A RISK AND DEFORMATION-BASED DESIGN APPROACH FOR MSE WALLS ON ROCK FILL SLOPES ALONG THE SEA TO SKY HIGHWAY TEST SECTION

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ABSTRACT

The Sea to Sky Highway will be upgraded in preparation for the Vancouver 2010 Olympic and Paralympic Games. An 800 m long test section was constructed to demonstrate that the proposed upgrades to the Sea to Sky Highway were achievable across very difficult terrain while at the same time minimizing impacts to the traveling public. Mechanically Stabilized Earth (MSE) retaining walls are required along substantial lengths of the highway alignment to support the southbound lanes of the proposed highway widening. This paper describes the design approach adopted for the MSE walls on rock fill slopes along the Sea to Sky Highway Test Section that includes both reliability and deformation (FLAC) analysis. Some construction considerations are also described.

INTRODUCTION

The Sea to Sky Highway (also referred to as Highway 99N) extends along the eastern side of Howe Sound connecting the communities of Horseshoe Bay, Squamish and Whistler in British Columbia. The route traverses very steep terrain, and the existing highway was developed by excavating significant cuts into the bedrock slopes above the highway and placing fill on the down slope side. Major upgrading of the highway will be carried out in preparation for the Vancouver 2010 Winter Olympics and Paralympic Games including further excavation on the upslope side of the highway and construction of fill slopes and Mechanically Stabilized Earth (MSE) retaining walls on the downslope side along significant portions of the 100 km long route. In 2003, the British Columbia Ministry of Transportation (BC MoT) committed to design and construction of an 800 m long Test Section located some 5 km north of Horseshoe Bay (See Figure 1). The Test Section would serve to provide valuable insights into the design and construction challenges that would face the BC MoT and Highway Designers as they embarked on this major highway upgrade project.

Retaining walls are required along a substantial portion of the highway alignment to support the southbound lanes of the proposed highway widening. The foundation conditions along the downslope side of the highway are highly variable including weathered soils and colluvium cover, rock fill, and bedrock.

SITE CHARACTERIZATION

Within the study area, the Sea to Sky Highway follows the eastern side of Howe Sound. Generally steep terrain exists above and below the highway alignment, requiring rock cuts up to some 35-m in height or more for development of the highway. Rock fill slopes extend below the western edge of the southbound lanes of the existing highway all the way down to the CN Rail tracks located at the toe of the slope along a substantial length of the highway. Beyond the tracks, the ground slopes down further over an estimated vertical height of about 30 m or more to Howe Sound. The existing rock fill slopes on the west (downslope) side of the highway are inferred to have been constructed by end-dumping techniques during initial construction of the highway. These rock fills are inferred to be heterogeneous and in a state of loose to compact in-situ relative density and considered to have a low factor of safety against instability. The extent of stripping of deleterious materials prior to fill placement during initial highway construction is unknown, and considered to be a risk to construction on the fills.

The subsurface conditions in the general area of the proposed MSE retaining walls were previously investigated by geophysical techniques and test holes put down by BC MoT with the results summarized in a detailed report prepared by Golder Associates Ltd. (1). Because of difficult access and highly variable surface and subsurface conditions, it is extremely difficult and expensive to obtain reliable information on the subsurface stratigraphy. In general, the materials underlying the west (downslope) edge of the existing highway comprise random rock fill overlying, colluvium, till-like soils and bedrock.

The approximate geometry and thickness of the rock fill was transferred onto appropriate sections from the highway design drawings using data from site investigations as well as recent and historic air photos.

A visual assessment was carried out of the rock fill slopes and clinometer measurements were collected along the slopes to obtain a better appreciation of the conditions and angle of repose of the existing random highway rock fill.

STABILITY OF EXISTING ROCK FILL SLOPES

From field observations, the nominal diameter of existing rock fill particles is estimated to vary from about 100 mm to 1.1 m or more. Areas having larger blocks tended to have open voids and their loose state suggests that the blocks were not placed but more likely side-cast or end-dumped. The existing rock fill is generally comprised of granite and quartz diorite rock compositions.

The surficial stability of rock fill slopes is largely controlled by the effective friction angle. As the fill slopes in the Test Section appeared to be reasonably stable under existing conditions, and no significant roadway distress was observed or reported in the vicinity of the Test Section, it was assumed that the factor of safety (FS) of existing end-dumped rock fill slopes was greater than unity (i.e. FS>1), but unlikely to be greater than 1.1 for potential near-surface failure surfaces. The mean value of friction angle for the materials forming the surficial layers of the existing rock fill slopes was back-calculated using the Mohr-Coulomb relationship:

$$\phi = \tan^{-1}(FS \times \tan \alpha) \tag{1}$$

where:

ϕ = friction angle

FS = factor of safety

α = slope angle

Clinometer measurements of the existing rock fill and talus slopes were carried out in the field and supplemented by a review of topographic survey data for the highway section between Stations 1105+930 and 1105+990. A statistical analysis was carried out on a data set that included 26 field measurements yielding a mean angle of repose of 38.4° with a standard deviation and coefficient of variation of 1.6º and 4%, respectively. This agrees well with the angle of repose near a talus slope apex which is typically in the order of 36º to 38º (Evans and Hungr (2)). If one assumes a FS value of 1, then the mean angle of internal friction is about 38.4 º.

In the analysis for the test section, a mean angle of internal friction, ϕ of 41° was assumed for the existing end-dumped rock fill corresponding to an assumed FS of 1.1. The computed standard deviation from the available data set of 1.6º indicates a very narrow spread of the rock fill friction angle which does not reflect the expected significant variation in material properties related to the assumed placement methods. Consequently, based on engineering judgment, a standard deviation of 4.5 ° was assigned to the end-dumped rock fill material. A unit weight of 17 kN/m^3 was assumed for the existing rock fill material.

DESIGN CONSIDERATIONS

Geometry

The existing downslope site conditions along the southern most 200 m of the test section comprised predominately rock fill with some sand infill within the voids. The results of two test holes indicated that the depth (thickness) of the rock fill along the crest of the existing rock fill slopes exceeded 10 m. It was not considered practical or feasible to remove the large quantity of existing rock fill material from this area and hence it was decided to construct a flexible MSE retaining wall system on the existing rock fill slopes to support the southbound lanes. The highway widening along this area of the highway consisted of constructing an MSE retaining wall near the crest of existing rock fill slopes but only after these slopes were enhanced with a high strength engineered rock fill slope widening (buttress) placed along the outside of the existing slopes. The proprietary MSE retaining wall systems were constructed using welded wire forms with crushed basalt rock infill as fascia, and reinforced with uniaxial polymeric geogrid. Crushed sand and gravel processed from a nearby borrow source was used as backfill. Figure 2 shows a general detail of the MSE retaining wall constructed on the improved rock fill slope.

At the time when the Test Section was being designed, BC MoT design criteria limited the height of vertical MSE retaining walls constructed using polymeric reinforcement to 5 m. Spatial constraints such as the proximity of the CN Railway tracks near the base of the existing slope together with vertical and horizontal alignment considerations and the requirement for a 2 m setback at the toe of the proposed MSE retaining wall dictated that the rock fill slope widening be constructed at a slope of about 1.2 Horizontal to 1 Vertical. The overall height of the rock fill slope supporting the MSE retaining wall varied from about 10 to 12 m. The sections analyzed were at Stations 1105+945 and 1105+965.

Initial design options for this section of highway considered a split-grade configuration whereby the southbound lanes would be constructed lower in elevation than the northbound lanes with a median retaining structure between the southbound and northbound lanes. A benefit of the split-grade cross-section is to unload some of the existing rock fill slopes hence improving stability. However, significant cost savings could be realized by eliminating the median walls that would be required and there was sufficient space to accommodate a four-lane wide template at one grade. The split-grade concept was therefore not pursued in detail.

Rock Fill

Parameters assumed for the existing end-dumped rock fill were previously described. A literature review of published and readily available data was carried out in order to determine representative engineered rock fill parameters for use in the analysis. Pertinent data from work by Saboya & Byrne (3), Barton & Kjaernsli (4), Charles & Watts (5), Marsal (6), Marachi et al. (7), Leps (8) and others were reviewed. Data in the above publications are primarily related to rock fill dams, with particle sizes ranging between about 20 mm and 200 mm.

The relationship between shear strength, ϕ and effective normal stress, σ_N for rock fill is nonlinear. A range of effective normal stress between 0 and 300 kPa is considered to be representative for the highway sections analyzed and hence a data subset was compiled of reported shear strength values where testing was conducted at normal stresses, σ_N of less than 300 kPa. Extreme values of friction angle and unit weight were excluded from this data subset. The results of basic statistical analysis of the data subset provided a friction angle, ϕ of about 46.8º with a standard deviation of 3.8º and a unit weight, γ of 17.8 kN/m^3 with a standard deviation of about 2.05 kN/m^3.

Colluvium

Although the two test holes put down by BC MoT along the crest of the existing rock fill slopes did not penetrate through the entire thickness of rock fill, it was assumed in the analysis that a weaker soil (colluvium) strata was present at the base of the existing rock fill. A review of historic air photos indicated that a portion of the test section was flat-lying and located in a terrain depression. Colluvial soils on slopes in Coastal BC are typically loose to compact, heterogeneous, relatively, organic-rich, coarse-grained soils with a typical thickness of about 1 m. They can typically be classified as sandy gravel or gravelly sand with fines seldom exceeding 15% to 20% by weight. If not removed, the organic component of the soil is subject to decay and can lead to reduction of the soil strength over time. The shear strength of the weaker colluvial soil layer was assumed to correspond to granular soil having a small component of apparent cohesion. A literature survey of the probabilistic distribution of sand and gravel strength properties, presented for example in work by Lacasse and Nadim (9) and Phoon (10) provides an estimate for the coefficient of variation of friction angle between 2% and 11%. If a coefficient of variation of 5% to 6% and a mean friction of 36º are assumed, this provides a standard deviation of about 2º. An apparent cohesion with a mean value of 5 kPa and a standard deviation of 1 kPa was also included in the stability models where this layer is assumed to be present. A unit weight of 16 kN/m^3 with a standard deviation of 1.5 kN/m^3 was assumed for the colluvial soil unit along the steeper portions of the original terrain (prior to filling).

A second type of weak layer was assumed for the flat-lying portion of the original ground surface where the presence of a more organic rich, fine-grained soil is feasible. The parameters for this soil, based on judgment, comprised a mean friction angle of 25º with a standard deviation of 2º, and an apparent cohesion of 10 kPa with a standard deviation of 1.5 kPa. A unit weight of 17 kN/m^3 and a standard deviation of 1 kN/m^3 was assumed for the colluvium along the flatter portions of the original ground (prior to filling).

Groundwater Conditions

No site information was available on the groundwater conditions within and below the rock fill slope. Signs of seasonal water ponding were observed in the CN Rail ditch along the toe of the highway rock fill slope. The water appears to be seasonal surface flow from highway culverts. Assumptions were made of both dry conditions and the condition where a 1 m column of water is present in the weaker soil layer (where applicable) or on top of the original ground surface (i.e. within the rock fill slope). Such ground water levels may only occur following or during sustained and/or high-intensity rainstorms. In our analysis we have assumed a 200-year return period for this condition.

MSE Retaining Wall Loading

Loading of the rock fill slope by the MSE retaining wall was analyzed explicitly by including the retaining structure as a structural unit in the geomechanical model. The mean unit weight of the MSE wall was assumed to be 22 kN/m^3 with a standard deviation of 0.5 kN/m^3, which corresponds to well compacted crushed sand and gravel fill.

Seismic Loading

A peak horizontal acceleration of 0.19 g for firm ground, corresponding to a 1:475-year ground motions, was obtained from the Pacific Geoscience Centre for the site area. An additional allowance for a 20 % increase over the firm ground acceleration was made for topographic effects, providing a peak horizontal surface acceleration of 0.23 g. To account for uncertainty in the input parameters, a mean value of 0.23 g was assumed with a standard deviation of 0.02 g.

Traffic Loading

To assess the performance of the slope and the retaining wall under possible scenarios of operating conditions, the dead load exerted by vehicles (assuming to be AASHTO H20 loading) on the structure was considered. The worst case scenario for such loading would represent traffic stopped on the highway due to highway maintenance or vehicle breakdown. To simulate the dead loading, an areal load of 25 kPa uniformly distributed over a width of 2.5 m in both of the south-bound lanes was assumed.

RELIABILITY ANALYSIS

Given the level of uncertainty associated with many of the material parameters and factors influencing stability, a reliability analysis was carried out to assess the probability of failure for the proposed MSE retaining wall constructed on a rock fill slope. Details of the reliability analysis are provided in Golder Associates Ltd. (11). The procedure for the reliability analysis includes three primary steps; i) identification of possible slope failure scenarios, ii) assignment of material properties, groundwater conditions and external loading, and iii) calculation of the probability of failure for possible failure scenarios.

The reliability analysis for the Test Section study considered potential shear failure occurring along the base of the rock fill in a weaker soil stratum (colluvium) beneath the rock fill or within the rock fill itself. Ravelling and shallow surficial failures were not considered, since their effects are not expected to be critical to highway performance. Furthermore, only geotechnical parameters and/or possible causes of failure were considered. Only geotechnical uncertainty was addressed in the analysis.

Possible Slope Failure Scenarios

In the first step of the reliability analysis, the possible slope failure scenarios that were deemed as likely to occur, or represent conditions present at the site, were identified including:

- presence of colluvium or other weak soil layer at the base of the rock fill;
- groundwater conditions;
- vehicle loadings; and,
- seismic loading.

The above scenarios can be graphically presented in an event tree as shown in Figure 4. Each branch of the event tree is assigned a conditional probability of occurrence. The probabilities were assigned based on either the occurrence period of a specific scenario (eg. 1:475-year earthquake is assigned a probability of occurrence of 0.21%) or our engineering judgement.

Assign Input Parameters

Input parameters for the geomechanical slope model, many of which were described previously, were treated as random variables and assigned a range of values in accordance with a probability density distribution. A normal probability distribution was assumed for all of the random variables.

Calculate Probability of Outcome and Slope Failure

Failure refers to any occurrence of an adverse event under consideration where the demand on a facility exceeds the facility's capacity and the factor of safety becomes less than unity (i.e. FS<1). The probability of failure was evaluated as a function of uncertainties in key input parameters, for each branch of the event tree as described below.

For each branch of the event tree, the probability of the given scenario is calculated. For each scenario, the frequency distribution of the factor of safety for slope stability is calculated with input values as previously described. Based on the mean and standard deviation of the factor of safety, the reliability index is calculated. The total probability for slope failure is calculated by summing up the probability of failure for each outcome multiplied by the probability of the occurrence of the outcome. The probabilistic stability analyses were carried out using the computer program SLOPE/W™ (Version 5.1).

Results of Analysis

The results of the probabilistic slope stability analysis carried out for the Test Section indicated that the presence or absence of a weaker soil stratum along the base of the rock fill had little or no influence on the probability of failure or the calculated mean factor of safety for the sections analyzed. The presence or absence of groundwater along the base of the rock fill or within the weaker soil stratum did not have any influence on the probability of failure and the calculated mean factor of safety for the section analyzed.

The reason for the observed insensitivity in the analysis to both the weaker soil layer (colluvium) beneath the rock fill and the water table is considered due to the location of the critical slip (failure) surfaces being within the upper portion of the rock fill as shown on the stability analysis model on Figure 3. The critical slip surfaces shown on Figure 3 indicate that a potential failure would extend from approximately the centre of the two southbound lanes, pass behind the heel (back) of the MSE retaining wall and extend down a short distance above the toe of the highway fill slope adjacent to the CN Rail line. The results of our site-specific slope stability analysis also suggest that providing the depth to bedrock is about 3 m or more, then, the base (colluvium) layer will likely not significantly influence stability. However, this assumes that the fill is buttressed with a zone of engineered rock fill along the outside edge of the slope and that this buttress is terraced into the existing slope and keyed into a competent base at the bottom of the slope, in accordance with BC MoT specifications.

Attempts to force the slip surface deeper into the rock fill in order to intersect the relatively weaker soil stratum (either dry or saturated) resulted in higher mean calculated factors of safety, and hence lower probability of slope failure. The results of the probabilistic slope stability analyses are presented in Table 1 below.

The design factor of safety typically accepted by BC MoT is 1.5 under static conditions and 1.1 under seismic conditions. As seen from the last two columns of Table 1 there is only 1.6% to 4.6% probability that these criteria would be achieved for static conditions and less than 0.5% chance for seismic loading conditions.

The results of the probabilistic slope stability analysis are summarized in Table 2. When the probability of failure of all likely outcomes for the site-specific analyses is summed up (as shown in Figure 4) the annual probability of failure of the slope is in the order of 1.86×10^{-3} (0.186%). This failure probability combines both the static and seismic conditions. It is noted, however, that a factor of safety of less than unity (FS<1) for seismic conditions may not necessarily represent a catastrophic slope failure. The slope could be subject to permanent deformation that may be repairable. A detailed analysis of the potential seismic deformations was carried out to evaluate the likely magnitude and pattern of deformations and is discussed in the next section of this paper. When the annual probability of slope failure is computed for static conditions only, a value of 3.83×10^{-5} (0.00383%) is obtained.

The results obtained on the probability of slope failure can be compared with other published information. Risk-related safety criteria have been developed by BC Hydro for assessing the risk of dam failure, and are used for comparison purposes for this study. The BC Hydro risk criteria relate the annual probability of occurrence of a hazard (failure) to the potential number of fatalities. For one potential fatality, the suggested acceptable societal risk is associated with an annual probability of occurrence of 10^{-3} (0.1%). The annual probability of slope failure for the section along the Test Section, as described herein, is about twice this value.

El-Ramly et al. (12, 13) reported probability of slope failure and associated reliability index for a tailings dyke at Syncrude operations in Fort McMurray (Alberta) and for the James Bay dykes (Quebec). Although the dykes at James Bay were not constructed, their design by a team of international experts is considered accurate. The adequate performance of the Syncrude dyke had an estimated probability of unsatisfactory performance of 0.0016 (0.16%) and a reliability index (β) of 2.31. At James Bay the probability of unsatisfactory performance was 0.0047 (0.47%) and the reliability index (β) was 2.32. It is of interest that both of the above reliability indices rate between categories 'poor' and 'below average' on the proposed US Army Corps of Engineers reliability index chart (see Table 3 below). El-Ramly et al. (13) noted that the significant difference between the estimated values from a structure performing adequately and the recommended values may suggest that practice is less reliable than commonly thought.

The same authors also reported a back-calculated probability of slope failure for the Lodalen landslide that occurred in 1954 in Norway, El-Ramly et al. (14). It is of interest that the back-calculated probability of unsatisfactory performance for the slope was computed to be 76%.

A comparison of the calculated annual probability of slope failure for the site-specific sections analyzed along the Test Section to the US Army Corps of Engineers criteria (see Table 3) suggests that the performance of the proposed MSE walls on the deep rock fill slopes will be above average for static conditions and average under the design seismic loading conditions. In areas where the rock fill slopes are shallower, the probability of failure will be higher. Additional analyses were carried out to assess various mitigation options that may be used to improve performance for these conditions and the results are presented in the Golder Associates Ltd. (11).

In areas of shallow rock fill underlain by steeply sloping bedrock, which may possibly be covered with some colluvium or deleterious materials, it may be more feasible and practical to remove the fill materials and found the proposed MSE wall directly onto competent bedrock. In other areas where complete removal of the fill materials is not practical or feasible, specialized techniques such as engineered buttress fills, light weight fill materials and/or anchoring to bedrock will likely be required to improve performance.

SEISMIC DEFORMATION ANALYSIS

Since the pseudo-static analysis indicated a high probability that MSE walls founded on rock fill will have a factor of safety less than 1 under design seismic loading, a detailed dynamic analysis was carried out to evaluate the likely magnitude and pattern of deformations.

Methodology

The earthquake-induced deformations of the roadway and slopes were computed for the 475-year level of earthquake shaking, in accordance with BC MoT requirements. The dynamic ground response analyses were carried out using the finite difference code FLAC^{2D}. These analyses were carried out neglecting 3-D effects such as those resulting from the curvature of the original topography and the highway; given the relatively gentle curvature, 3-D effects would be expected to be minor. The externally specified UBCSAND stress-strain model developed by Dr. Peter Byrne at the Dept. of Civil Engineering, University of British Columbia, was used to model the non-linear and in-elastic behaviour of rock fill and the underlying colluvium layer. UBCSAND models the dynamic load-deformation behaviour of geologic materials using an elasto-plastic stress-strain model, and permits coupled stress and flow analyses where required. The MSE wall and associated reinforced soil zone were modeled as a relatively rigid block obeying Mohr Coulomb material behaviour.

One-dimensional (1-D) ground response analyses were carried out initially to assess the applicable damping parameters for the different material zones. Parallel analyses were carried out using the computer programs DESRA-2C and FLAC for a soil column located midway up the slope. The peak ground surface accelerations were used as the bench-marking variable to establish the Raleigh damping to be used in the FLAC analysis, in addition to the hysteresis damping originating from material non-linearity. The damping variations so established were used in the subsequent two-dimensional (2-D) analysis of the slope.

Proposed Model Configuration and Assumptions

The FLAC^{2D} finite difference model developed for the analysis of dynamic deformations is shown on Figure 5. The various material zones considered in the analyses together with the nodal points that were monitored for detailed performance are also shown on this figure.

The section selected for analysis was considered to be one of the more critical sections with respect to general embankment and wall deformations which will be most difficult to mitigate. It includes steep slopes and thick random rock fill, and also has the highest MSE wall along this section. While stability at the transition zones from rock fill to more competent bedrock foundation may be more critical, options to improve performance at these shallow bedrock locations are expected to be more readily available.

For the purposes of analysis, the geometric domain of the slope was discretized into 1080 zones from elevation 20 m to 50 m. The domain included bedrock, colluvium, random rock fill comprising the majority of the slope, an engineered rock fill buttress on the downslope side, a 5 m high MSE wall section, and an anchored reinforced concrete retaining wall at the split grade (which was not constructed in the final design).

A layer of colluvium some 1 to 1.5 m in thickness, comprising compact to dense sand and gravel with silt, was assumed above the bedrock surface. The groundwater table has been assumed to be located at, or slightly above, the bedrock surface. It is therefore considered possible that a portion of the colluvium could be saturated at times of wet weather conditions. However, in view of the limited thickness of the colluvium layer, the expected free-draining nature of the overlying rock fill, and the inferred compact to dense consistency, a low risk of liquefaction was assigned to this layer under the design seismic loading conditions. Should liquefaction occur, the deformations could be much larger.

The model adopted in the analysis consisted of a split-grade section with 4 lanes (two lanes southbound and two lanes northbound), an MSE wall that was 5 m in height constructed on a stabilizing rock fill buttress (berm) that was about 8 m in vertical height and extending about 2 m out from the front face of the MSE wall. The stabilizing rock fill buttress was assumed to be constructed of well-compacted select rock fill materials that will be supported on the existing rock fill comprising the slope. (*It is noted that this was the original design concept for the Test Section. Subsequent to completing the detailed analysis, design and contract award, the split-grade concept was replaced with a design section where both southbound and northbound lanes are located at one elevation supported by a slightly higher engineered rock fill slope.*)

Material Properties

Shear Strength
Median values of shear strength parameters and unit weights were used in the deformation analyses consistent with the material properties used in the static reliability analysis of the rock fill slopes described above.

Moduli of Deformation
The shear and bulk moduli of deformation of loose rock fill were established based on data reported by Duncan et al (15) for comparable materials. The properties of random rockfill are difficult to estimate and are likely to vary from location to location; the parameters used in the analysis are considered to be conservative.

The shear moduli of deformation of the well-compacted rock fill buttress were estimated based on average values reported in the literature for the dynamic analysis of rock fill dams by Seed et al (16), Gazetas & Dakoulos (17), Iwashita et al (18), and Denge et al (19). The corresponding bulk moduli were computed using a Poisson's ratio of 0.2.

The shear and bulk moduli of deformation for the soils comprising the MSE wall were estimated using the Seed et al (20) data for well-compacted sands and gravels and a Poisson's ratio of about 0.4. Further details on the input parameters adopted in the analysis are provided in Golder Associates Ltd. (11).

Assumed Structural Parameters for the Median Retaining Wall
An anchored reinforced concrete L-shaped retaining wall was proposed to support the split grade of the highway separating the northbound and southbound lanes and included in the original analysis. Further details on the median retaining wall properties are provided in Golder Associates Ltd. (11).

Rock Anchors
Details on the material properties of the rock anchors used to support the L-shaped median retaining wall are provided in Golder Associates Ltd. (11). The material parameters considered for the retaining wall and the rock anchors are not considered to have a significant impact on the ground deformations computed for the southbound lanes or the MSE wall. It was necessary, however, to consider a reasonable set of parameters for the structural elements in order to develop the FLAC^{2D} model with a vertical split grade option.

Ground Motions
Dynamic analyses were carried out for the 475-year level of earthquake loading that corresponds to a peak horizontal firm-ground acceleration of about 0.20 g. Consistent with the standard of practice for the Fraser Lower Mainland area and seismic ground motion parameters developed for major bridge structures in the general area, an M7 event that is representative of 10 to 15 cycles of effective loading, and a duration of strong shaking of about 10 to 15 seconds was considered for the design earthquake.

The input ground motions were spectrally-matched to the 1:475-year firm-ground response spectrum provided by the Geological Survey of Canada (GSC) in April 2003 and uniformly scaled to a peak ground acceleration (PGA) of about 0.20 g.

Two sets of spectrally-matched input ground motions were used in the analysis of the slope. They consist of motions from the 1971 M6.5 San Fernando Earthquake recorded at the Caltech Station (N-S Component) and the 1989 M7.1 Loma Prieta Earthquake recorded at the Capitola Station (E-W Component).

Results of Seismic Deformation Analysis

Predicted Deformations

The computed peak and residual ground deformations are presented in tabular form for the horizontal and vertical directions at some of the locations shown on Figure 5. A range of values is indicated for the various elevations, reflecting the variations that occur at each level. The results of the horizontal computed ground deformation time-history at the MSE wall are shown on Figure 6 (Horizontal Displacements at Nodal Point 5).

The results indicate that peak transient lateral deformations of the southbound lanes are likely to vary from 150 to 370 mm. The largest lateral deformations of the order of 330 to 370 mm are predicted to occur immediately behind the MSE wall, reflecting the flexibility of the wall and soil-structure interaction response. The corresponding residual lateral deformations are about 50 mm smaller than the transient deformations. The computed maximum vertical deformations are 60 mm.

The large lateral deformations occur as a result of the outward rotation of the MSE wall and soil movements associated with the resulting void. The differential lateral and vertical deformations in the vicinity of the MSE wall are expected to be in the order of 220 mm. Further behind the MSE wall, the differential movements reduce to 40 to 50 mm. These large differential lateral and vertical deformations will induce cracking of the pavement structure that may render the southbound lanes unsafe, and hence unusable, immediately after the design earthquake.

Peak Ground Surface Accelerations

Peak ground accelerations in the order of 0.5 to 0.8 g have been computed at the crests of the MSE and reinforced concrete walls. These peak acceleration pulses are the result of both material behaviour as modeled using UBCSAND and effects of topographic amplifications. These somewhat larger localized high frequency peak accelerations do not have a significant impact on the computed deformations, since they occur over very small time increments in the order of $1\text{x}\ 10^{-5}$ seconds. They may be of interest to the designers of the walls.

Highway Damage from Past Earthquakes

A brief literature review was conducted to assess the level of damage to highway slopes and embankments caused by major earthquakes involving ground shaking comparable to that anticipated at the subject site to provide a qualitative validation of the predicted ground movements. The 1994 Northridge Earthquake (M6.8) caused damage to a portion of Mulholland Drive that runs along the crest of Santa Monica Mountains. The site of damage is located about 13 km from the epicenter of the earthquake. Damage in the form of vertical offsets of 300 mm and ground fissures that are up to 100 mm wide have been reported (EERI Preliminary Reconnaissance Report, March 1994). The peak horizontal ground acceleration experienced near the area of roadway damage is estimated to be about 0.4 g.

Damage has also been reported to the on-ramp access to northbound Highway 101 at Universal City where the roadway was supported using a high crib wall. Cracks about 25 mm wide occurred at a distance of some 2 m the face of the wall (see Figure 7).

The information contained in the various reports examined as part of this study indicated that damage to highways constructed in steep terrain and comprising high MSE walls is likely to be concentrated along the shoulder or the outer lane(s) of the roadway, providing that liquefaction does not occur beneath significant areas. Overall, lateral and vertical ground deformations of up to 300 mm are highly likely in roadways constructed on poor quality fill materials, where soil liquefaction is not a concern. Failure of steep slopes and localized rock falls from the area above the highway are also likely to result in damage and disruption to traffic on the highway.

The seismic deformation analysis and review of comparative case histories indicted that the deformation that would likely occur as a result of the design seismic event will be of sufficient magnitude to likely render the southbound lanes unusable following the seismic event; however, the northbound lanes are expected to remain relatively unaffected and hence will provide access for emergency vehicles.

CONSTRUCTION CONSIDERATIONS

As much as possible, existing rock fill removed from some areas of the site was re-used to construct rock fill slope widening required along the west edge of the highway. Deleterious materials, where encountered, were disposed of and replaced with suitable rock fill materials. The rock fill was also obtained from rock cuts in other areas of the Test Section. The new rock fill used to widen the existing fill slope was constructed by placing the rock fill in carefully controlled lifts and benched into the existing slope. Compaction was achieved by trafficking with heavy construction equipment (See Figure 8). Following completion of construction of the rock fill slope, the MSE retaining wall was constructed in accordance with the recommendations of the proprietary wall suppliers.

Not all of the rock along the highway corridor is considered suitable for re-use to construct rock fill slopes and fills. Potentially Acid Generating (PAG) rock exists along several sections of the highway alignment. PAG rock is typically associated with rock where mineralization is present in ore rich zones. Exposure of PAG rock to atmospheric oxygen and water, results in generation of sulphuric acid with significant detrimental effects on the surrounding environment. Although no PAG rock was encountered along the test section, if such rock had been encountered, it would either need to be appropriately disposed of or could be re-used providing it was encapsulated to prevent continued long-term exposure to the atmosphere.

Construction of the initial section of the Sea to Sky Highway was completed in August of 2004. Figure 9 shows the completed MSE retaining wall construction on the engineered rock fill slope.

CONCLUSIONS

The existing rock fill slopes along the west edge of the Sea to Sky Highway are only marginally stable. Results of detailed slope stability and seismic deformation analyses carried out for a specific proposed design section consisting of a 5 m high MSE retaining wall constructed on an improved (widened) rock fill slope located along the Sea to Sky Test Section have been presented and a number of conclusions can be drawn from the results of the analyses.

For deep rockfill (greater than about 5 m), constructed with an engineered rock buttress fill zone, a 1.2H:1V slope geometry and subjected to loading from a 5 m high MSE wall at the crest, the mean calculated static factor of safety is about 1.3, with a probability of failure less than about 0.02 percent. The mean calculated seismic factor of safety using pseudo-static analysis methods is about 0.93, with a probability of failure of about 87 percent. In most instances, "failure" is more likely to involve limited deformations during/following a seismic event rather than complete collapse. Results of detailed analysis indicates that the calculated FS is not likely to be influenced by an underlying sloping bedrock surface or colluvium layer where the thickness of rock fills (depth to underlying bedrock surface) is greater than about 5 m beneath the rear of the reinforced mass and the critical slip surfaces pass within the upper zone of the rock fill.

The performance of the proposed MSE walls on an improved deep rock fill slope is expected to be above average under static conditions and average under the design seismic event when comparing the calculated annual probability of failure for the specific section analysed (i.e. between Stations 1105+945 and 1105+965) and the US Army Corps of Engineers criteria. In areas of shallow rock fill underlain by steeply sloping bedrock, which may possibly be covered with some deleterious materials, the probability of slope failure will be higher. In some areas, depending on the depth to bedrock, it may be feasible and more practical to remove the fill materials and found the proposed MSE walls directly onto competent bedrock. In other areas, where complete removal of the fill materials is not practical nor feasible, specialized techniques such as engineered buttress fills, lighter weight fill materials and/or anchors may be considered to improve performance.

Results of detailed analysis indicate that it is not possible to achieve a calculated factor of safety of 1.5 under static conditions and 1.1 under design seismic conditions without including specialized techniques such as buttress fill, lighter weight fill materials and/or anchor support.

Results of dynamic ground response analyses carried out using the finite difference code FLAC^{2D} indicate that under the design (1:475 year) earthquake loading, the peak lateral and vertical ground movements of the southbound roadway section will be about 150 to 330 mm and 60 mm, respectively. The MSE wall located at the edge of the southbound lanes is likely to experience slightly larger seismic lateral and vertical movements of 190 to 370 mm and 90 to 100 mm, respectively. The corresponding permanent movements will be about 40 mm smaller than the peak transient movements. The results of the deformation analyses are consistent with field observations of recent earthquake damage to roadways constructed in steep terrain and comprising loose fill materials where soil liquefaction is not a concern. The results also suggest that the northbound lanes will remain functional following the design seismic event allowing for passage of emergency vehicles. The deformations predicted by the FLAC^{2D} analysis carried out as part of the design of the Test Section were subsequently adopted as design criteria by BC MoT for the Sea to Sky Highway Upgrade Project.

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Table 1 Results of Probabilistic Slope Stability Analysis

Seismic Loading	Vehicle Loading	Probability of Failure (%)	Probability of Non-Failure (%)	Mean FS	Standard deviation	Reliability Index (β)	Probability of FS<1.5 (per cent) static	Probability of FS<1.1 (per cent) dynamic
No	Live Load	0.003	99.996	1.35	0.088	3.98	95.4	----
No	Dead Load	0.015	99.984	1.31	0.087	3.98	98.4	----
Yes	Live Load	86.796	13.203	0.93	0.061	< 1	----	99.6
Yes	Dead Load	87.847	12.153	0.93	0.061	< 1	----	99.7

Table 2 Probability of Slope Failure

Case	Annual Probability of Failure
Static stability of slope [A]	3.833×10^{-5}
Seismic stability of slope [B] (pseudo-static analysis)	1.823×10^{-3}
Total stability of slope [A+B]	1.861×10^{-3}

Table 3 Relationship between Reliability Index (β) and Probability of Failure (P_f) and Expected Performance Level (US Army Corps of Engineers, 1997)

Reliability Index (β)	Probability of Failure (P_f)	Expected Performance Level
5.0	3×10^{-7}	High
4.0	3×10^{-5}	Good
3.0	0.001	Above Average
2.5	0.006	Below Average
2.0	0.023	Poor
1.5	0.07	Unsatisfactory
1.0	0.16	Hazardous

Figure 1 Project Location

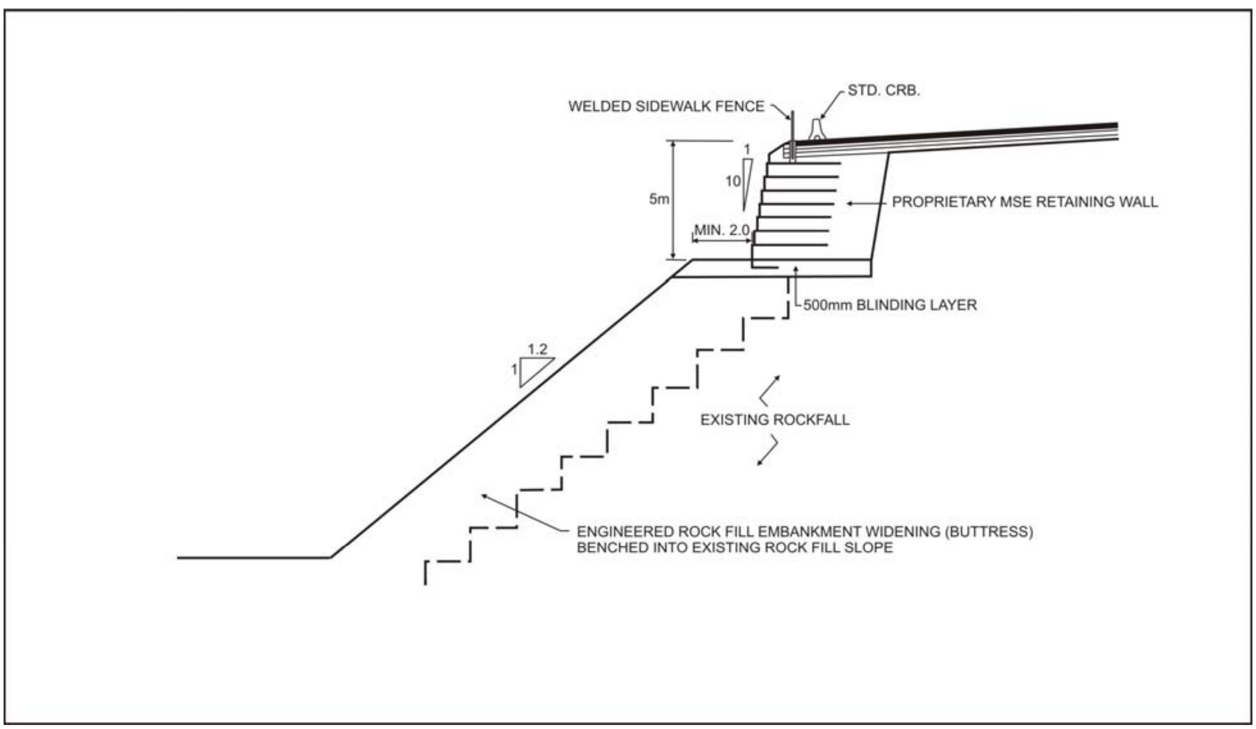

Figure 2 General Detail of MSE Retaining Wall Constructed on an Improved Rock Fill Slope

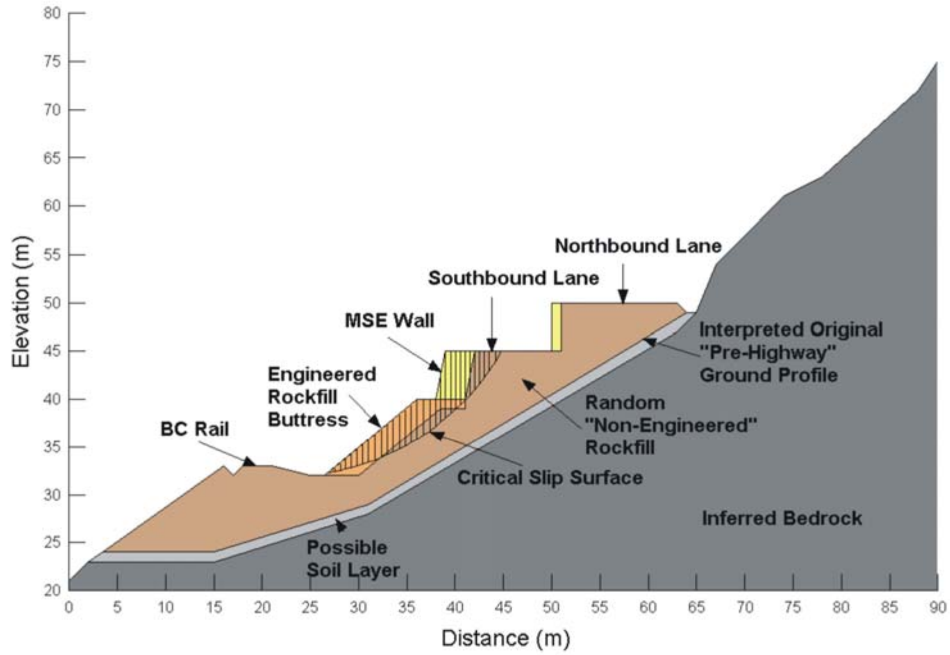

Figure 3 Slope Stability Model

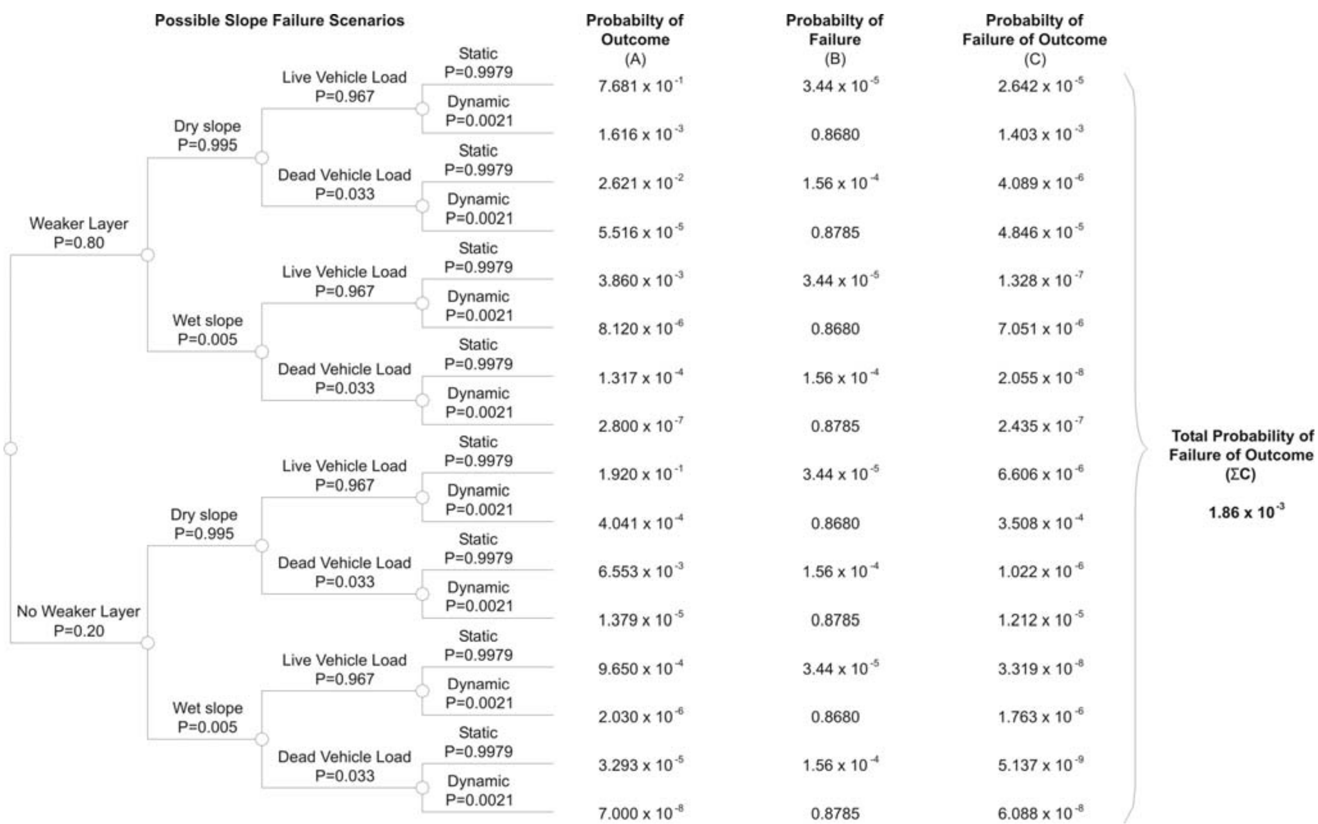

Figure 4 Event Tree for Probability of Slope

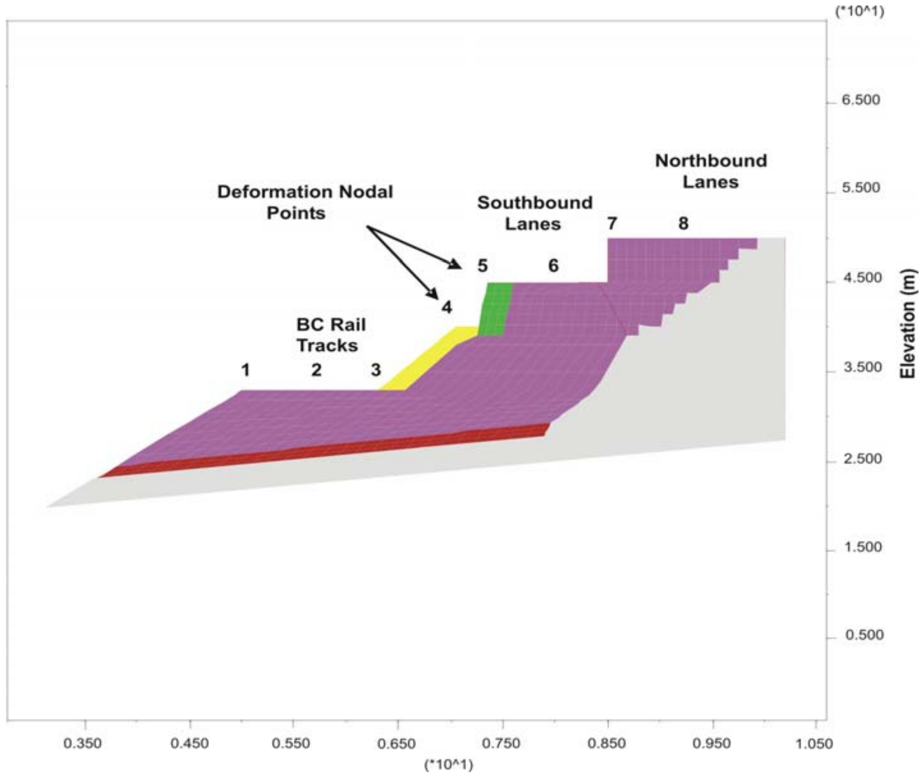

Figure 5 FLAC^{2D} Finite Difference Model

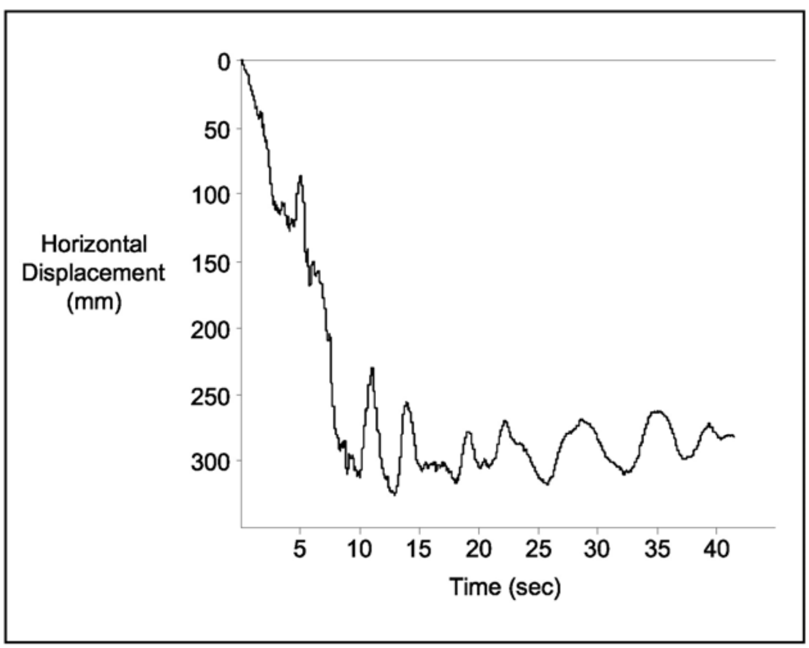

Figure 6 Computed Horizontal Displacements at MSE Wall (FLAC^{2D})

Figure 7 Damage to Road Shoulder on an On-Ramp to Highway 101, Universal City, LA, USA (1994 Northridge Earthquake)

Figure 8 Construction of Rock Fill Slope to Support MSE Retaining Wall

Figure 9 Completed MSE Retaining Wall on Rock Fill Slope