Tauroa residential subdivision: Landslide remediation and hill slope stabilisation for earthquake resistance

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ABSTRACT: The Tauroa residential subdivision is situated in the hills of southern Havelock North in Hawke's Bay, New Zealand. The development proposal includes removal of a moderately deep (<10 m) landslide that failed within mid-Pleistocene estuarine mudstone and its replacement with an engineered fill slope. As a result of a pseudo-static seismic stability assessment utilising a design 7.5 *M* earthquake, specific application of NZS 1170 for an ultimate limit state (ULS) seismic event and an f_{eq} factor developed via the "seismic screening analysis", engineered fill slope design resulted in a stable surface through removal of the landslide materials, subsoil drainage control and placement of a buttress fill. The seismic design philosophy incorporated reducing the NZS 1170 peak ground acceleration by f_{eq} thus allowing for limited slope deformations to occur as a result of the ULS seismic event. The magnitudes of the allowable slope deformations were set with respect to the tolerance level of the proposed structures.

1 INTRODUCTION

Geologically, some of the most dramatic aspects of the hills to the south of Havelock North, New Zealand include the deep-seated to moderately deep landslides within the Kidnappers Group strata. Many landslides are visible in this area, and these can pose risks to nearby development. Assessment of the engineering stability of these landslides and of unfailed slopes in similar conditions is complex.

Stage 7 of the Tauroa residential subdivision is currently on-going, and the assessment of the stability of an engineered fill slope proposed at the site of an existing moderately deep (< 10 m) landslide was required. The assessment and design incorporated previous work as well as additional geotechnical investigations that were conducted during detailed design.

2 SITE LOCATION AND DESCRIPTION

The Tauroa subdivision is located at the southern end of the Havelock North township in the Hawke's Bay region of New Zealand. Stage 7 of the subdivision comprises approximately 10 ha of generally hilly terrain between Tauroa Road and the Mangarau stream. Several incised ephemeral gullies and two significant landslides are located within the area. The larger landslide ("Landslide 1") is approximately 7 ha in size and abuts the site to the northwest. The smaller landslide ("Landslide 2", hereafter referred to as *LS2*) is located in the northern half of the site and is approximately 0.2 ha in size. The hills are moderately steep, with slope angles generally of the order of 12 to 20 degrees to the horizontal, and are dissected by the ephemeral gullies. Refer to Figure 1 for the locations of these landslides and for a presentation of the site topography in relation to the development proposal.

3 DEVELOPMENT PROPOSAL

The development proposal for Stage 7 of the Tauroa subdivision includes subdividing the property in order to construct 12 new residential building platforms and associated infrastructure. Due to the hilly nature of the existing site topography, some moderately significant earthworks will be carried out to form the roads and building platforms. During the initial stages of the project development, it was proposed to leave *LS2* largely untouched. During the detailed design phase of the project, however, the design was altered to include removing *LS2* and replacing it with an engineered fill slope. This design change allowed for a more stable road alignment, the construction of an access road for Lot 12 on top of *LS2* and the formation of more robust building platforms on Lots 3 and 7. Refer to Figure 2 for the current development proposal.

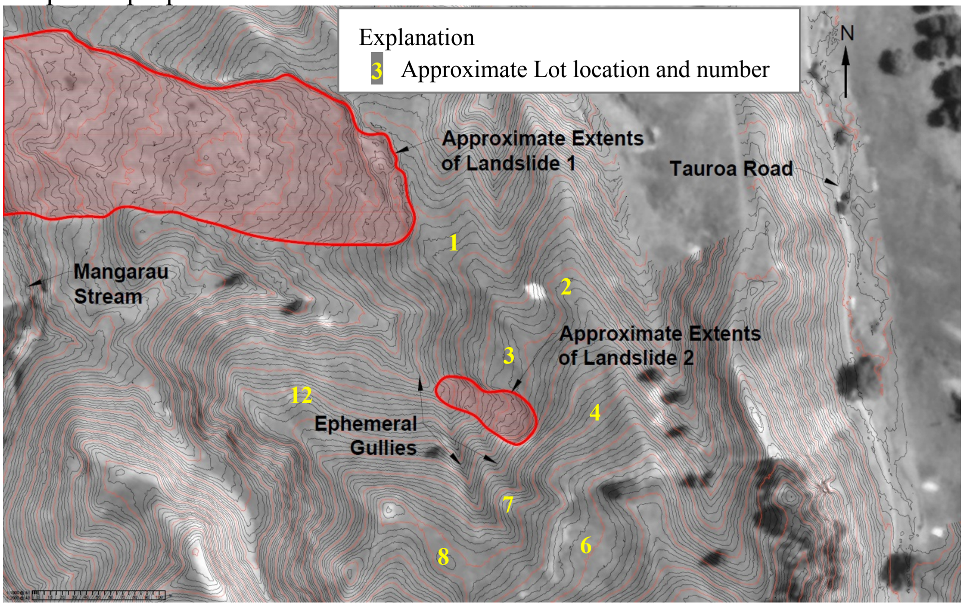

Figure 1. LiDAR topography of site on 1950 aerial photograph and showing key features within the vicinity.

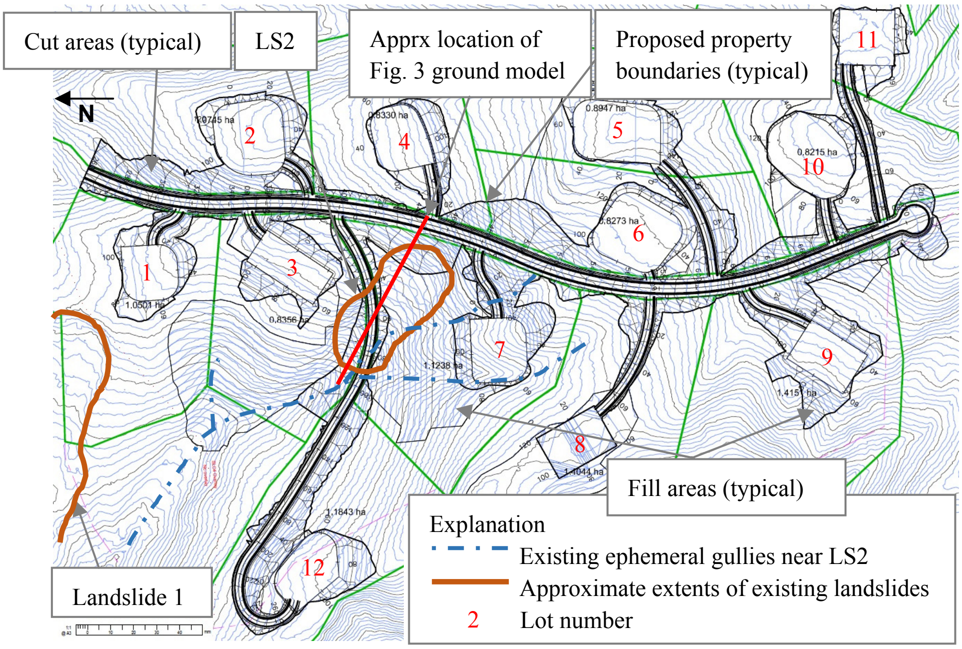

Figure 2. Tauroa residential subdivision, Stage 7 development proposal.

4 GEOLOGIC SETTING

4.1 Geology of the Area

Tauroa subdivision is located within the western extreme of the forearc basin structural high that is associated with the Australian-Pacific plate margin. From the Miocene epoch through the middle Pleistocene epoch, the area was subjected to cyclic sea inundation and emergence as a result of tectonic movements and glacial periods (Kingma, 1971). During periods in which the land was submerged, sediments were deposited within the epeiric shelf (Begg, Hull & Downes, 1994). The sedimentation thus principally resulted in the formation of mudstone, limestone and conglomerate rocks that were deposited within a shallow sea environment. Since the mid Pleistocene, the area has been above sea level, and the present drainage patterns were established (Kingma, 1971).

In the area of *LS2*, the rock strata comprise calcareous sandstones overlain by Kidnappers Group estuarine mudstone that dip approximately 10 to 15 degrees to the west-northwest. The presence of layered carbonaceous materials (derived from vegetation) within the mud may provide low-strength layers on which slope movement can occur (Bell, 1991). These strata are overlain by interbedded volcanic tephra (typically fine pumiceous sands) and glacial loess silts, the most recent of which is typically 2 to 3 m thick and often contains an approximately 1 m thick cemented fragipan (Bell, 1991).

4.2 **Seismic History**

The Hawke's Bay region is one of the most earthquake prone in New Zealand. Oblique subduction of the Pacific plate beneath the Australian plate dominates the tectonic activity of the region, which hosts a minimum of 22 known active faults and folds that are capable of producing very strong earthquake shaking. As strong earthquake shaking (i.e. maximum felt intensities of ≥ 7 on the Modified Mercalli Intensity scale) has been felt in Hawke's Bay on at least 19 occasions during the last approximately 150 years (Begg, Hull & Downes, 1994), earthquake shaking must be considered when assessing the slope stability hazard of developments.

4.3 **Description of *LS2***

LS2 is a moderately deep (<10 m) landslide characterised by classic translational movement within a locally weak layer of the estuarine mudstone. The triggering mechanisms of similar, but larger, translational and deep-seated landslides some 500 and 150 m to the northeast are likely attributed to a combination of rainfall infiltration and of downcutting and lateral erosion of the toe by the Mangarau stream (Bell, 1991; Gray & Jowett, 1998). Seismic movement may also be a contributing factor. Due to the presence of two ephemeral drainage gullies near the toe of *LS2*, it can be postulated that the failure may have been triggered by a similar combination of factors.

5 **PREVIOUS WORK**

5.1 **1991 Work by Bell**

Bell published the initial Tauroa subdivision work in 1991. He mapped numerous landslide features and, utilising tephra and pumice markers as well as aerial photo interpretation, dated two secondary slide failures as having occurred between approximately 1800 and 100 years before present (BP). Bell (1991) focused considerable attention on the Goat Shed Slide, which is located approximately 500 m northwest of *LS2*. Several polished and/or slickensided surfaces were identified within borehole cores and were logged as failure surfaces, with the failure planes dipping approximately six degrees to the west. Samples of these failure surfaces were tested in the laboratory and returned average residual soil strength parameters of c_r = 5 kPa and ϕ_r = 11 degrees. A separate un-failed estuarine mudstone sampled from an adjacent borehole returned c_r = 6 kPa and ϕ_r = 16 degrees.

5.2 **Work by Gray & Jowett (1998) and Gray & Watson (2012)**

The subject property was first studied in the preliminary geotechnical appraisal (Gray & Jowett, 1998), and it was later assessed in more detail in a preliminary geotechnical report (Gray & Watson, 2012). Subsoil investigations identified the failure plane near the centre of *LS2* at approximately 6.65 to 6.90 m below the existing ground surface. Groundwater was not encountered in an exploratory borehole that extended to 15.45 m.

6 ASSESSMENT OF THE STABILITY OF *LS2*

6.1 New Geotechnical Investigations

As discussed previously in this paper, changes were made to the site development proposal between the publication of the referenced preliminary geotechnical report and commencement of the detailed design phase. In order to produce a substantially better product, these changes included remediation of *LS2* by undercutting and removing the landslide debris and replacing it with an engineered fill. In order to design the remedial works and the engineered fill, additional geotechnical site investigation and laboratory testing was required.

6.1.1 *Field Work*

Three mechanically excavated TPs were conducted at the site during September 2013. "*TP.1*" was conducted approximately 190 m to the south-southeast of *LS2*. "*TP.2*" was excavated immediately above the *LS2* head scarp to a depth of 3.4 m and exposed loess SILT interbedded with pumice sand; a cemented fragipan ("hardpan") was logged between 1.1 and 1.5 m below the existing ground surface. "*TP.3*" was excavated on the toe of *LS2* to a depth of 3.0 m, and it exposed silty CLAY colluvium (i.e. landslide material) and undisturbed estuarine muds; a potential slickensided slip surface was logged at 2.4 m below the existing ground surface. The silty CLAY encountered in *TP.3* was described as being stiff, moist and highly plastic. Shear vane tests with a Pilcon vane shear device were conducted during the excavation of the third TP. Average returned undrained shear strength (S_U) values at a depth of 2.5 m were 74 kPa (peak) and 23 kPa (remoulded), while S_U values at a depth of 3.0 m were > 175 kPa (peak) and 32 kPa (remoulded). Bulk samples were obtained from the material excavated from the TPs, and pushtube samples were obtained from *TP.3* at a depth of 2.5 m.

6.1.2 *Laboratory Testing*

Samples retrieved during the additional geotechnical site investigation were tested in an Opus IANZ accredited soil laboratory. In order to evaluate the soil strength parameters for the engineered fill, consolidated drained direct shear tests were conducted on remoulded samples of the bulk soils that were obtained from *TP.1* and *TP.2*. A three stage consolidated undrained triaxial compression test with pore pressure recording was conducted on the pushtube sample to evaluate the in-situ soil strength parameters for the estuarine mud deposits. Utilising the laboratory test results, design ϕ' and c' values of 28 degrees and 15 kPa, respectively, were estimated for the sandy SILT (fill), while design ϕ' and c' values of 23 degrees and 9 kPa, respectively, were estimated for the in-situ stiff clayey SILT.

6.2 Slope Stability Assessment

The geotechnical data was used to prepare an interpretive sub-surface ground model for the proposed engineered fill. Refer to Figure 3 for the presentation of this interpretive ground model, which formed the basis of the slope stability assessments. The slope stability assessments were conducted utilising numerical analysis using the computer programme SLOPE/W (GEO-SLOPE, 2012). In accordance with the local Engineering Code of Practice (Hastings District Council, 2011) and standard engineering practice for slopes that provide integral support for or direct loading on a structure, target factors of safety (*FS*) against failure of 1.5, 1.2 and 1.0 were established for static, transient seepage and seismic loading conditions, respectively.

Both circular and block-specified failure surfaces were analysed. The Morgenstern-Price method of slices was utilised to determine the *FS* for each failure surface. Groundwater was modelled within SLOPE/W using a phreatic surface(s) that, depending on the condition modelled, was either constrained to a certain (or several certain) strata or was unconstrained.

The development proposal includes a road above the engineered fill, an access road for Lot 12 down the face of the engineered fill and a residential building platform within the southern portion of the engineered fill. Therefore, critical case vehicle and building loads were applied as appropriate. It was assumed that the landslide colluvium would be completely removed below the slip surface and that properly constructed keyways and benches with subsoil drains would be formed during construction of the engineered fill. Furthermore, it was also assumed that quality assurance construction monitoring would be performed to ensure that the design compaction specifications would be achieved. The model was assessed at two cross-sections down the length (in profile) of the engineered fill slope and at two cross-sections across the width of the engineered fill.

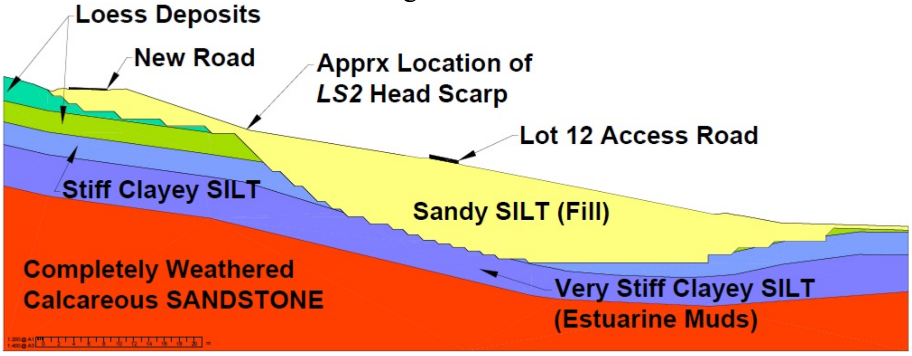

Figure 3. *LS2* ground model for SLOPE/W (refer to Fig. 2 for location)

6.2.1 *Static Slope Stability Analysis*

Geotechnical soil properties were assigned to the interpretive ground model strata using the results of the laboratory test data and experience with similar soils in similar geological conditions. Whilst groundwater was not encountered in the referenced site investigations, a prevailing groundwater surface was assumed at the interface of the completely weathered SANDSTONE and the clayey SILT.

6.2.2 *Transient Seepage Slope Stability Analysis*

The transient seepage condition was modelled first utilising an elevated groundwater level (i.e. phreatic surface) applied above the prevailing typical groundwater table. This surface was applied to all soil types. As the presence of groundwater perched above the fragipan is believed to have caused many of the numerous shallow surface failures observed within the Tauroa subdivision (Bell, 1991), the transient seepage condition was secondly modelled utilising the elevated phreatic surface (as above) in conjunction with a phreatic surface perched above the fragipan in the natural soils. Thirdly, failure of the slope subsoil drains was also modelled by applying the elevated phreatic surface in conjunction with a phreatic surface perched above both the natural fragipan and the engineered fill.

6.2.3 *Dynamic (Seismic) Slope Stability Analysis*

6.2.3.1 Methodology

Due to the absence of loose, saturated sandy soils and sensitive fine-grained soils at the site, the engineered fill was assessed for inertial seismic stability utilising the pseudo-static method (refer to Terzaghi, 1950; Seed & Martin, 1966; Seed, 1979; Makdisi & Seed, 1979; Hynes-Griffin & Franklin, 1984). In this method, a horizontal seismic load (k_{eq}) is applied to the slope to approximate earthquake shaking. Depending on which variant of the method is applied, k_{eq} is either equal to a specific acceleration (e.g. 0.1 g, 0.15 g, 0.20 g) or is equal to the peak ground acceleration (PGA) multiplied by a site seismicity factor (f_{eq}). The pseudo-static method can be further utilised to estimate the Newmark-type sliding block displacement (Newmark, 1965) resulting from the design seismic event.

As an alternative to conducting a Newmark-type displacement analysis of the slope, a simplified screen analysis procedure was adopted. In the absence of specific guidance on the use of the simplified screen analysis procedure within New Zealand, the "screen analysis procedure" presented in SP117A (California Geological Survey, 2008) was utilised as a best practice basis for which to develop appropriate f_{eq} values. This method was formulated to identify geographic areas that are potentially susceptible to earthquake-induced landslides (Blake, Hollingsworth & Stewart, 2002; Stewart, Blake & Hollingsworth, 2003). The Stewart, Blake & Hollingsworth (2003) equation for f_{eq} is

$$feq = \frac{NRF}{3.477} \times \left[1.87 - \log_{10} \left(\frac{u}{\left(MHA_r / g \right) \times NRF \times D_{5-95,m}} \right) \right] \quad (1)$$

where MHA_r/g = maximum horizontal acceleration of base rock; NRF = nonlinear response factor to correlate MHA_r/g to the spatially averaged peak amplitude of shaking within the soil slide mass (since earthquake shaking is often amplified or de-amplified by the soil overlying the bedrock); u = 5 or 15 cm; and $D_{5\text{-}95,\mathrm{m}}$ = median duration from Abrahamson and Silva (1996) relationship.

Refer to Stewart, Blake & Hollingsworth (2003) for the equations for NRF and $D_{5\text{-}95,m}$. These equations require values for expected magnitude (M) and distance to the source (r). A 7.5 M event on either the Poukawa fault zone or the Mohaka fault is considered the likely design event for the site (Berryman, McVerry & Villamor, 1997; Hengesh et al., 1998; Hawke's Bay Engineering Lifelines Project, 2001), so worst-case values of 7.5 for M and 48 km for r were utilised to calculate f_{eq}.

6.2.3.2 Calculation of k_{eq}

For the analysis, the PGA was calculated utilising NZS 1170:2002 for residential dwellings with a 50-year design life and a Site Subsoil Class of Class C ("Shallow Soil"). A PGA of 0.52 g was determined for the ultimate limit state (ULS) condition, which corresponds to a 500-year return period.

Using Equation 1, f_{eq} values of 0.60 and 0.47 were returned for these respective situations: maximum slope displacements of less than 50 mm for slopes providing integral support of a building platform, and maximum slope displacements of less than 150 mm for slopes not providing integral support of a building platform. These threshold displacements are presented within the "screen analysis procedure" methodology and reflect the fact that landslides are capable of accommodating a limited amount of displacement (i.e. partial mobilisation) before complete mobilisation of the basal rupture surface and catastrophic ground failure occurs (Murphy & Mankelow, 2004). Multiplying the PGA by the f_{eq} values produced k_{eq} values of 0.31 g and 0.25 g for use in the analyses for allowable deformations of 50 and 150 mm, respectively.

For this study, MHA_r in Equation 1 was set equivalent to PGAs determined by application of NZS 1170 for Site Subsoil Class C, rather than for Site Subsoil Class A or B. This was done to provide appropriate conservatism consistent with the nature of the development proposal.

6.2.3.3 Slip Surface Location and Modelled Soil Strengths

It was assumed that the location of the seismic condition slip surface is generally consistent with the location of the corresponding static condition slip surface. As earthquake loadings are generally applied so rapidly that all but the most permeable of soils (e.g. coarse gravels and/or cobbles) are loaded in an undrained manner (Seed, 1979; Kramer, 1996; Duncan & Wright, 2002), it was assumed that the sub-surface soils at the site will not drain appreciably during earthquake loading. Thus, undrained shear strengths were used within the SLOPE/W analyses by utilising the programme's Mohr-Coulomb soil strength setting together with its option to keep the slice base shear strength unaltered when the dynamic force is applied. The accuracy of this option within SLOPE/W was tested on a homogenous slope model comprising numerous thin horizontal soil strata, and the option was found to be sufficiently accurate for the intended purpose.

Lastly, it is widely accepted that degradation of soil strength occurs during the dynamic (e.g. earthquake) loading of a slope (Makdisi & Seed, 1977; Makdisi & Seed, 1979; Seed, 1979; Kramer, 1996; Duncan & Wright, 2002; Murphy & Mankelow, 2004) and that a "dynamic yield strength" may be utilised to represent such strength degradation when complete mobilisation of the basal rupture surface does not occur (Makdisi & Seed, 1977; Seed, 1979). As modelling the slope utilising soil "dynamic yield strengths" would require lengthy iterative computations, the author elected to increase the seismic condition target FS from 1.0 to 1.2 to approximate soil dynamic strength degradation.

6.3 **Summary of Stability Analyses and Design of Slope Surface**

Save for the failure surfaces initiating from the north-western half of the Lot 7 building platform and extending north across the engineered fill slope, the *FS* values returned from the slope stability analyses all met or exceeded the target *FS* values. For the excepted failure surfaces, *FS* values were less than 1.2 for a k_{eq} of 0.31 g. In order to mitigate the potential for a slope failure resulting in moderate displacements to occur, the access road for Lot 12 was shifted slightly north and the toe elevation at this location was raised by 2 m. When re-assessed, these changes resulted in slope stability *FS* values that exceeded 1.2.

7 DESIGN OF REMEDIATION PROGRAMME FOR *LS2*

The remedial design includes undercutting and removing the existing *LS2* colluvium to expose undisturbed deposits, forming keyways and benches with subsoil drainage, placing controlled engineered sandy SILT fill in 0.2 m (compacted) lifts and forming a new ground surface. A cut-fill building platform is to be constructed for Lot 7 at the top of the engineered fill slope, and a buttress fill of approximately 2 m in height is to be constructed at the toe and along the lower face of the slope to the north (below Lot 3) of *LS2*.

Control of both surface water and subsoil seepage is considered critical to the success of the *LS2* remedial works. Therefore, the remedial design also includes pavement cut-off drains along both the new roadway and the Lot 12 access road as well as subsoil drains placed within the ephemeral gullies and seepage areas to the south and west of *LS2*. The subsoil drains are to discharge via solid pipes to the stormwater detention dam to be constructed approximately 100 m northwest of the existing *LS2* toe. Surface water control is to be provided by drainage swales along the uphill side of the Lot 12 access road, around the back and eastern side of the Lot 7 building platform and along the uphill (eastern) side of the main subdivision road; the road is immediately east of the existing *LS2* head scarp. These drainage swales are to discharge to the reticulated stormwater system beneath the new road.

The sandy SILT material for use as engineered fill is to be sourced from borrow areas within the subdivision. The earthworks specification produced for the works provides appropriately stringent materials testing and compaction controls for the works. In addition to including material requirements for the bulk fill, the document specifies minimum values and testing frequency for bulk fill air voids, water content, material relative density and undrained shear strength. Monitoring of the fill-induced settlements of the subsoils and engineered fill is planned during placement of the engineered fill. Settlement monitoring is to continue after completion of the works and until at least 90 per cent of the settlement has occurred. Quality assurance testing and engineer observation is also specified so that preparation of a geotechnical completion report can be prepared following completion of the works.

Upon preparation of this paper, the proposed works have been initiated, and the area of *LS2* has been cleared and stripped. Subsoil drains have been installed within several of the ephemeral gullies near *LS2*, and a keyway has been cut into competent undisturbed materials near the toe of *LS2*. However, undercutting of the landslide colluvium has not yet commenced.

8 SUMMARY AND CONCLUSIONS

The seismic design of the remedial engineered fill slope, which is to replace *LS2* at Tauroa subdivision, employed the pseudo-static analysis method with k_{eq} determined utilising the simplified “seismic screening analysis” (Stewart, Blake & Hollingsworth, 2003). Allowable slope displacement thresholds were adopted to reflect that landslides are capable of accommodating a limited amount of displacement before complete mobilisation of the basal rupture surface and catastrophic ground failure occurs (Murphy & Mankelow, 2004). Displacement thresholds consistent with the Stewart, Blake & Hollingsworth (2003) method and appropriate for the importance level of each portion of the engineered fill slope were adopted for this study. The resulting remedial design and construction programme is within the bounds of normally accepted earthworks programmes for similar development proposals, and the risk level is considered appropriate to the importance level of the proposed structures. Therefore, the analyses and remedial design approaches described in this paper can find useful application in similar hillside developments within New Zealand where the subsoils are not considered susceptible to weakening instabilities (e.g. flow liquefaction and cyclic mobility) and where brittle rock slope failures are not anticipated.

9 ACKNOWLEDGEMENTS

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