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SEISMIC DESIGN AND ANALYSIS OF SINGLE-STOREY CONCRETE MASONRY UNIT BUILDINGS FOR THE EVERGREEN LINE

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ABSTRACT: The Evergreen Line Rapid Transit Project is located in the lower mainland of British Columbia, a region which is prone to earthquakes. The transit system required the design and construction of five, single-storey Propulsion Power Substations (PPS) at various locations along the line, with each location composed of different ground conditions and different geometry. Each PPS building was designed for the "High" importance category as defined in the BC Building Code. Site conditions varied between stiff competent soils, to loose liquefiable soils, which had varying levels of spectral acceleration. The Seismic Force Resisting System (SFRS) of the PPSs are composed of Concrete Masonry Unit (CMU) shear walls integrated with a roof diaphragm. The shear walls were detailed for the moderately ductile level and as a result are expected to exhibit minimum plastic hinging without shear failure and local buckling during a seismic event. Diaphragms and their connections were designed to remain elastic during a seismic event. Foundations were designed to accommodate significant ground liquefaction movements at three of the five sites. This paper presents the analyses performed for the design of the PPS buildings based on the Project's Seismic Peer Review Process and the various site conditions for each building.

1. Introduction

The Evergreen Line is an 11-km extension of Vancouver's current light rail transit system, the SkyTrain network. The new line connects to the existing Millennium Line at Lougheed Town Centre Station in Burnaby and extends to Douglas College in Coquitlam. Included in the project are five propulsion power substations (PPS) which house the electrical equipment to power the trains. The five PPS buildings are located along the alignment with various site conditions. The buildings are single-storey and are composed of reinforced concrete foundations, concrete masonry unit walls and either precast hollowcore roof slabs or a reinforced concrete roof slab supported on steel beams. Refer to Fig. 1 for the foundation plan and building cross-section of the Lougheed PPS (first of the five PPS buildings). The remaining four PPS buildings have different geometry, structural wall thicknesses, roof types and foundations types. Refer to Table 1 for a comparison between the five PPS buildings.

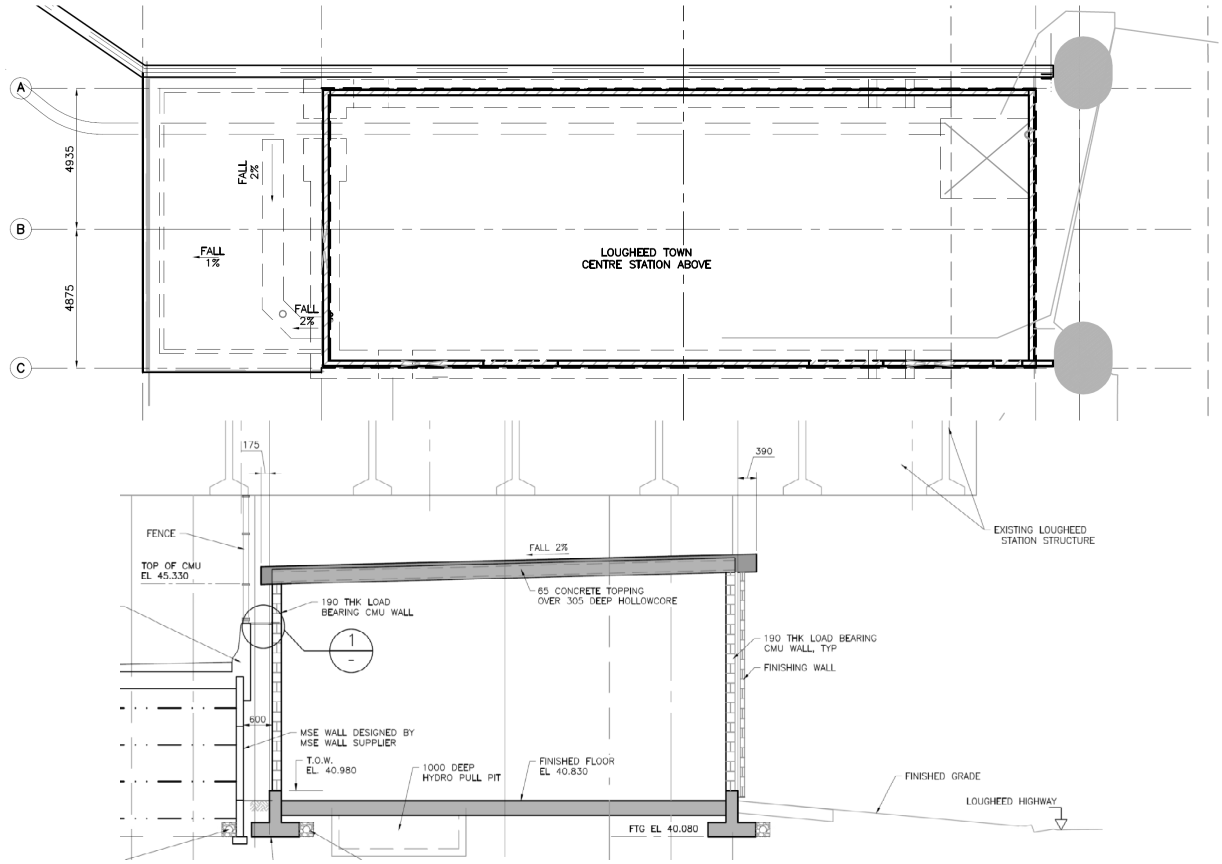


Fig. 1 – Lougheed PPS Foundation Plan and Building Cross-Section

Table 1 – Building Characteristics.

PPS ID	Location	Dimensions	Wall Type	Roof Type	Foundation Type
Lougheed	Burnaby, BC	9.8-m wide x 25.0-m long x 5.0-m tall	190 mm thick CMU	Precast hollowcore slab	Strip footing
Burquitlam	Coquitlam, BC	9.3-m wide x 40.1-m long x 5.8-m tall	190 mm thick CMU	Precast hollowcore slab	Basement walls bearing on tunnel below
North Portal	Port Moody, BC	5.5-m (min), 13.8-m (max) wide x 36.5-m long x 5.6-m tall	290 mm thick CMU	Concrete slab on steel beams	Raft slab
Falcon	Port Moody, BC	9.6-m wide x 36.8-m long x 5.6-m tall	290 mm thick CMU	Concrete slab on steel beams	Raft slab
Lafarge Lake – Douglas	Coquitlam, BC	8.1-m wide x 28.6-m long x 4.8-m tall	190 mm thick CMU	Precast hollowcore slab	Raft slab

This paper presents the analyses performed for the design of the five PPS buildings based on the Project's Seismic Peer Review Process and the various site conditions for each building.

2. Seismic Design Approach

2.1. Seismic Peer Review Panel

A seismic peer review panel (SPRP) was composed of three members, one appointed by the client, one appointed by the contractor and one appointed by the first two members. All members are Professional Engineers recognized as leading experts in the areas of structural and geotechnical seismic design and analysis. The SPRP acted as an independent peer reviewer, which conducted weekly meetings to review and approve all seismic design strategies and preliminary results. Seismic design strategies and preliminary results were presented to the SPRP in Seismic Design Strategy Memorandums (SDSM).

For every structure constructed on the Evergreen Line Project, a SDSM was prepared and approved by the SPRP before the designer could complete final detailed design of the structure. The SDSM details the seismic design approach for each structure, including the assumptions, required seismic performance levels, seismic ground motion inputs, structural and geotechnical design strategy, type of Seismic Force Resisting System (SFRS) also referred to as the Earthquake Resisting System, seismic load paths, step-by-step design procedures and analysis methodology and preliminary analysis results.

2.2. Required Seismic Performance Levels

The design life of the PPS buildings is 100 years as required by the Evergreen Line Rapid Transit Project Agreement. The required seismic performance level for the PPS buildings is a 2475-year return period with the “High” Importance Category.

2.3. Codes and Standards

The following codes and standards were followed during the design of the PPS buildings as required:

- Evergreen Line Rapid Transit Project Agreement, Schedule 4, part 2, Articles 4, 5 and 6 (Structures, Seismic and Geotechnical);
- British Columbia Building Code (BCBC) 2006;
- CAN/CSA A23.3-04, Design of Concrete Structures;
- CAN/CSA S16-06, Design of Steel Structures;
- CAN/CSA S304.1-04, Design of Masonry Structures;
- Canadian Foundation Engineering Manual.

2.4. Analysis Procedures

Initially, each structure was designed for all non-seismic loads. Subsequently the seismic loading was determined in accordance with the BCBC 2006 and was applied to the structure. The five buildings are comprised of shear walls in orthogonal directions (longitudinal and transverse building axis) which form a rectangular geometry with the exception of the North Portal PPS which has an additional external room and a diagonal wall. The variation at the North Portal PPS was due to geometrical site constraints and resulted in an irregular building classification. Linear dynamic analysis as specified in the BCBC 2006 was performed for the North Portal PPS due to its irregularity. The equivalent static force procedure was also applied for the North Portal PPS to validate results. As the degree of irregularity was minor, the results of the linear dynamic analysis were similar to that of the equivalent static force procedure. At the four remaining building locations, the equivalent static force procedure for regular structures was followed as specified in the BCBC 2006. Due to the simplicity of the buildings, the equivalent static force procedure was acceptable for analysis.

In-plane flexure and shear forces of the Seismic Force Resisting System (SFRS) due to lateral earthquake loads applied to the building system were checked. The force demands were the result of the base shear being applied in the two orthogonal directions associated with the perimeter walls of the building. Torsional effects were applied where the center of rigidity was offset laterally from the center of mass. The center of mass is adjusted by a factor of 10% of the building plan dimension in the direction opposite the center of rigidity as specified in the BCBC 2006.

The walls were then checked for out-of-plane flexure and shear forces due to lateral wind, earthquake loads and/or eccentric vertical loads. Once detailed for out-of-plane force effects, the SFRS was once again checked to ensure it was detailed to the code requirements.

The in plane force effects applied to the diaphragms were assessed as specified in the BCBC 2006.

A global stability check of the building in both the longitudinal and transverse directions was performed by considering overturning and sliding due to lateral wind and seismic loads.

Additional miscellaneous structural components such as the masonry rain screen veneer wall ties, masonry rain screen veneer supporting angles and lintel beams were checked and detailed as specified in the appropriate design codes.

3. Site Conditions

3.1. Location and Soil Conditions

The five PPS locations are listed below, table 2 below compares the site conditions of each building.

- The Lougheed PPS is located below the existing Millennium Line guideway adjacent to Lougheed Town Centre Station in Burnaby.
- The Burquitlam PPS is located in Coquitlam and is constructed above the Evergreen Line guideway's cut-and-cover transition tunnel.
- The North Portal PPS is located in Port Moody, at the foot of Burnaby Mountain adjacent to Burrard Inlet in soft liquefiable soils with shallow ground water levels.
- The Falcon PPS is located in Port Moody adjacent to the earth cut constructed by CP Railway (CPR).
- The Lafarge Lake-Douglas PPS is located in Coquitlam adjacent to Lafarge Lake.

Table 2 – Site Conditions.

PPS ID	Soil	Ground water level	Site Class	Ground Elevation
Lougheed	mixture of sands, silts and clay above underlying till-like deposits	2.0 m to 5.8 m below the surface	C	40.8-m
Burquitlam	sands and silts above underlying till-like deposits	0.3 m to 3.5 m below the surface	C	115.4-m
North Portal	mixture of clay, silts, sands, gravels, organics and soft clayey silt with zones of sandy silt above underlying till-like deposits	0.3 m to 0.8 m below the surface	F	7.4-m
Falcon	mixture of interlayered sand, silty sand, silt and gravel layers which vary from loose to dense levels of compaction, firm to stiff clay, silt and sand with silt layers, above compact to very dense sand and gravel with variable silt above till-like deposits	3.5 m below the surface	F	26.3-m
Lafarge Lake – Douglas	mixture of compact interlayered sand and gravel above soft to firm silty clay and clayey silt above compact to very dense sand and gravel above till-like deposits	3.0 m below the surface	F	33.4-m

3.2. Response Spectra

Based on pre-project investigations, response spectra and associated ground motion time-histories analyses were performed by the Geotechnical Engineer, seismic input ground motion parameters were developed for a reference site, which was geographically located at the start of the Evergreen Line. During detailed design, site-specific ground response analyses were performed by the geotechnical engineer. The period for each building was calculated using the Priestley and Hart equation described in Anderson and Brzev (2009).

3.2.1. Lougheed PPS and Burquitlam PPS

Liquefaction was determined to be unlikely at these sites (using Youd et al., 2001, method). As a result, the design spectral acceleration values determined using the BCBC 2006 were used for seismic analysis of the PPS building at these two sites.

3.2.2. North Portal PPS

A site specific spectrum shown in Fig. 2 – (a) was prepared for six input motions obtained from the Project Agreement for the 2475-year seismic event. The average response of the six input motions was used for the analysis of the building. The code developed spectrum for both Site Class D and E were compared with the site specific spectrum. The site-specific spectrum resulted in a higher spectral acceleration than that for Site Class D and E.

The site-specific spectrum has two peaks, one at a period of roughly 0.18 seconds, and the second at a period of roughly 0.45 seconds. The period of the PPS structure is calculated as roughly 0.1 seconds and may elongate to roughly 0.18 seconds if the SFRS experiences yielding. It was determined that the first peak of the spectrum would be used as the upper bound of the short period plateau employed by the BCBC to obtain base seismic shear forces applied to the structure.

3.2.3. Falcon PPs

Site-specific ground response analysis resulted in a site specific spectral acceleration curve shown in Fig. 2 – (b). The code developed spectrum for both Site Class C, D and E were compared with the site specific spectrum. The site-specific spectrum resulted in a higher spectral acceleration than that for Site Class C, D and E.

The site-specific spectrum's peak was slightly beyond the expected period range of the building. The peak was used as the upper bound of the short period plateau employed by the BCBC to obtain base seismic shear forces applied to the structure.

3.2.4. Lafarge Lake-Douglas PPS

Site-specific ground response analysis resulted in a site specific spectral acceleration curve shown in Fig. 2 – (c). The code developed spectrum for both Site Class D and E were compared with the site specific spectrum. The site-specific spectrum resulted in a spectral acceleration equivalent to that for Site Class E.

The site-specific spectrum has two peaks, one at a period of roughly 0.33 seconds, and the second at a period of roughly 0.8 seconds. The period of the PPS structure is calculated as roughly 0.12 seconds and may elongate to roughly 0.21 seconds if the SFRS experiences yielding. It was determined that the Sa(0.2) value which governed over the first peak of the site specific spectrum would be used as the upper bound of the short period plateau employed by the BCBC to obtain base seismic shear forces applied to the structure.

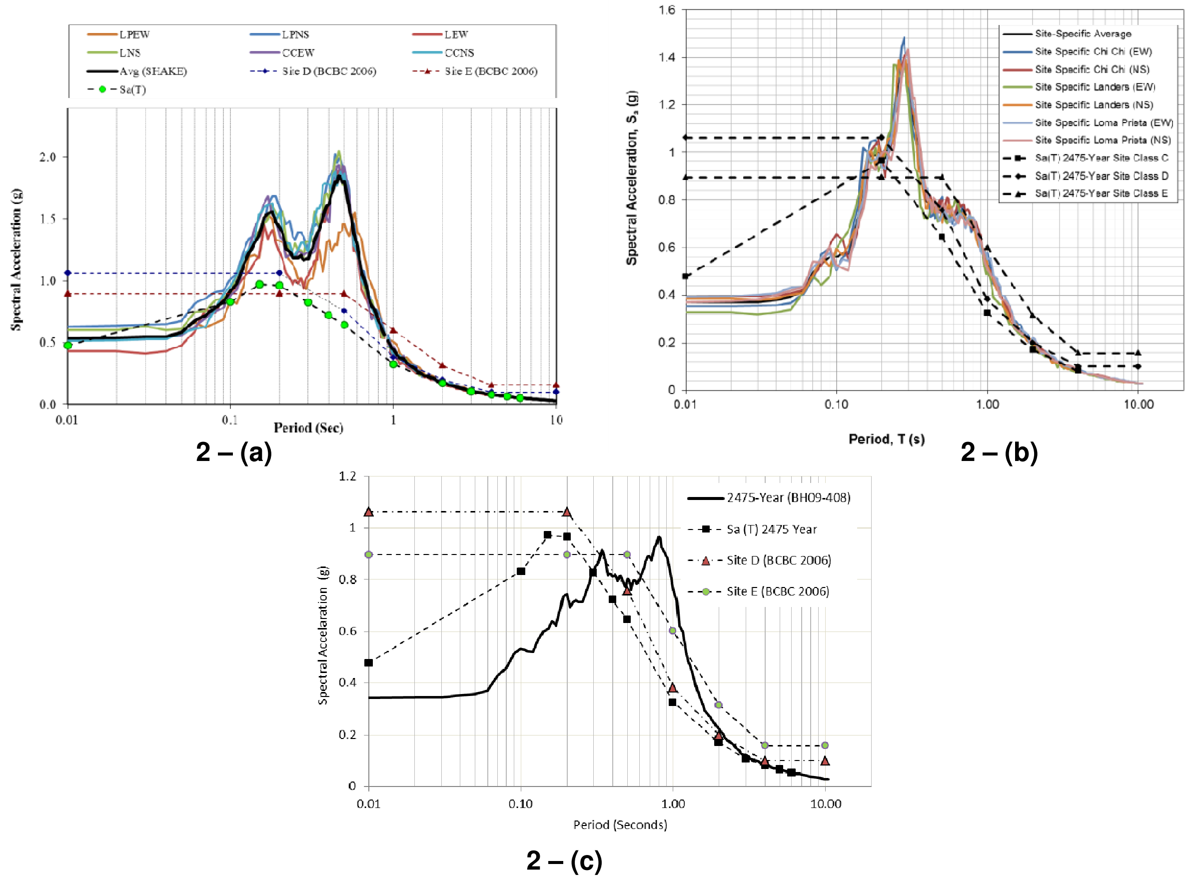


Fig. 2 – (a) Site Specific Response Spectra at North Portal PPS; (b) Site Specific Response Spectra at Falcon PPS; and (c) Site Specific Response Spectra at Lafarge Lake-Douglas PPS

4. Building Enclosure Structure

4.1. Seismic Force Resisting System

The SFRS – or Earthquake Resisting System (ERS), as defined in the project agreement – for the PPS buildings comprise of reinforced concrete masonry unit (CMU) walls, that support the roof of the building. The reinforced CMU walls are detailed to a moderate ductility level using the applicable ductility-related and overstrength-related force modification factors in accordance with the BCBC 2006 and CSA S304.1-04.

The force modification factors used are:

- Ductility-related force modification factor, R_d=2.0; and
- Overstrength-related force modification factor, R_o=1.5.

By using the moderate ductility level, the SFRS will exhibit minimum plastic hinging without shear failure and local buckling during the seismic event. All other components of the structure were detailed to remain elastic during the seismic event. As required by the BCBC 2006, the structures were designed to limit the lateral deflection to 1% of the inter-storey height.

4.2. Seismic Load Paths

The inertial loads from the structure are transferred through the SFRS into the foundation and in turn into the soil. Roof structure types were selected to meet the needs of the structural design and accepted input from the contractor to suit constructability. Typically the roof structures are comprised of precast hollowcore slabs with a reinforced concrete overlay. At the sites with large spectral accelerations, steel beams composite with a reinforced concrete slab were utilized to reduce the mass of the roof structure.

The roof structure diaphragms were designed to remain elastic and the connection with the shear walls was detailed per the requirements of the BCBC 2006.

The North Portal PPS site is unique to the other sites as it was estimated that it has the potential for large seismically induced lateral spread displacements for the design seismic event. The equipment in the PPS building must connect with the guideway system through electrical cables. At this location, the cables hang from the adjacent bridge structure and penetrate through the PPS building enclosure. Coordination with the guideway designer was required to determine the maximum relative movements between the two structures. Due to their locations, the lateral spread displacements will cause the building to travel away from the pile supported guideway structure. The two structures had different seismic performance levels and were required to be detailed for different return periods, as a result, the governing return period was considered when determining the relative displacement of the structures.

5. Foundation

5.1. Conventional Strip Footing

At the Lougheed PPS where liquefaction is not expected to occur, conventional strip footing foundations are provided to transfer both vertical and lateral loads from the building structure, to the soil. At Burquitlam PPS reinforced concrete walls bearing on the guideway's cut-and-cover tunnel situated below the PPS are provided. All vertical and lateral loads from the building structure are transferred into the tunnel.

For both cases, the reinforced concrete foundation walls are designed for force effects as specified in the BCBC 2006 and are detailed per the requirements of seismic design provisions of CSA A23.3-04.

5.2. Raft Slab in Liquefiable Soils

Due to the potential for liquefaction at the three PPS sites, a raft slab foundation was considered suitable for supporting the structure, transferring seismically induced inertial loads, and resisting ground displacements. For the three sites with this foundation option, ground improvements were not considered necessary.

Using the empirical methods the geotechnical engineer provided approximate estimates of seismically induced lateral spreading displacements. For the 2475-year event, these displacements were in the order of 1.5 m to 2 m at the North Portal PPS, 2.5 m at the Falcon PPS and 1.2 m at the Lafarge Lake - Douglas PPS.

The post-seismic settlements for sand and fine-grained soils were providing by the geotechnical engineer using methods based on Wu (2002) as recommended by Idriss and Boulanger (2008). These settlements were in the order of 0.15 m to 0.35 m at the North Portal PPS for the 2475 year event. It was advised by the geotechnical engineer that differential settlement equal to half of these total settlements should be considered in designing the raft slab foundations.

Per recommendations of the Task Force Report (2007), it is important for communication to occur between the structural engineer and geotechnical engineer and for them to review the foundation design at liquefiable sites together. The result of this discussion with the geotechnical engineer was to design the three PPS buildings with a stiff raft slab to uniformly support the building in the event of post-seismic soil softening or loss of support.

Each raft slab foundation was analyzed using a shell model supported by linear soil springs with stiffness equivalent to the subgrade modulus. The self weight and floor loads were distributed over the footprint of the slab, and wall and roof loads were applied around the perimeter, as seen in Fig. 3. Each of the buildings had a cable pulling pit which drops below the floor elevation. Pull pits were modeled as openings to determine the force effects in the surrounding raft slab.

It was recommended by the geotechnical engineer that the raft slab be modelled with a zone of softened subgrade modulus over a horizontal distance of 10 m. Three modes were considered by the structural engineer to assess the slab performance, the 'Simple Support Mode', the 'Hogging Mode' and the 'Loss of Corner Support Mode'. Refer to Fig. 4 through Fig. 6 where the hatched colour indicates the zone of softened subgrade modulus.

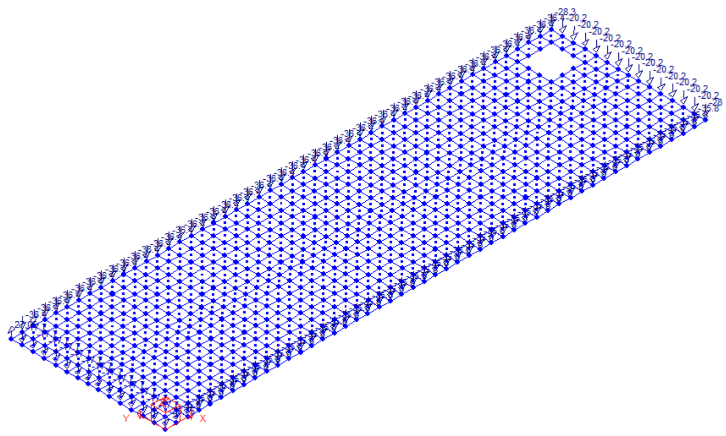
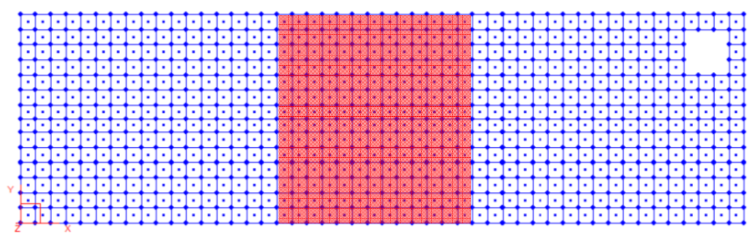


Fig. 3 – Typical Raft Slab Model of the Reinforced Concrete Raft Slab

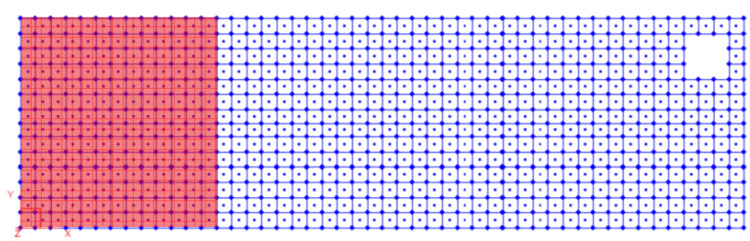
(Lafarge Lake – Douglas PPS Case is shown above)



= Loss of Support (Liquefied Soil)

Fig. 4 – Liquefied Soil: Simple Support Mode

The simple support mode considers a softened subgrade modulus for a 10 m long zone at the center of the building.



= Loss of Support (Liquefied Soil)

Fig. 5 – Liquefied Soil: Hogging Mode

The hogging mode considers a softened subgrade modulus for a 10 m long zone at either end of the building.

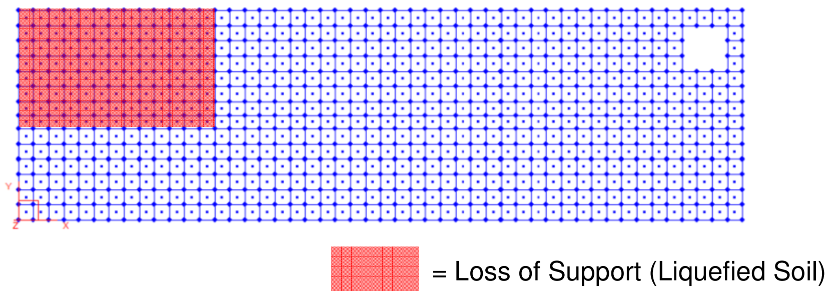


Fig. 6 – Liquefied Soil: Loss of Corner Support Mode

The loss of corner support mode considers a softened subgrade modulus for a 10 m long zone by half the width of the building located at the corner of the building's footprint.

The raft slab analysis resulted in vertical deflections which were largest around the perimeter of the building for the static case. For each loss of support condition, vertical deflections were largest in the zone of reduced subgrade modulus. This resulted in peak bending moments and shear forces around the perimeter of the slab for the static case and peak bending moments and shear forces at the transition from the original subgrade modulus to the softened modulus.

The raft slab was detailed to remain elastic under the force effects resulting from the three liquefied soil modes. As a result, the stiff raft slab has uniform thickness in the center, with a locally thickened beam around the perimeter. The perimeter beam provides additional stiffness to support the vertical loads transferred from the building's perimeter walls into the foundation.

6. Lessons Learned

6.1. Concrete Masonry Design

It was observed that due to the conditions of certain building structures, the out-of-plane force effects may govern the CMU wall design, particularly for high seismic spectral accelerations. As required by CSA S304.1-04, there exists a relationship between the vertical and horizontal reinforcement steel for moderately ductile squat shear walls, where the horizontal reinforcement is a function of the vertical reinforcement. If the CMU wall is strengthened in the vertical direction to resist out-of-plane force effects, the code requires the horizontal reinforcement to be increased, which may result in the wall's in-plane shear capacity to significantly exceed the seismic force effects. Careful consideration should be given when detailing the SFRS to ensure the CMU walls remain economical and exceptional capacity does not increase theoretical demands on the supporting diaphragms.

6.2. Alternate Foundation Analysis for Liquefiable Sites

The three PPS buildings situated above liquefiable soils were designed with reinforced concrete raft slab foundations, but steel pipe piles were also discussed during preliminary design development.

The requirements of the Evergreen Line Project Agreement and design codes allow for ERS components to undergo an inelastic response, but that capacity-protected components shall remain elastic during the seismic event. Typically for pile supported structures, such as the guideway bridge structures this means that plastic hinging will form in the reinforced concrete columns, and piles will remain elastic. However, due to the high lateral spread values associated with the design earthquake, the above concept resulted in inefficient foundation designs to meet the strict project requirements. As a result, the Province has accepted a request to allow for minor inelastic response of concrete filled steel pipe pile foundations. The piles were to remain elastic in the event of inertial loads from the above ground structure, but could exhibit minor inelastic deformation below ground in the event of post seismic lateral spread. The plastic hinging is controlled by limiting the maximum plastic rotation as per ATC-49. During the detailed design stage, pushover analyses was performed by the various designers for the pile supported guideway structures through the Port Moody area, where high lateral spread values during the design seismic event are expected.

While this change to the project requirements resulted in a more economical design for the guideway structures, it was determined that a reinforced concrete raft slab foundation was acceptable, and more economical than a pile supported slab for the three PPS buildings situated in liquefiable soils.

7. Conclusion

The five propulsion power substations required for the Evergreen Line were designed and detailed to meet the seismic design requirements of the Evergreen Line Project Agreement and applicable codes. The building's SFRS consists of reinforced concrete masonry unit (CMU) walls, that support the roof of the building. The reinforced CMU walls were detailed to a moderate ductility level using the applicable ductility-related and overstrength-related force modification factors in accordance with the BCBC 2006 and CSA S304.1-04. Roof structure types were selected to meet the needs of the structural design and accepted input from the contractor to suit constructability. Each building foundation was designed for its unique site condition, three of which may experience liquefaction and lateral spreading during a seismic event.

At the time of drafting this paper, all five PPS buildings have been substantially completed to the design drawings. Refer to Fig. 7 for photos of the substantially completed structures.

Fig. 7 – Substantial completion of Lougheed PPS (left) and North Portal PPS (right)

8. Acknowledgements

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9. References

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