The paper was published in the proceedings of the 9th Australia New Zealand Conference on Geomechanics and was edited by Geoffrey Farquhar, Philip Kelsey, John Marsh and Debbie Fellows. The conference was held in Auckland, New Zealand, 8 – 11 February 2004.

Earthquake induced slope displacements and the shear strength of closely jointed rock masses

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Summary: This paper continues work on the assessment of the shear strength parameters for the closely jointed greywacke rock masses around Wellington. Greywacke (an indurated sandstone of Mesozoic age in which the unweathered rock material is very strong and hard) is one of the principal basement rocks in New Zealand and, because of complex tectonic history, it is closely jointed. In previous papers the Casagrande resistance envelope for Wellington greywacke slopes was estimated from the back analysis of existing stable slopes. The Casagrande envelope lies well below what would seem, from the descriptions in the literature, to be a possible Hoek-Brown failure envelope for these rock masses. The effects of seepage and earthquake loading have also been investigated and it was found that a suitable earthquake peak ground acceleration moves the mobilised shear strength curve close to the estimated Hoek-Brown envelope. Calculation of the response of numerical slope models to earthquake acceleration time histories extends the previous work. Satisfactory performance of the slopes is found possible, in that the post-earthquake permanent displacements are modest, even when the shear strength parameters for the rock mass allow some failure during earthquake loading.

INTRODUCTION

This paper continues work reported earlier (Pender 1999, Pender & Free 1993, and Pender 1990) in which back analysis of existing slope height – slope angle data was used to obtain a lower bound on the shear strength envelope of a closely jointed rock mass. The particular slopes are those around the city of Wellington in New Zealand in which the rock type is greywacke – a highly indurated sandstone of Mesozoic age. The standard method for assessing the strength of a geotechnical material is to recover a sample and test it in the laboratory, or, alternatively, conduct *in situ* tests. In the case of a closely jointed rock mass neither of these approaches is feasible; consequently back analysis is used to obtain some indication of the shear strength properties of the rock masses.

Slope height – slope angle data for Wellington slopes (Grant-Taylor, 1964), are shown in Figure 1. Two limiting cases are evident in this figure, an upper limit for the best material and a lower limit for the material near the Wellington Fault. As the context of this paper is stability analysis, there is a temptation to consider all the points on Figure 1 as the result of slope instability. However, Grant-Taylor emphasises that other mechanisms, for example erosion, are likely to be responsible for at least some of the points in the diagram. Thus we have concentrated on the upper bound of all the points in Figure 1 as this leads to the highest values for the mobilised shear strength of the rock mass that can be obtained by back analysis of the data. All the work reported in this paper was based on four points along the upper bound curve in Figure 1: a 16 m high slope at 75°, a 50 m high slope at 65°, a 110 m high slope at 55°, and 200 m high slope at 45°. Previous back analysis showed that it is not possible to model these four combinations of slope height and angle with a single linear c and ϕ failure envelope. The mobilised shear strength curve obtained (Pender & Free 1993) for a dry rock mass under static conditions is plotted in Figure 2. Clearly the rock close to the Wellington Fault cannot be expected to stand as well as material remote from the fault, as near the Wellington Fault the rock is likely to be more closely jointed with interlocking much reduced. Unlike the upper bound in Figure 1, the data for slopes near the Wellington Fault can be matched reasonably well with a single set of Mohr-Coulomb shear strength parameters: c = 30kPa and ϕ = 26°. As this friction angle is considered too low for a typical closely jointed rock mass, the heights of the slopes adjacent to the Wellington Fault must be controlled either by the properties of the fault zone or possibly by a gradual surface deterioration of the rock.

The Hoek-Brown failure criterion is, in principle, capable of describing rock masses such as those in closely jointed Wellington greywacke. It has been presented in a number of forms. The modified version of the criterion (Hoek at al

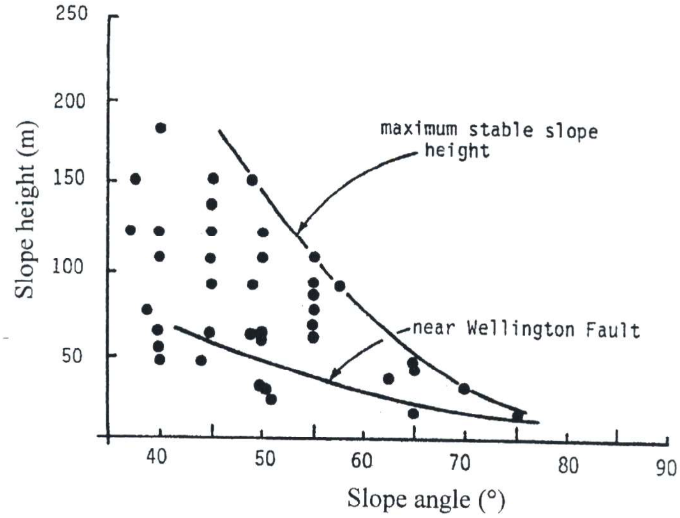

Figure 1. Wellington slope-height slope-angle data (after Grant-Taylor 1964).

1992) is expressed in terms of principal stresses:

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a \tag{1}$$

The parameters m_b, s and a depend on the intensity of jointing in the rock mass. A basic input parameter is the unconfined compression strength, σ_c, of the unweathered rock; for NZ greywacke this is frequently well in excess of 100 MPa. However, in the earlier paper, Pender (1999), it seemed more appropriate to use a smaller value of 50 MPa. (It is of interest that independent work on the shear strength properties of greywacke rock masses (Read et al, 1999) found that a value of 60 MPa for the unconfined compressive strength was needed to achieve a reasonable Hoek-Brown failure envelope.) Using the guidance given in by Hoek et al (1992), and the comments of Hoek (1998), values were chosen for the parameters a and m_b (0.5 and 1.2). A material with zero tensile strength is obtained by setting s to zero. The corresponding failure envelope for the rock mass, obtained using calculations outlined by Hoek et al (1992), is plotted in Figure 2 along with the mobilised shear strength curve for the Wellington slopes. In passing, it is of note that the shear and normal stresses for the Mohr-Coulomb failure envelope are a very small fraction of the assumed unconfined compression strength for the intact rock.

An early back analysis (Pender & Free, 1993) was done on the assumption that the slope is dry and that there is no earthquake acceleration present; this gives a lower bound on the mobilised shear strength curve. Inclusion of steady state seepage was found to increase by only a modest amount the size of the mobilised shear strength envelope (Pender, 1999).

EARTHQUAKES IN WELLINGTON

Wellington has a history of earthquake occurrences. The most recent major events were in the nineteenth century and caused uplift of part of the present city area. However, geological and geomorphological investigations reveal that major events have occurred many times in the past. Thus the slopes represented in Figure 1 have probably been subjected to many severe earthquakes, which, assuming satisfactory performance, is an another pointer to the shear strength of the rock masses. Unfortunately recorded ground motions, or even peak ground accelerations, are not available for these events. However, studies of earthquake risk in Wellington give an indication of likely peak ground accelerations that can be expected in the region. These studies are the basis of the design spectra in the existing and draft NZ Loading Standard (Standards New Zealand, 2002). For a rock site this document gives a PGA (peak ground acceleration) of about 0.4 g at a return period of 500 years and about 0.8 g at a return period of 2000 years. These values are used below to represent the levels of PGA that the Wellington slopes might have been subject to in the past.

EARTHQUAKE EFFECTS

Two approaches are used to estimate the demands earthquake excitation places the slopes: a pseudo static method, which estimates the horizontal acceleration to induce limiting equilibrium, and a computational approach, which calculates the response of the slopes to an earthquake acceleration time history.

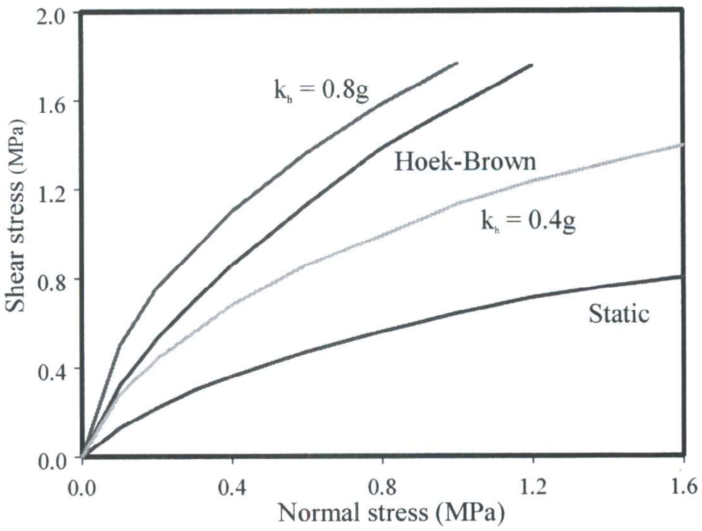

Figure 2. Hoek-Brown failure envelope, static mobilised shear strength envelope, and mobilised shear strength envelopes for 0.4 g and 0.8 g horizontal accelerations.

Pseudo static estimates

Prater (1979) and Chen and Liu (1990) present a pseudo-static approach based on a logarithmic spiral failure surface to assess the effects of horizontal and vertical accelerations in a slope. For given values of friction angle and cohesion Prater tabulated values of the horizontal acceleration coefficient, k_h, which will take a slope of a certain height and angle to limiting equilibrium, these values are very similar to those given by Chen and Liu. The range of tabulated values was extended by coding in Mathcad the expressions given by Prater. One thus obtains from Prater's equations a range of linear c, ϕ failure envelopes corresponding to the limiting equilibrium of a particular slope configuration at a given horizontal acceleration. The Casagrande resistance envelope (Casagrande 1950) for the particular geometry is then obtained by sketching an inner envelope to these failure envelopes. The mobilised shear strength curve is obtained by enveloping the resistance envelopes for the individual slopes. The mobilised shear strength curves are plotted in Figure 2 for horizontal accelerations of 0.4 g and 0.8 g. It is seen that the shear strength mobilised by a 0.4 g horizontal acceleration lies below the Hoek-Brown envelope but that that corresponding to 0.8 g lies above the Hoek-Brown envelope. The 0.4 g mobilised shear strength curve is determined at lower normal stresses by the steep slopes and at high stresses by the shallower slopes, Pender (1999). The 0.8 g curve is mostly influenced by the 45 degree slope with no contribution from the 76 degree slope.

The above pseudo-static estimates of the mobilised rock mass shear strength simply find what stresses are required to equilibrate the gravity load induced by the weight of the rock mass and an additional inertia load generated by a particular value of horizontal acceleration. The conclusion is that if the rock mass has that amount of shear strength then failure will not be initiated in the slope when a horizontal acceleration pulse of the same magnitude is applied. This ignores the effect of any stress concentration in the slope and dynamic effects on the stress distribution during the earthquake. More significantly it overlooks the fact that even if shear failure of the rock mass is initiated during the earthquake motion the displacements generated may not be significant. In other words the pseudo static approach is a very conservative way of considering slope behaviour during earthquake loading.

FLAC MODELLING OF SLOPE BEHAVIOUR

In the remainder of this paper results of subjecting numerical slope models to earthquake excitation are presented. This was done to investigate residual displacements generated during the earthquake motion when some shear failure was induced in the rock mass. Calculations were done using the *FLAC* (Fast Lagrangian Analysis of Continua) software (Itasca, 2000), which is able to perform linear dynamic analyses of elastic materials and nonlinear analysis of Hoek-Brown and other inelastic materials. Space limitations preclude a detailed description of the work, which can be found in Kong (2003). The earthquake motion used was a subduction zone record of duration about 40 seconds, scaled to the required PGA. This and other records considered to be appropriate for earthquake motions in the Wellington area were provided by McVerry (2002).

Hoek-Brown failure envelopes and static factors of safety

First, the pre-earthquake static factors of safety of the slopes were evaluated with *FLAC* and then the factors of safety following the application of the earthquake to the slope. The permanent deformations induced by the

Table 1. Parameter values for the Hoek-Brown failure envelopes

Parameter	Hoek-Brown	Hoek-Brown enhanced
σ_c (MPa)	50	50
m_b	1.2	6.0
s (low value for no-tension)	10^{-5}	10^{-5}
a	0.5	0.5

Table 2. Pre- and post-earthquake static factors of safety

Slope angle (degrees)	Slope height (m)	$m_b = 1.2$		$m_b = 6.0$	
		Pre-EQ	Post-EQ	Pre-EQ	Post-EQ
45	200	2.11	2.12	3.45	4.06
55	110	2.59	2.60	3.24	3.47
65	50	2.49	2.41	3.30	3.25
75	16	2.22	-	3.50	2.64

earthquake alter the slope geometry and the hence the final factor of safety. Two sets of parameters were used for these calculations: one corresponding to the curve labelled Hoek-Brown in Figure 2 (derived from the information given in papers by Hoek) and the other with parameters that give an envelope beyond the 0.8 g mobilised strength curve in Figure 2 (the reason for this is explained below). In both cases the parameter s has a very small value so the rock masses are modelled as no-tension materials. The pre- and post- earthquake factors of safety for the four slope geometries are given in Table 2. It is apparent for the flatter slopes that the small changes in geometry induced by permanent displacements during the earthquake actually increase the static factor of safety slightly. In the case of the steeper slopes the permanent deformations alter the slope geometry sufficiently to decrease the factor of safety.

Calculated slope response

Initial calculations were done using the Hoek-Brown failure envelope on all four slope geometries with the input acceleration time history scaled to a PGA of 0.8 g. The proportions of the slope models and underlying rock layer were as shown in Figure 3. The acceleration time history is applied all along the base of the model which had quiet boundaries along the base and sides. The numerical models for the flatter higher slopes performed satisfactorily during these calculations and indicated small to modest residual displacements. However, the calculations for the 75 degree slope model halted before the completion of the earthquake record with a "bad geometry" message. Examination of the displacement vectors in the 76 degree slope during the earthquake indicates a toppling failure mechanism rather than shearing. To eliminate this problem the Hoek-Brown m_b parameter was increased from 1.2 to 6.0 (labelled "Hoek-Brown enhanced" in Table 1). In addition the damping in the model was increased so that the horizontal displacements at the quiet side boundaries were small. Calculations repeated with different values of damping showed that local damping in excess of 5%, and preferably 10%, is needed to give small displacements at the lateral boundaries. Displacement vectors for the 55 degree and 75 degree slopes are given in Figure 4. From this figure it is apparent that relatively small permanent lateral deformations will induce substantial changes in the geometry, and hence on the stability, of steep slopes, but have lesser effects on the flatter slopes.

Residual displacements

In Figure 5 the permanent displacements after the earthquake, termed residual displacements herein, are given for three positions on the slope. The PGA for the acceleration time history was 0.8 g and the strength parameters for the rock mass were Hoek-Brown enhanced. Notice that for all four slope models there are residual displacements even though the shear strength envelope for this case lies well beyond the 0.8 g mobilised shear strength curve in Figure 2.

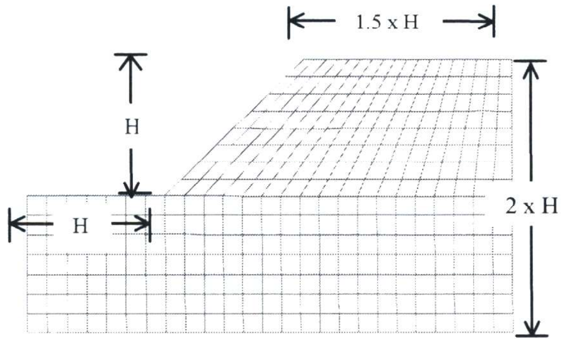

Figure 3. Proportions of the slope models.

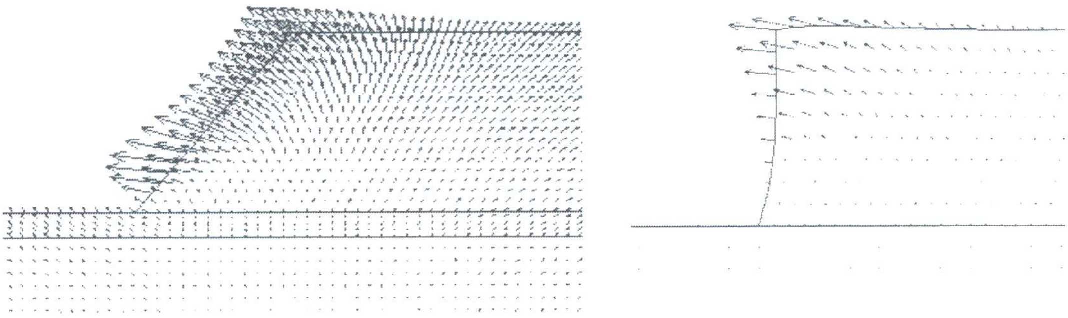

Figure 4. Displacement vectors after the earthquake for the 55 degree slope (left) and 75 degree slope (right).

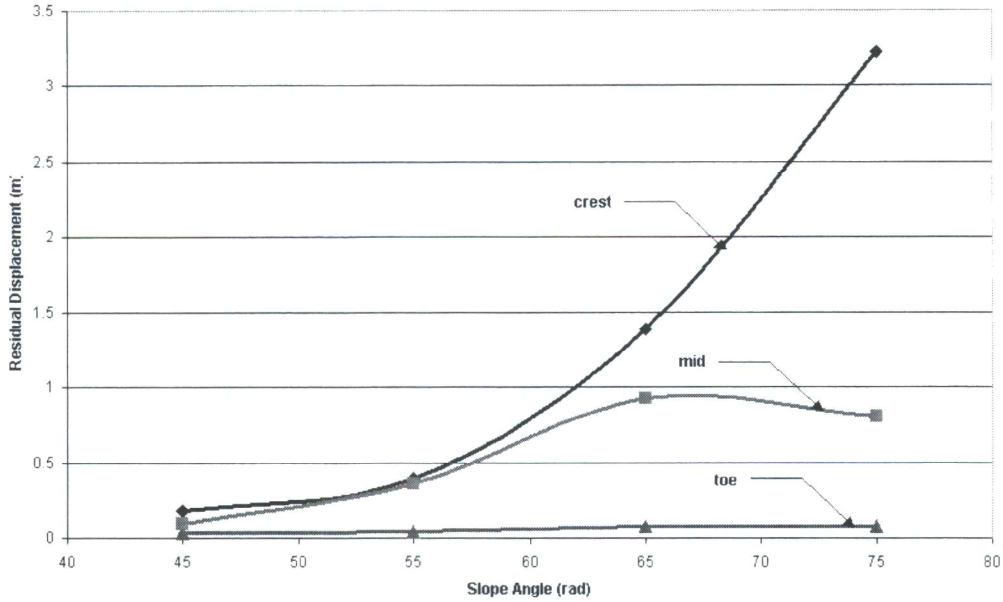

Figure 5. Residual displacements versus slope angle for the 0.80 PGA.

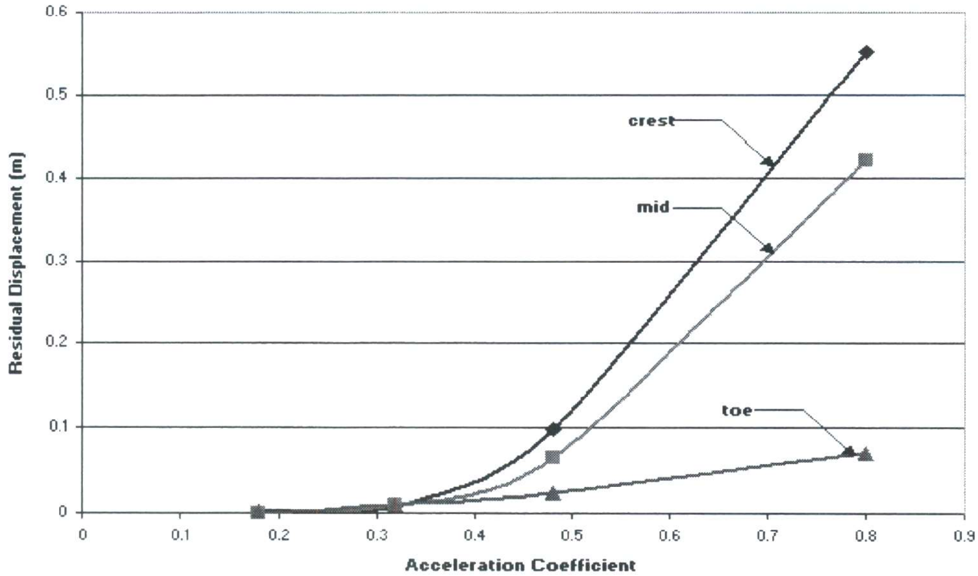

Figure 6. Residual displacements versus PGA for the 55 degree slope model.

On the basis of the pseudo static back analysis one might expect that that there would be no residual displacements in the slopes as the Hoek-Brown enhanced failure envelope lies beyond the mobilized strength curve for 0.8 g. However, the moblised strength envelope is based on limiting equilibrium analyses and does not account for the

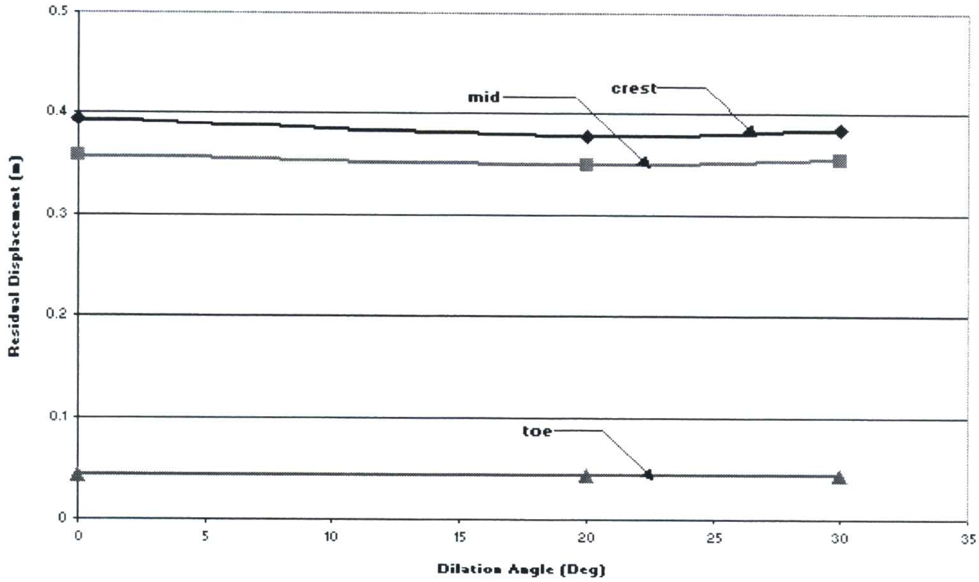

Figure 7. Effect of dilation angle on the post-earthquake residual displacements for the 55 degree slope model.

details of the actual stress distribution in the slope other than the requirement that equilibrium be satisfied. The *FLAC* calculations take account of the amplification of the earthquake motion as it travels upwards through the rock mass and also any stress concentrations in the slope. As a consequence the available shear resistance of the rock mass is exceeded at some positions in the slope during the shaking. The number of times that shear failure was initiated in the rock mass was tracked during the analysis; it varied from model to model with the extremes being 90 and 300 times. Even so, the residual displacements after the earthquake are quite modest in comparison with the slope heights with the exception of the 75 degree slope.

Figure 6 gives the residual displacements for the 55 degree slope as a function of PGA of the input earthquake motion. It is apparent for this 110 m high slope that the residual displacements are negligible for PGAs up to about 0.5g and even beyond this the displacements are still quite small.

Figure 7 gives the residual displacements for the 55 degree slope as a function of the dilation angle of the rock mass. This parameter comes into action when failure occurs in the rock mass. It is important as small angles mean that the stress distribution in the slope becomes localized and appears to be highly erratic. When the angle of dilation is about 20 degrees or above localization does not occur and the stress distribution remains smooth. Figure 7 indicates that localization does not effect the residual displacements significantly.

Not shown diagrammatically, because of space limitations, are the shear strain increment contours in the slope at the end of the shaking. Generally these are concentrated near the slope face. In this regard it is of interest to note that many of the photographs of post-earthquake slope conditions in New Zealand, Hancox et al (1997), indicate, with some exceptions, damage which appears to be shallow sliding. It is also of interest that this report, which documents earthquake induced slope instability in New Zealand, has only one example from the Wellington area.

The slope performance was also calculated in some cases for in which the vertical component of the earthquake motion as well as the horizontal was applied to the slope model. As expected this increased the residual deformations of the slopes but not markedly so.

At the outset of this work the assumption was made that the closely jointed rock mass would have zero tensile strength, which is achieved in the modelling by setting the parameter s to a very small value. In view of the results for the 75 degree slope with the Hoek-Brown envelope an alternative to the Hoek-Brown enhanced envelope used herein would be to increase the value of s rather than m_b so giving the rock mass tensile strength. One could argue that the closely jointed rock mass would have tensile strength as the material is tightly interlocked.

CONCLUSIONS

The following conclusions are reached:

- The back analysis method of inferring the mobilised shear strength curve during earthquake loading is conservative in that it assumes no failure occurs during the shaking. The *FLAC* calculations show that, even through the earthquake record exceeds the critical acceleration of the slope many times during the event, and therefore induces some failure in the rock mass, the residual displacements are modest.

- It is the steep slopes which are most critically affected by earthquake shaking. Even small changes in the slope geometry cause a substantial reduction in the post-earthquake static factor of safety. On the other hand the high but shallower slopes were found to be respond to earthquake loading in a surprisingly robust manner.
- It appears that the Hoek-Brown envelope is probably a reasonable estimate for the Wellington closely jointed greywacke rock slopes.
- Another step in the work will be to investigate the effect of allowing some tensile strength in the rock masses. This could be justified because of the tightness and close interlocking of the joints in the Wellington rock slopes.

ACKNOWLEDGEMENTS

Assistance from the following is gratefully acknowledged:
Dr G. H. McVerry, of the Institute of Geological and Nuclear Sciences, for provision of earthquake records.
Dr P. A. Cundall, Itasca Consulting Group, for advice on several aspects of dynamic analysis in *FLAC*.
Dr M. A. Coulthard, M. A. Coulthard and Associates, for advice on the linking of zones with differing mesh concentrations in the slope models.
The University of Auckland School of Engineering for the award of a Masters Scholarship to Vickie Kong.

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