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SEISMIC DESIGN AND ANALYSIS FOR THE CUT-AND-COVER TUNNELS OF THE EVERGREEN LINE RAPID TRANSIT PROJECT

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ABSTRACT: Construction of the Evergreen Line Rapid Transit Project is currently underway in Vancouver, BC. There are two sections of cut-and-cover tunnel structures within the project. The major seismic load resisting system in the cut-and-cover tunnels is the moment connections between slabs and walls. Member capacity and behavior beyond elastic range were studied by conducting pushover analysis to more accurately detail the design for structural capacity. This paper discusses the structural design and analysis approaches of the cut-and-cover tunnels to satisfy the seismic performance requirements for different levels of earthquakes, including 100-year, 975-year return periods and subduction events. Linear and non-linear behavior of the tunnels were considered in racking deformation and pushover analyses. Soil-structure interaction analysis and dynamic earth pressure methods were used to obtain critical racking deformation demand. Design forces calculated from racking deformations were applied to the structure based on both pseudo-triangular pressure and pseudo-concentrated force models. In the case of inelastic behavior, minor hinging is allowed in tunnel walls during a high seismic event. The final design of the cut-and-cover tunnels is based on this documented seismic design strategy, which must go through a unique review process and be approved by a Seismic Review Panel.

1. Introduction

1.1. Scope

The North and South Portals of the Evergreen Line connect the bored tunnel to the elevated guideway (Fig. 1). The portals are made up of a section of cut-and-cover tunnel, open trench and at-grade guideway. The scope of this paper will focus only on the cut-and-cover tunnels at the portals.

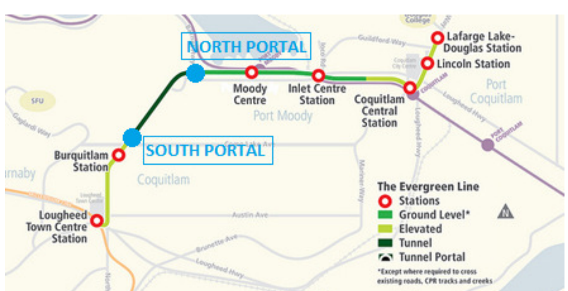

Fig. 1 – Evergreen Line North and South Portals Locations

The cut-and-cover tunnel at the North Portal is a 91-metre long buried reinforced concrete box structure which height varies over 8 m from the entrance of the bored tunnel at a maximum of 7 m to a constant height of 5 m. The tunnel has a constant width and centre wall. The cut-and-cover tunnel at the South Portal has similar cross sectional dimensions, with a length of 100 m. (Fig. 2)

1.2. Site Conditions

The North Portal cut-and-cover tunnel crosses underneath Barnet Highway at the east side of Burnaby Mountain. Several small creeks are located within the vicinity, with Schoolhouse Creek right at the east side of the Portal. Surficial geology around this area includes dense to very dense glacially overridden deposits (Vashon and Pre-Vashon Sediments). Soil layers consist of fill of variable thicknesses, overlaying a thin layer of sandy alluvium and weathered surficial soil, overlying dense to very dense soils. Groundwater levels at this site are within 2 m from ground surface, and hydrostatic pressure was included in the design.

The South Portal cut-and-cover tunnel is located on a relatively flat plain of till, along Clarke Road on the south side of Burnaby Mountain. Surficial geology in the area includes dense to very dense glacially overridden deposits (Vashon and Pre-Vashon Sediments). Soil layers consist of fill of variable thicknesses, overlying a discontinuous layer of post-glacial deposits, overlying dense to very dense soils. Groundwater levels are approximately 1.5 m from ground surface, and hydrostatic pressure was included in the design.

1.3. Seismic Review Panel

The seismic design of the Evergreen Line structures is subject to an independent peer review by the Seismic Peer Review Panel, made up of three Professional Engineers recognized as leading experts in the areas of structural and geotechnical seismic design and analysis. A Seismic Design Strategy Memorandum (SDSM) was prepared for each structure along the Evergreen Line. The documents detail the seismic design approach proposed for each structure, including the assumptions, design strategy, earthquake resisting systems, step-by-step design and analysis methodology, strains and deformations , component and foundation properties, ground motion input, global modal characteristics and seismic demands for the structure. The Seismic Review Panel reviews and comments on the SDSM, and the design can only commence when the SDSM is approved.

2. Seismic Design Approach

2.1. Design Strategy

2.1.1. Earthquake Resisting Systems

The main earthquake resisting system in the cut-and-cover tunnels is by moment frame action from the moment connections between the tunnel roof, base slabs and walls. Resistance to seismic forces is provided by developing moment and shear forces at the joints of the frame and resisting them with structural capacity of the members. The rigidity and strength of the members contributes to the lateral stiffness of the tunnel when it is displaced laterally.

For high earthquake events, minor plastic hinges were allowed in the tunnel exterior walls. All other components were designed as capacity-protected elements which will remain elastic through an earthquake.

2.1.2. Load Paths

The seismic forces encountered by the tunnel will be translated to bending moment and shear forces in the tunnel members, which will be taken by the members' structural capacity and transferred to the surrounding soil. Transverse soil displacement will impose a load on the structure, causing it to deform and transfer the load back to the surrounding soil. Longitudinal ground movement will cause axial and curvature strains in the tunnel, which will be resisted as stress in the tunnel elements and transferred to the surrounding soil.

2.1.3. Geotechnical Design Strategy

Results of liquefaction assessment conducted by the Geotechnical Engineers using the "simplified method" (Youd et al., 2001) showed that liquefaction is unlikely at both portal sites. Estimated factors of safety against liquefaction triggering were found to be greater than 2.

2.2. Codes and Standards

Seismic design if the cut-and-cover tunnels follows the following codes and standards:

- CAN/CSA S6-06, Canadian Highway Bridge Design Code 2006 and BC Supplement to CAN/CSA S6-06
- CAN/CSA A23.3, Design of Concrete Structures

2.3. Seismic Performance Levels

The design of cut-and-cover tunnels shall meet the following seismic performance levels:

- 100-year Return Period Earthquake Event Level – Immediate Use Performance
- 975-year Return Period Earthquake Event Level – Repairable Damage Performance
- Subduction Earthquake Event Level – Repairable Damage Performance

Immediate Use Performance level requires a structure to remain essentially elastic and be available for immediate passenger service. Only minor damage within a Permitted earthquake resisting system is allowed.

For Repairable Performance Level, inelastic response damage of a Permitted earthquake resisting system is limited to the strain range specified in Section 2.5. The structure is restorable to its pre-earthquake condition without replacement of primary structural members. Passenger service interruption for inspection and immediate temporary repair time is limited within one month, and permanent repairs are limited to within three months.

2.4. Seismic Ground Motion

The site-specific ground motion parameters for the seismic design of the cut-and-cover tunnel is given in the Evergreen Line Project Agreement (Province of BC, 2012) and is applicable to the whole alignment. Peak vertical ground acceleration was considered as two-thirds of the peak horizontal ground acceleration.

The following table shows the design seismic ground motion input. These ground motions were also used in liquefaction assessment of the site. Magnitudes for 100-year and 975-year Earthquake Events are based on the mean spectral values of 1 or 2 seconds plus 0.2 units, and values for Subduction Event are based on a Magnitude 8.2 earthquake.

Table 1 – Seismic Ground Motion Parameters [g]

Return Period	PGA	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Magnitude (M)
100-year	0.116	0.224	0.150	0.078	0.039	6.7
975-year	0.343	0.688	0.456	0.230	0.119	6.9
Subduction	0.160	0.370	0.310	0.170	0.077	8.2

2.5. Inelastic Response Limits

Seismic deformation was kept within elastic range for Immediate Use Performance Level. Allowable concrete strain limit of 0.004 and allowable steel strain limit of 0.006 based on unconfined concrete were used for static and pseudo-static loading for Repairable Performance Level, as approved by the Seismic Review Panel.

3. Seismic Analysis

The seismic design of the cut-and-cover tunnels followed the method as described in Hashash et al. (2001) and Wang (1993). The design was based on the Deformation Method that focuses on the displacement/deformation aspects of the ground and structures.

3.1. Material Properties

Concrete used in all tunnel structures are of Class C-1, with minimum concrete strength at 28 days (f'_c) of 35 MPa per CAN/CSA A23.1. Modulus of elastic of concrete (E_c) was calculated according to CAN/CSA A23.3 to be 28165 MPa.

Reinforcing steel is of grade 400W per CAN/CSA G30.18, with minimum yield strength (f_y) of 400 MPa. Modulus of elasticity of steel (E_s) is 200 000 MPa.

The effective moment of inertia of the structure (I_{eff}) for all seismic performance levels was determined based on moment-curvature methods.

3.2. Tunnel Design – Deformation Method

The Deformation Method seismic design and analysis were done according to the following steps, following the de-coupled soil-structure interaction analysis using racking deformation (Hashash et al., 2001; Wang, 1993):

STEP 1: Geotechnical Engineers determined the subsurface conditions and soil properties of the site from field investigations.

STEP 2: Earthquake design parameters were derived based on geotechnical input, which includes peak ground acceleration and velocities for the required Seismic Performance Level for the 100-year, 975-year and Subduction Earthquake Event Levels.

STEP 3: Preliminary design of the structure and initial sizing of members based on static loading conditions.

STEP 4: Free-field shear strains/deformations of the ground at the depth of interest were estimated using one-dimensional site response analysis.

STEP 5: Relative stiffness (flexibility ratio) between the free-field medium and the structure was determined.

STEP 6: Racking coefficient, R, was obtained based on the flexibility ratio.

STEP 7: Actual racking deformation of the structure, $\triangle_s$, was calculated using free-field shear deformation $\triangle_{free\text{-}field}$, which was calculated using one-dimensional analysis, and racking coefficient, R. ($\triangle_s = R^* \triangle_{free\text{-}field}$)

STEP 8: The seismically induced racking deformation, $\triangle$s, was imposed in a simple frame analysis. The Pseudo-Concentrated Force Model for Deep Tunnels gives a more critical moment response at the roof-wall joints, while the Pseudo-Triangular Pressure Distribution Model for Shallow Tunnels provides a more critical evaluation of the moment capacity of a rectangular structure at its bottom joints. For design purposes, both models were employed in the frame analyses.

STEP 9: In addition to the horizontal forces, the loads due to vertical accelerations were also accounted for. Vertical seismic forces exerted on the roof of the tunnel was estimated by multiplying the estimated peak vertical ground acceleration by the backfill mass.

STEP 10: Racking-induced internal member forces and vertical seismic forces were added to the other loading components by using the loading criteria for the project.

STEP 11: If the results from STEP 10 show that the capacity of the structure exceeds the demand for all seismic performance levels, ie. concrete and steel strains in all members are within limits as per Section 2.5, the design is considered satisfactory.

STEP 12: If the results from STEP 10 show that the structure does not have adequate capacity, the structural members' ductility was checked if the structure's flexural strength was exceeded. Redistribution of moments and consideration of plastic hinges is acceptable. If plastic hinges develop the flexibility ratio was re-computed and the analysis restarted at STEP 5.

STEP 13: The structure was redesigned if the strength and ductility requirements, based on strain limits as per Section 2.5, were not met and/or the resulting inelastic deformations exceed allowable levels.

STEP 14: Sizes of the structural members were modified as necessary. The design was complete if the allowable strain criteria were met.

3.2.1. Soil-structure Model

The external forces acting on the tunnel exert shear stresses and normal pressures on the tunnel walls, roof and base slabs. The pseudo-static earthquake forces were superimposed on the existing static state of stresses in the structure members with alternating directions.

The tunnel cross-sections were modelled using beam elements under a two-dimensional plane strain condition. Analyses were performed for two sections of each of the North and South portal tunnels – one section at maximum height and one section at constant height. The dimensions and design of the constant height typical sections are shown in Fig. 2.

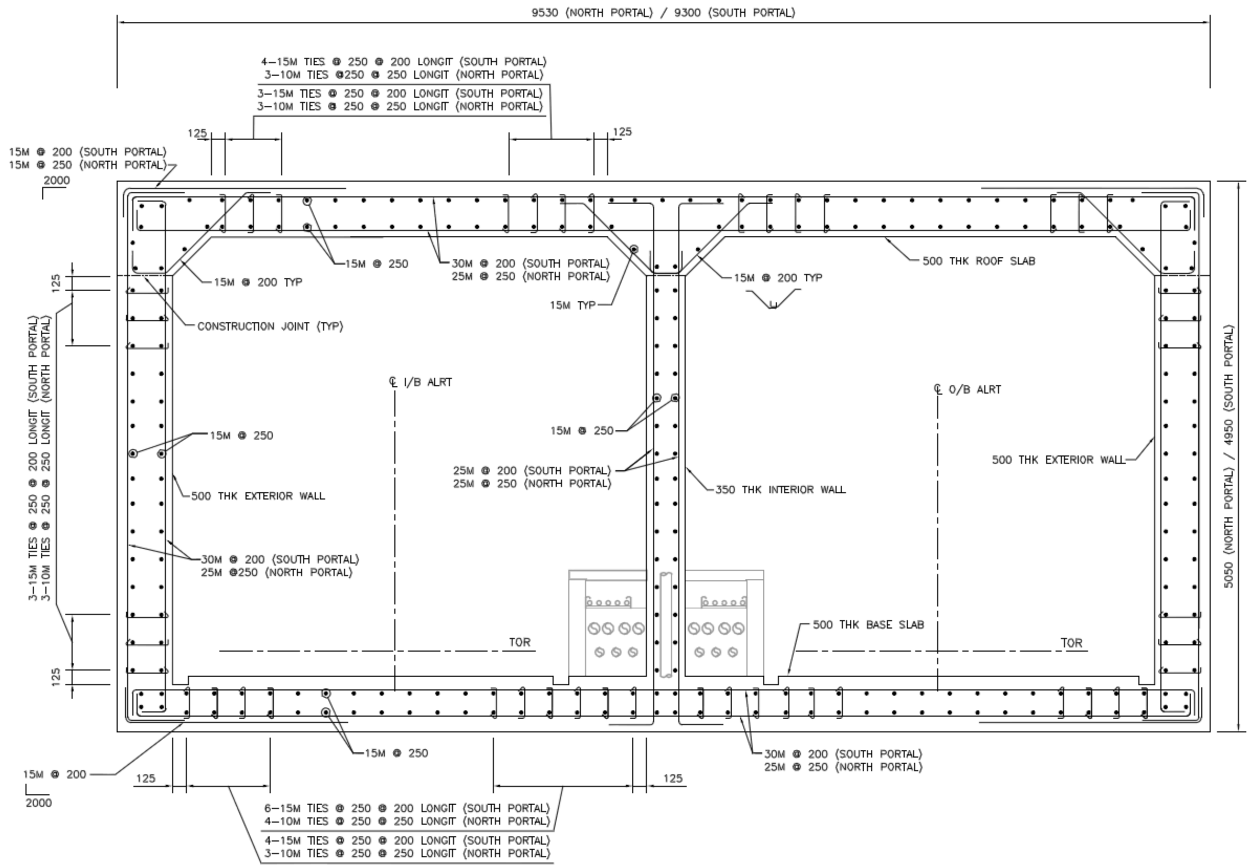

Fig. 2 – Evergreen Line Cut-and-cover Tunnel Section

The following design requirements and assumptions were incorporated into the model:

- Preliminary analysis for the tunnel was carried out by ignoring the stiffness of the tunnel and imposing “free-field” ground deformations on the tunnel.
- Seismic loads due to racking deformations and dynamic earth pressures were included.
- Ground accelerations were accounted for.
- The maximum and minimum load factors for soil pressure were based on CAN/CSA S6-06.
- Structural members were modelled by continuous flexural beam elements of linear elasticity. Structural frames with rigid connections were considered. Rigid links were defined within the geometry of the joints.
- No-slip condition was considered along the soil/structure interface.
- Geometry of the model was defined at the centreline of the physical walls and slabs.
- The stiffness of the members was reduced to account for the cracked state of the structure. Walls and slabs were divided into multiple elements and the effective moment of inertia use in analysis was derived based on bending moment demand/moment-curvature analysis of the particular element.
- In the longitudinal direction, the tunnel structure was assumed to experience the same strains as the ground in the free-field and the presence of structures and disturbance due to excavation were ignored.

Longitudinal strains were not expected to govern the displacement of the tunnel due to the configuration. Analyses were done based on equations recommended by Hashash et al. (2001) and the strain caused by axial and curvature deformation was found to be +/- 0.0006. The axial strain is dominant over the curvature strain, but is still very small compared to the concrete compression strain limit of 0.0035 specified in CAN/CSA A23.3-04. Concrete may crack at this strain in the case of tension, but reinforcing steel will close the cracks at the end of shaking as long as the strain is within the steel yield strain of 0.002.

Shear displacement from S-waves was expected to govern the racking displacement.

3.2.2. Flexibility Ratio

The flexibility ratio for a rectangular tunnel is a measure of the medium relative to that of the tunnel structure. Using the shear strain (or angular distortion) equation for a soil element subjected to simple shear condition and converting the angular distortion into a concentrated force, P, together with the flexural (or racking) stiffness of the structure, the flexibility ratio, F, was obtained as

$$F = \frac{G_{max} W}{S_1 H}$$

where G_{max} = maximum shear modulus of soil = $\rho_m C_m^2$

ρ_m = density of soil
C_m = shear wave velocity
W = tunnel width
H = tunnel height
S_1 = the force required to cause a unit racking deflection of the structure = $1/\Delta_1$
Δ_1 = lateral racking deflection caused by a unit concentrated force

From the results of the flexibility ratio, there are three cases for tunnel response:
F = 1.0: Tunnel should distort the same magnitude as the ground in the free-field
F < 1.0: Tunnel is stiffer than ground and should distort less
F > 1.0: Racking distortion of the tunnel is amplified
F → ∞: Structure has no stiffness and shall have identical deformations to the perforated ground

Δ_1 was obtained from two-dimensional frame analysis. The following table shows the design parameters used to obtain the flexibility ratio.

Table 2 – Obtaining Flexibility Ratios for the Tunnel Sections for 1 m Length of Tunnel

	North Portal		South Portal	
	Const. Height Section	Max. Height Section	Const. Height Section	Max. Height Section
Soil Density, ρ_m [kg/m^3]	2000	2000	2018	2018
Shear wave velocity, C_m [m/s]	250	250	350	350
Max Shear Modulus, G_{max} [kPa]	125000	125000	247200	247200
Tunnel Width, W [m]	9.53	9.85	9.30	9.84
Tunnel Height, H [m]	5.05	7.00	4.95	7.00
Lateral Racking Deformation, Δ_1 [m]	0.0171×10^{-3}	0.0433×10^{-3}	0.0139×10^{-3}	0.0436×10^{-3}
Force to cause 1 m of racking deformation , S_1 [kN]	58480	23095	71788	22931
Flexibility Ratio, F	4.03	7.62	6.41	15.02

3.2.3. Racking Coefficient and Racking Deformation

The results from finite element analysis by Wang (1993) showed that the flexibility ratio has the most significant influence on the distortion of the structure due to racking deformations. The racking ratio, defined as the normalized structure racking distortion with respect to the free-field ground distortion, is

$$R = \frac{\Delta_s}{\Delta_{free-field}} = \frac{\gamma_s}{\gamma_{free-field}}$$

where Δ_s = lateral normalized racking deformation of structure
$\Delta_{free-field}$ = free-field ground distortion
γ_s = angular distortion of structure
$\gamma_{free-field}$ = free-field angular distortion

Fig. 3 was used to determine the racking deformation based on F and R. The figure was plotted using varying moduli of elasticity, tunnel radius and wall thickness based on circular tunnels, and Poisson Ratios for the portals confirmed by the Geotechnical Engineer. The flexibility ratios for circular tunnels are conservative upper bounds of rectangular tunnels (Wang 1993).

$$F = \frac{E_m(1-\nu_l^2)R^3}{6E_l I(1+\nu_m)}$$

where E_m = Modulus of elasticity of medium
ν_m = Poisson's ratio of medium
R = Radius of tunnel lining
E_l = modulus of elasticity of tunnel lining

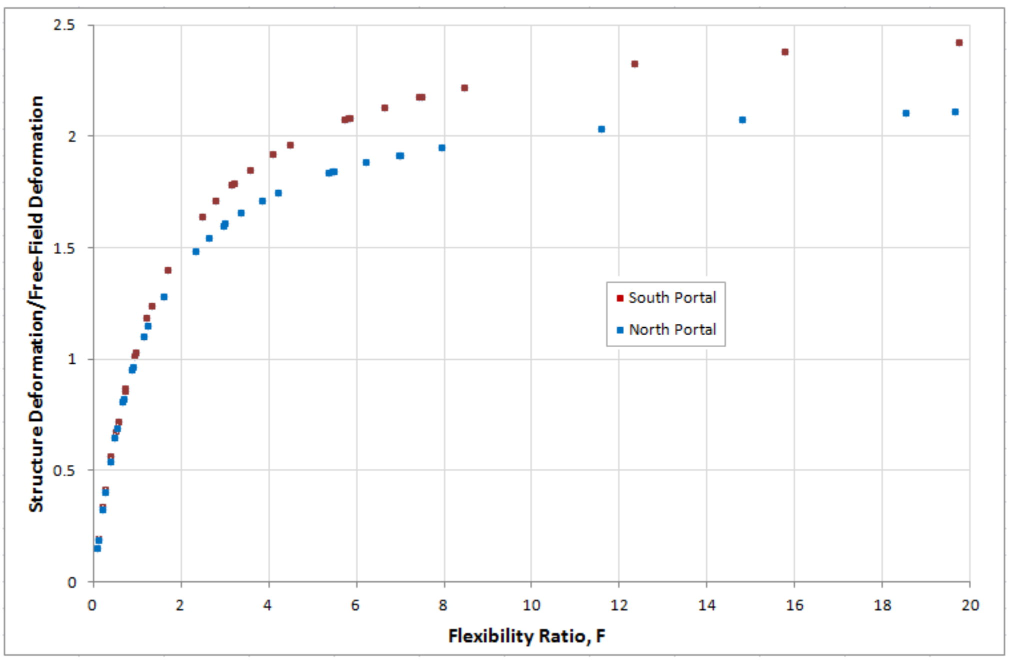

Fig. 3 – Racking Coefficient vs. Flexibility Ratio

The structure deformations due to seismic were approximated by the peak ground velocity (PGV) of the different earthquake event levels. The deformation is calculated as the maximum shear strain (γ_{max}) multiplied by the height of structure, where γ_{max} is obtained by PGV divided by shear wave velocity. The structure deformation was found by using the racking ratios obtained above. The following table shows the PGV, maximum shear stress, free-field deformation and estimated structure deformation for the different earthquake event levels.

$$\gamma_{max} = \frac{PGV}{C_m} \quad where\ C_m = shear\ wave\ velocity[m/s]$$

$$\Delta_{free-field} = \gamma_{max} H \quad where\ H = height\ of\ structure$$

$$\Delta_s = R\Delta_{free-field}$$

Table 3 – Obtaining Structure Deformations for the Tunnel Sections

Earthquake Event Level		North Portal		South Portal	
		Const.Height Section	Max. Height Section	Const. Height Section	Max. Height Section
	R	1.75	1.95	2.10	2.30
100-year	PGV [m/s]	0.09	0.10	0.09	0.09
	γ_s [rad]	0.00036	0.00040	0.00026	0.00026
	$\Delta_{free\text{-}field}$ [mm]	1.82	2.80	1.27	1.80
	Δ_s [mm]	3.19	5.46	2.67	4.14
975-year	PGV [m/s]	0.28	0.28	0.25	0.25
	γ_s [rad]	0.00112	0.00112	0.00071	0.00071
	$\Delta_{free\text{-}field}$ [mm]	5.66	7.84	3.53	5.00
	Δ_s [mm]	9.90	15.29	7.41	11.50
Subduction	PGV [m/s]	0.17	0.18	0.15	0.15
	γ_s [rad]	0.00072	0.00072	0.00043	0.00043
	$\Delta_{free\text{-}field}$ [mm]	3.43	5.04	2.12	3.00
	Δ_s [mm]	6.01	9.83	4.45	6.90

3.2.4. Frame Analysis Models

The racking deformation, Δ_s, was imposed onto the tunnel in a simple frame analysis. The governing case of the following two models (Fig. 4) was used in the analysis.

Pseudo-Triangular Pressure Distribution Model

The shear force developed at the soil/roof interface will decrease as the soil cover decreases for shallow tunnels. The predominant external force which causes the structure to rack may gradually shift from the shear force at the soil/roof interface to the normal earth pressures at the side of the tunnel walls. A triangularly distributed force model on one side of the tunnel is used in the analysis.

Pseudo-Concentrated Force Model

The triangular pressure model described above yield satisfactory results in the bottom slab-wall joints, but generally it tends to underestimate the bending moment response in the upper roof-wall joints for a given racking deformation. The concentrated force model is also used in the analysis of the transition tunnel to model the forces in the upper joints more accurately.

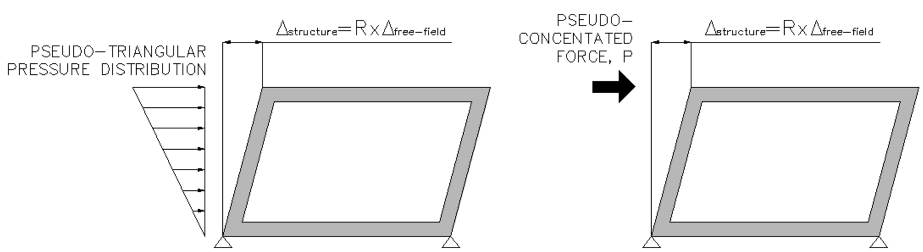

Fig. 4 – Frame Analysis Models

In addition to the racking deformations, loads due to vertical accelerations and for longitudinal strain resulting from frictional soil drag were also accounted for. Vertical seismic forces exerted on the roof was estimated by multiplying the estimated peak vertical ground acceleration by the backfill mass.

3.2.5. Pushover Analysis

The pushover analysis models assumed pinned supports at three locations under each wall. Pushover analysis was done with one meter length of the tunnel. Loading defined in the model include dead load (both structural members and soil backfill), lateral at-rest soil pressure applied on the exterior faces of both exterior tunnel walls (including hydrostatic pressures over the full height) as trapezoidal distributed pressures based on soil covers and water levels, and vertical acceleration. At-rest soil pressure is considered due to the tunnel being restrained by soil on both sides and deformation will be too minimal to mobilize passive pressure.

The base shear verses displacement plots from the pushover analysis are shown in Fig. 5. It is noted that the pseudo-triangular pressure distribution model has a greater base reaction compared to the pseudo-concentrated load model at the same displacement. In addition, the heavier reinforced sections show higher base reactions, which is expected due to the sections' higher plastic moments.

It is observed that the structures are slightly past the elastic range for the displacements caused by racking in the 975-year earthquake. The first and main hinge is formed at the top of the exterior wall on the side opposite the load application (Fig. 6). The bending moment in the hinge was obtained from the moment diagram corresponding to the calculated racking deformation. Table 4 shows the moment demands at these displacements, and the corresponding strains obtained from moment-curvature relationships.

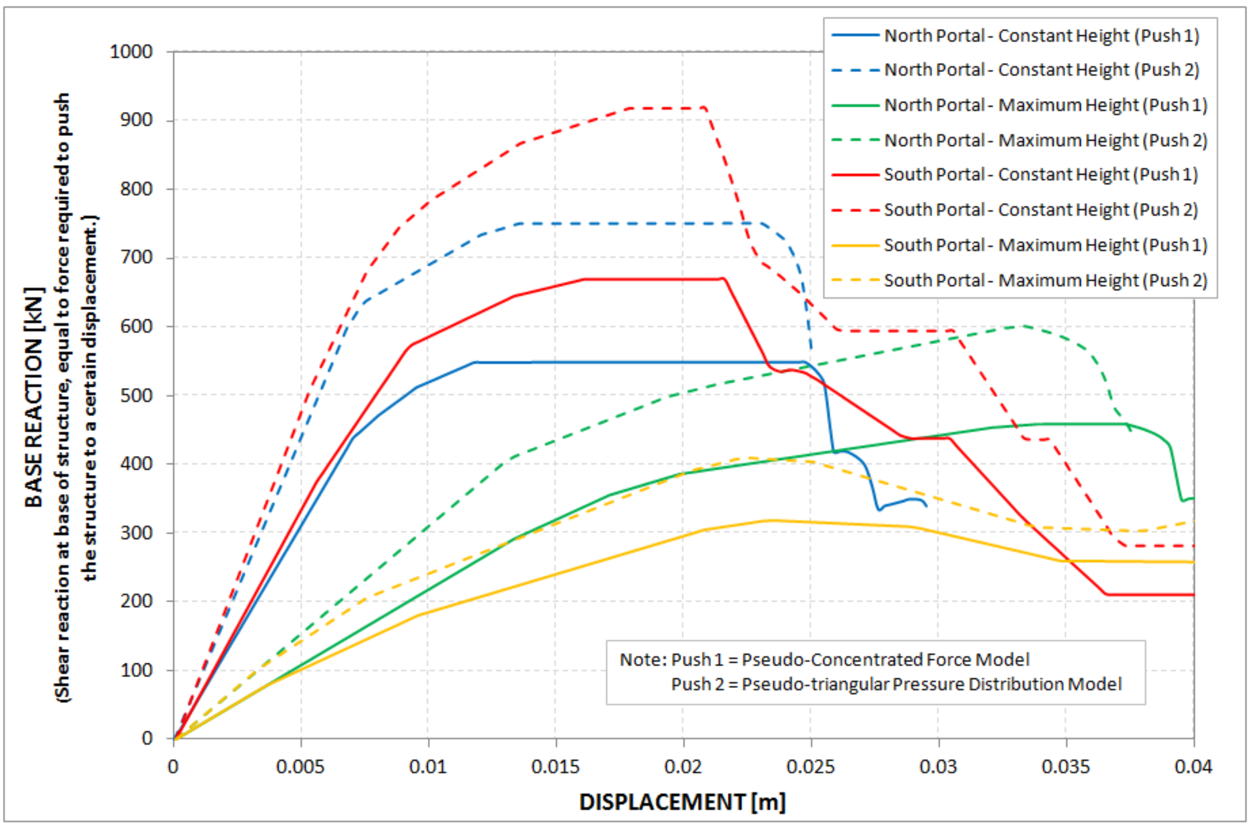

Fig. 5 – Pushover Analysis Base Shear vs. Displacement

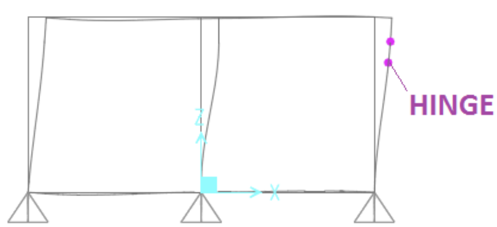

Fig. 6 – Hinging during 975-year Earthquake Event from Pushover Analysis

Table 4 – Moment Demand and Strains at Hinges

	North Portal		South Portal	
	Const. Height Section	Max. Height Section	Const. Height Section	Max. Height Section
Moment Demand [kNm/m]	395	504	454	447
Concrete Strain	0.00066	0.00080	0.00078	0.00076
Steel Strain	0.00204	0.00229	0.00225	0.00216

4. Conclusion

The Evergreen Line cut-and-cover tunnels were designed to meet the seismic requirements for all 100-year and 975-year return periods earthquake events and the subduction earthquake event. Material strains obtained from pushover analyses were within design limits for unconfined concrete. The introduction of the Seismic Review Panel brings a lot of value to the project through discussions and comments to achieve a safe and state of the art design. The benefit of having the review panel is to allow for independent professional input during the design stage. However, the review process turned out to be much longer than expected due to questions arising during the approval process. Analysis parameters were updated a few times as geotechnical investigations proceeded to arrive at the final design.

Comparing the Evergreen Line cut-and-cover tunnel to other similar projects in Vancouver, such as the Canada Line, which was designed for the 475-year earthquake based on the same racking deformation method but without pushover analysis, it is found that in general the final amount of reinforcing used is similar. Given that the Evergreen Line is designed to a higher level of earthquake (975-year), this shows that using pushover analysis that allows plastic hinging results in savings in reinforcement.

Fig. 7 – Construction of the Portal Tunnels

5. Acknowledgements

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