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PERFORMANCE BASED DESIGN FOR UNSTABLE SLOPES IN HIGHLY SEISMIC AREA

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Abstract: The design of slopes under seismic loading is typically undertaken by simplified pseudo-static methods. These approaches are usually conservative, as they fail to account for the real earthquake representation and the ground behaviour, therefore unnecessary costs for slope strengthening measures are introduced. Methods for assessing the seismic performance of geotechnical structures and soil-structure systems have evolved significantly in the last few years. This paper presents a case study of slope engineering design using a performance based approach in a highly seismic area affected by historic landslides. The approach is a deviation from routine slope design by aligning the performance of the slope with different deformation levels. The site response analyses, performed to select appropriate design acceleration values for the slopes, demonstrated that the presence of a shear zone within the landslide mass has a profound effect on the peak ground acceleration by acting as an isolator. The slope deformations were assessed using numerical and empirical methods. Substantial cost savings were achieved through the adoption of a performance based approach.

Introduction - Seismic Design of Slopes

The design of slopes under seismic loading is typically undertaken by simplified pseudo-static methods using limit equilibrium principles. However, experience has demonstrated that these methods provide very conservative results. Important clues about the performance of slopes in seismic conditions can be obtained from case histories from past earthquakes. Pacoima Dam is a 113 metre high concrete arch dam located north of the city of Los Angeles. A well-known ground motion record obtained above the south abutment during the 1971 San Fernando earthquake showed large accelerations of 1.25g horizontal and 0.70g vertical which have been attributed to topographical amplification. Originally this dam was designed on a pseudo-static basis for an assumed horizontal ground acceleration of 0.15g. This dam was also shaken by the 17 January 1994 Northridge Earthquake of M 6.8. Despite being subjected to high accelerations, the arch dam survived the Northridge earthquake well with the main damage being an opening of the contraction joint between the arch dam and the thrust block at the left abutment of approximately 50 mm. This suggests that slopes can often survive bigger magnitude events compared to the design events.

In designing such slopes the selection of an appropriate pseudo-static coefficient (k_h) is the most important and challenging aspect of the pseudo-static stability analysis. In general engineering judgment should be applied in the selection of an appropriate value of k_h. Further uncertainties are introduced by the selection of the Peak Ground Acceleration (PGA), which is usually defined without performing site specific response analyses, using conservative amplification factors that increase the bedrock acceleration and ignore the beneficial effect of the non-linear soil behaviour. In highly seismic areas, this behaviour can in fact result in a significant reduction in the PGA. Based on the foregoing, due to the conservatism of these approaches the slope design typically introduces significant costs for

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the strengthening measures. The Performance Based Earthquake Engineering (PBEE) concept has emerged with emphasis on the performance based design for geotechnical structures (ISO-23469, 2005). In broad terms, this general framework implies an engineering evaluation and design of structures ensuring that their seismic performance meets the objectives of modern society (Cubrinovski, 2009). In order to achieve a more economic design, it allows Factor of Safety (FOS) smaller than unity during earthquake shaking, provided that the resulting displacements post-earthquake shaking are within acceptable limits. The displacement calculation can be performed with different approaches, including empirical methods and rigorous analyses. Both methods are largely dependent on the yield acceleration (k_y), which is the pseudo-static coefficient associated with a FOS equal to unity. The empirical methods calculate displacements based on the slope and earthquake characteristics, while the rigorous methods involve the direct use of earthquake time histories. A more advanced way to examine the slope stability is by performing a time domain non-linear numerical analysis, modelling the 2D slope geometry. Due to the complexity and cost associated with this solution it is not often adopted for routine design. This paper will discuss the application of the above methods through a case study of an unstable slope in a highly seismic area. The cost saving that can be achieved with the PBEE methods in comparison with standard simplified pseudo-static methods will also be highlighted.

Design Procedure

The examined case study comprises of a slope having a height of typically 110 metres, an inclination between 15^o and 20^o and a length of 400 metres. The slope had experienced historic landslide activity. The proposed design comprises of re-profiling the slope and installing seven rows of ground anchors at 2 to 6 metre centres, inclined downwards at either 15^o or 20^o to the horizontal.

Figure 1 illustrates the iterative design process undertaken to design the remediation of the slopes. The first step in the method was to check the static stability of the slopes. The analysis was undertaken adopting a proprietary limit equilibrium slope stability software package GeoStudio Slope/W, with Morgenstern and Price method. The software allows for the ground and groundwater profile to be modelled along with structural elements (ground anchors and soil nails) used to provide support to the slope in static conditions. A target FOS equal to 1.25 was needed to satisfy the project requirements. To attain this FOS, the capacities of the structural elements (ground anchors and soil nails) were progressively increased until the target FOS was achieved. The soil parameters were verified by conducting appropriate back analysis of the previous slope failures.

The next step in the method was to check the seismic stability of the slopes. The pseudo-static approach was based upon a two-stage process comprising of the following: stability analysis using GeoStudio Slope/W; and, where the FOS was less than unity, evaluation of the slope displacements. It should be noted that the design was not based on fulfilling a target factor of safety in pseudo-static conditions. The slope stability analyses were undertaken by introducing k_h to represent the seismic loading. The provisions in Eurocode 8 section 4.1.3.3, suggesting a value of 0.5*PGA for the k_h value were adopted. Site response analyses (discussed later) were carried out to evaluate the shear stresses generated within the soil mass and the appropriate PGA values.

The displacements associated with the failure masses identified by the slope stability calculations were then assessed. An iterative procedure was then undertaken to balance the slope displacements with the forces in structural elements. The resulting deformations within the soil mass were determined by using both empirical and rigorous methods. If the deformations due to earthquake loading are considered acceptable, then additional measures to stabilise the slope would not be required. Conversely, when deformations exceeded the performance threshold (set at 500mm, Figure 1), additional strengthening measures were implemented. Throughout the design process, several sensitivity analyses were undertaken to introduce a degree of conservatism and ensure that the design solutions provide robustness, recognising the complexity of the ground conditions and the inherent variability that exists across the site.

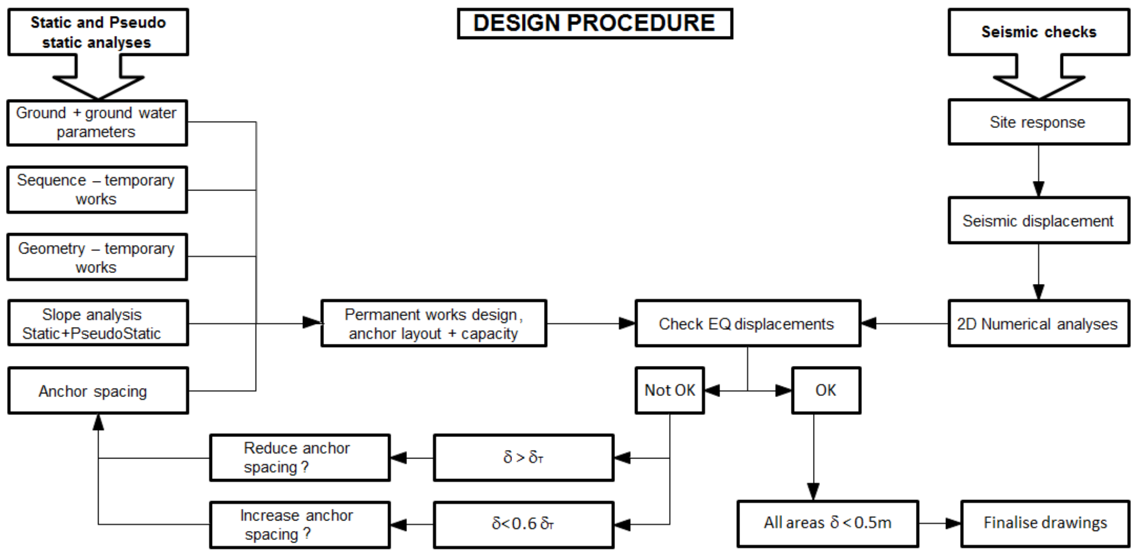

Figure 1: Overall design methodology adopted for all slopes

Geology and Groundwater Conditions

The geology of the site is highly variable, dominated by volcano-clastic tuff sediments, which vary in colour, grading and mineralogy. A complex history of tectonics has folded and faulted these units, displacing them vertically and laterally around the site, juxtaposing different rock types along unconformable boundaries. The geology of the examined area can mainly be divided into four different rock units: coarse ash crystal tuff unit; Basalt/Andesite/Rhyolite; Mudstone/Siltstone Unit; and Fault zone material. A ground model was developed based upon geological and geomorphological mapping of the site, interrogation of the historic and recent ground investigation data, evaluation of ground monitoring instrumentation, and assessments of shear surface locations from borehole records (e.g. changes in material strengths, in-situ testing and core recoveries). The simplified ground model used for design consists of a geological sequence of a "weaker rock" overlying a "stronger rock" . A shear zone was assumed between the weaker and the stronger rock locally. The examined area is split into two zones with different geological conditions; a northern zone designated as "Area A" where the shear zone is present and a southern zone designated "Area B", where the shear zone is absent. An example of the SLOPE/W model used for Area B is shown in Figure 2.

For the definition of geotechnical parameters (Table 1), lower bound unconfined compressive strengths derived from field and lab tests were adopted. Specialist XRF/XRD testing also confirmed the presence of expansive smectite rich minerals, which had a significant effect on the material strength when exposed to water and exhibited very low residual shear strengths. Residual strength parameters were adopted for the fault zone. No benefit was taken from the strain rate effects or material dilation that occur during seismic events. Discounting such considerations introduces a degree of conservatism in the design. Displacement and groundwater monitoring of landslide areas facilitated back analysis and verification of material parameters, especially the shear strength being mobilised across shear zones. The back analysis has also allowed for the assessment of the influence of matters such as variations in groundwater tables and differing failure surface geometries. The groundwater table has been defined based on piezometer measurements.

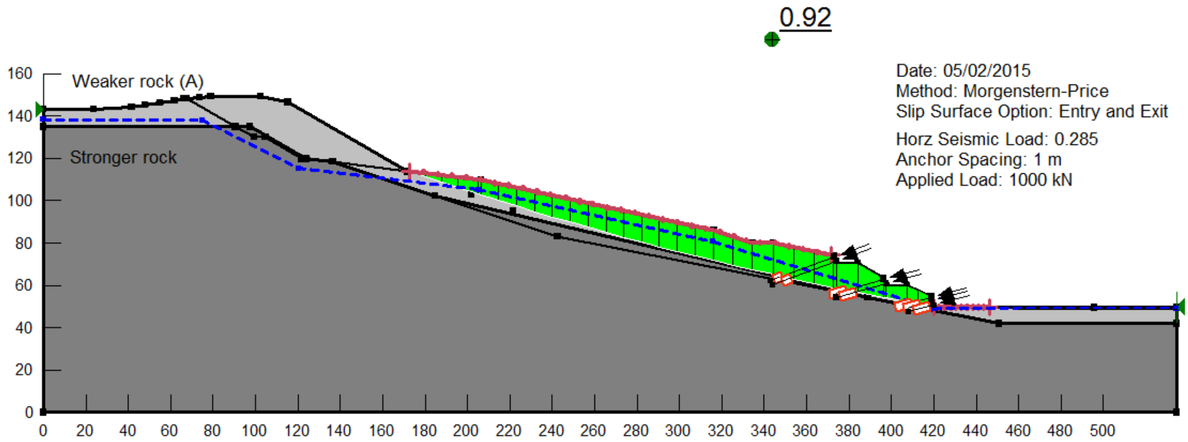

Figure 2: Geological units and design profile of Area B

Table 1: Summary of geotechnical design parameters: uniaxial compressive strength (UCS), geological strength index (GSI), (mi) and disturbance factor (D)

Material	**σ_c or UCS (kPa)**	**GSI**	**mi**	**D**
Weaker Rock	3500	20	10	a=0.7
				b=0.2
Shear Zone	3500	20	10	0.7
Stronger Rock	10000	40	20	0.7

Site Response Analysis

One dimensional site response analysis was undertaken in order to define the PGA variation within the sliding ground mass. The analysis was performed using the software DEEPSOIL (Hashash, 2010), with a modified hyperbolic model and non-Masing criteria to represent hysteretic loading and unloading behaviour of the ground. In order to perform the site response analysis, the following parameters have been defined:

- Input time histories;
- Dynamic ground properties (shear wave velocity, stiffness and damping behaviour with strain); and
- Bedrock properties.

Time histories selection

The project is located in a tectonically active region with evidence of a number of fault systems being present. However, there is no evidence that any of these faults is active today. A probabilistic seismic hazard assessment (PSHA) has been performed for this specific area, concluding that the peak bedrock acceleration for different return periods varies from 0.21g to 0.82g. A peak bedrock acceleration equal to 0.5g was adopted to define the base seismicity at the site corresponding to a return period of 475 years or 10% probability of exceedance in the next 50 years, satisfying the local seismic code requirements. In order to perform the site response analysis and the dynamic non-linear analysis, appropriate ground motions were selected. Seven time histories consistent with the prevailing hazard of the area in terms of magnitude, epicentral distance, fault mechanism and bedrock shear wave velocity, were chosen from different earthquake databases (PEER and European). These motions were linearly scaled to match the desired peak bedrock acceleration of 0.5g. A time-domain spectral matching procedure was also implemented with the software RSPmatch (Hancock et al., 2006) in order to modify the original accelerograms to be compatible with the target bedrock response spectrum. The software performs a time domain modification of an acceleration time history to make it compatible with a user specified target spectrum. Figure 3 shows one of the ground motions, first linearly scaled and then spectrally matched. It can been seen that the spectral demand is underestimated for short and long periods and overestimated for a period range of 0.2s to 1s when the motion is just scaled, while the ground motion after the spectral matching fits the design spectrum for all periods.

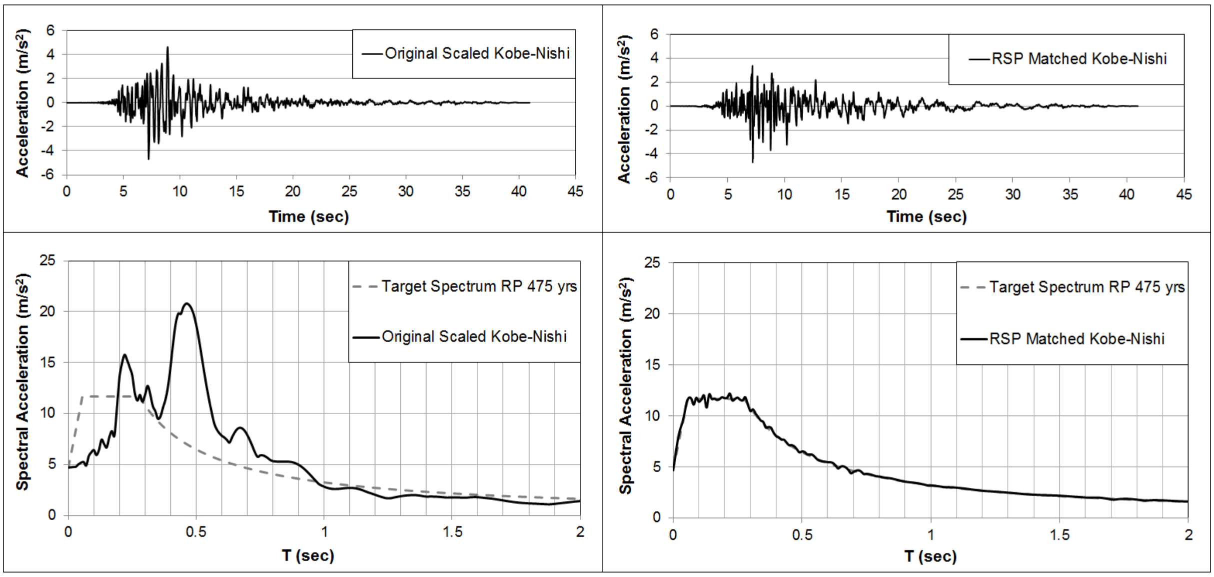

Figure 3: Example of ground motion modified to match the target spectrum; left figures show the original motion, linearly scaled to the target PGA; right figures show the motion after spectral matching

Dynamic Ground Properties

Two different profiles have been investigated, one representative of Area A, including a shear zone, and one representative of Area B. The shear wave velocity profiles were designed after a thorough review of the existing geophysical tests, SPT tests and ground strength parameters.

The following shear wave velocity values were adopted for the different layers: Weaker Rock-V_s=400m/s, Shear Zone-V_s=300m/s and Stronger Rock- V_s=750m/s.

In the absence of advanced laboratory testing and published stiffness degradation and damping curves for fractured rock, the derivation of the stiffness degradation curves has been performed on the basis of the proposed A* framework discussed by Eadington and O'Brien (2011).

The damping curves of each layer were defined using the relationship proposed by Zhang et al (2005), based on a modified hyperbolic model and a statistical analysis of existing resonant column and torsional shear test results from 122 specimens.

The derived stiffness and degradation curves were used in order to calibrate the hyperbolic model parameters used in DEEPSOIL. The parameters of the model that provide a best fit to the A*curves and do not alter the implied shear strength of the ground were adopted. The adopted stiffness degradation and damping curves used in the site response analyses of Area A are shown in Figure 4.

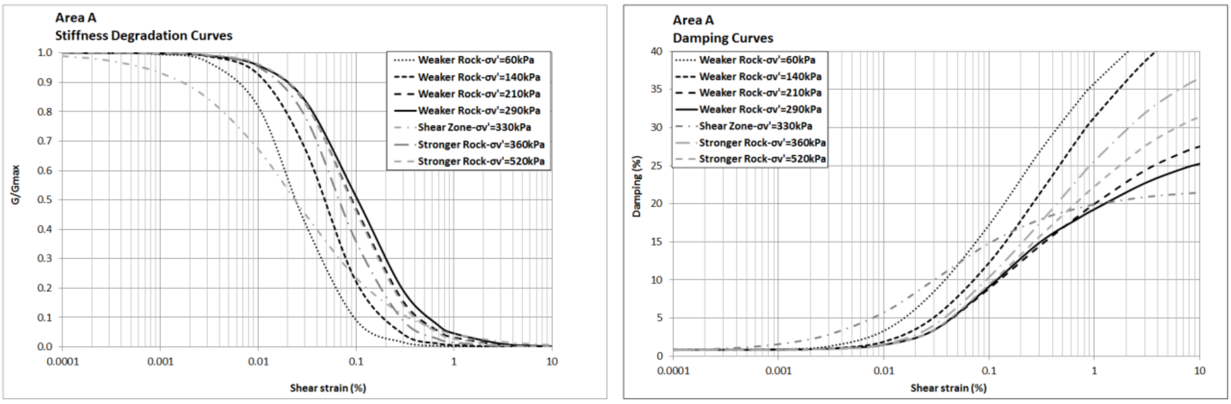

Figure 4: Stiffness Degradation and Damping Curves adopted for the site response analyses of Area A.

Site response results

The variation of PGA and shear strains with depth is presented in Figure 5 and Figure 6 for Area A and Area B respectively.

Several comments can be made from these results:

- The response of the two areas is very different. The maximum PGA occurs at the interface between the weaker rock and the shear zone for Area A, and at the surface for Area B, where the shear zone is absent;
- The presence of the shear zone plays a significant role in the definition of the surface PGA. For Area A, due to the high magnitude of peak bedrock acceleration, the shear zone develops large strains, resulting in high damping and hence in the attenuation of the PGA to an average value of about 0.31g at the surface. The shear zone effectively acts as an "isolator" zone;
- The absence of the "isolator" zone in Area B and the presence of a thin layer of weaker rock underlined by stronger rock, produce a shallow amplification of the ground motion, resulting in a high PGA value equal to 0.57g at the surface.

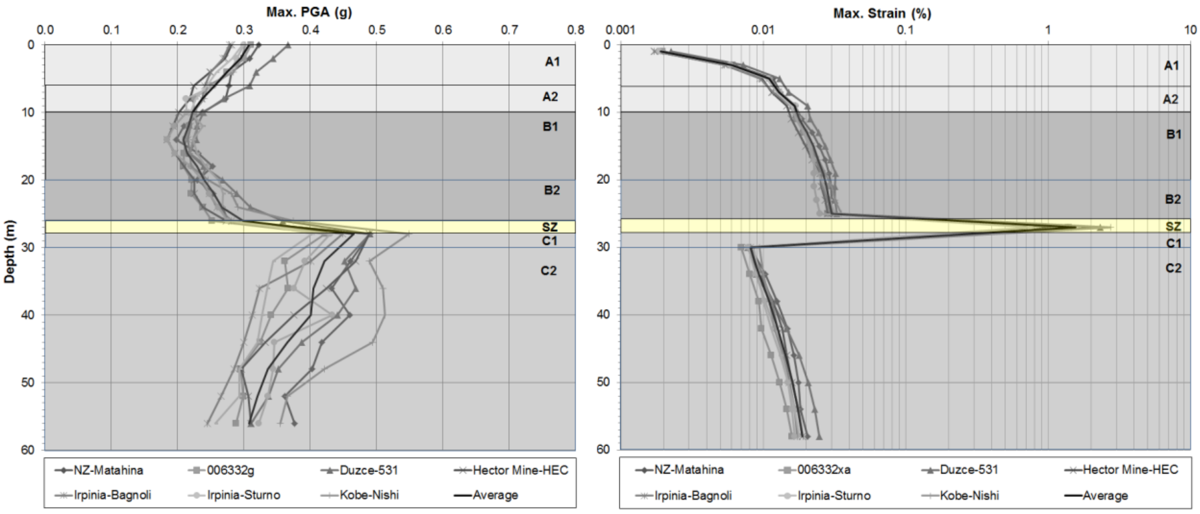

Figure 5: PGA and maximum shear strain with depth for Area A

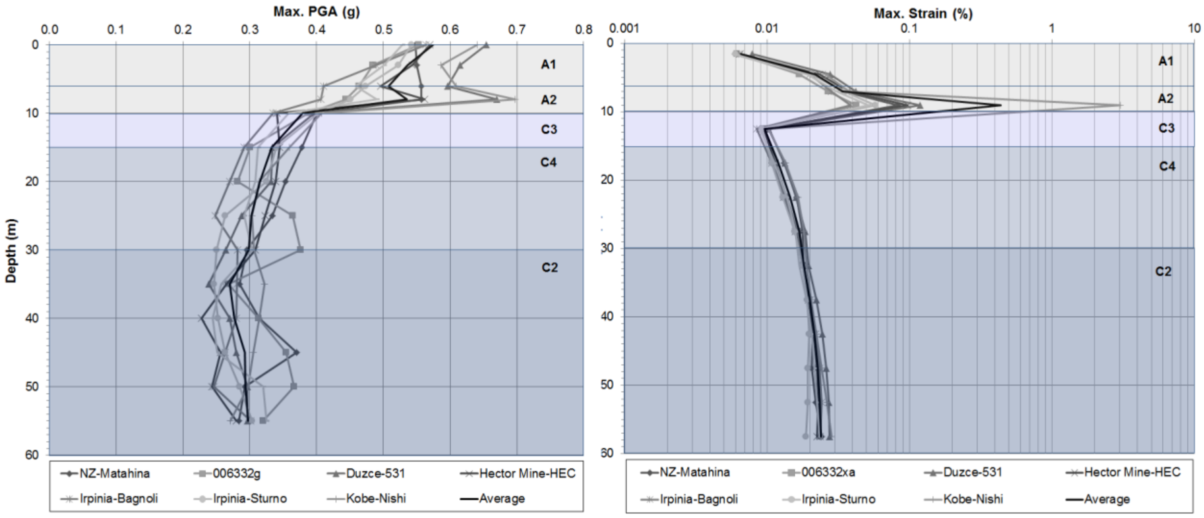

Figure 6: PGA and maximum shear strain with depth for Area B

Displacement Calculations

The results presented in this paragraph concentrate on Area B, as in this area the PGA values obtained with depth were larger. After performing pseudo-static analyses with a k_h=0.285 a FOS=0.92 was obtained **(Figure 2**). The expected earthquake induced sliding displacements were initially predicted using the software SLAMMER (Jibson et al., 2013), with the aid of various empirical methods. Among the relationships implemented in SLAMMER the Rathje and Antonakos (2011) relationship is considered the most sophisticated for slope displacement calculations, as it is applicable to both rigid and flexible sliding masses. In general, if the sliding mass is relatively shallow and stiff, a rigid sliding block analysis is considered appropriate because the period of the sliding mass is zero and its dynamic response could be ignored. Deeper and softer sliding masses are flexible and have natural periods greater than zero, thus the rigid sliding block model is not appropriate. A decoupled sliding block analysis computes the dynamic response without consideration of the sliding displacement and then uses the dynamic response results to compute sliding while a coupled analysis simultaneously computes the dynamic and sliding responses.

In order to perform the displacement calculations, a k_y of 0.25g was obtained for Area B with the aid of SLOPE/W analysis.

The calculated displacements using different empirical methods were varying between 6cm-15cm for the examined slip mechanism, with an average value of 8cm and were below the performance limits.

The expected earthquake induced sliding displacements were also predicted using rigorous (Newmark type) models with the use of the spectrally matched time histories. Three types of analyses were performed to calculate the average displacement: rigid block analysis with only downslope displacements and coupled and decoupled analysis of flexible sliding block with equivalent linear soil model. The resulting displacements with the addition of one standard deviation were around 2cm and hence smaller than the ones calculated with the empirical equations.

A sensitivity study was undertaken in order to understand the influence of the shear wave velocity and the thickness of the failing mass on the calculated displacements. The characteristics of the previous examined landslide were adopted. The results of the sensitivity are illustrated in Figure 7. It can be seen that the maximum of the average displacements for the lower shear wave velocities (V_s=300&400m/s) are reached for a slope height of 6m, while they tend to reduce for smaller and larger failing masses. Increasing the shear wave velocity, the displacements tend to be constant after a certain value of the failing mass thickness. For V_s=750m/s, the maximum average displacement is around 8cm with a height of the failing mass equal to 10m or thicker.

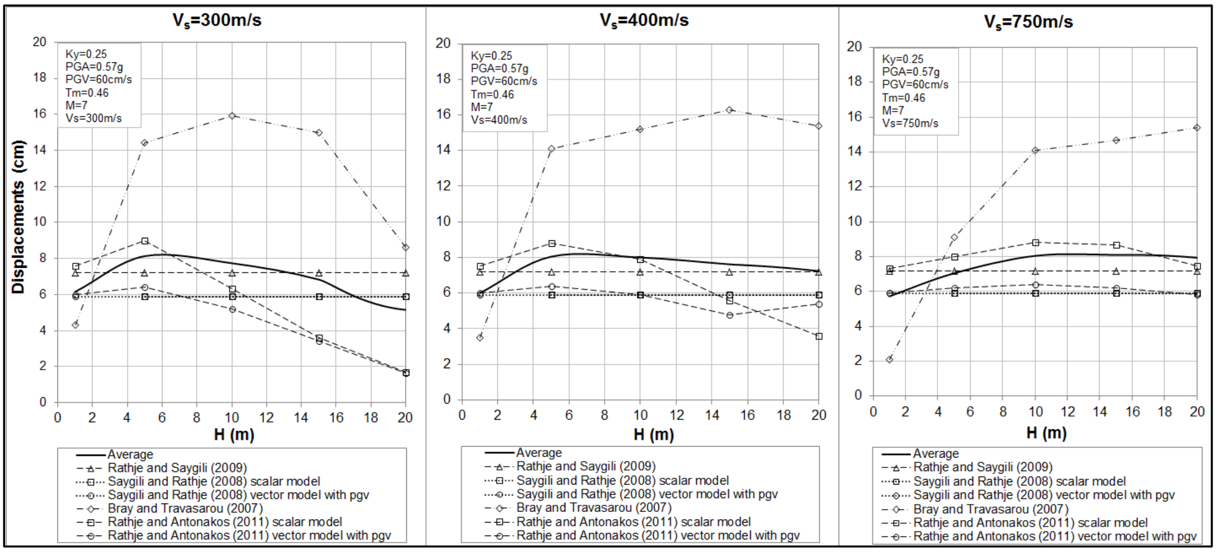

Figure 7: Displacement vs. height of the landslide for Area B considering different shear wave velocities: a) V_s=300m/s, b) V_s=400m/s, c) V_s= 750m/s

Full Dynamic Analysis

A numerical analysis using the proprietary finite difference software, FLAC Version 7.0 (Itasca 2011), was performed in order to corroborate the results obtained with the simplified methods. The numerical modelling has incorporated a three-stage process comprising of: back analysis of the existing slope stability, static stability of the design profile, and seismic performance of the slopes. It is recognised that the sig3 hysteretic damping model used to describe the ground behaviour in FLAC follows the Masing criteria, which allow a simplification of the ground response during dynamic loading. As a result, DEEPSOIL was used to compare the sig3 model prediction in 1D site response analysis. Adopting this approach, the compatibility in terms of ground response (attenuation or acceleration) between the DEEPSOIL and the numerical analysis provides a simple calibration check and adds the confidence that the numerical modelling is generating a similar ground response as derived from an independent method that uses a more advanced constitutive ground model. Following calibration of the numerical model, the dynamic model was run with three earthquake time histories, noting that each time history was used twice with the principal direction independently analysed both 'away' from the slope (opposite direction) and 'towards' the slope. Hence, overall there were six dynamic analysis runs for each analysis section. The resulting displacements are shown in Figure 8, for one of the analysis runs based on shear wave velocity of 400m/s. It can be seen from this figure that the 3-tier ground anchor system progressively reduces the displacement from a maximum value of approximately 30cm around the mid-height of the sliding mass to less than 10cm at the toe level. Furthermore, it should be noted that the toe displacement of between 5cm and 10cm predicted by the FLAC dynamic analysis is commensurate with those determined by the SLAMMER analyses (see Figure 7(b)).

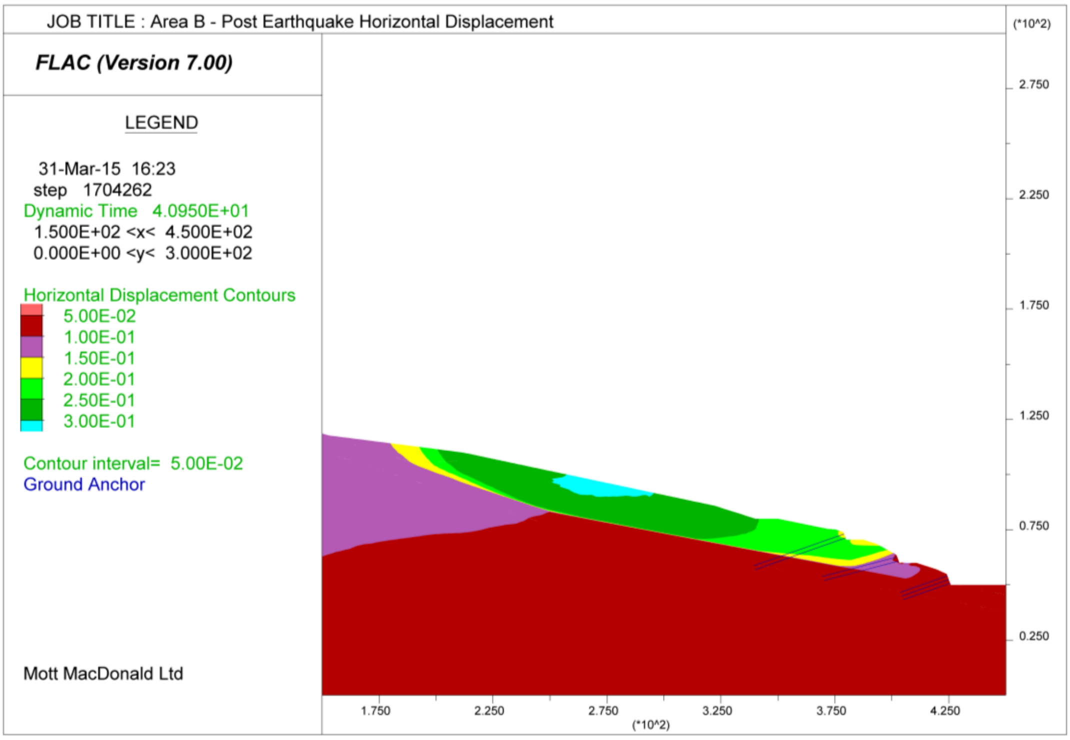

Figure 8: Horizontal displacements from FLAC

Conclusions

The paper illustrated the iterative approach adopted for the detailed design of hillside stabilisation measures for a site with two active landslides. The landslides occurred during excavation works and subsequently Mott MacDonald was commissioned to develop a detailed design for construction of 1800 linear metres of hillside up to 120 metres high. The overall design followed a performance-based approach.

The earthquake demand imposed on the slope was defined with the aid of non-linear site response analyses. This allowed substantial reduction in the earthquake demand at areas where a fault zone was identified. In case the design was performed by using the local seismic code specified PGA values significant cost would be introduced in the remediation works.

The seismic performance of the slopes was satisfied by keeping the displacement below a limit value. The limit value was selected in accordance with the project requirements. It has to be mentioned, however, that most of the codes do not specify the deformation limits for different earthquake scenarios for routine design, therefore their selection can be challenging. Seismic displacements were calculated with empirical, rigorous and numerical analyses. The different methods gave confidence in achieving the performance criteria.

The slope stabilisation measures encompassed the use of approximately 8000 soil nails, with a total length of around 80km and 1500 ground anchors, with a total length of around 52km. For the particular examined 'Area B' discussed above, the adoption of the performance based design procedure resulted in the use of 40% less anchors than would have been used if pseudo-static approaches with a target FOS=1.1 were adopted. Given the size of the length of site, the performance based design approach saved in excess of US$15 million in cost associated with the ground anchors.

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