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SEISMIC SOIL-STRUCTURE INTERACTION ANALYSES AND COMPUTATIONAL SIMULATION OF TUNNELS IN SOFT GROUND SUBJECT TO CASCADIA SUBDUCTION ZONE EARTHQUAKE

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Abstract

The main objective of this paper is to describe the framework and main assumptions for the nonlinear dynamic analysis and design of a tunnel lining in a cohesionless sand and gravel soil. The tunnel is subjected to heavy earthquake excitation in the Cascadia subduction zone. In order to study the influence of the design parameters, Finite Element and Finite Difference Methods (FEM and FDM) and Soil-Structure Interaction (SSI) analyses were used. Also included in this paper is the selection of the lining shape, using a variety of assumptions, including a zone of improved material due to the use of umbrella pipes above the tunnel crown and the effects of the excavation sequence. This study is based on simulated ground motions and other parameters from various earthquake records and highlights the aspects of SSI analyses leading to the support required to accommodate the excavation sequence. This approach provides a practical method for the design and construction of tunnel linings in cohesionless soft ground. From a case study, a feature, interpreted as an unexpectedly deep moraine, was encountered while driving a hard rock tunnel using drill and blast techniques. The depth of the tunnel crown was 60 m below ground surface. This moraine is dense sand with rounded boulders, cobbles and gravel. In order to deal with the effects associated with this cohesionless moraine material, an umbrella pipe system with ports for grouting was driven in order to provide the cover required to install the lattice girders and shotcrete required as initial support to provide a safe and stable opening during the construction. The initial support was designed to support the static loads during the construction. The permanent support, incorporated the temporary support, consisted of additional shotcrete and rebar, and was designed to perform satisfactory for the earthquake of Annual Exceedence Frequency (AEF) of 1/2,475 year. Grouted hollow bar anchors were used in the sidewalls. Four earthquake time histories, including a subduction earthquake, were used to check the adequacy of the design. For construction control various types of geotechnical instruments such as convergence arrays and MPBX (Multi Purpose Borehole Extensometers) should be installed and monitored to check the design assumptions for deformations.

Keywords: Soil Structure Interaction; Finite Difference Method; Drill and Blast Technique; Nonlinear Dynamic Analysis; Earthquake Excitation; Soft Ground Tunnel Lining

1 Introduction

An access to the Underground Powerhouse was developed through the Access Tunnel (AT) in a sample project as shown in Figure 1. The AT is a 9x6 meter section tunnel with vertical side walls and an elliptical crown. The length of this access tunnel is about 400 m. This tunnel connects the surface to the service bay of the underground powerhouse. This tunnel also provides emergency personnel egress from the powerhouse in a segregated manway.

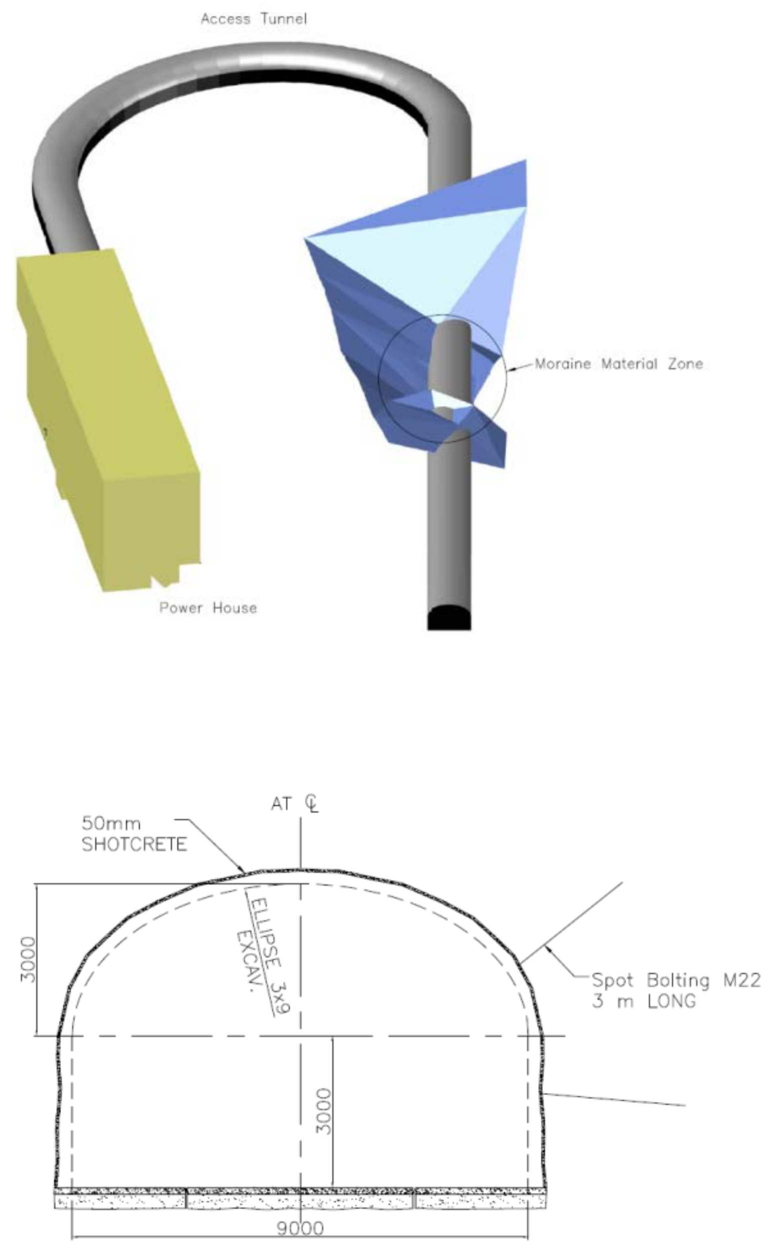


Figure 1 – Access Tunnel Layout and Typical Section in Hard Rock

Regionally the project site and surrounding area are underlain by volcanic basaltic rock crossed by several geological faults. The rock mass was relatively good quality, strong basalt with two sub-vertical joint sets and one sub-horizontal joint set. In order to design the excavation and support for the AT, a pattern of rock bolts were installed on the crown and Fiber Reinforced Shotcrete (FRS) was placed in the crown and side walls.

The excavation of the AT unexpectedly encountered a zone of moraine accompanied by a heavy inflow of ground water. Additional probe drilling inside the tunnel and from the surface indicated that this feature extends at least 50 m ahead of the current position of the tunnel face. Ground water inflows into the tunnel reduced over time, indicating that drained conditions were to be encountered when construction proceeded. A time consuming and detailed geologic and geotechnical investigation was carried out and, as a result, the zone of cobbles and boulders was assumed to exist to the full depth of overburden above the tunnel (i.e. approximately 60 m), over the full face of the tunnel, to the north and south as well as 20 meters below the invert. This condition was materially different than what was foreseen based on the information provided. To continue construction of the tunnel, the excavation method was changed from drill and blast to sequential excavation. One of the main challenges was to describe the framework and main assumptions for the nonlinear dynamic analysis and design of the tunnel lining in the encountered cohesionless moraine. The tunnel is subjected to heavy earthquake excitation, as the project is located in the Cascadia subduction zone in the North American west coast area.

2 Background

The AT starts at the portal and proceeds to the powerhouse service bay (a difference in elevation of 30.5 m). The average ground elevation is 115 m. Based on the average rock conditions observed in the available boreholes, the tunnel would pass through mainly good quality rock. A relatively small area of the AT passes through poor rock with faulted and/or sheared zones. The shape of the AT excavated section was a D shaped section with a width of 9 m, a height of 6 m and a length of about 400 m. The crown forms an ellipse with a width to rise ratio of 3:1. The AT also incorporate side access tunnels to a gate gallery, an adit connected to the power tunnel and a vehicle turnaround.

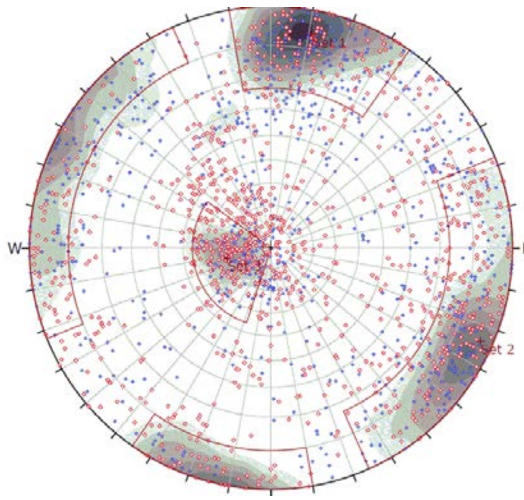
The AT in the rock was excavated by driving a smaller pilot tunnel which was then “slashed” to the final dimension over the first 15 meters. Thereafter the tunnel was excavated full face. Approximately 80 m from the entrance of the tunnel, the excavation encountered the cohesionless moraine material.

3 Site Geology

Regionally, the site and surrounding area are underline by basaltic volcanic rocks that are late Triassic in geological age. This formation is comprised of three sub units – a lower unit of pillowed basalt, a middle unit of massive basalt with amygdales, and an upper unit interlayered with sedimentary rocks. Most of the basalts in the study area have undergone low grade regional metamorphism. A variety of original scale geological faults have been mapped in the region, but none appeared to cross the study area. Most of these faults are to the south and west of the study area and are comprised of NW-SE striking faults, which are interpreted as having normal (dip-slip) displacements. A thrust fault was also indicated to the southwest. Minor faulting and sheared rock were anticipated along the AT alignment.

Basalt Mafic was the only lithology type encountered in the area. The bedrock along the tunnel alignment mostly consisted of basalt that has been subjected to low grade metamorphism, but was generally fresh. Minor alteration (e.g. carbonate, epidote, chlorite) was observed. The altered rock, where present, generally exhibited lower strength and slightly reduced rock mass quality values and exhibited lower strength.

No faults were anticipated to intersect the AT alignment. Based on a geological investigation, there are three major joint sets in the area of the project, two sub-vertical and one sub-horizontal. Figure 2 shows the orientation and characteristics of these three joint sets.



Symbol		Quantity
	3	803
	4	1052

Color	Density Concentrations
	< 1.50
	1.50 - 2.00
	2.00 - 2.50
	2.50 - 3.00
	3.00 - 3.50
	3.50 - 4.00
	4.00 - 4.50
	4.50 - 5.00
	5.00 - 5.50
	5.50 <
Maximum Density	5.25%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1w		84	191	Set 1
2w		87	294	Set 2
3w		21	77	Set 3

Plot Mode	Pole Vectors
Vector Count	1855 (1855 Entries)
Terzaghi Weighting	Minimum Bias Angle 15°
Hemisphere	Lower
Projection	Equal Angle

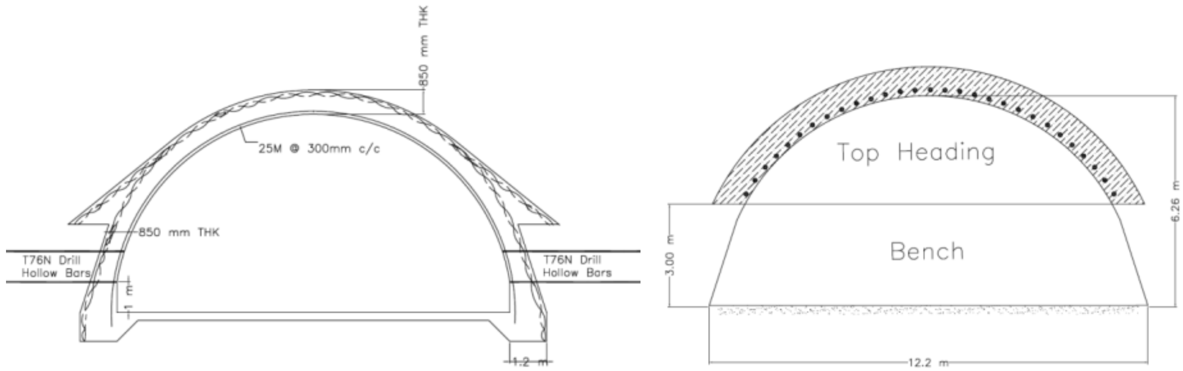
Figure 2 – Stereonets, Joint Sets and Join Orientation at the AT alignment

4 Excavation Methodology

At approximately 80 m of what was excavated from the entrance of the tunnel, a void was encountered with a diameter of approximately 2 m at the 10 o'clock position of the tunnel showing a sand and cobbled filled zone. In order to tunnel through this moraine, additional rock supports such as lattice girders and shotecrete were used. A sequential excavation method was proposed, the excavation methods were radically changed, as shown in Figure 3. The design shape was changed to a semi-circle with an excavation sequence of a top heading followed by the bench/invert. A combination of ground support alternatives were developed and coordinated between designers, subcontractors, and material suppliers.

5 Application of Umbrella Pipes

In order to continue constructing the tunnel with the semi-circle excavation shape and the revised sequence, the ground support elements included a canopy tube arch as shown in Figure 3. The chosen canopy tubes were DSI AT-139 pipes, fully grouted along the entire length, spaced at 400 mm center to center, length 12 m, installed after every 4 m of excavation along the tunnel axis. Lattice girders were placed beneath the canopy tubes and encased in shotecrete.



(a) Initial and Permanent Support

(b) Excavation Sequences

Figure 3 – Excavation Section and Permanent Liner

6 Initial FRS Lining

As an initial support, FRS (Polyolefin fiber reinforced wet-mix shotcrete) was applied with a total thickness of 350 mm including 75 mm shotcrete cover on both faces of the tunnel excavation. The shotcrete was reinforced with CP130/8/11 steel lattice girders spaced 1 m center to center along the tunnel axis and welded wire fabric (152 × 152 MW 13.3 ×13.3 - 152 mm squares, cross-sectional area of wires 13.3 mm^2) between lattice girders. Side wall dowels and various temporary shotcrete facings were incorporated. Details and sequencing of the excavation and ground support system are shown in Figure 3.

7 Numerical analysis

Numerical analyses were carried out to design the support for the moraine zone both during and after excavation and during the design earthquake event as well. FDM software (FLAC) was used to model the field stresses, excavation sequences and stress displacement analyses of the coupled support and rock mass interaction. The in-situ stress prior to excavation was defined by gravity and the ratio of the horizontal stress over the vertical stress was assumed to be K=0.4. This is a typical value for a soil with Poisson ratio of the order of 0.3.
This paper also discusses the application of the substructure method using FEM software (Sap2000) for verification of the analysis for the permanent support.

8 Material Properties

For of tunnel which was located in the moraine material, the properties for the rock masses, the moraine, the section and the shotcrete were assumed as shown in Table 1.

Type	Unit Weight [kN/m3]	Ground UCS [MPa]	Deformation Modulus [GPa]	Poisson's Ratio	Cohesion [MPa]	Friction Angle [Degree]	Bulk Modulus K[GPa]	Shear Modulus G[GPa]	Tensile Strength (MPa)
Rock Mass	29.50	26.00	40.00	0.29	2.00	65.00	31.75	15.50	0.10
Boulder and Cobble Filled Void	21.00	-	0.45	0.30	0.00	41.00	0.38	0.17	0.00
Shotcrete	23.50	40.00	28.00	0.20	-	-	15.56	11.67	3.70

Table 1: Material Properties Used for Numerical Analyses

9 Arch Effect Estimation using FLAC

In order to estimate the stresses and displacement of the boulder filled zone and also to calculate ground arch action, the excavation without any support installation was modeled in FLAC. This model shows the depth of plastic zone above the tunnel, see Figure 4. The ground arch action depth was calculated as 24 meters. The ground cover above the tunnel crown was assumed to be 60m. A comparison of this result was carried out with Terzaghi's empirical arch load method [1]. The arch action depth value as predicted using FLAC is 40% higher than Terzaghi's arch load.

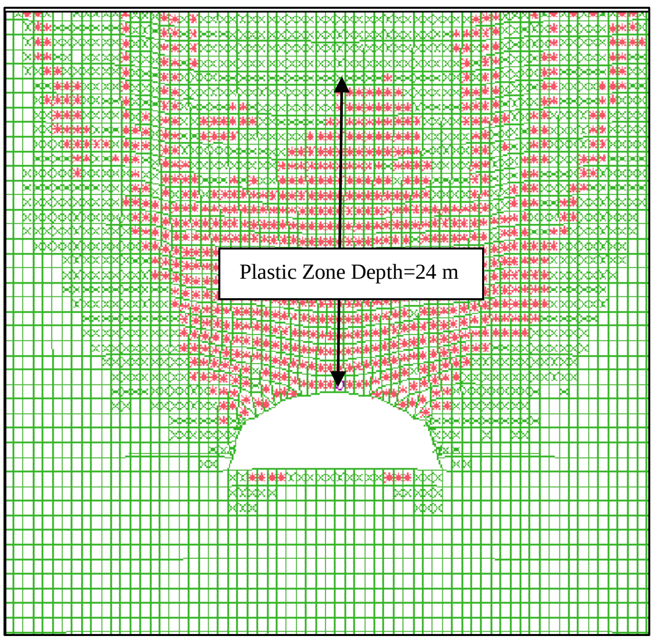


Figure 4 – Plastic Zone above the AT, as predicted using FLAC

10 Pipe Umbrella

A zone of improved material above the tunnel crown was used to simulate the pipe umbrella support, using the method prepared by Hoek (2001) [2]. The improved material representing the pipe umbrella support layer consists of grout-filled steel pipes and the moraine material. The layer was defined as an arch above the excavated tunnel face, see Figure 3. Based on the cross sectional area, the material properties of the improved layer were estimated by a weighted average of the strength and the deformation properties of the components. A formula of the assumed weighting of the three components is represented in Equation 1.

$$\text{Improved Layer} = (\text{soil} \times 0.8) + (\text{steel pipe} \times 0.01) + (\text{concrete} \times 0.19) \quad (1)$$

To keep the model simple, an arch of 1.0 m width, was assumed to have the improved strength as illustrated in Figure 3. The moraine, steel pipes and grouted area are given a weighting of the improved layer of 80%, 1% and 19%, respectively.

During construction the grout did not penetrate through the entire area surrounding the pipes, so the advantage of the improved zone was considered only for initial support and was not taken into account for the design of the permanent liner.

11 Ground Reaction and Initial Support

The method introduced by Hoek et al. 1995 [3] and Brady & Brown 2006 [4] was used for estimating the deformations of a circular equivalent section of the AT in the moraine with the pipe umbrella and lattice girder support system. The ground reaction curve shows an approximate convergence of 0.17 % at the face of tunnel. This value yields approximately 20 mm of deformation at the face of the tunnel. The Factor of Safety is calculated as 1.7 for the initial support system with no earthquake loading effects.

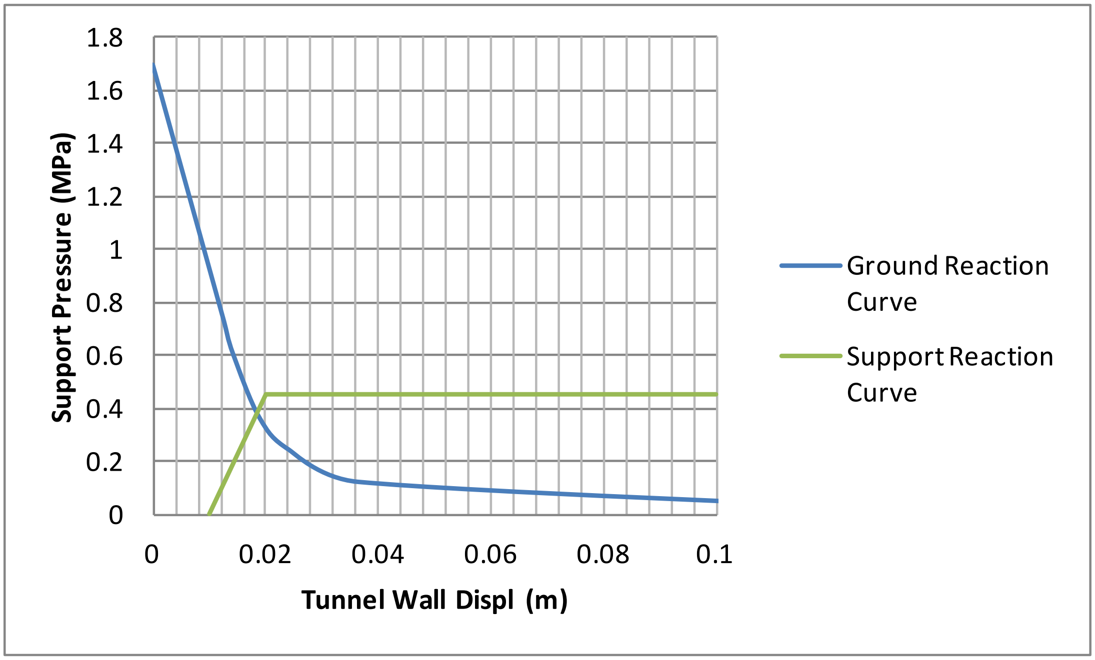


Figure 5 – Ground and Support Interaction Curve for Equivalent Section of the AT

12 Permanent Support

The stability analysis of the tunnel for the permanent condition needs to include the effects of the earthquake event. Time history analyses were carried out to estimate the tunnel stability and the required liner strength. The design of the permanent liner assures to avoid collapses due to excessive deformations or failure during and after a seismic event.

13 Earthquake Ground Motion

A total of four (4) time histories from Pacific Earthquake Engineering Research (PEER-NGA) and UBC (University of British Columbia) databases in similar tectonic regions were selected for the earthquake analyses. The magnitude range is 6 to 9, and a rupture distance less than 100 km. The estimated or measured time-averaged shear-wave velocities in the top 30 m at the recording sites (Vs30) were above 600 m/sec. Earthquake records were selected based on their contributions to the hazard from active crustal and subduction interface earthquakes. The duration of strong shaking, earthquake magnitude, faulting style, site class and Vs_{30} were used as selection criteria to identify suitable candidate records. The selected earthquakes and records used are as listed in Table 2.

Event Name	Date	Source	Magnitude
Tabas, Iran	1978	Active Crustal	M7.4
Tohoko, Japan	2011	Subduction Interface	M 9.0
Kobe, Japan	1995	Active Crustal	M6.9
San Fernando, Lake Huges#4	1971	Active Crustal	M6.5

Table 2: Selected Earthquake for Numerical Time History Analyses

In order to reflect the computed hazard for the site, the earthquake time-history records were selected from recording stations having Vs_{30} of no less than 600 m/sec, having spectral shapes similar to the design spectrum and included near-field records.

The geometric mean of the horizontal components of the selected earthquake time-history records was linearly scaled to NBCC Post-Disaster Earthquake [5] PGA=0.42g (0.28g x 1.5) using time-domain techniques and spectrally matched to the horizontal design spectrum. The PGA was also decreased to 0.3g (0.42g*0.7), according to Power et al. 1996 and M.A. Hashasha [6], considering the AT depth is greater than 50 meters. This recommendation was adopted by Hashsha et. al, ITA [6] and FHWA-NHI-1-034 [7].

As shown in Table 2, the Tohoko earthquake was a major shaking event caused by a slip between subducting tectonic plates. This regime is the most intense earthquake that might happen during the AT life period. The acceleration record of this earthquake is shown in Figure 6.

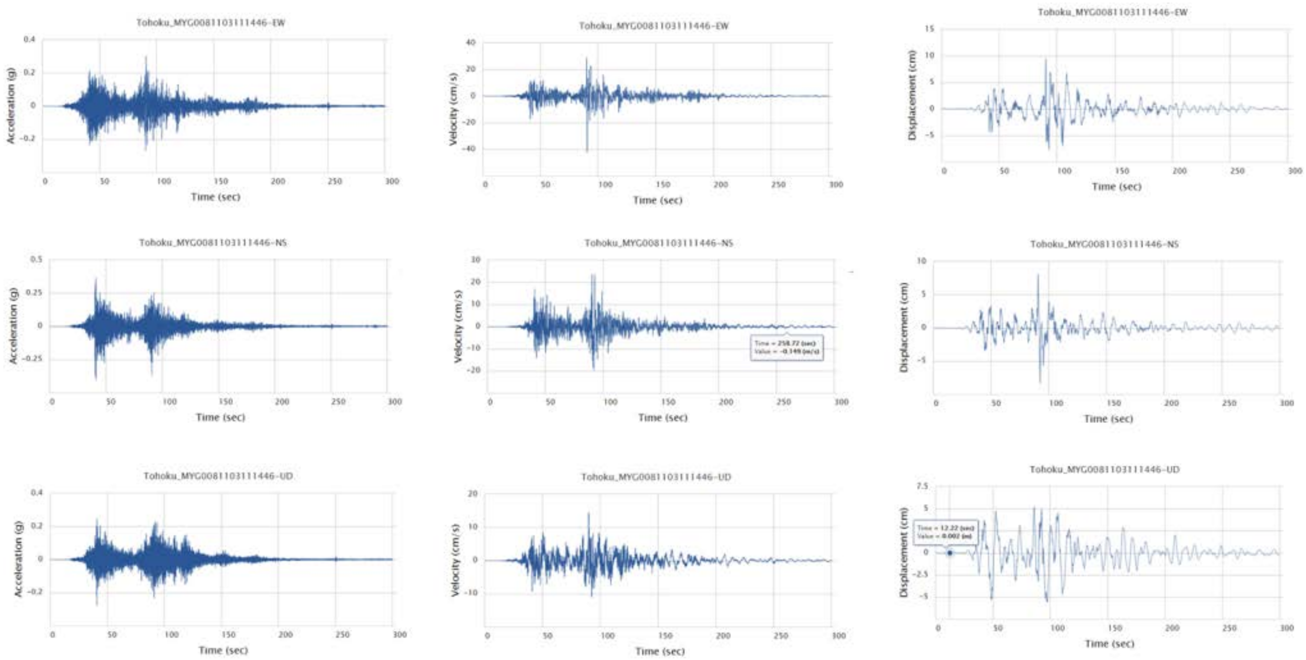


Figure 6 – Tohoko, Japan, 2011, M9.0, Earthquake Records

14 Subgrade Reaction to be used for Soil/Liner Interface

In the Numerical model, the interface stiffness between the liner and surrounding soil of the tunnel, as recommend in the FLAC manual [8], was set to ten times the equivalent stiffness of the stiffest neighboring zone, see Equation 2.

$$K_n = K_s = 10\left(\frac{K+\frac{4}{3}G}{\Delta Z_{min}}\right) \tag{2}$$

The interfaces were assumed to have no slip or opening. However elastic displacement was allowed according to the given stiffness. The interface stiffness for assumed minimum $\Delta Z = 0.5$ m yields $K_n = K_s = 21$ GPa/m

15 Shear Modulus and Damping Ratio for Dynamic Analysis

The model used Rayleigh damping in place of hysteretic damping. The parameters for shear modulus and damping ratio were selected to correspond to an equivalent uniform strain of 0.003 according to the average elastic deformation at the AT. The damping ratio of 7% and 30% of shear modulus reduction were estimated according to Kramer 1996 [9].

16 Time History Analyses and Results

The 2D nonlinear FLAC analysis of ground deformation shows that the resultant deformation and the level of stresses are within the acceptable limits. Structural analysis of the permanent liner shows that the reinforced concrete liner (Thickness=850 mm) and the associated rockbolts in the side walls have strength adequate to withstand the earthquake of Annual Exceedence Frequency (AEF) of 1/2,457. Figures 7 and 8 present the deformation results of the tunnel crown during and after the four (4) earthquake events. Figure 9 shows the shear failure index in the AT after the Tohoko strong ground motion. As can be seen, the plastic zone would extend up to the ground surface after the Tohoko Megathrust Earthquake. This is most likely due to the fact that Tohoko earthquake has a large duration and two major peaks.

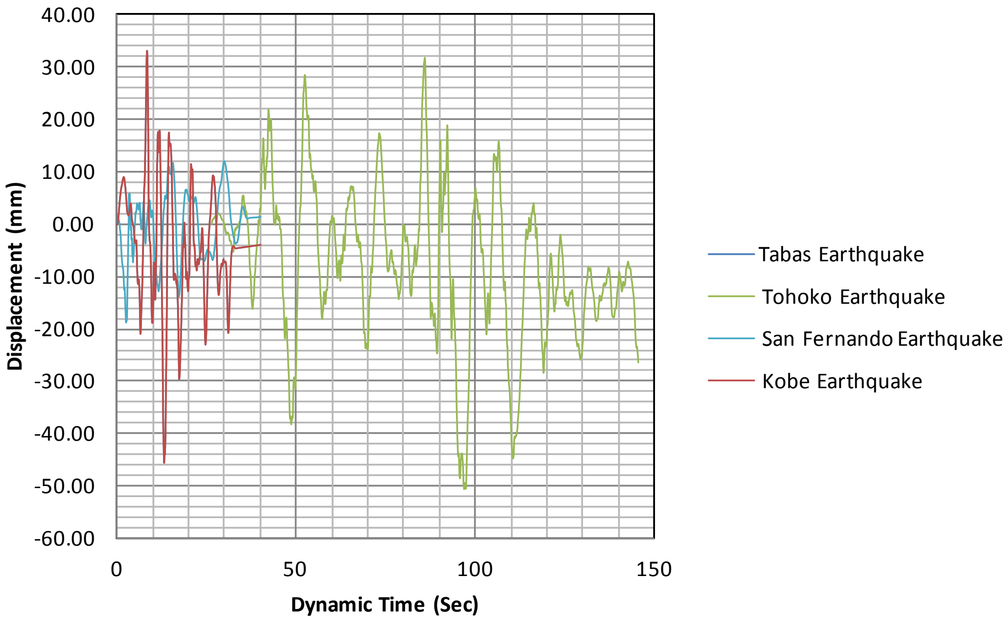


Figure 7 – Displacement Time History Analysis of the AT Crown scaled to PGA=0.42g

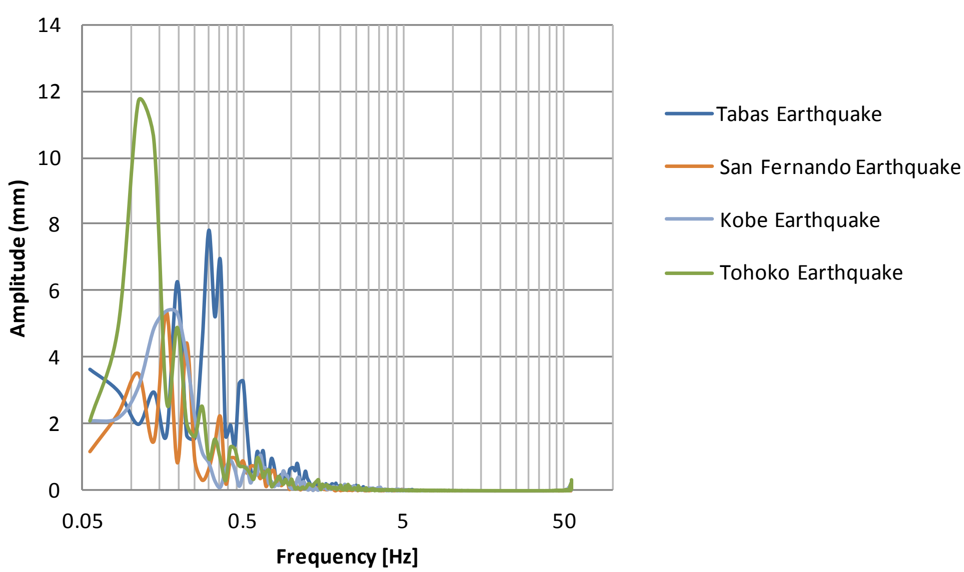


Figure 8 – Fast Fourier Transformation of Displacement Responses of the Applied Earthquakes

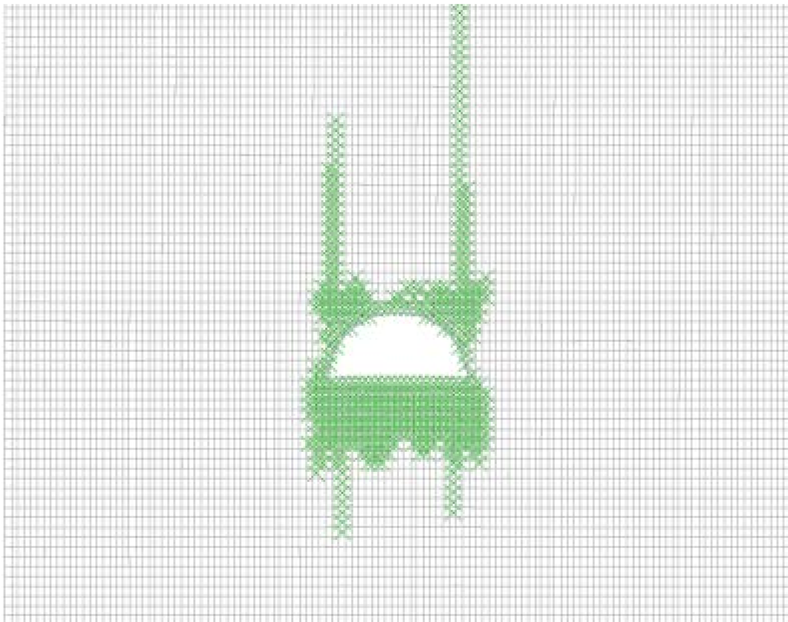


Figure 9 – Plastic Zone Extension after Tohoko Subduction Earthquake

17 FEM Model

Seismic stability assessment of the AT was also compared with those of time history analyses made in the earlier section, using Sap2000 software (FEM). A ground load assessment was performed to quantify the external loads on the tunnel lining and the ground support components. The full overburden weight, assuming the ground failure during a seismic event was applied and the structural capacity of the shotcrete lining was calculated based on the reinforced concrete strength design methodology described in EM110-2-2104 [10]. The radial and tangential spring stiffnes, expressed in units of force/displacement per square meter were estimated according to EM 1110-2-2901 [11].

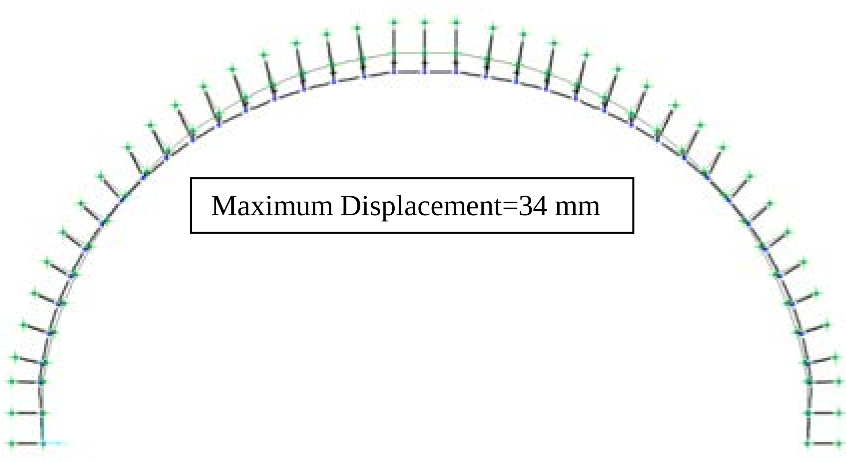


Figure 10 – Sap2000 Substructure Model

The results of the Finite Element Model illustrate that the performance of the tunnel as it was estimated through the time history analyses, is acceptable during the earthquake. As shown in Figure 10, maximum vertical displacement in the crown is calculated as 34 mm. Design of the liner is obtained from the method described in EM 1110-2-2104 [10].

18 Conclusion

When subjected to a moderate or major earthquake, the AT undergoes a large deformation which without permanent lining would almost certainly lead to failure of the tunnel. Nonlinear dynamic FDM and FEM analyses showed the deformations that would be caused in the surrounded moraine material and structural interaction in the permanent support. The tunnel has been assessed for identified earthquakes with a mean Annual Exceedence Frequency (AEF) of 1/2,475 (a PGA of 0.42g). The tunnel was stabilized using 850 mm of reinforced concrete lining with a semi-circular shape. The invert of the tunnel would crack following the high seismic event, however theses cracks in the invert stabilize after shaking and thus are not considered failure of the lining and the tunnel overall stability, as these minor damages can be easily repaired.

19 Acknowledgements

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20 Copyrights

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