Evergreen Line Rapid Transit project: deep foundation and ground improvement solutions

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A. Azizian & B.E. Hall

Tetra Tech Canada, Vancouver, BC, Canada

M.L. Preece

SNC-Lavalin Inc., Vancouver, BC, Canada

ABSTRACT: The Evergreen Line Rapid Transit Project is the most recent addition to the SkyTrain network in Greater Vancouver, British Columbia, Canada. The 11 km alignment consists of elevated and at-grade guideway sections as well as cut-and-cover and bored tunnels. Because of the high seismicity of the region, liquefiable and soft soils posed the main challenges for the design of structures. A performance-based design approach with varying levels of performance requirements were specified for the project. Several deep foundation and ground improvement solutions were used to meet the project design objectives, while satisfying cost, schedule and site constraints. Foundation types included drilled shafts and driven steel pipe piles. Ground improvement solutions included Cement Deep Soil Mixing (CDSM) and driven timber piles. This paper presents an overview of the subsurface ground conditions along the alignment, seismic analyses (e.g. FLAC modelling), and the selected foundation or ground improvement types.

1 INTRODUCTION

The Evergreen Line Rapid Transit (ELRT) is an 11 km extension of the SkyTrain system through Burnaby, Coquitlam and Port Moody in Greater Vancouver (Lower Mainland), British Columbia (BC), Canada (Fig. 1). The ELRT consists of approximately 6 km of elevated guideway, 3 km of at-grade (ground level) guideway, 2 km of bored tunnel, two cut-and-cover tunnel approach structures, seven passenger stations, six propulsion power substations (PPS), and a vehicle storage and maintenance facility (Figs. 1 and 2). The elevated guideway sections consist of two types of super-structure: precast segments and steel/concrete girders (Fig. 3).

The ELRT was delivered as a Design/Build project by Evergreen Rapid Transit Construction (EGRT), an SNC-Lavalin consortium, to the Province of British Columbia. Design and construction of several sections were subcontracted to a joint venture of SNC-Lavalin Constructors (Western), Inc. (SLCW), and Graham Construction (SGJV). The pursuit phase of the project started in November 2011. Project was awarded in October 2012, followed by an intense geotechnical investigation, design and construction schedule in 2013-2014. The last precast segment of the elevated guideway was in place in March 2015. The ELRT was officially opened to public in December 2016. Now BC has the longest fully-automated rapid transit line in the world (http://www.evergreenline.gov.bc.ca).

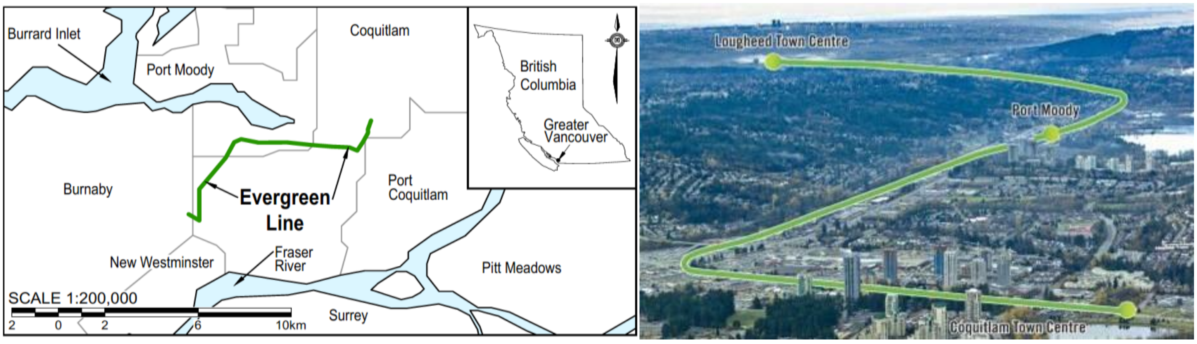

Figure 1. ELRT location and aerial view (looking east).

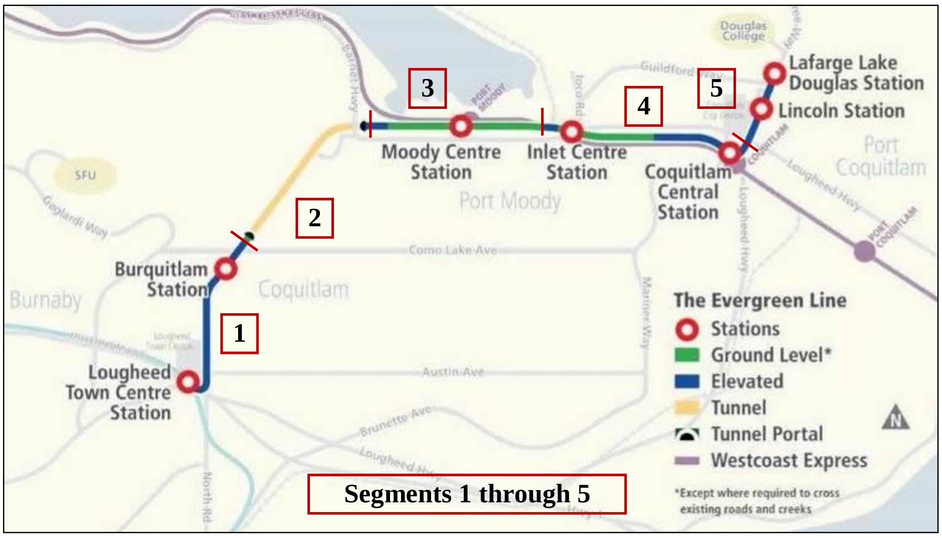

Figure 2. ELRT alignment, segments 1 through 5, and station locations.

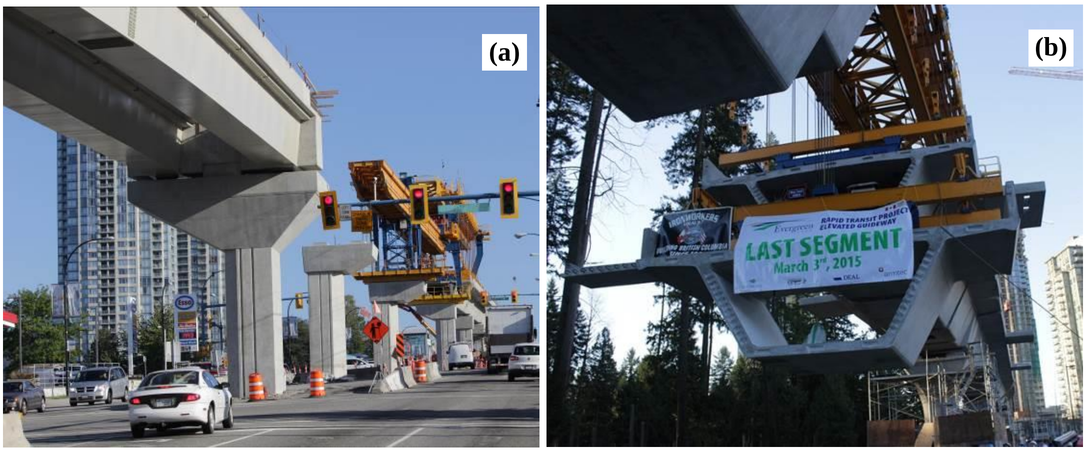

Figure 3. Views of elevated guideway: a) steel girder (Segment 1), and b) precast segments.

2 SUBSURFACE CONDITIONS

The ELRT was divided into 5 Segments (Fig. 2), starting at the following locations from south to north: 1) Lougheed Town Centre Station; 2) south tunnel portal; 3) elevated guideway north-west of the bored tunnel; 4) elevated guideway east of Inlet Centre Station; and, 5) elevated guideway north-east of Coquitlam Central Station. The alignment runs through highly variable topographic and subsurface ground conditions. The geological profiles along Segments 1 through 5 (Fig. 2) can be broadly divided in 3 categories as follows (see Pellett et al., 2016, for further details):

- Segments 1 and 2 (Fig. 4a): Variable fill, over post-glacial silty sand (Capilano Sediments), over very dense glacially overridden till-like deposits (Vashon Sediments), over interbedded sand and silt grading downwards into silty clay (Quadra Sediments), over a basal glacial till-like layer (possibly Semiahmoo Sediments). Groundwater is confined by the upper till-like material and piezometric levels were observed to be within a few meters of ground surface in some areas. Localized artesian pressures (in the Quadra Sediments) and shallow perched water were also encountered.
- Segments 3 and 4 (Fig. 4b): Variable fill, over sandy debris fan, marine and shoreline deposits (Salish Sediments), over post-glacial marine silty clay and sandy glaciomarine deposits (Capilano Sediments), over very dense till-like deposits (Vashon Sediments). Groundwater is shallow and nominally at the base of the fill, and in places, is tidally-influenced. Artesian pressures up to about 13 m above ground surface were measured in granular layers underlying the marine clay.

- Segment 5 (Fig. 4c): Variable fill, over coarse deltaic sands and gravels grading with depth into marine interlayered sand, silt and clay (Capilano Sediments), overlying very dense glaciofluvial deposits and till-like deposits (Vashon Sediments). Groundwater is shallow and nominally at the base of the fill. Artesian pressures up to about 5 m above ground surface occur in the till-like deposits.

3 GEOTECHNICAL CHALLENGES

The primary geotechnical design and construction challenges are summarized in Table 1.

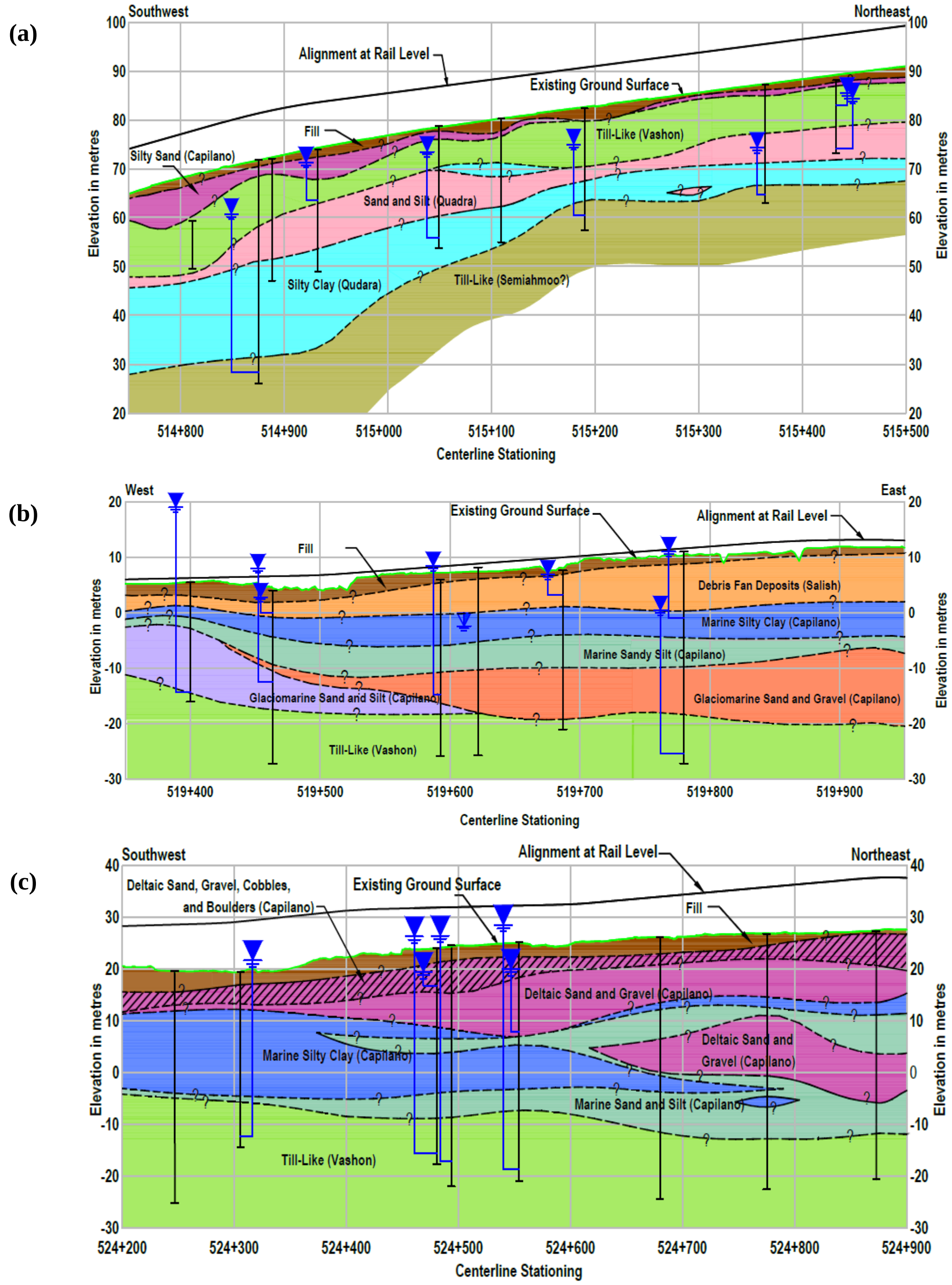

Figure 4. Typical soil profile for a) Segments 1 and 2; b) Segments 3 and 4; and c) Segment 5.

Table 1. Summary of the geotechnical challenges.

Issue/Challenge	Segment Number 1 & 2	3 & 4	5
Loose debris fan / deltaic deposits that are potentially liquefiable or subject to cyclic mobility causing post-seismic vertical and lateral spreading displacements	-	x	x
Soft marine clay sediments causing settlement of track slab and/or MSE embankment	-	x	-
Ground slope, proximity to creeks/ravines	-	x	-
Obstructions (cobbles, boulders, glacial erratics, and buried logs or stumps) causing delays and difficulties during ground improvement and/or pile driving	-	x	x
Difficult groundwater conditions and artesian pressures	x	x	x
Impact to other structures/facilities	x	x	x
Impact to the adjacent Canadian Pacific Rail (CPR) tracks	-	x	-
Extremely limited right of way for construction activities	x	x	x
Large variability in depth to competent/stable material for pile bearing, etc.	x	x	x

4 SEISMIC DESIGN CRITERIA

The Project Agreement (PA) for the ELRT provided an extensive set of criteria, which included detailed requirements for seismic, geotechnical and structural design.

4.1 Design standards and review process

The following design codes, standards and guidelines were specified by the PA: BC Ministry of Transportation and Infrastructure Bridge Standards and Procedures Manual, Supplement to CAN/CSA–S6–06 (2007); Canadian Highway Bridge Design Code, CAN/CSA-S6-06 (2006); ATC-32 Improved Seismic Design Criteria for California Bridges: Provisional Recommendations (1996); ATC-49/MCEER Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (2003); AASHTO LRFD Bridge Design Specifications (2010); Canadian Foundation Engineering Manual (2006); and, British Columbia Building Code (BCBC, 2006).

The PA mandated several stages of review, checking, approval and/or acceptance: review and approval of the Seismic Design Strategy Memoranda (SDSM) for each structure (or structure type) by the Seismic Peer Review Panel (SPRP); review and acceptance of Geotechnical Design Reports by the Owner's Engineers; and, checking and issuance of Design Certificates by independent firms for Category III Structures (e.g. a guideway foundation in liquefiable soil).

The PA specified that design finalization and construction could not commence without prior approval of the corresponding SDSM. Preparing the strategy level and detailed analysis/design components of the SDSMs for each structure was concurrent with the geotechnical investigation program and other mandatory detailed design and construction activities. This concurrence imposed major challenges on meeting the project design and construction schedule.

4.2 Seismic Performance Levels

The PA required either 3 or 4 Seismic Performance Levels, as summarized in Table 2, to be met for each Seismic Event Level depending on the location/type of structures. Generally, the 3-Level criteria were applicable to the elevated guideway structures and were more onerous than the 4-Level criteria that were applicable to the at-grade sections (Segments 3 and 4, Fig. 2). The BC Building Code (2006) requirements and seismic return period were specified for passenger station and PPS buildings.

Table 2 presents the Peak Ground Accelerations (PGA) and spectral accelerations, Sa(T), corresponding to Site Class C (firm ground) as per BCBC (2006) for the seismic events specified by the PA. The PA also provided the ground motion time-histories, which included three sets of one vertical and two horizontal components, except for the Subduction event with only one set. The PA required accounting for the effects of the simultaneous application of horizontal and vertical ground motions.

Table 2. Seismic Performance Levels, PGA, selected spectral accelerations (Sa), and Magnitude (M).

Seismic Event Level	Seismic Performance Level for Guideway Structure		Buildings (Stations/ PPS)	PGA (g)	Sa(0.2) (g)	Sa(2.0) (g)	M
	3- Level Criteria	4-Level Criteria					
100-Year	Immediate Use		N/A	0.116	0.224	0.039	6.7
475-Year	N/A	Repairable	N/A	0.254	0.502	0.086	6.8
975-Year	Repairable	Life-Safety/ No-Collapse	N/A	0.343	0.688	0.119	6.9
2475-Year	N/A to Guideway		Safe Egress	0.478	0.965	0.170	6.9
Subduction	Repairable		N/A	0.160	0.370	0.077	8.2

4.3 Structural requirements

Performance-based seismic design outlined in the PA was primarily displacement-based and a significant departure from the traditional force-based methods (Khan and Jiang, 2015). The PA specified two types of Earthquake Resisting Systems (ERS), as described below:

- Permitted ERS: The Permitted ERS were limited to ductile substructure elements, such as columns, braced frames, and moment resisting frames along with base isolation and energy absorption devices. The design philosophy was to limit the inelastic response in the Permitted ERS to a specified range corresponding to each Seismic Performance Level. All other components were to be Capacity-Protected as per ATC-32 (1996), and to remain essentially elastic.
- Potentially Permitted ERS: The PA also stipulated that piles with minor inelastic response could be used as Potentially Permitted ERS, on a case-by-case basis, along with demonstration of compliant performance, subject to acceptance/approval by the SPRP.

Project-specific limits were developed and approved for pile inelastic response and reinforcement steel strains. Allowable displacement limits were also developed and approved by the SPRP for each Seismic Event Level, based on discussions with designers from other disciplines (e.g., structural, guideway tracks, MSE wall). Refer to Azizian et al. (2015 and 2017b) for more details.

5 ANALYSIS/DESIGN METHODOLGY

The PA specified the following related to the foundation design:

- Foundations in soils that are likely to liquefy, or experience partial liquefaction (cyclic mobility) and/or undergo post-liquefaction movements, must be assessed using the site-specific ground response and soil-structure interaction analyses.
- Soil-structure interaction analyses for foundation design should be performed using de-coupled methods of analysis as a minimum. For selected sections of the ELRT, coupled analyses were also required to confirm the results of the de-coupled analyses.
- Inertial loading from the structure and the loading from ground displacements (e.g. settlements and lateral spreading) must be accounted for in foundation design.

Coupled soil-structure interaction analyses were performed using the computer program FLAC (Itasca, 2012). Constitutive models UBCSAND and UBCHYST/Hyper-U were used for simulating the behavior of liquefiable sands and soft clays, respectively (e.g., Beaty and Byrne, 2011). De-coupled analyses were performed using programs such as LPILE/GROUP and Slope/W, and empirical methods of estimating lateral spreading displacements for liquefiable soils (e.g. Idriss and Boulanger, 2008; Youd et al., 2009). Site-specific response spectra were calculated using program SHAKE2000.

The PA required consideration of both kinematic and inertial loads but did not specify if/how these should be combined. Adopting the Caltrans (2012) approach was proposed by the design team and approved by the SPRP: 100% inertial; 100% kinematic; 100% kinematic with +/- 50% inertial.

6 FOUNDATION TYPES AND GROUND IMPROVEMENTS

6.1 Drilled shafts

The precast segmental guideway in Segment 1 was supported on single drilled shafts (Figs. 2 and 5). The drilled shafts were installed dry in areas where minimal groundwater was encountered, or using a drilling slurry to prevent the hole from collapsing in areas where groundwater was encountered. The shafts were typically 2.4 m in diameter and generally varied from about 15 m to 25 m in length. Larger shallow footings with tie-down anchors were originally considered for this section but were later abandoned because of the negative impact on traffic in a busy urban corridor (Fig. 3). No load testing was performed but results of load tests on a similar nearby project were used in conjunction with conservative resistance factors. Crosshole Sonic Logging (CSL) tests were performed to check the integrity of the drilled shafts. Pressure grouting at the shaft base was used in a few cases where water upwelling or soil sloughing were encountered near the base.

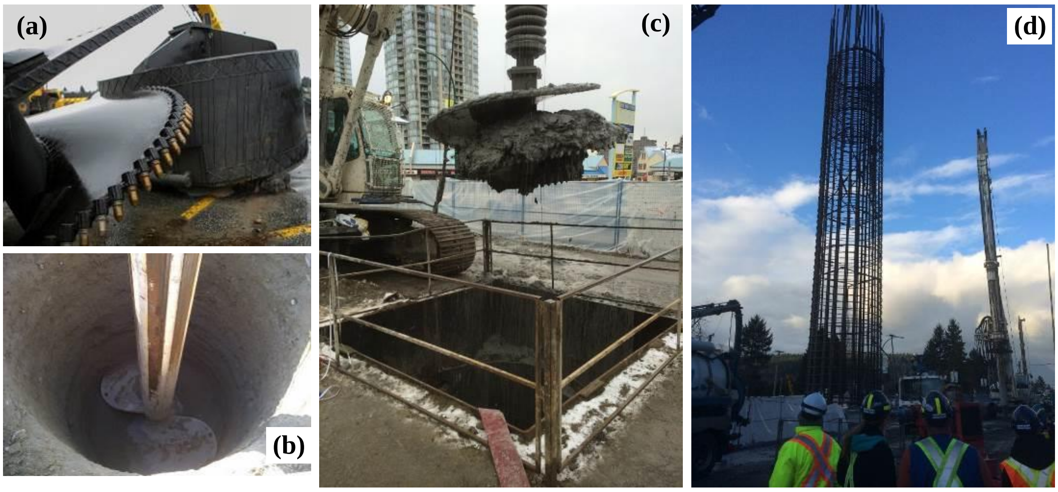

Figure 5. Drilled shafts: a) drilling and cleanout tools, b) dry drilling, c) drilling rig; and d) reinforcement cage.

6.2 Driven steel pipe piles

Elevated guideway structures in Segments 3, 4 and 5 were supported on groups of driven open-ended steel pipe piles (Fig. 6). Piles were vibrated down through soft deposits and then impact-driven into till-like soils using diesel hammers. Pile lengths varied significantly, from about 10 m to 40 m, because of the large variability in depth to till-like soils as well as the strength/composition of these soils. Pile diameters ranged from 610 mm to 1.8 m. Pile group arrangements included: 2x1, 4x1, 2x2, 3x2, etc. Piles were filled with reinforced concrete and connected either by a pile cap, or extended above ground and framed into a pier cap.

High-strain dynamic testing (e.g. PDA) were required by the PA for at least one pile per pier. These tests were used to confirm that adequate pile capacities were achieved, and to detect damage due to driving into till-like materials with high potential of encountering obstructions (e.g. boulders). Damaged piles were typically removed and re-installed. In a few cases, tie-down anchors or H-piles were installed through or adjacent to the pipe pile to supplement its tensile (uplift) capacity.

Only structural solutions were used to resist liquefaction-induced displacements mainly because ground improvement would have caused unacceptable disruption to nearby structures (e.g. a chemical facility) or environmentally sensitive lands/creeks. Pile demands due to liquefaction and associated lateral spreading were calculated using de-coupled and/or coupled (FLAC) analyses. Azizian et al. (2015) and Khan et al. (2015) present the main geotechnical and structural design considerations for some of the elevated guideway structures. The design/analyses focused on incorporating more flexibility/ductility into the structural system (e.g. by varying pile sizes) thus avoiding hard points and concentration of extremely high local demands due to liquefaction variability.

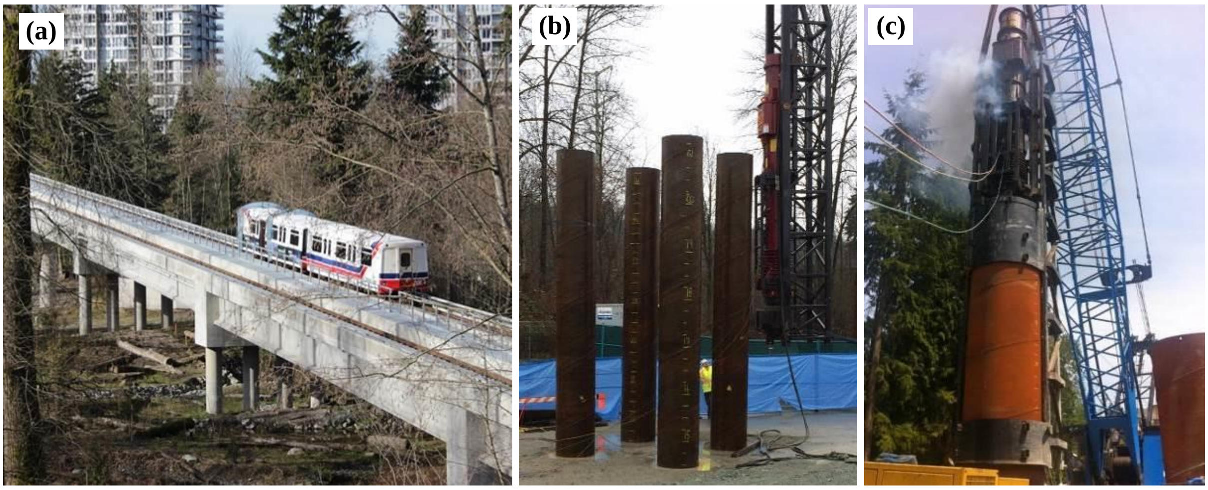

Figure 6. a) Elevated guideway west of Inlet Centre, b) a typical steel pipe pile group; c) pile driving.

6.3 **Cement deep soil mix (CDSM) panels**

The at-grade section in Segment 3 (Fig. 2) was supported on improved ground consisting of 1.83 m diameter cement deep soil mix (CDSM) columns. These columns were overlapped to form shear panels (Figure 7) spaced nominally at 5 m on centers. Over 2000 CDSM columns were installed. The CDSM columns extended to depths of 10 m to 24 m below ground level. In places, a 2.5 m high by 0.75 m wide concrete crash wall was required to protect the ELRT from the adjacent CPR heavy freight railway lines. CDSM columns were installed using a purpose-built, hollow stem bar and mixing tool arrangement slung from a purpose-built bored-pile rig (Fig. 7). The mixing tool was rotated in and out of the ground while injecting grout slurry.

CDSM is a geomaterial that has stress-strain characteristics analogous to that of an over-consolidated clay, or a weak sandstone. The computer program FLAC was used to simulate the behavior of the CDSM columns/panels. The seismic loading demands subjected the CDSM panels to bending and potentially cracking; therefore, the Ubiquitous-Joint model that accounts for planes of weakness was used to estimate maximum displacements in the case of progressive failure due to cracking (Fig. 7b). Stress-strain properties were obtained from triaxial compression tests on soil-cement samples. See Hall et al. (2015) and Azizian et al. (2017a) for further details.

Prior to commencing production, two test sections were completed to verify mix proportions and mixing parameters for the production work, and confirm minimal impacts to existing facilities. Quality Control (QC) testing included both UCS testing of samples obtained by wet grab methods (immediately following installation of a column) and 83 mm (PQ-sized) core holes, which were drilled through the columns after a specified period of time following installation. In addition, three full-scale loading tests were conducted at three locations to confirm that CDSM columns in clay had adequate vertical capacity and met settlement criteria. The load response curves showed evidence of strain-hardening.

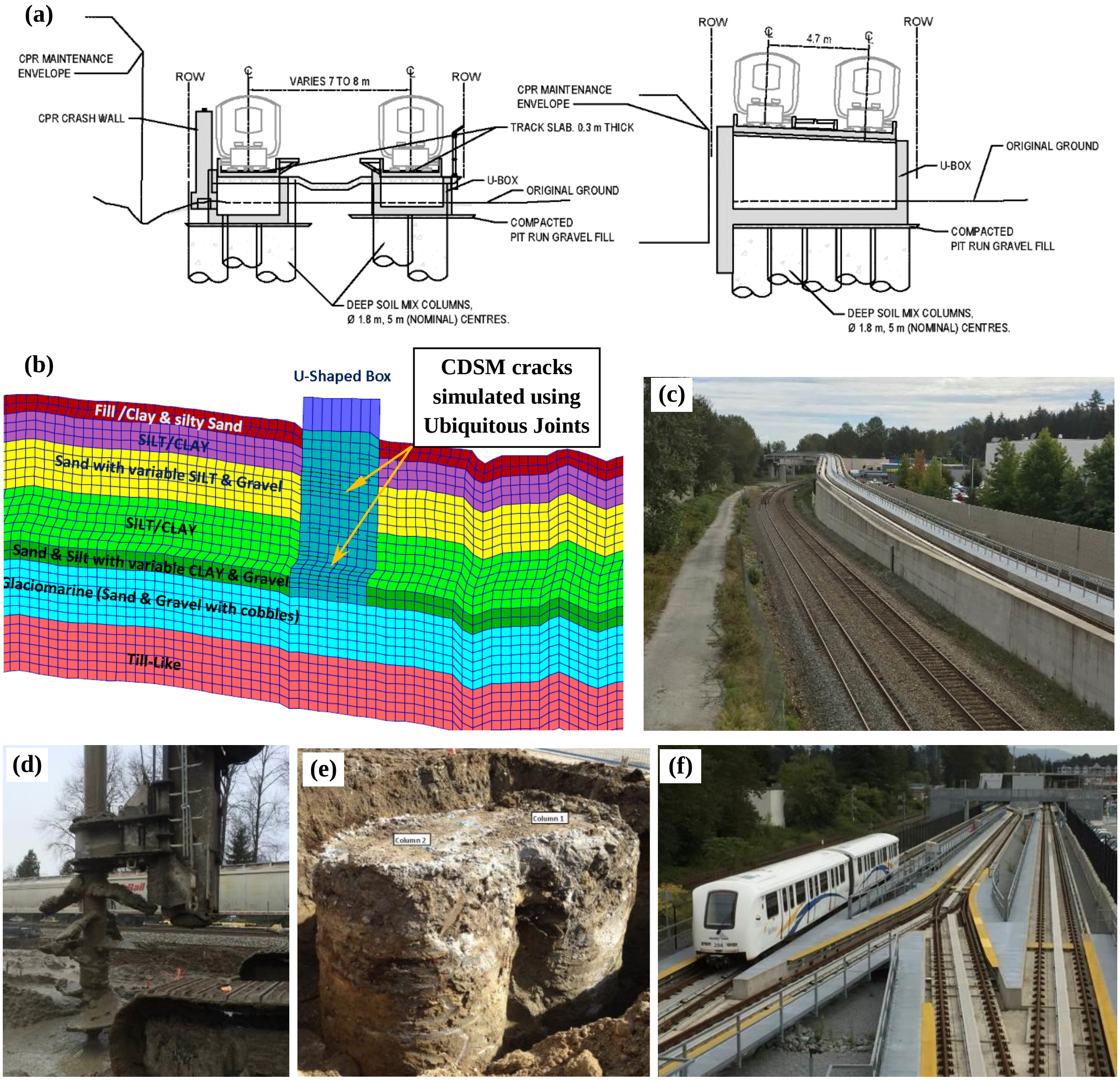

Figure 7. a) Details of CDSM ground improvement, tracks, crash wall and a U-shaped approach structure; b) FLAC model deformed shape, c) transition from at-grade to elevated guideway, d) soil mixing auger, e) exposed hardened CDSM columns; and, f) cross-over at-grade section approaching Moody Centre Station.

6.4 Timber pile ground improvement

A nearly 1 km section of the project runs through the CPR cut (Segment 4, Fig. 2). Along this section, the guideway track slab is founded on MSE embankment supported on ground improved by driving timber piles (Fig. 8). This improvement method (i.e. composite pile-soil system) was selected after analyzing several options including CDSM and vibro-replacement densification by stone columns. One of the key considerations for the use of timber piles was the need to keep their tops below the permanent groundwater level to meet the durability requirements of the PA. Local experience indicates that untreated timber piles installed below permanent groundwater level meet the 100-year design life required by the PA.

The timber piles were unpeeled, untreated, and installed on a 1.2 m square grid over a nominal width of about 10 m. Nominal timber pile length was about 10 m to 15 m. Piles were driven using drop hammers, and cut-off at 0.6 m below the permanent groundwater level. Timber piles were not spliced.

Coupled seismic deformation analyses were performed using the program FLAC and the UBCSAND constitutive model (Figs. 8c, d), and were selected as the design basis (Azizian et al., 2017b). Comparisons were made to analysis results from simplified approaches.

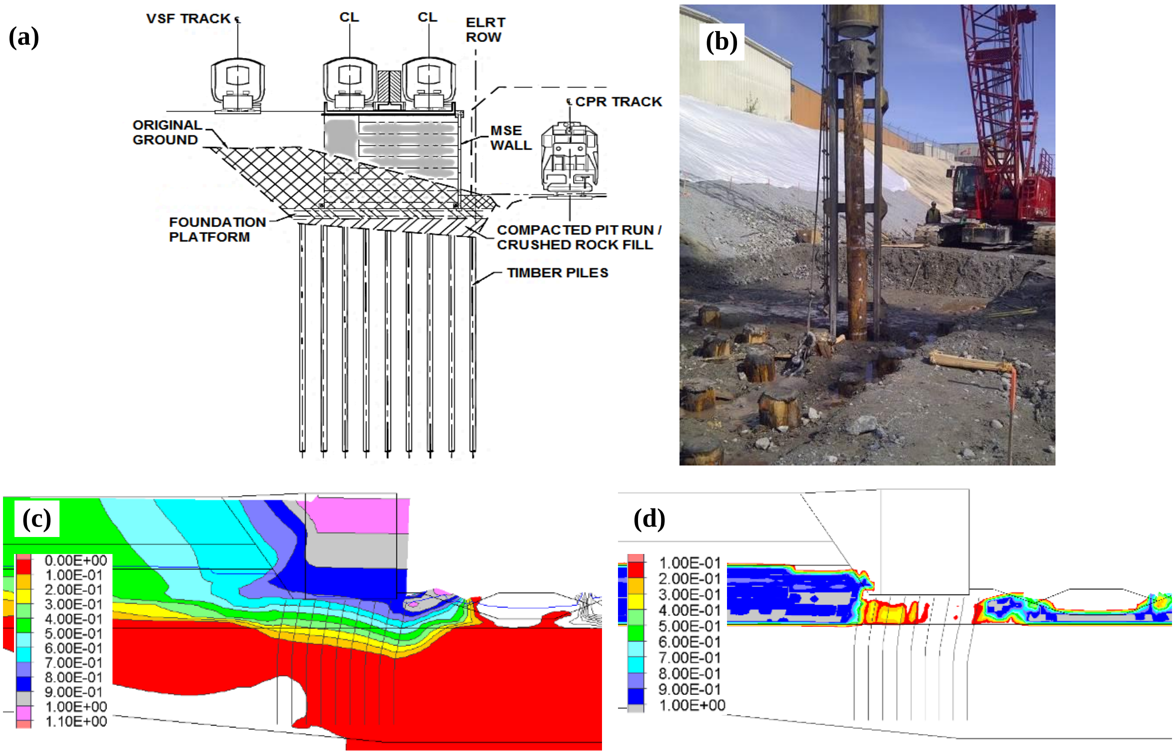

Figure 8. a) Layout of guideway MSE embankment and timber piles, b) timber pile driving, c) FLAC displacements and, d) FLAC excess pore water pressure ratios (liquefied if >0.75).

6.5 Other foundations

Passenger station buildings were supported on the same foundation type as the adjacent guideway, except for the Inlet Centre Station that was built on a ridge of till-like soils. Mat foundations (raft slabs) and strip footings were used for PPS buildings located in liquefiable and non-liquefiable sites, respectively (Brignall et al., 2015). Cut-and-cover tunnel approaches consisted of box structures founded on very dense soils (Chin et al., 2015).

7 TRANSITION ZONES

Transition zones between various structure types (i.e. elevated to at-grade guideway), ground conditions (liquefiable to non-liquefiable), and foundation / ground improvement solutions (e.g. piles to CDSM) required more analytical effort. These efforts focused on estimating guideway displacements for various seismic events and adjusting foundation parameters (e.g. pile size and number/spacing of CDSM columns) to allow for compatible displacements.

8 CONCLUSIONS

Complex and variable ground conditions require a variety of foundation options and/or ground improvement solutions, particularly when satisfying stringent design criteria. A single solution is often not the best solution for the entire alignment of a long, linear project. Ground improvement options are also often limited or unpractical because of their impacts on existing facilities and infrastructure. Alternative solutions and contingency plans should be in place in case preliminary design assumptions related to site conditions or soil type/characteristics/properties change during detailed design or construction.

9 ACKNOWLEDGMENT

We gratefully acknowledge the permission to publish this paper. Many present and former Tetra Tech (EBA) colleagues as well as sub-consultants and sub-contractors were involved in the project during subsurface exploration, design and construction phases. Their contributions are greatly appreciated.

10 IN MEMORIAM – BRIAN HALL (1953-2016)

Brian Hall, Tetra Tech EBA's Chief Engineer and Evergreen project's Geotechnical Manager, passed away on December 25, 2016. Brian was a brilliant geotechnical engineer and mentor. His great depth and breadth of knowledge and nearly 40 years of experience, from sub-Saharan Africa to North America, were unique. His legacy includes over 60 technical papers and many engineers who have been profoundly influenced by his mentorship. He encouraged us to publish our project experience for the engineering community to advance our collective knowledge. May his soul rest in peace.

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