The performance of hillside earth fills under earthquake loading

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ABSTRACT: Earth fills located on hillsides in areas of high seismicity have potential for shear failure (sliding) and seismically induced deformations at ground surface due to compression of the fill. A fill developed for residential purposes in the Wellington area has been analysed and shown to be of concern for potential instability as a result of fill geometry, properties, and proximity to the Wellington Fault. Conclusions are drawn about the performance of the fill during a 450 year return period earthquake, and a recommendation made to revisit applicable New Zealand Standards so that such fills are designed to provide a higher level of earthquake resistance.

1 INTRODUCTION

As part of a request for further information pursuant to Section 92 of the Resource Management Act 1991, the owners of property at 35 Domanski Crescent, Wellington were required to provide Wellington City Council with *"a stability assessment and assurance that there is sufficient reserve of stability to sustain the development"*. The owners had made application to Wellington City Council to construct a new dwelling, which was judged to be a Discretionary (Restricted) Activity under various rules of the District Plan. One such rule related to proposed earthworks which involved excavation with a maximum height of 5.1m.

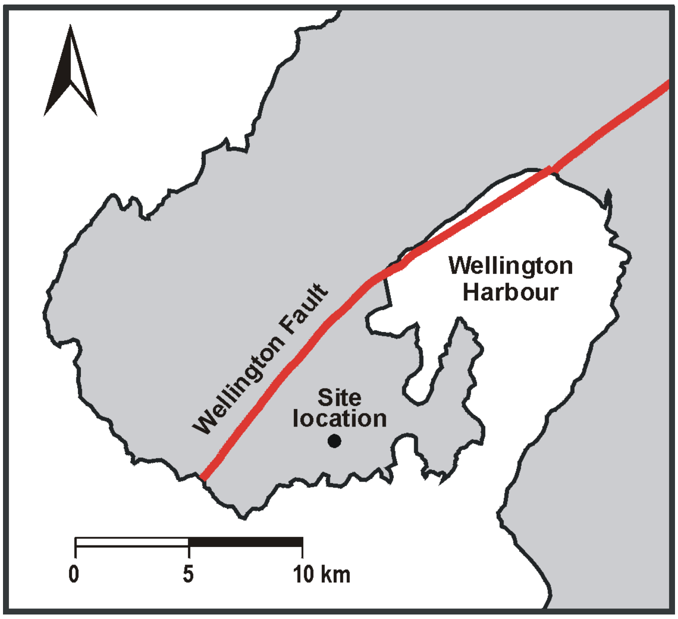

Figure 1. Site location map

The site is located in the Wellington urban area, about 3.5km from the active Wellington Fault (Figure 1). This fault is an important element in the context of Wellington earthquake hazards. The moment magnitude of the earthquake expected to accompany rupture of the fault is 7.3, with an estimated recurrence interval of 600 years (Stirling *et al.* 2002). The last displacement event on the Wellington Fault occurred about 300-450 years ago (Van Dissen *et al.* 1996). Ground motions arising from a Wellington Fault displacement earthquake need to be taken into account when carrying out a site stability assessment. Peak acceleration at the site during such an event is expected to be as high as 0.9g (Centre for Advanced Engineering, 1991).

The site is on a moderately steep slope comprising fill. It is believed to be typical of a number of fills constructed in the Wellington region, where the steep natural topography has been modified by cutting and filling to provide sites suitable for conventional suburban housing development. With the advent of efficient excavation and earthmoving equipment, from about the 1960s there has been a steady expansion of the urban area facilitated by such earthworks.

Earthworks for residential developments are constructed according to the provisions of NZS 4431:1989 (Standards New Zealand 1989). The standard requires that the effects of earthquake are considered, but only in relation to slope stability, without providing any guidance in this matter. The design of earthworks may not require the advice of specialist geotechnical engineers. We understand that specific geotechnical earthquake engineering studies are not usually carried out on residential earth fills in the Wellington region.

Experience of earth fill behaviour in the Northridge (1994) earthquake showed permanent ground deformations in unsaturated, compacted hillside fills (Stewart *et al.* 2001). Relatively modest deformations induced widespread damage totalling hundreds of millions of dollars. The levels of shaking that gave rise to damage at Northridge would be comparable with a moderate earthquake affecting Wellington, much less than the earthquake associated with rupture of the Wellington Fault.

2 SITE DESCRIPTION

A topographic map of the site area is shown on Figure 2. The area was extensively modified by earthworks, as part of a development project that would provide two platforms for housing development. This was completed in 1991. The face of the fill is relatively steep, and at the time was not considered attractive for housing, however in 2001, Wellington City Council granted subdivision consent for the face of the fill with subsequent property boundaries as shown on Figure 2.

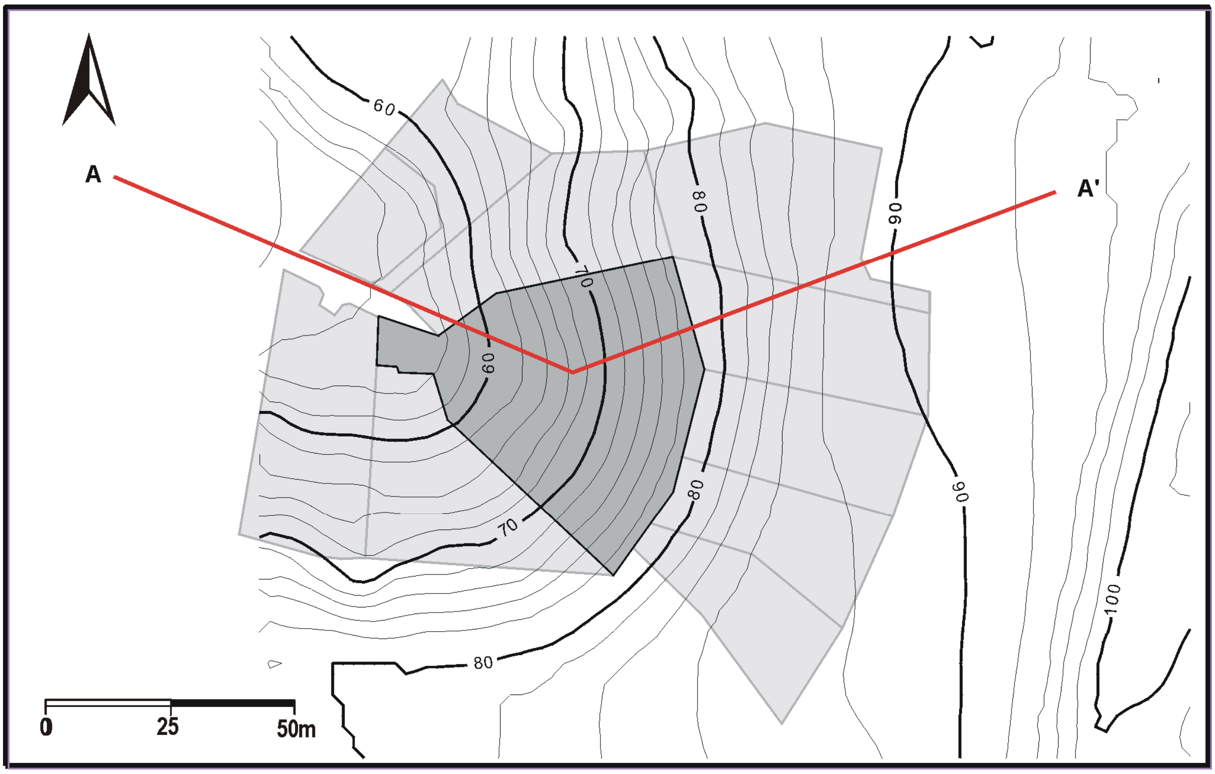

Figure 2. Site plan, with ground surface contours

We have constructed a 3-D model of the fill, using as built drawings. The interface between *in situ* ground (variably weathered sandstone and siltstone, commonly known as greywacke) and fill shows that up to 18m of fill has been placed in a topographic depression, as can be seen in the cross section (Figure 3). The fill comprises a variety of materials including weathered greywacke, silts, and clays. There was engineering supervision of the fill construction; compaction was required to be 95% of the Modified Proctor maximum dry density, and field density and moisture content testing was carried out on a daily basis whenever filling was in progress. As required by NZS 4431:1989 (Standards New Zealand 1989) a "Statement of suitability of earth fill for residential development" was completed by a registered engineer.

The fill construction report refers to a number of areas with groundwater seepage, and over 300m of subsoil drains were laid to collect this water. We were not able to locate the discharge points of these drains to check whether they are still functioning. Inspection of the fill shows surface drains have not been maintained, and they are now disrupted. Surface water is not controlled, and is able to infiltrate and locally saturate the fill. There is water seeping from the face of the fill indicating that it is saturated in places, and at one location there is a deeply eroded gully in the fill, possibly due to piping failure.

We have yet to carry out any subsurface investigations at the site. Instead we have relied on the as built drawings, the compaction information contained in the registered engineer's report, and laboratory strength testing that we and others have carried out on fill derived from similar material in the Wellington region.

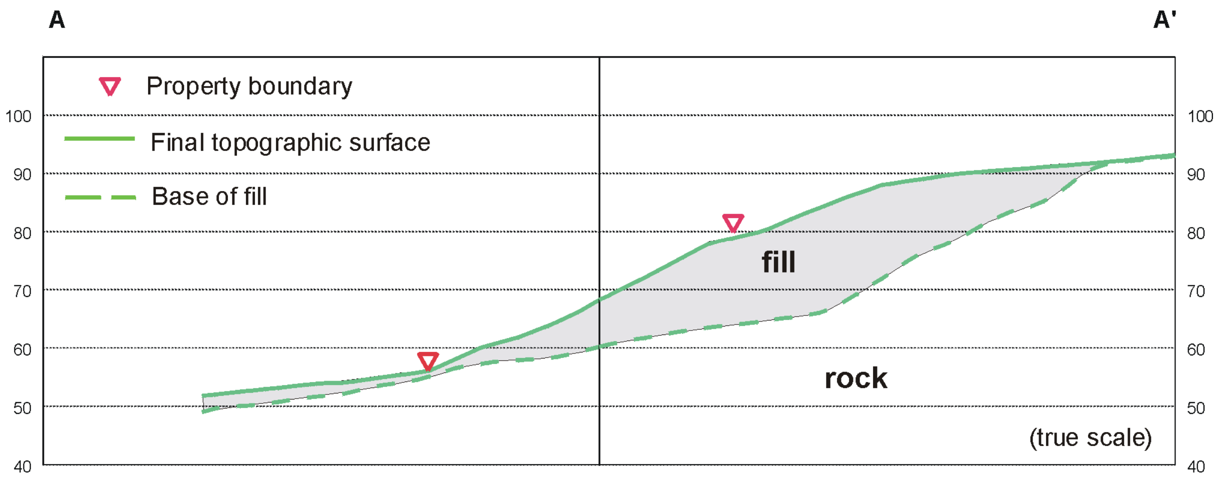

Figure 3. Cross section A-A' through fill.

3 STABILITY ANALYSES

Using the computer based 3-D model of the fill, various stability analyses were carried out. The conventional methods used for stability analyses involve representing the slope with a 2-D cross section which is then analysed using limit equilibrium methods. However the stability of this fill is influenced by the 3-D shape of the interface between rock and fill, hence we used a computer program TSLOPE3 (Engineering Portal Ltd, 2005) that uses a 3-D force equilibrium method of analysis.

Back analysis of the fill was carried out with shear failure assumed to occur at the fill / rock interface. This enabled us to evaluate the sensitivity of variation in effective shear strength parameters, friction angle and cohesion, relative to calculated factor of safety. The results of the case of static conditions and no pore water pressure acting on the fill / rock interface are shown on Figure 4. Under these conditions, the fill shows adequate stability over a wide range of strengths. Our best estimate of Mohr Coulomb effective shear strength parameters for the fill is friction angle of 32^{o}, and effective cohesion of 5kPa; with these values applied to the fill / rock interface there is a factor of safety of about 2.2.

The appropriate factors of safety that should be used to judge slope performance are not always clear. Crawford and Millar (1998) have provided a useful summary of approaches used in New Zealand.

They recommended a minimum factor of safety of 1.5 for design conditions (as required by the NZ Building Code) and a lower minimum factor of safety of 1.2 under extreme conditions (such as full saturation, and maximum credible earthquake – but not necessarily at the same time). For a site such as this, where material behaviours are not well defined and there are significant consequences of failure, we require a minimum factor of safety of 1.5 under earthquake loading.

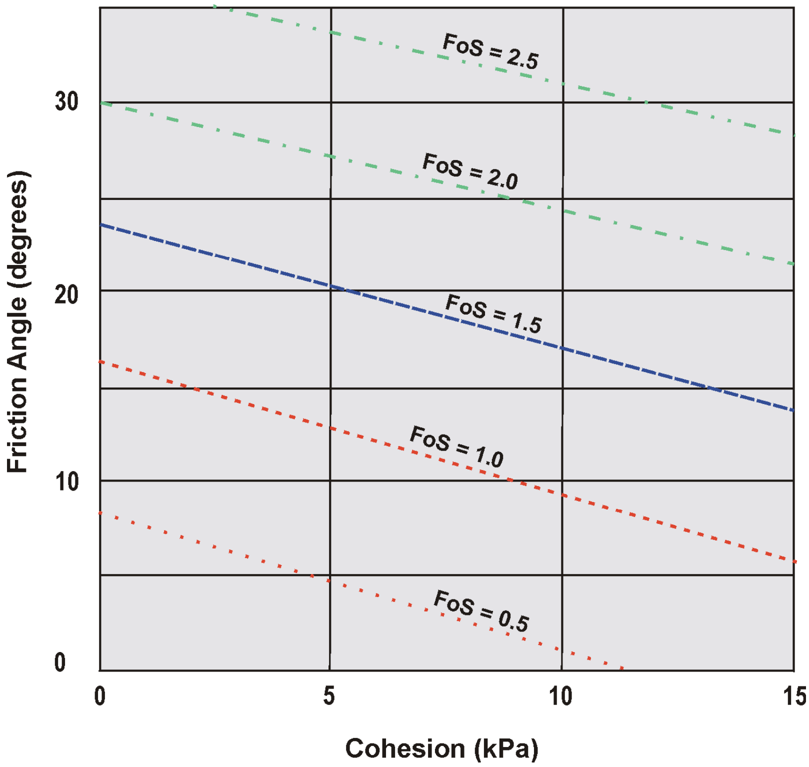

Figure 4. Calculated factor of safety against sliding, dry slope, no earthquake.

The fill appears to be saturated in part, despite construction of subsurface and surface drains. We did not have any groundwater measurements that would allow us to estimate pore water pressures acting on the fill / rock interface. Therefore we carried out a parametric study of the effect of varying r_u, the pore pressure ratio, defined as the pore pressure divided by the total vertical stress.

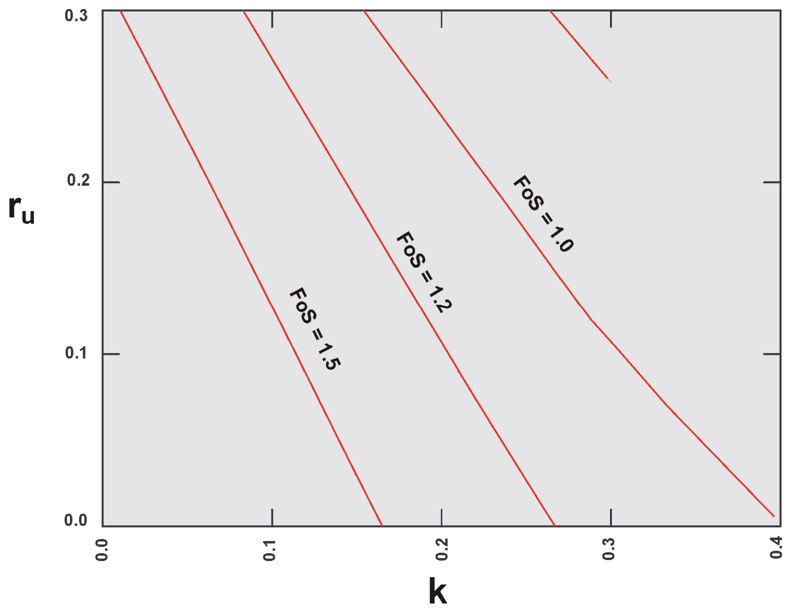

Figure 5. Calculated factor of safety for range of pore pressure ratio and seismic coefficient k

For the assumed effective shear strength $\varphi=32^{o}$ and c'=5kPa, we have investigated the factor of safety as we vary pore pressure ratio, and the pseudo-static seismic coefficient k (which controls the pseudo-static force on the failure mass, so its value should be related to a measure of the amplitude of the inertial force induced in the potentially unstable material). The results of these analyses are given in Figure 5.

The selection of seismic coefficients for use in pseudo-static slope stability analyses, such as is implemented in TSLOPE3, is related to the expected peak earthquake acceleration, and the magnitude of the earthquake. For this site, an appropriate seismic coefficient is judged to be 0.25.

The results presented in Figure 5 show that the calculated factor of safety is less than 1.5, about 1.25 for the dry slope case (r_u = 0), when k = 0.25. With increasing pore pressure ratio r_u reflecting changing groundwater conditions, the factor of safety decreases further.

When the pseudo-static factor of safety is calculated as 1.0, the coefficient is the critical or yield coefficient k_y.

4 DYNAMIC ANALYSES

Dynamic finite element studies were carried out to assess the accelerations of the potentially unstable fill. These data were then used to assess the magnitude of the non-recoverable deformations of the slope under seismic loading. The computer finite element computer program QUAKE/W (version 5, Geo-Slope International 2001) was used. This program is a time domain equivalent linear strain compatible solution with viscous damping. The solution is iterative in that several trial analyses are performed to assess the seismic response with the properties re-assessed after each analysis on the basis of the calculated strains. For this situation, where the input motion was large, five iterations were needed to achieve a solution.

The site location near the Wellington Fault required use of a suitable rock acceleration record. We used the Santa Cruz record from the University of California Santa Cruz Lick Observatory during the Loma Prieta 1989 event. This site had a hypocentral distance of 18km and was thus a near source record judged to be suitable for our work. The geology of the recording site was sandstone and the peak ground acceleration was 0.41g. The record was scaled to a maximum acceleration of 0.85g, with an interval of strong ground shaking of 15 seconds. The first 20 seconds of the acceleration record was used in the analysis.

A value of 180 m/sec^2 was used for the small strain shear wave velocity of the fill. Strain dependent viscous damping was incorporated. The excitation resulted in peak ground surface accelerations of approximately 10 m/sec/sec, peak ground surface displacements of 40mm, and shear strains in the fill of up to 0.5%. The calculation of shear strains of this order implies volumetric strain will occur over a significant depth of the fill leading to surface settlement from seismic compression. For clean sands such settlements may be of the order of 1% of the fill thickness (Stewart *et al.* 2001), but decreases with plasticity and water content at the time of compaction, for fills with significant fines (Whang 2001). The imposed shear stresses at the fill-rock interface were high with values up to 300 kPa. The analysis was carried out with simultaneous vertical acceleration using a scaled vertical component (maximum acceleration 0.57g) of the same earthquake record as used for the horizontal component. The vertical component had little effect on the horizontal ground response.

The pseudo-static method of stability analysis does not provide information on the deformations associated with slope failure, and further analyses are required to provide such data. Newmark (1965) suggested a procedure whereby the displacement of a potential sliding mass is calculated by double-integrating the difference between the applied acceleration history and the yield acceleration.

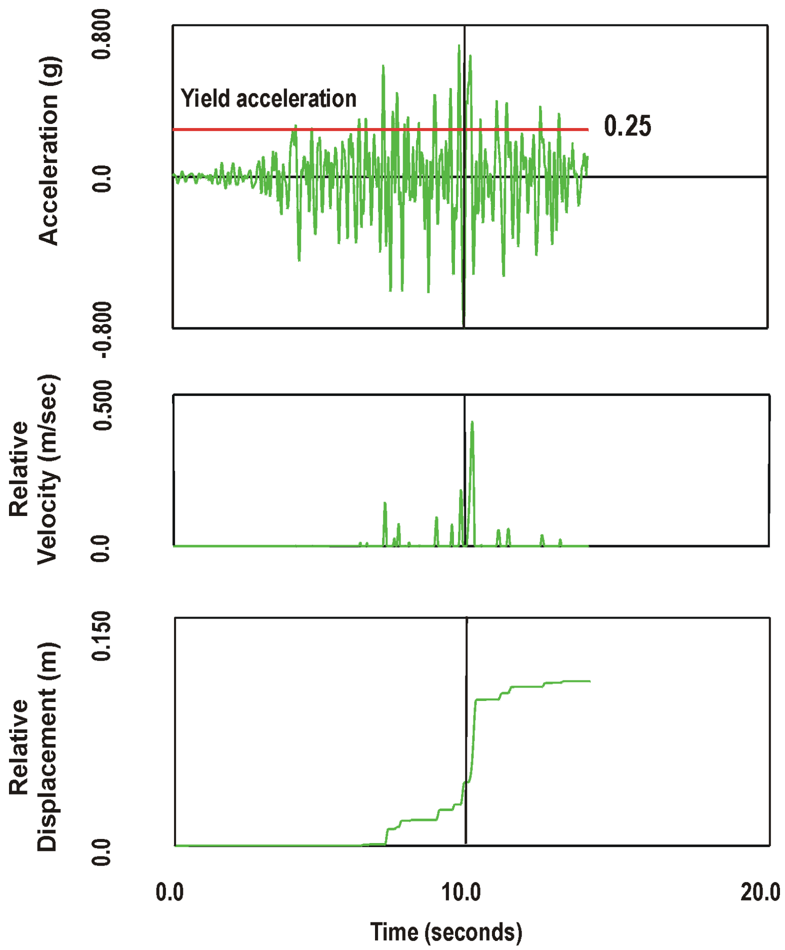

Figure 6. Calculated relative displacement and velocity from program TNMN.

We have used a program, TNMN (Engineering Portal Ltd 2002) that computes displacement of a potential sliding mass as suggested by Newmark (1965). Output from this analysis is shown on Figure 6, with a yield acceleration of 0.25g. The acceleration record used in the TNMN analysis was that calculated by QUAKE/W at a nodal point at the centre of mass of the fill.

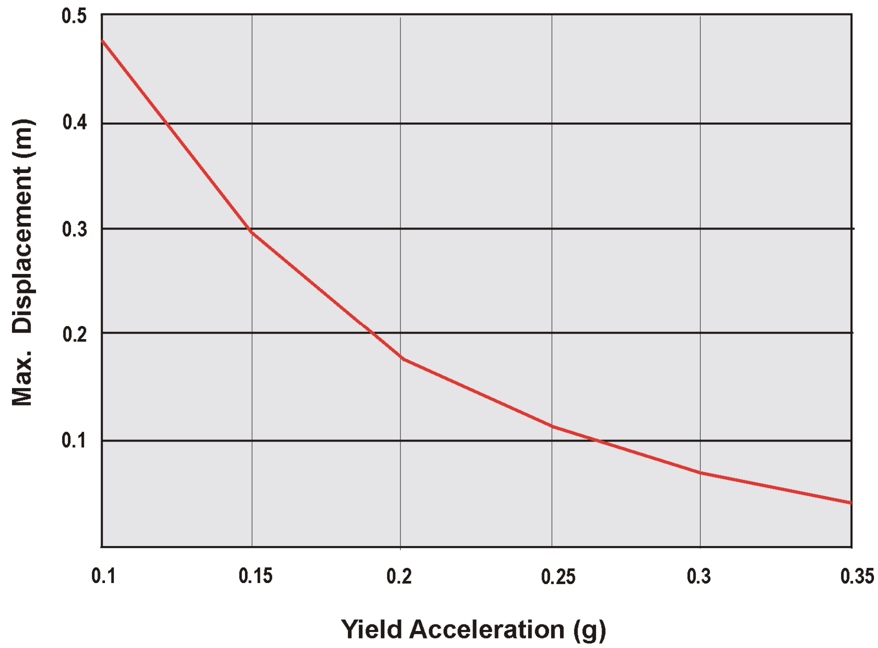

Figure 7 Displacement versus yield acceleration calculated from program TNMN.

The increase in calculated displacement with decrease in yield acceleration that will occur as r_u increases or if actual soil strength is lower than assumed, is shown on Figure 7. The yield acceleration is also dependent on the value of r_u, increasing r_u leading to a reduction in the yield acceleration and an increase in the non-recoverable displacement. The value of yield acceleration adopted reflects a low value of r_u and hence a low estimate of the deformations.

5 DISCUSSION

Based on the analyses we have carried out, we believe that there is inadequate resistance to shear failure at the interface between fill and rock given the loading conditions that should be considered. This puts considerable volume of the fill at risk of slope failure. Significant landsliding along well-defined slip surfaces was generally not observed following the Northridge earthquake (Stewart *et al.* 2001), which suggests that the pseudo-static analysis procedures used to design some of these slopes were "correct". A reason for apparent stability may be the provisions of the local Californian grading codes, with detailed requirements for over-excavation and benching, subsurface drainage, and toe-of-fill keyways.

We have also shown that seismic compression of the fill should be expected during strong shaking. The occurrence of shear strains of the order of 0.5% implies volumetric strain will occur over a significant depth of the fill, leading to surface settlement from seismic compression.

The experience from the Northridge earthquake (Stewart *et al.* 2001) was that localised horizontal and vertical crack offsets from deformation of fills were typically less than 80mm, similar to the displacements we have calculated. The cracking did not generally damage structures to the extent that life safety was threatened. However there were significant economic losses as a result of homeowner expectation that damaged houses would be returned to their pre-earthquake condition.

We do not believe we have been conservative with our approach to predicting the behaviour of this fill to strong shaking. We have carried out a fully drained analysis, even though there are fines present which would be expected to give rise to pore water pressure effects during shaking. We have used a 3-D method of stability analysis which provides a higher factor of safety than conventional 2-D analyses. We have used laboratory derived strength parameters for fill, which are possibly higher than might apply at the fill / rock interface. Coincidentally the fill properties, both static and dynamic, are very similar to those used by Stewart *et al.* (2001) for fills in California. The 15 seconds of strong ground shaking might also be considered short for a nearby M 7.3 earthquake. Two aspects of soil behaviour under seismic loading will aid the performance of the fill. These are the increase in shear strength under dynamic loading conditions, and strain hardening. An estimate of the effects of these two factors is that strength could increase by as much as 20%.

It is not clear from the Building Code or New Zealand Standards the appropriate loading conditions an earth fill for residential development should be designed for. The 450 year earthquake should be considered, which would be consistent with the design of structures that may be constructed on the fill. We might also need to consider the fully saturated case, to provide for the time when subsurface drains have clogged up and are no longer effective. It could also be argued that both conditions (earthquake, high groundwater level) are considered at the same time. The question of appropriate factors of safety to be used is also an area for debate.

6 CONCLUSIONS

The earth fill described in this paper is believed to be similar to a number of other fills constructed for residential development in the Wellington region. Although constructed and certified according to NZS 4431:1989, they are unlikely to have benefited from geotechnical earthquake engineering analysis. This then leads to the possibility that such fills, particularly where there is a dipping fill / rock interface, may not have adequate stability under earthquake loading conditions, particularly if there is poor drainage leading to high pore water pressures in the fill.

Seismic compression of a fill for residential purposes is not generally considered in New Zealand. As shown by our analyses, and as demonstrated during the Northridge earthquake, surface cracking of fills should be expected during strong shaking. Our analyses indicated high acceleration at the surface of the fill during a 450 year return period earthquake. Surface mounted structures can expect accelerations of the order of 1g, which may cause problems in some modern houses that are relatively stiff and brittle.

Present Standards related to earth fills should be revisited with respect to geotechnical earthquake engineering issues, to provide a comparable level of earthquake resistance as is now required of structures.

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