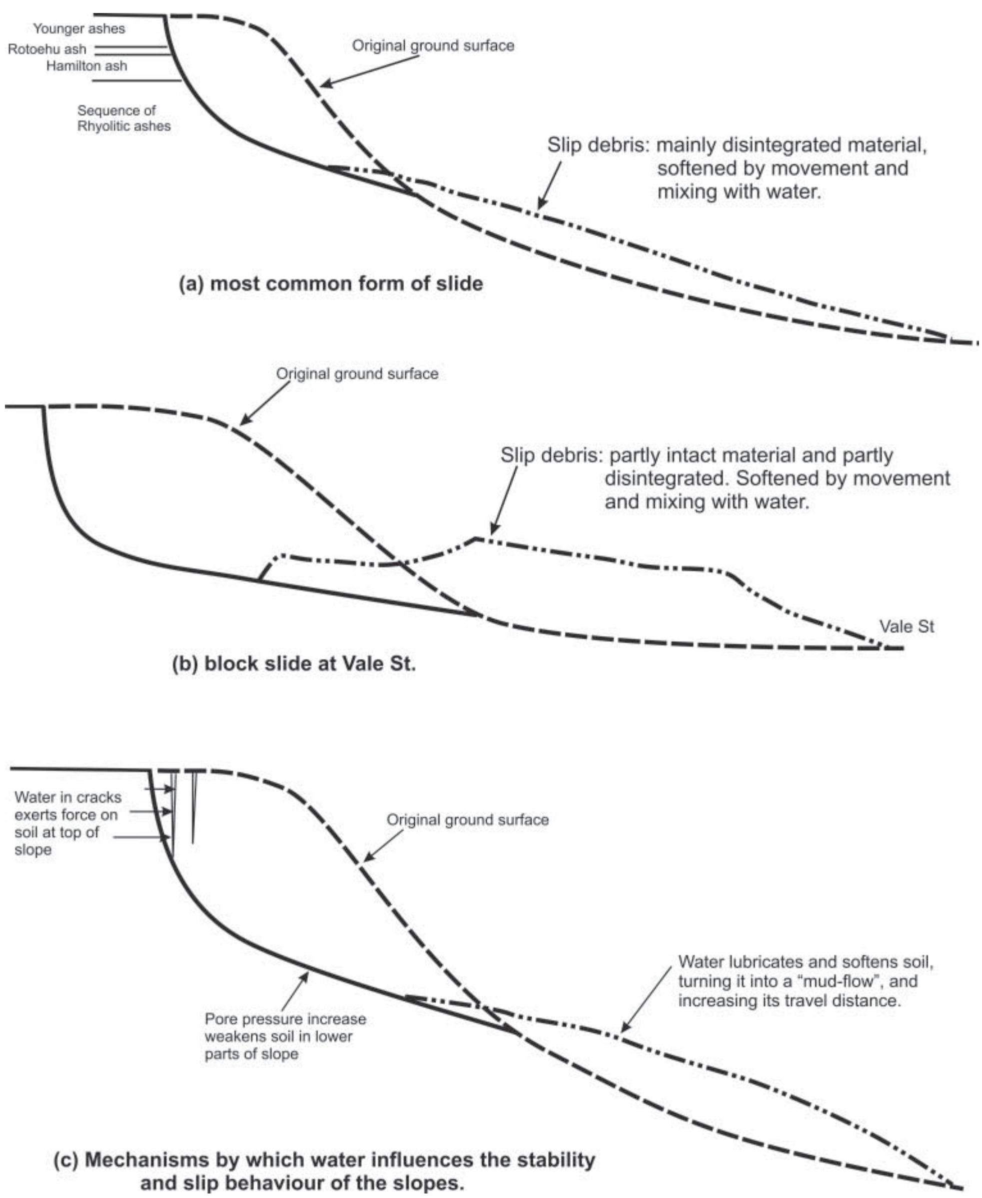

Figure 1. Forms of the slope failures.

6. The majority of the houses that suffered severe damage were at the foot of slopes rather than at the top of them. The damage was done by the impact of the slip debris material against these houses. A number of houses at the top of the slopes were left in precarious positions, although they had not actually suffered material damage. Many of these have since been removed.
7. The run-out distance of the landslip debris in general fell within a line projected at about 1V to 4H (14°) from the head-scarp (the top edge) of the slip.
8. Surface water run-off appeared to be an important contributing factor to many of the failures. A number of streets ended in cul-de-sacs sloping down toward the top of steep slopes, followed by driveways ending almost at the edge of the slope. These acted as surface flow paths, and local residents spoke of torrents of water flowing down these driveways and then onto the slopes. In effect the streets formed streams and many driveways became spillways discharging the flow at the top of the slopes.

Despite what is said in Point 1 above, the slopes in volcanic ash soils in Tauranga seem to be more prone to slips than similar slopes in Taranaki or the central plateau. While most of the slips at Otumoetai occurred in old cliff slopes, there were a substantial number (possibly the majority) that occurred in the slopes of valleys that run "inland" from the old cliff line. The precise explanation for these valleys is unclear, but the horse-shoe shaped head of these valleys tends to concentrate surface run-off as well as seepage flow through the ground, so that these areas become particularly susceptible to continuing slope instability.

Geology and Soil Conditions

In very general terms the volcanic ash sequence, from the surface downwards, (as I understand it) in the Otumoetai area consists of the following:

(a) Pale brown "younger ashes", including the very sandy Rotoehu ash. The Rotoehu ash is found at an average depth of about 2.5m and is usually between about 0.5m and 1m thick.

(b) The Hamilton "brown" ash, which is found immediately beneath the Rotoehu ash. This material is generally stiff to hard in consistency and about 1m thick.

(c) Below the Hamilton ash there is a series of layers known as the Matua Subgroup. These consist mainly of creamy white fine grained rhyolitic ash, but which also contain a number of paleosols (old ground surfaces). These paleosols appear to be very similar to the Hamilton brown ash, being generally of higher strength, lower sensitivity, and lower permeability, than the intervening layers. At Otumoetai there are no true "basement" rocks and the ash sequence extends down to sea level. Elsewhere within Tauranga city, variably weathered ignimbrite and weakly cemented volcaniclastic sediments underlie the ash sequence.

The term "ash" is used here only to distinguish the various layers; they are all well weathered and geotechnically are now clays or silts, or something in between. The properties of the ash layers vary considerably and they no doubt influence to some extent the way failures occur. The Hamilton ash is believed to be derived from andesitic parent material, and appears to have very similar properties to other "brown ashes" found in Taranaki and the central plateau. They are assumed to contain a considerable amount of the clay mineral allophone. As some readers will know, I have been involved with allophone clays for some years (forty seven, to be precise). Allophane clays generally perform very well in slopes, and their geotechnical properties are good. However, the layers beneath the Hamilton ash are believed to be derived from rhyolitic ash, and have some rather different properties to the brown ash. In particular they are weaker and are often of high to very high sensitivity. These differences in properties are described in greater detail in the next section, giving an account of an investigation into one particular slip.

The Slip at 198 Grange Road

General Description

An opportunity to carry out some specific (but very limited) tests was provided by a reasonably large slip that occurred at 198 Grange Road. This slip did not immediately threaten any houses, and for this reason has been left more or less as it was when the slip occurred. A cross section of the slope is shown in Figure 2. This cross section is not highly accurate, but is a good approximation, having been determined with an electronic "Abney" level, backed up by direct tape measurements. As evidenced by the profile, the slope had been benched at some time prior to the slip. Two slips actually occurred in May 2005. The lower slip was by far the largest and extended a considerable distance laterally along the slope. The upper slip was of very limited lateral extent and had the form of a narrow gully. This upper slip may have been more the result of erosion than shear failure. The lower slip had the appearance of a typical circular arc failure. Hardly any of the material that slipped remained on the site; it disintegrated and disappeared into the scrub covered marshy area below the site. There was very little indication from the general appearance of the failure surface that its shape had been significantly influenced by any particular soil layer. It seemed to cut through all the layers in its path.

Figure 2 also shows descriptions of the layers, and the approximate boundaries between them. The arrows show the locations from which disturbed samples were taken for laboratory testing. Four of the layers had the appearance of typical "brown ash". These are labelled as brown ash, and the samples taken from there were S1, S5, S9, and S11. It was very apparent at the time the samples were taken that the brown ash was strong (stiff to hard) and of low sensitivity, while the intervening light coloured soil was weaker and generally highly sensitive. The laboratory tests included natural water content and Atterberg Limits (natural and oven dried), on all the samples, particle size on six samples, and residual friction angle on four samples. A hand vane was used to measure the in situ undrained shear strength, and also the remoulded strength. These strength measurements were done later than the sampling, by which time the upper slip had been largely filled in so as to buttress the driveway above, making strength measurements no longer possible.

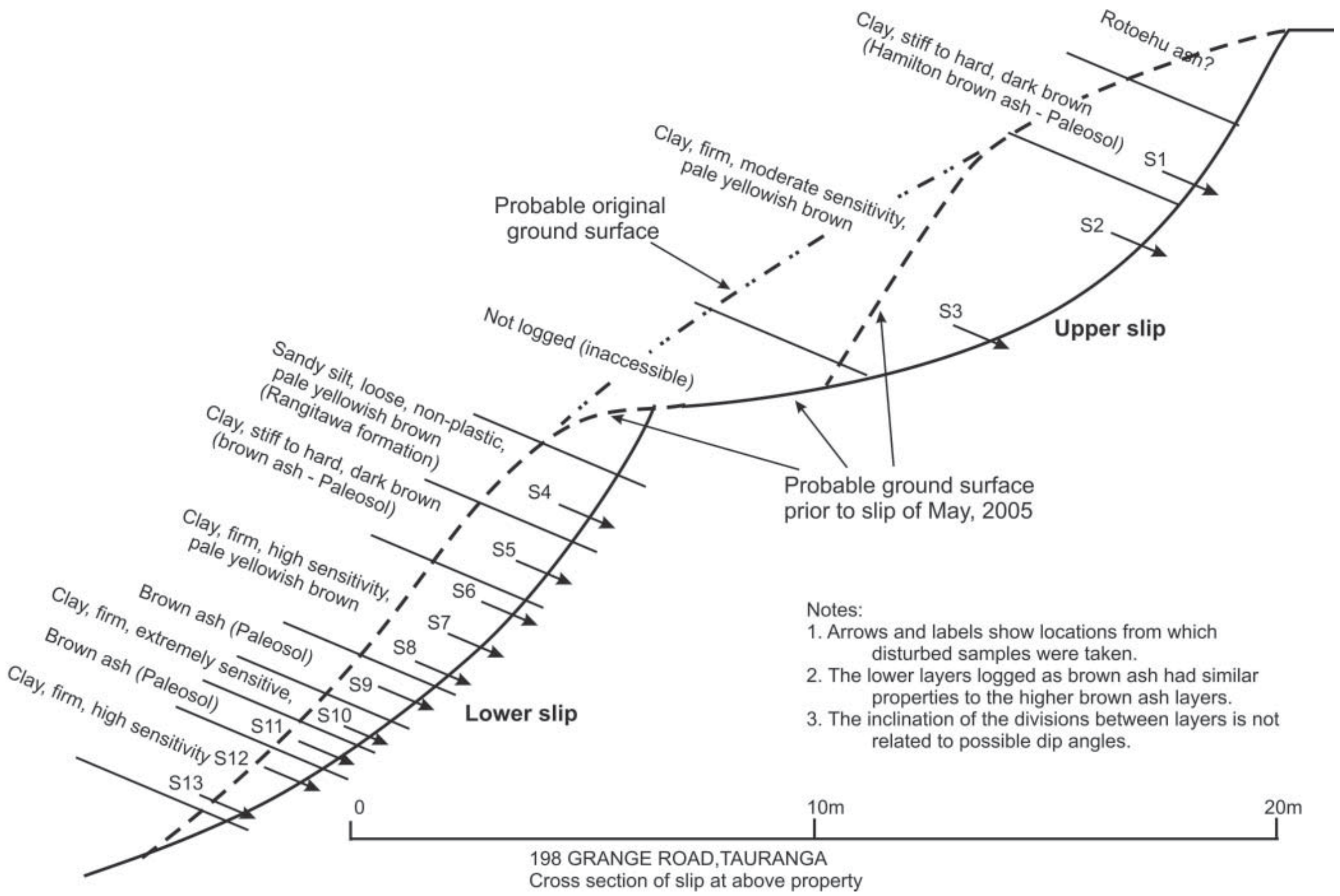

Figure 2. Cross section of the slope at 198 Grange Road, Tauranga.

Results of the Laboratory Tests and Undrained Strength Measurements

The results of the tests are summarised in Tables 1 and 2, and set out in graphical form in Figures 3 to 6. Comments on these results are as follows:

1. In Figure 4 the results are plotted against vertical depth below the highest point, this being the crest of the scarp at the upper slip. Plotted in this way the vertical distance between the samples is quite small, although when taken the distance between samples was much greater because of the angle of the slope. Samples S1 to S3 were from the upper slip and S4 to S13 from the lower slip. The top sample from the main slip, S4, was non-plastic sandy silt; only natural water content and particle size measurements were made on this soil. It is believed to be from a layer called the Rangitawa formation. The dip of the layers is not known.
2. The position of the Rotoehu ash layer in this profile is somewhat uncertain. It is probably the top layer of the upper slip. It was difficult to get access here but it seemed probable that the top brown ash layer was the Hamilton ash, in which case the layer immediately above it should be the Rotoehu ash.
3. The most striking features of the tests are the rapid changes in properties over short distances, and the extremely high sensitivity of some of the light coloured (rhyolitic) samples. Samples S1, S5, S9 and S11 were “brown ash” and generally of low sensitivity. The other (light coloured) samples were of high to very high sensitivity, as mentioned earlier. Samples S10 and S12 had natural water contents significantly above the Liquid Limit, giving them sensitivities around 100 and 70 respectively.
4. The samples occupy the normal position of volcanic ash soils on the Plasticity Chart (Figure 3) and all undergo some loss of plasticity after oven drying. In this respect, there is no significant difference in behaviour between the brown samples and the light coloured samples. Loss of plasticity is generally a good indicator of allophone content, at least in the writer’s view. These tests do not suggest that the allophone content is significantly different between the two soil types.

Sample No	Soil description	Natural w/c (%)	Atterberg Limits			Liquidity Index
			LL	PL	PI	
S1	CLAY, hard, low to medium sensitivity, homogeneous, brown ("Brown ash")	70.1	91	55	36	0.42
S2	CLAY, stiff, moderate sensitivity, pale yellowish brown, black specks	67.0	76	54	22	0.59
S3	CLAY, stiff, moderate to high sensitivity, pale yellowish brown, black specks.	85.3	93	50	43	0.82
S4	SANDY SILT, non-plastic, loose, pale yellowish brown (Rangitawa formation)	33.9	NP	NP	NP	-
S5	CLAY, stiff to very stiff, low sensitivity, homogeneous, brown ("Brown ash")	81.5	116	58	58	0.41
S6	CLAY, stiff to very stiff, high sensitivity, yellowish brown with many black specks.	94.8	96	55	41	0.97
S7	CLAY, stiff, high sensitivity, very pale yellowish brown with many black specks.	86.5	96	52	44	0.78
S8	CLAY, stiff to very stiff, high sensitivity, yellowish white with many black specks.	83.4	91	48	43	0.82
S9	CLAY, hard, homogeneous, non-sensitive, brown ("Brown ash")	61.0	90	58	32	0.09
S10	SILTY CLAY, stiff, low plasticity, very high sensitivity, pale yellowish brown with black specks.	68.7	60	55	5	2.7
S11	CLAY, hard, low sensitivity, homogenous, brown ("Brown ash")	75.7	104	65	39	0.27
S12	CLAY, stiff, very high sensitivity, pale yellowish brown.	92.3	85	52	33	1.22
S13	CLAY, stiff, low sensitivity, pale yellowish brown with some specks.	82.1	98	66	32	0.50

Table 1. Soil Descriptions and Basic Properties

Sample No	Undrained shear strength (kPa)		Sensitivity	Particle size			Residual friction angle ϕ'_r
	Undisturbed	Remoulded		Clay fraction (%)	Silt fraction (%)	Sand fraction (%)	
S1	-	-	-				
S2	-	-	-				
S3	-	-	-				
S4	-	-	-	19	13	68	
S5	129	15	7	44	41	15	20.7
S6	107	2	54				
S7	97	4	24	25	28	47	16.6
S8	100	3	34				
S9	207	60	3.5				
S10	97	1	97	16	34	50	
S11	210	38	5	44	43	13	24.2
S12	72	1	72	18	35	47	19.1
S13	77	13	6				

Table 2. Additional Soil Properties

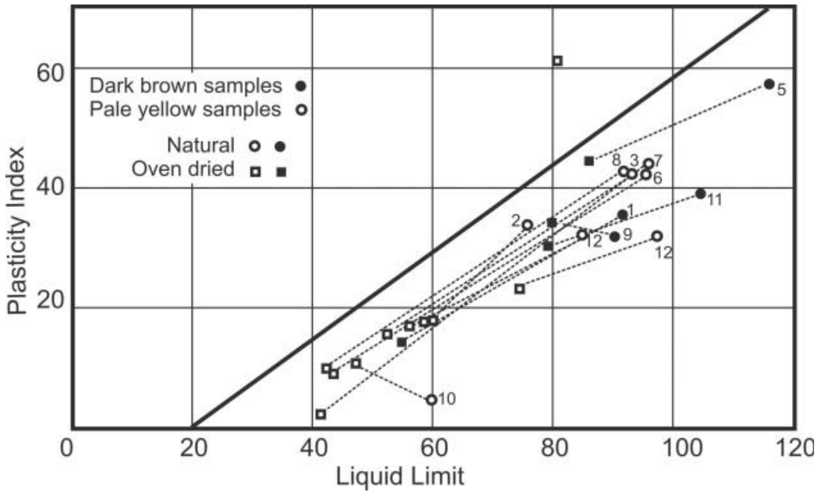

Figure 3. Atterberg limits on the Plasticity Chart.

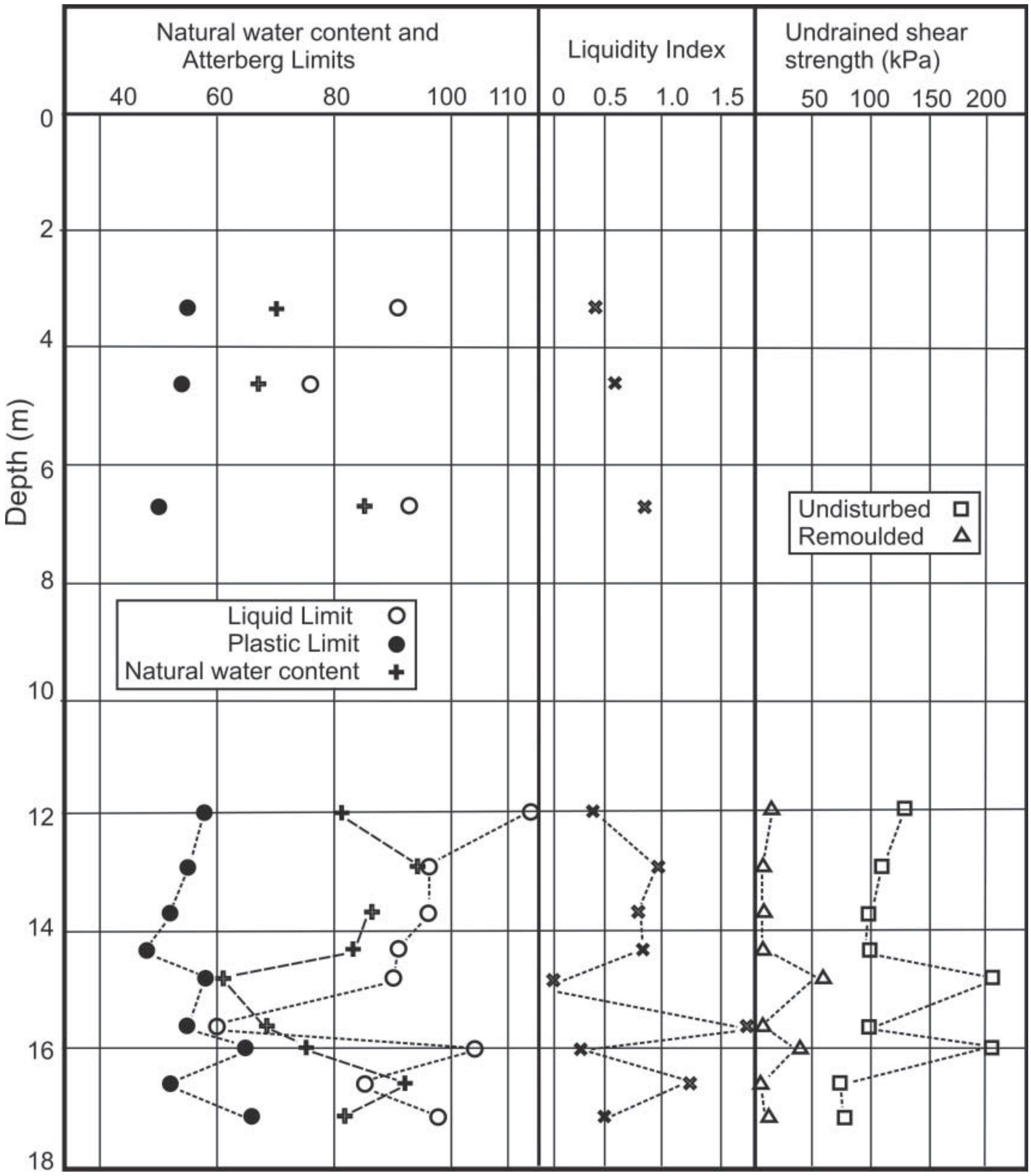

Figure 4. Results of tests on disturbed samples, plotted against depth.

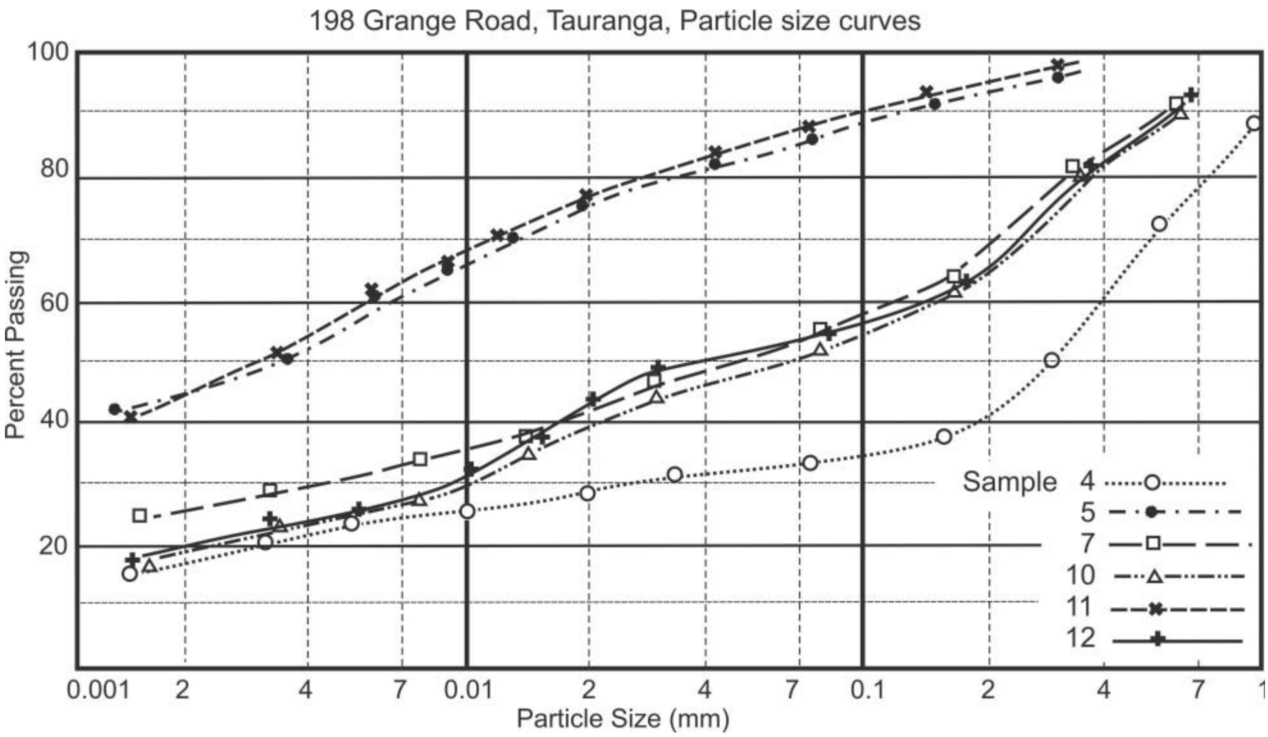

Figure 5. Particle size curves from four samples.

5. Two of the particle size curves (Figure 5) were from brown ash samples; these were S5 and S11. Another three were from the pale coloured samples, and the last from the non-plastic layer (Rangitawa formation). There is surprising similarity between the curves from the two soil groups.

To summarise, the most interesting finding from these tests was the rapid change in properties with depth because of the presence of the two soil types, and the extremely high sensitivity of the light coloured, rhyolitic soils.

Stability Analysis

Whether there is much to be gained from stability analysis in this situation is open to debate, since reliable information is not available on the seepage state in the slope or the shear strength parameters of the soil. Regardless of that, use has been made of the programmes SEEP/W and SLOPE/W to see what a theoretical analysis comes up with. Firstly, SEEP/W was used to establish a flow net and the way it would develop, assuming an initial low water table and continuous rainfall at the ground surface. This is easily done using the transient capability in the programme. The soil parameters adopted (somewhat arbitrarily, and subject to adjustments) for the seepage analysis and the stability analysis that followed are as follows:

Seepage analysis (the soil was assumed to be fully saturated):

Coefficient of permeability, k = 0.05m/day
Coefficient of compressibility,
$m_v = 1 \times 10^{-4}$ kPa^{-1}

Stability analysis:

Unit weight, $\gamma = 15.3$ kN/m^3
Cohesion intercept, $c' = 10$ kPa
Angle of shearing resistance, $\phi' = 35^o$

Despite the very high value of k and low value of m_v adopted, the time needed to establish an equilibrium seepage pattern was about 20 days. The seepage analysis therefore extended over 20 days with daily time steps. The long term seepage pattern this leads to is shown in Figure 6. It is of interest to note that the water table reaches the ground surface sometime between the first and second time step, that is between one and two days, but it takes a long time after this for the final steady state seepage situation to develop. The same phenomenon occurs with level sites. The time for the water table to reach the ground surface as a result of prolonged rainfall may not be very great, but the time for the pore pressures beneath the water table to reach equilibrium is very much longer.

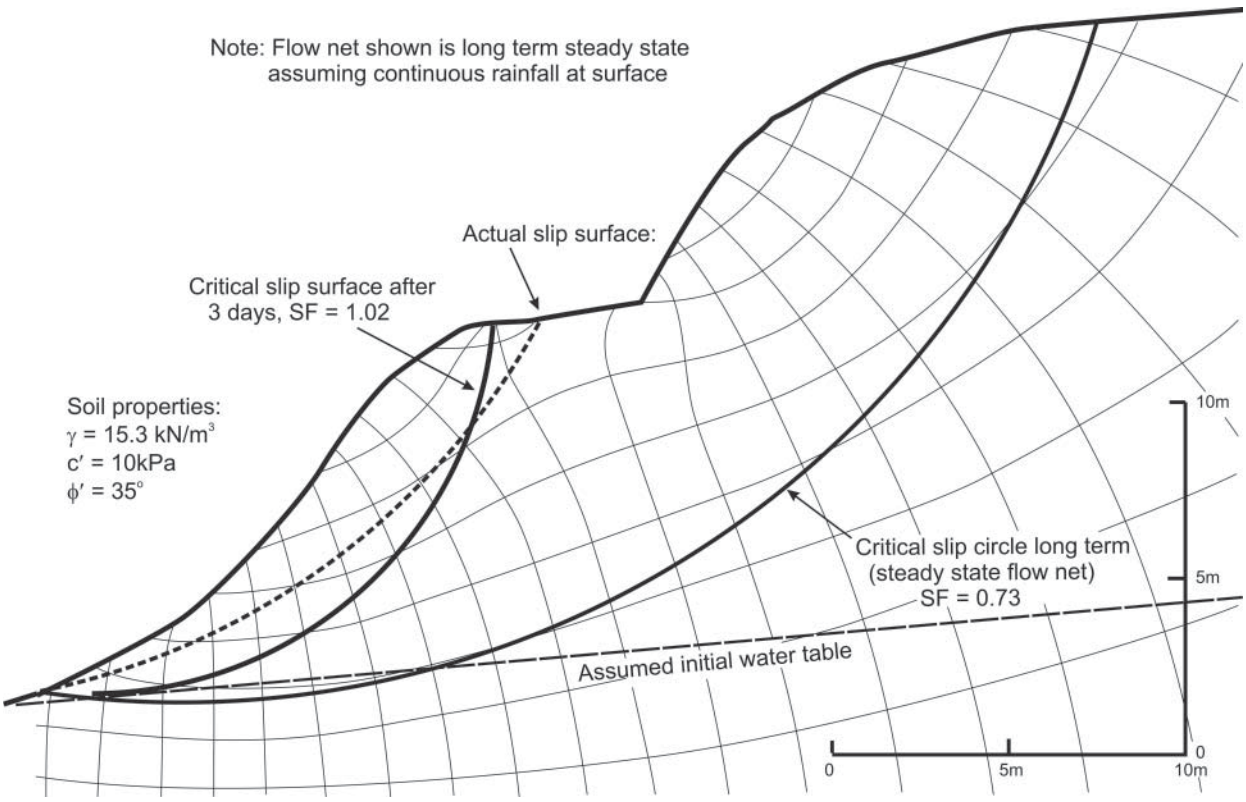

Figure 6. Slope cross section with flow net and failure surfaces.

As users of these programmes will know, SLOPE/W can carry out its stability analysis using the flow patterns determined from SEEP/W. This is a very convenient feature of the programmes made use of here. Safety factors were calculated at time steps of 1 to 4, then 6, 10 and 20. The steady decrease in the safety factor with time is illustrated in Figure 7.

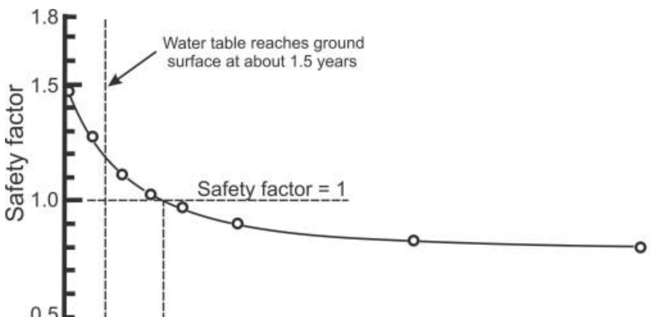

Figure 7. Decline in safety factor with time.

After 3 days the safety factor has decreased to unity; the corresponding critical circle is shown in Figure 6. If the rain continues to fall for 20 days (and the slope doesn't fail) then the steady state flow pattern would develop; the safety factor drops further to only 0.73 and the critical circle is in the new position in Figure 6. The critical circle remains within the lower slope until day 4 when it moves to encompass the complete slope. It is seen that the critical circle at the time the safety factor falls to unity agrees surprisingly well with the actual failure surface.

It is possible, by trial and error, to determine a combination of parameters that give an even better fit to the actual failure surface. The combination is: $c' = 6$ kPa, $\phi' = 35^{o}$, $r_u = 0.2$. The relatively shallow failure surface on the steepest part of the slope is indicative of a material in which the frictional component of shear strength is more influential than the cohesive component. An even better fit might be obtained by further reducing the c′ value and increasing the ϕ′ value. The time steps used in the analysis are rather arbitrary. It may be that the m_v parameter used was much too high. No one has measured m_v values in situ for any soil using the natural rise and fall of the pore pressure in the ground as the loading and unloading mechanism, so adopting a laboratory value may be a long way from the true value. Adjusting the parameters will alter the duration of the time steps but everything else will remain the same.

Discussion and Conclusion

The most interesting information to come from the study at Grange Road is the extreme sensitivity of some of the layers, and the rapid changes in sensitivity with depth. The presence of these sensitive layers is clearly the most significant property of the rhyolitic ashes, but the extent

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to which sensitivity influences slope stability is not at all well understood. It is undoubtedly true that it strongly influences post failure behaviour. One of the reasons the slips in the May 2005 storm travelled as far as they did was the high sensitivity of the soil. The other reason was the abundant supply of water to further soften the soil after failure and lubricate the failure surface. The extent to which sensitivity influences pre-failure behaviour is problematical. It is certainly not the case that sensitive soils lose strength because they become wet. Neither is it they case that these soils liquefy and this causes the failure. Like any other soil, sensitive soils lose strength as pore pressures rise within them, eventually to the point of causing failure, and then liquefy as shearing continues. Their dramatic loss of strength occurs after failure.

Sensitive soils do not all behave in a similar manner. Their behaviour depends both on their sensitivity and on the ease with which they are remoulded. Some sensitive soils require very little energy to completely remould them while others require a lot of effort (or energy) to reduce their strength to its fully remoulded value. Not enough careful laboratory testing has been done on highly sensitive soils. This is probably partly due to the difficulty of obtaining good quality undisturbed samples, but also because relevant testing would require stress controlled triaxial equipment capable of measuring loads and pore pressures at low stress levels. The average confining pressure on the slip surface at Grange Road is probably about 20 to 30 kPa, so the measuring equipment would need to be accurate to about 0.1 kPa or 0.2 kPa. The most informative tests on the Tauranga rhyolitic ashes would probably be triaxial tests in which the sample is set up and subject to the effective stress state it exists under in the ground, and the pore pressure then slowly raised to induce failure. Such tests would be stress controlled tests rather than conventional strain controlled tests. Cyclic undrained triaxial tests would also be useful in evaluating their behaviour, especially their susceptibility to strength loss during earthquakes.

Apart from this sophisticated laboratory testing, it would be helpful to have a better understanding of the weathering process in these ashes. In particular the following questions arise from the situation at 198 Grange Road:

(a) Are the brown ash layers identified here as paleosols derived from essentially the same parent material as the intervening pale coloured layers?

(b) Why are the properties of these brown layers distinctly different from the pale coloured layers? Is it because exposure at their original surface results in a different weathering process, or does it simply accelerate the weathering so that these layers represent a more advanced stage of weathering?

Not too much should be read into the theoretical analysis above, but the following points should be noted:

(1) The theoretical rate (based on parameters from laboratory tests) at which pore pressures rise in slopes is generally much slower than that indicated by storm events like that of May 2005, in Tauranga. As mentioned above, the coefficient of permeability used in the analysis was far higher than that obtained from laboratory measurements, but still indicated a relatively slow response time for the pore pressure state. It is also possible that the true m_v value of the soil may be much lower than that used here.

(2) It is likely that the actual seepage and pore pressure state in the slope may not bear much resemblance to that indicated by theoretical analysis. The theoretical analysis here assumes uniform soil properties, which is clearly not the case. The presence of fissures and discontinuities further complicates the picture and these may be the predominant channels by which the pore pressure rises in the slope.

As for practical relevance, the conclusion to be drawn from the above investigation is that the soil characteristic most likely to be an indicator of the susceptibility of a slope to failure would be the presence of highly sensitive layers dipping in a direction favourable to slip movement. This is probably not saying anything more than is already known by those familiar with the Tauranga situation. However, exactly how these sensitive layers should be taken into account in assessing slope stability is not clear. At an analytical level, my observation would be that if a safety factor of 1.5 is considered acceptable for a non sensitive soil, then a safety factor of 1.7 or 1.8 would be appropriate for a highly sensitive soil. This is not entirely an arbitrary judgement. My understanding is that there are always overstressed zones within a slope if the safety factor is less than about 1.7. In other words, some yield occurs, but does not normally lead to failure. With highly sensitive soils this internal yielding may be more serious, and lead to loss of strength and possibly failure. Using a higher safety factor would minimise this possibility.

Acknowledgement

I want to express my thanks to the following: Firstly, Tauranga City Council for their invitation to be involved in the aftermath of the May 2005 storm. Secondly, a number of people from the geotechnical fraternity in Tauranga for their observations and assistance, in particular I want to thank David Milner, Sally Hargraves, and Marriane O'Halloran. Finally, I want to thank Mr and Mrs French, the owners of the property at Grange Road for allowing me access to their property.