Ministry of Transportation of British Columbia

George Massey Tunnel # 1509

Prediction of Tunnel Performance with No Ground Improvement

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Amendments Record Sheet

Executive Summary

The purpose of this study is to estimate a level of earthquake that the structurally retrofitted George Massey Tunnel can tolerate without life safety damage, under the current ground and soil conditions without any ground improvement. The return period of this reduced level of earthquake is expected to be less than the original 475 yr return period design seismic event with the Peak Ground Acceleration (PGA) of 0.25 g at the firm ground level, as specified in the 2000 seismic strategy and assessment criteria. The tunnel displacements, laterally 0.3 m and upward 0.09 m, under the reduced level of earthquake shaking are predicted to be in the same order as the tunnel displacements under the 475 yr return period events with the previously proposed (2006 design) ground densification and seismic drains.

In this study, both the displacements calculated by dynamic analysis using the program 2D FLAC and liquefaction triggering calculated using the program SHAKE2000 were used. Predictions of tunnel performance (i.e. tunnel lateral and upward displacements) without ground improvement at different reduced shaking levels were made and compared to the performance of the tunnel under the 475 year return period event with improved ground.

Based on our assessment, similar tunnel performance occurs at earthquake shaking with peak ground acceleration (PGA) in the range of 0.13 g to 0.16 g at the firm ground level. The return periods of these levels of earthquake shaking would be approximately 150 to 240 years. PGAs at the ground surface (riverbed) level for the predicted smaller earthquakes with 0.13 g to 0.16 g at the firm ground level are estimated to be about 0.1 g to 0.12 g.

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1 Introduction

The seismic retrofit design and upgrade work for the George Massey Tunnel was carried out in several phases from 2000 through 2006:

i. Seismic strategy and safety retrofit assessment – from 2000 to 2001;

ii. Final seismic retrofit and rehabilitation design – from 2001 to 2003;

iii. Structural retrofit contract awarded and executed – from 2004 to 2006; and

iv. Additional site investigation with 12 CPTs and two boreholes, and subsequent ground densification and seismic drain design optimization 2006.

The seismic design philosophy adopted in this project was based on both ground improvement and structural retrofit. The ground improvement was aimed at reducing the potential for tunnel flotation and/or excessive lateral movement during the 475 yr design seismic event. The structural retrofit, combined with upgrading the emergency system, was designed to improve the ductility of the tunnel section and increase the emergency pump capacity to reduce the flood risk to the public in case of a 475 yr design seismic event.

The structural retrofit work was completed in 2006. The plan, at that time, was to start the ground improvement during the following years. However, after the completion of the structural retrofit, MoT decided not to go ahead with the ground improvement based on a second value engineering (VE) exercise taking into consideration a new plan of replacing the tunnel with a new crossing in the next 20 years.

Then MoT requested, in order to be able to assess the risk of eliminating the ground improvement work, a study that would define a reduced level of earthquake that the current structurally retrofitted tunnel can tolerate without lift safety damage.

The purpose of this study is to estimate a level of earthquake shaking that the structurally retrofitted George Massey Tunnel can tolerate without life safety damage, under the current ground and soil conditions (i.e. without any ground improvement). The scope of work was given in our proposal dated 2008 September 22.

2 Background of Analysis

In 2000 and 2001, Buckland & Taylor Ltd. was retained by MoT to conduct a seismic strategy and safety retrofit assessment on George Massey Tunnel for the 475 yr return period design earthquakes. The findings and results from this study are summarized in the report (Buckland & Taylor Ltd. 2001) entitled “George Massey Tunnel No. 1509 – Seismic Safety Retrofit and Rehabilitation – Assessment Phase – Seismic Retrofit Strategy Report” dated 2001 March 26. The comprehensive geotechnical assessment was performed mainly by FLAC total stress based soil liquefaction triggering and lateral spreading analyses, as well as SHAKE analyses. It was found that the loose sands around and underneath the tunnel would liquefy, causing severe tunnel flotation and/or lateral movement if the riverbed is sloped transversely across the tunnel due to riverbed scour. As part of the study, the preliminary ground improvement design which included stone column vibro-replacement ground densification and seismic drains was done by FLAC analyses.

This study was followed by the Final Seismic Retrofit and Rehabilitation Design in 2001 and 2002. The ground densification and gravel drain concepts for mitigating soil liquefaction and lateral spreading hazards were verified by the centrifuge tunnel model tests conducted at Rensselaer Polytechnic Institute (2002) and by blast induced soil liquefaction and gravel drain testing at the south end of George Massey Tunnel (Pacific Geodynamics Inc., 2002). The state-of-the-art FLAC numerical models with effective stress based UBCSAND soil constitutive model were calibrated against the results of the centrifuge tests and the field gravel drain tests. The final ground densification and seismic drain design were done by parametric studies conducted using the calibrated FLAC numerical models. The calibration of the FLAC numerical models, the results of FLAC parametric analyses for the final ground improvement design and the tunnel structural ABAQUS model analyses using the ground displacement patterns, soil springs and tunnel displacement time histories predicted by the FLAC analyses are summarized in Yang et al. (2003, 2004, 2006), Naesgaard et al. (2004) and Taylor et al. (2004).

In 2006, MoT carried out a geotechnical exploration at the tunnel site under the contract with Trow and ConeTec. This exploration included 12 Cone Penetration Tests (CPT) along both sides of the tunnel and two boreholes near the north bank and the south bank. The CPT results are included in Trow’s report entitled “Geotechnical Exploration and Report – George Massey Tunnel – CPT & Drillhole Testing 2006” dated October 18, 2006.

The 2006 site investigation confirmed the soil stratigraphy determined by the 1991 MoT site investigation including 6 CPTs/SCPTs and 2 boreholes, as well as previous site investigations for the original tunnel construction.

In the same year, three detailed transverse soil profiles based on additional 2006 CPTs were analyzed in a series of updated FLAC models using a new version of UBCSAND 904a. These 2006 results (Buckland & Taylor Ltd. 2006) led to a revised ground improvement scheme with a reduced densification width and reduced number of seismic drains from the scheme which was originated in 2001 to 2002 during the final seismic retrofit design.

3 Analysis Procedure Used in Current Study

Although uncertainties still remain in the seismic input motions used in our numerical analyses and in the numerical model itself simulating the soil liquefaction triggering mechanisms, the numerical models were carefully calibrated in our 2001 and 2006 FLAC analyses to the 2002 centrifuge tunnel model test results and field gravel drain test results commissioned by MoT. The numerical model calibration to the centrifuge models and field gravel drain test results has given us significant confidence that our FLAC models are capable of simulating the tunnel behavior and the soil behavior. The numerical exercise employed in this study can give useful insight into the tunnel and soil behavior when using the reduced earthquake input motions.

The following steps were taken for the current assignment.

i. Select Model 1 from our 2006 FLAC analyses based on Section A soil profile in Trow's report (2006) as the worst soil section in terms of 10 m thickness of loose sand underneath the tunnel. This transverse section model has a ground surface slope of 9% on one side of the tunnel to simulate the existing large scour hole that potentially can cause severe lateral spreading following soil liquefaction. Figure 1 shows the layout and mesh of Model 1. Figure 2 shows the layout and mesh of Model 1 with a continuous silt barrier at the underside of the tunnel;

ii. Create Model 1F to be a flat ground surface version of Model 1. This model was also used because previous work has indicated that tunnel heave or upward flotation is worse for portions of the alignment where the adjacent ground is flat. Figure 3 shows the layout and mesh of Model 1F;

iii. Run Model 1 and Model 1F with no ground improvement under the 475 yr return period design earthquake;

iv. Run Model 1 and Model 1F with ground improvement under the 475 yr return period design earthquake (the results reflect the effects of ground densification and seismic drains in the 2002 seismic retrofit design and 2006 revised ground densification scheme);

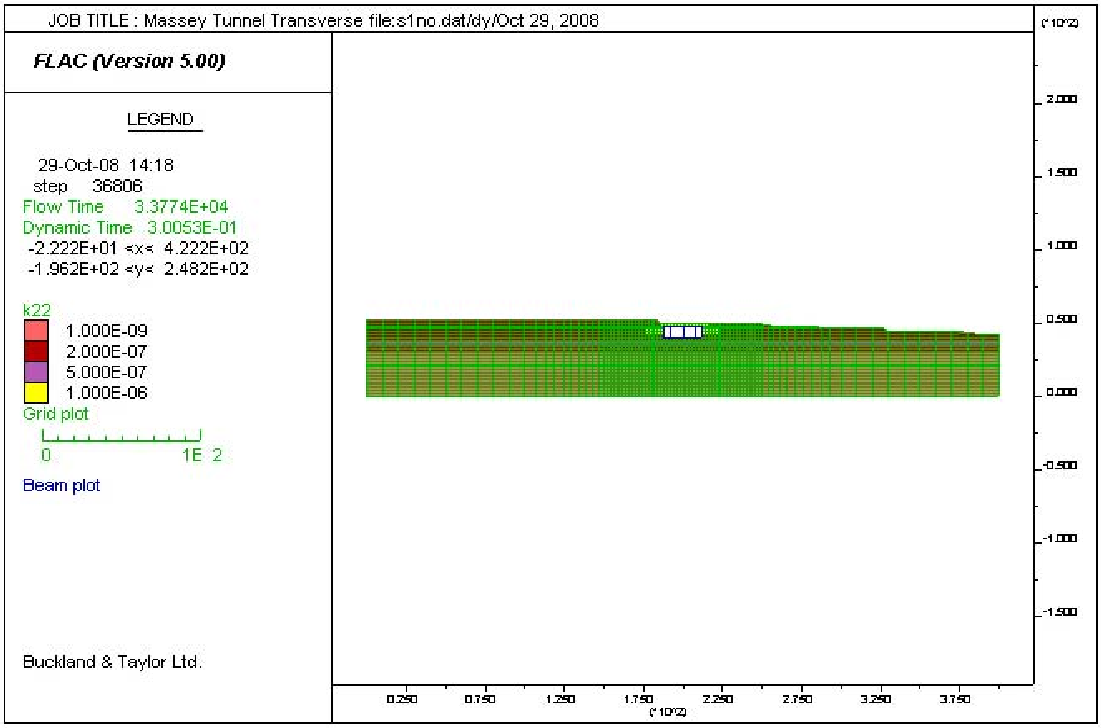

Figure 1: Model 1 (9% Ground Surface Slope on One Side) Layout and Mesh

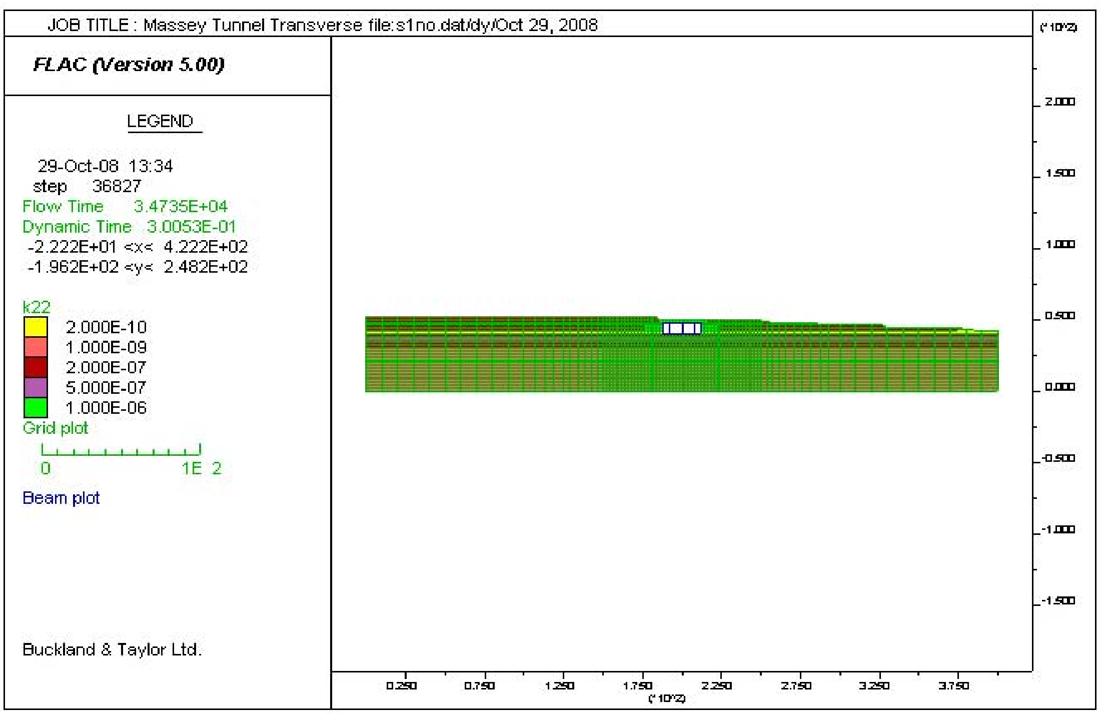

Figure 2: Model 1 (9% Ground Surface Slope on One Side) with a Continuous Silt Barrier at the Underside of Tunnel

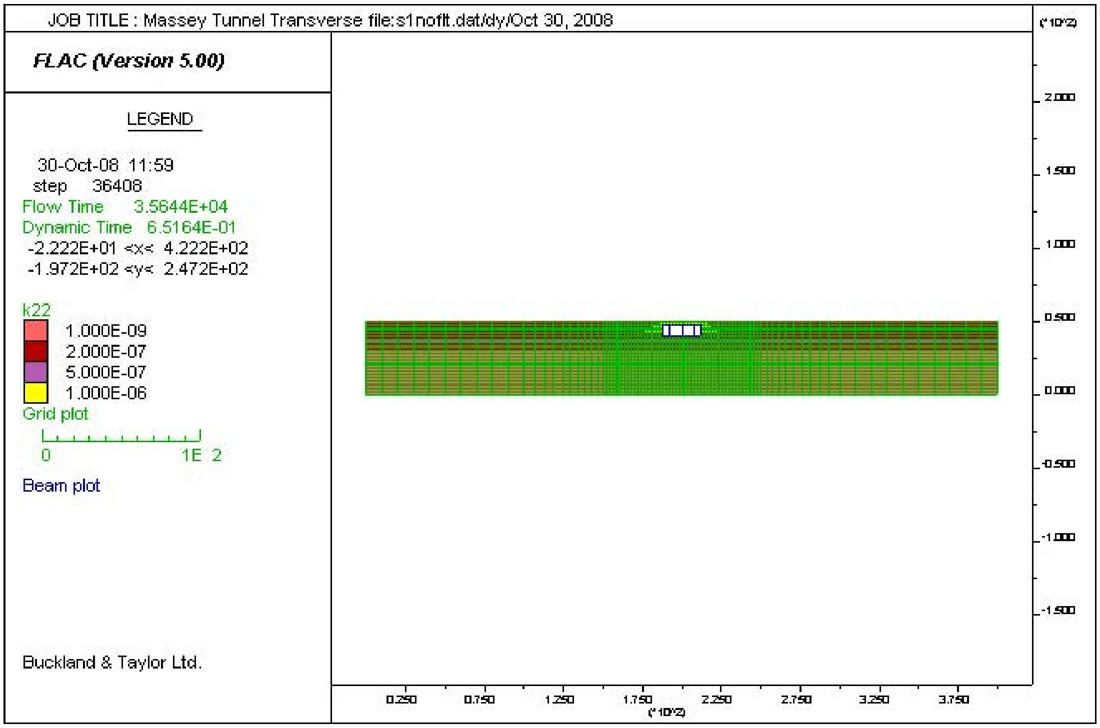

Figure 3: Model 1F (Flat Ground Surface) Layout and Mesh

v. The following simple scaling method was adopted for this study.

Rerun Model 1 with scaled earthquake records so that the horizontal peak ground acceleration (PGA) at the firm ground level is reduced as follows:

PGA = 0.25 g (factor = 1.0, i.e. the 475 yr return period original design earthquake);

PGA = 0.19 g (factor = 0.75, roughly corresponding to a 312 yr return period event);

PGA = 0.16 g (factor = 0.625, roughly corresponding to a 240 yr return period event);

PGA = 0.13 g (factor = 0.52, roughly corresponding to a 175 yr return period event); and

PGA = 0.09 g (factor = 0.375, PGA = 0.09 g taken from the 100 yr return period event at the nearby Canada Line North Arm Bridge site).

In each of the above FLAC runs, the horizontal and vertical input motions are scaled and applied to the FLAC model at the lower boundary.

Figure 4 shows the 475 yr return period original design earthquake spectrum at the firm ground level and the spectra for the scaled earthquake records used in this study. The underlining assumption of simply scaling the 475 yr design earthquake amplitude is that the source earthquakes contributing to the lesser events would have similar magnitude and frequency content to the 475 yr return period design event. Based on the Geological Survey of Canada (GSC) seismic model this assumption is not unreasonable. 100, 475 and 2475 year return period GSC design spectra are almost parallel and reasonable estimates of each can be obtained by direct scaling. Deaggregation of events of different return periods also does not show large changes in mean magnitude.

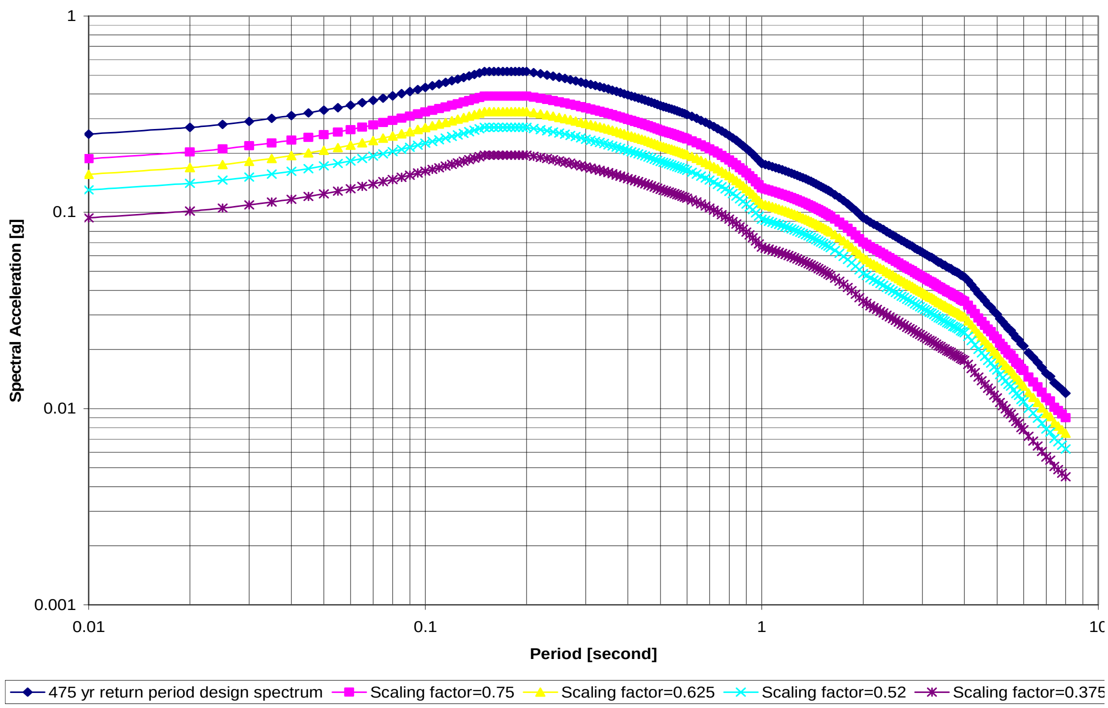

Figure 4 475 yr Return Period Original Design Spectrum and Spectra for Scaled Earthquake Records Used in This Study

The return periods are calculated based on the following mathematical Log-Log relationship (Adams et al, 2000, Lee et al, 2003 and Matheu et al, 2005) for the Vancouver region from the 475 yr event with PGA of 0.25 g and the 100 yr event with PGA of 0.09 g:

Log (PGA) = a*Log (return period) + b

Where a and b are the slope and intercept on the ordinate axis, respectively, with the values of a = 0.6557 and b = -2.357.

It is noted that some scholars believe that PGA and return period may be correlated using the following semi-Log relation when looking at different types of source earthquake, distances, characteristics of certain earthquakes and specific locations (Anderson 2008).

PGA = a* Log (return period) + b

Where a = 0.2364 and b = -0.383

Table 1 shows comparison of return periods calculated using Log-Log and semi-Log relations. Considering uncertainties in many aspects of earthquake engineering, the differences using the two different correlations are not considered to be significant for the purpose of this study.

Table 1 Comparison of Return Periods Calculated using Log-Log and Semi-Log Relations

	PGA at the firm ground level [g]				
	0.25	**0.19**	**0.16**	**0.13**	**0.09**
Return Period Log-Log [yr]	475	312	240	175	100
Return Period Semi-Log [yr]	475	265	200	150	100

vi. Use the target lateral tunnel deflection of 0.3 m which was the lateral drift limit used in the 2006 FLAC analyses and revised ground improvement design. Use the target tunnel uplift of 0.09 m to prevent tunnel flotation, which is consistent with the 2006 revised ground improvement design. From the tunnel safety perspective, the tunnel lateral drift is not as critical as the potential hazard of tunnel flotation, a scenario of tunnel popping up to the ground surface. This is due to the fact that added longitudinal and transverse reinforcement inside the tunnel air tubes in the structurally retrofitted tunnel could not do anything in mitigating the tunnel flotation hazard due to massive soil liquefaction under the 475 year design earthquake; and

vii. Perform SHAKE2000 analyses to assess liquefaction triggering based on the 2006 CPTs at Section A (i.e. CPT 06-01 and CPT 06-07 for Model 1) of the Trow's report (2006). The Cyclic Stress Ratios induced by the 475 year return period design earthquake from the SHAKE analyses are scaled by the same factors shown in Step 5. From the SHAKE results, levels of PGA that induces only minimal amount of soil liquefaction can be predicted by comparing scaled Cyclic Stress Ratios (CSR) to the soils Cyclic Resistance Ratios (CRR). This comparison was performed using CPT 06-01 and CPT 06-07.

4 Results of FLAC Analyses

4.1 Results of FLAC Analyses with No Ground Improvement

Three FLAC runs were carried out in Step 3. The results are shown in Table 2.

Table 2 Results of FLAC analyses with no ground improvement and 475 year return period design earthquake with PGA of 0.25 g

	Model 1	Model 1 rerun	Model 1F
FLAC File	S1no	S1siltno	S1noflt
Tunnel uplift [m]	0.29	0.18	0.38
Tunnel lateral disp [m]	1.13	0.5	0.03
Notes	• No densification or seismic drains • Without a continuous silt barrier	• No densification or seismic drains • With a continuous horizontal silt barrier layer 2m thick across the model at the elevation of underside of the tunnel	• No densification or seismic drains • Without a continuous silt barrier

It is noted that Model 1 was analyzed with and without a continuous silt barrier. The reason for doing so is that the silt barrier layer may or may not be present at the soil section selected for FLAC analyses. The worse tunnel lateral displacement from the two scenarios is being considered. Model 1 without ground improvement and without a continuous silt barrier predicts more than 1.1 m tunnel lateral displacement, considerably more than the target value of 0.3 m. Model 1F with no ground improvement and flat ground surface gives tunnel uplift of almost 0.4 m which also is more than the 0.09 m design limit.

The presence of a continuous silt barrier has one major detrimental effect to the tunnel and was a subject of the previous seismic design studies in 2001 and 2006. Large excess pore water pressure generated during the 475 yr return period design earthquake in loose sands at the locations under the tunnel and outside the ground improvement zones tends to migrate toward the lower effective stress region directly under the tunnel and potentially pushes the tunnel out of the ground gradually over time. This is because the silt barrier forces the excess pore water to dissipate towards the tunnel instead of upwards to the ground surface. This possible tunnel flotation failure phenomenon was demonstrated by the FLAC simulation and documented in Yang et al. (2004, 2006) and Naesgaard et al. (2004).

The silt barrier becomes a non-issue in this study because scaling the original design earthquake to limit soil liquefaction will also limit excess pore pressure buildup and migration.

4.2 Results of FLAC Analyses with Ground Improvement

Three FLAC runs were carried out in Step 4. The results are shown in Table 3.

With the 2006 revised ground improvement scheme of soil densification and seismic drains, the tunnel lateral displacement and uplift are less than the target values set in Step 6.

Table 3 Results of FLAC analyses with ground improvement and 475 year return period design earthquake with PGA of 0.25 g

	Model 1	**Model 1**	**Model 1F**
FLAC File	S1den	T516g5-904a	S1denflt
Tunnel uplift [m]	0.09	0.06	0.06
Tunnel lateral disp [m]	0.25	0.21	0.01
Notes	• With densification and seismic drains • Without a continuous silt barrier	• With densification and seismic drains • With a continuous horizontal silt barrier layer 2 m thick across the model at the elevation of underside of the tunnel • From the 2006 FLAC analysis	• With densification and seismic drains • Without a continuous silt barrier

4.3 Results of FLAC Analyses with No Ground Improvement and Scaled Earthquake Input Motions

Six FLAC runs were carried out in Step 5. The results are shown in Table 4.

Table 4 Results of FLAC analyses with no ground improvement and scaled earthquake input motions

	Model 1	Model 1	Model 1	Model 1	Model 1F	Model 1
FLAC File	S1no	S1no75	S1no625	S1no52	S1noflt52	S1no375
Scaling factor	1.0	0.75	0.625	0.52	0.52	0.375
PGA at the firm ground level [g]	0.25	0.19	0.16	0.13	0.13	0.09
Peak soil lateral (horizontal) excitation at underside of tunnel from FLAC runs [g]	0.214	0.122	0.098	0.08	0.074	0.054
Peak tunnel lateral excitation from FLAC runs [g]	0.102	0.1	0.097	0.089	0.061	0.072
Tunnel uplift [m]	0.29	0.153	0.14	0.075	0.051	0.007
Tunnel lateral disp [m]	1.13	0.75	0.57	0.33	0.013	0.02
Notes -No ground improvement -Without a continuous silt barrier						

It is noted that the FLAC models without a continuous silt barrier was used in Step 5 because this model predicts worse tunnel uplift and lateral displacement.

Based on the target lateral displacement of 0.3 m and tunnel uplift of 0.09 m described in Step 6, it is estimated that if a lower intensity earthquake motion does not exceed PGA of 0.13 g, the tunnel will undergo acceptable performance. Under the same PGA of 0.13 g, the model with a flat riverbed (Model 1F) predicted very small tunnel uplift and lateral displacement.

5 Results of SHAKE2000 Analyses

In Step 7, the Cyclic Resistance Ratios (CRR) of the in-situ soils were calculated using SHAKE2000 with a soil profile based on CPT06-01 and CPT06-07 (Section A of the Trow's 2006 report). The Cyclic Stress Ratios (CSR) induced by the 475 yr return period earthquake were calculated using SHAKE91 during the 2000 seismic strategy and assessment phase. For the current study, the CSRs were scaled using the same scaling factors shown in Step 5 to obtain the reduced CSRs for the reduced seismic events. By comparing the CRRs of the in-situ soils with the CSRs induced by different seismic events, one can determine if soil liquefaction in the in-situ soils will be triggered. The results from Step 7 are shown in Figure 5 for a factor of safety against soil liquefaction triggering of 1.0 and Figure 6 for a factor of safety against soil liquefaction triggering of 1.2. The 1.2 factor of safety was used in the previous SHAKE analyses and liquefaction triggering assessment (Buckland & Taylor Ltd., 2001). In these figures, if the values of CRR or CRR/1.2 are greater than CSR for each level of PGA, soil liquefaction is considered to be not triggered. On the other hand, if the values of CRR or CRR/1.2 are less than CSR for each level of PGA, soil liquefaction is considered to be triggered.

Figure 5 shows that if a factor of safety of 1.0 is considered, the earthquake with a PGA not exceeding 0.16 g will induce limited amount of soil liquefaction that leads to limited soil lateral spreading and tunnel uplift.

Figure 6 shows that if a factor of safety of 1.2 is considered the earthquake with a PGA not exceeding 0.13 g will induce limited soil liquefaction. Limited liquefaction should correspond to limited soil lateral spreading and limited tunnel movements.

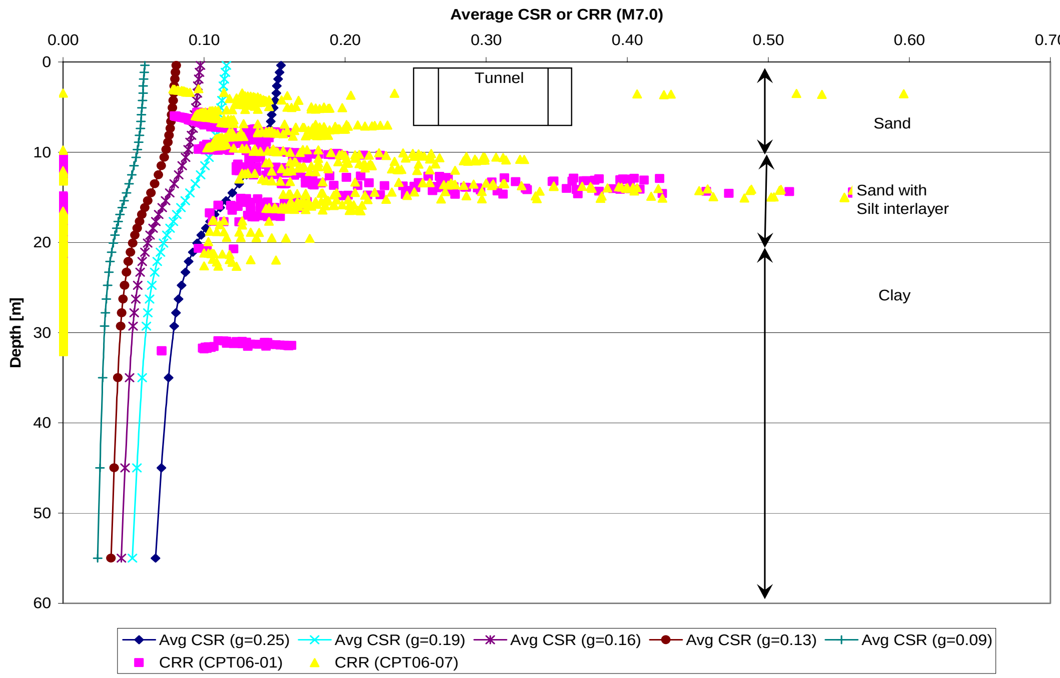

Figure 5 Average Cyclic Stress Ratios from Scaled Earthquake Motions versus CPT Inferred Cyclic Resistance with Factor of Safety of 1.0

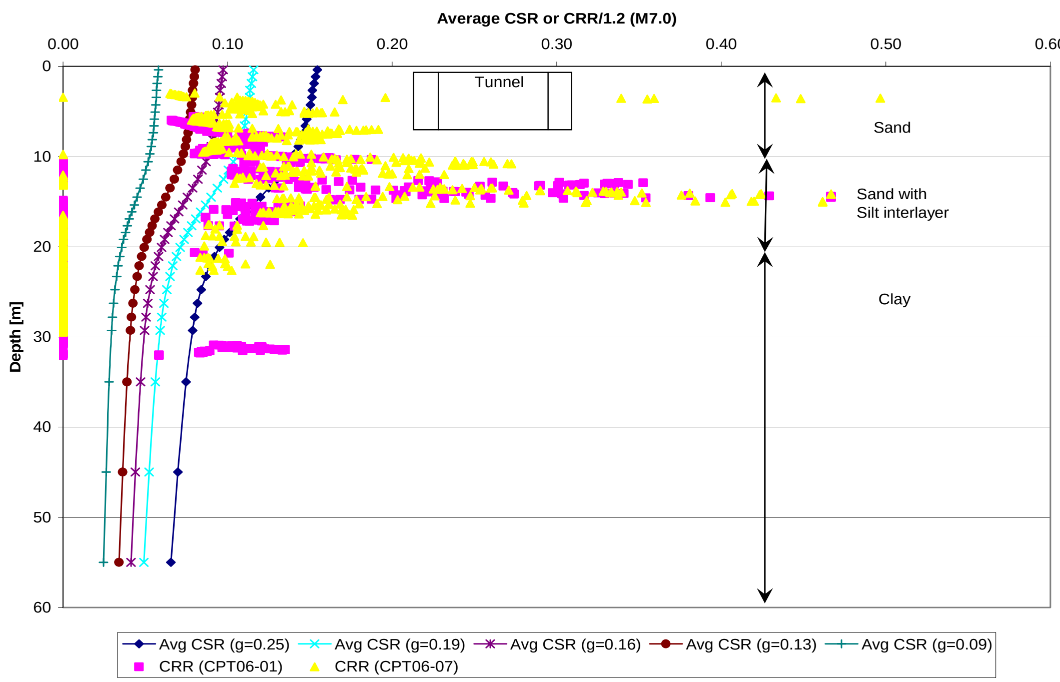

Figure 6 Average Cyclic Stress Ratios from Scaled Earthquake Motions versus CPT Inferred Cyclic Resistance with Factor of Safety of 1.2

6 Conclusions

The tunnel seismic performance criteria specified as life safety retrofit with no collapse of the tunnel structure was achieved by combined 2006 ground densification and seismic drain design and 2002 structural seismic retrofit design along with emergency pump system update inside the now completed structurally retrofitted tunnel. To meet the same seismic performance criteria without ground improvement scheme in place, a lower level of earthquake shaking was assessed in this study. The assessment was based on matching the lateral and upward tunnel displacements predicted under reduced levels of earthquake shaking without any ground improvement with those predicted under the 475 yr return period design earthquakes with the proposed 2006 ground densification and seismic drains. In both scenarios, the target tunnel displacements are 0.3 m laterally and 0.09 m upward.

The performance based FLAC analysis simultaneously assesses excess pore water pressure increase during seismic events, soil liquefaction triggering, associated soil lateral spreading and tunnel movements (i.e. tunnel performance).

The SHAKE2000 analysis assesses soil liquefaction triggering and gives no information on the consequences of liquefaction. However, previous work and past earthquake case histories indicate that ground displacements are much smaller if liquefaction is not triggered.

In this study, both the performance based dynamic FLAC analysis approach and the conventional liquefaction triggering assessment approach was used, to assess tunnel performance at different levels of seismic events when no ground improvement was considered.

Based on the results of our assessment, the tunnel under a reduced level of earthquake shaking between horizontal PGAs of 0.13 g and 0.16 g at the firm ground level with no ground improvement is predicted to perform similarly to the tunnel under the 475 yr return period design earthquake with the proposed ground improvement in 2006. As the SHAKE analyses performed during the 2001 assessment phase showed a PGA of about 0.19 g at the ground surface (riverbed) level after the wave propagation through 1D soil columns for the 475 yr return period design earthquake with PGA of 0.25 g at the firm ground level, a ratio of 0.76

(=0.19/0.25) was calculated between the firm ground level and the ground surface level. Using the same ratio, PGAs at the ground surface (riverbed) level for the predicted smaller earthquakes with 0.13 g to 0.16 g at the firm ground level are estimated to be about 0.1 g to 0.12 g.

The return periods at these levels of earthquake shaking, with PGAs between 0.13 g and 0.16 g, would be approximately 175 to 240 years using the Log-Log correlation between PGA and return period or approximately 150 to 200 years using the semi-Log correlation between PGA and return period.

7 References

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