Site Preparation for Vancouver Airport Designer Outlet Centre, Richmond, BC

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Challenges from North to South
Des défis du Nord au Sud

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ABSTRACT
This is a case history of the site preparation for a commercial development near Vancouver International Airport, British Columbia. The low elevation of the site required filling for flood protection from the adjacent Fraser River. The subsurface conditions included soft compressible deltaic deposits that can generate large settlement under fill loads and loose sand-silt that can liquefy under the design earthquake. The site investigation and analysis concentrated on estimating post-construction settlement and assessing soil liquefaction under the design seismic event. A preload was carried out to reduce post-construction settlement. In-depth soil densification was used to mitigate soil liquefaction. The site preparation was complicated by the proximity of a light rail transit and a jet fuel pipeline. The settlement analysis used calibration from historical field monitoring data from a nearby highway embankment. Both CPT and Tests results are presented.

RÉSUMÉ
Cet article présente l'histoire d'un cas de préparation d'un site pour un vaste développement commercial situé proche de l'Aéroport International de Vancouver, Colombie Britannique. La faible élévation du site a nécessité un remblayage afin de le protéger contre les inondations du Fleuve Fraser. Les conditions souterraines de ce site consistent en un dépôt deltaïque profond et compressible pouvant générer un tassement à long terme sous les charges d'un remblai, ainsi qu'un dépôt de sable et de silt lâches risquant de se liquéfier sous le séisme de référence de la région. La reconnaissance et l'analyse du site sont concentrées sur l'estimation du tassement à long terme après la construction et sur l'évaluation du risque de liquéfaction des sols en cas d'événement sismique. Un traitement de pré-chargement a été réalisé afin de réduire le tassement post-construction. Pour réduire le risque de liquéfaction des sols, une densification en profondeur a été choisie. La préparation du site a été compliquée en raison de la proximité d'une ligne de train léger récemment construite pour l'aéroport et d'un pipeline transportant le carburant des avions. Cet article décrit l'analyse du tassement qui a été faite en utilisant des observations historiques. Les résultats des essais CPT et SPT sont présentés.

1 INTRODUCTION

Over the past few years significant advances have been made in the site preparation of sites with deep compressible soils as well as loose and potentially liquefiable sand and silt. However, the post-construction performance of similar sites still requires to be documented in case histories for the benefit of geotechnical engineering practice (Crawford and Morrison, 1996; Ripley, 1995; Crawford et al, 1991). This paper describes a case history of the site preparation that was undertaken for a large commercial retail development located near the Vancouver International Airport, on Sea Island, Richmond, British Columbia.

The site is situated near the west end of Arthur Laing Bridge and north of the Grant McConachie Highway and covers an area of about 33,500 m^2 as shown in Figure 1. The site has an irregular shape and is bounded by a surface parking area and an undeveloped area to the west and south, respectively, and the Fraser River to the north and east. In 2012, at the time of our site investigation, the grade elevation ranged between 1.0 and 2.0 m (geodetic datum). The proposed development consisted of a large number of small two-storey buildings. The top of the floor slab was set at El. 3.25 m to address flood risk from the adjacent Fraser River and the landscape grade surrounding the buildings was set near El. 3.0 m.

2 SUBSURFACE CONDITIONS

Table 1 provides a summary of the generalized subsoil profile, which consisted of compact to dense sand fill overlying stiff to soft compressible clayey silt, overlying loose to compact sand and silt overlying soft and compressible interbedded silt and clayey silt to considerable depth (up to 100 m). The free water level was estimated at about El. -1.3 m with seasonal and tidal fluctuations of the order of 1 m.

3 PRELOADING

Earthworks within the development site included initial filling up to a working grade set at about El. 3.0 m. This was followed by a rolling preload in two phases to El. 5.3 m over building areas and to El. 4.0 m over adjacent paved areas.

Preloading was carried out to reduce, but not eliminate, long-term post-construction settlement of the proposed buildings and adjacent yard areas. The fill area was setback a minimum of 15 m distance from an active jet fuel pipeline, which bisected the site in the southeast to northwest direction.

Settlement survey monitoring during Phase 1 preloading was carried out from January 11, 2013 to June 6, 2013. Figure 1 shows a summary of observed net settlement versus log time during Phase 1 preloading over the building areas after subtracting the short term settlement that occurred during initial site filling. The linear extrapolation of the plotted preload settlement after about 100 days on the log time scale indicates projected long-term settlement ranging from 100 to 325 mm at 10,000 days.

Figure 2 shows that after surcharge removal, the extrapolated post-construction settlement at 10,000 days ranges from 20 to 65 mm assuming that settlements are proportional to the applied load increment.

This simple extrapolation method does not take into account expected deep seated settlement which is not reflected in the short duration preload. Estimated post-construction settlement based on theoretical settlement analysis are of the order of 200 to 300 mm.

Nevertheless, the graphical extrapolation of the Phase One preload data indicated that the projected post-construction settlement was equivalent to differential settlement of less than 1/500 based on the distance between observations points.

Phase One preload fill was removed down to slightly below design grade in June 2013 and rolled into Phase 2 preloading with an overlap of about 15 m between the crests of the two phases. The Phase Two preload data resulted in similar projected post-construction settlement.

4 SETTLEMENT ASSESSMENT OF ADJACENT JET FUEL PIPELINE AND LIGHT RAIL LINE

4.1 Settlement calculations

For the settlement assessment of the jet fuel pipeline and light rail line adjacent to the site, an idealized soil profile and associated geotechnical parameters were derived from available geotechnical test holes within the project site together with laboratory and field data from previous work near the site.

The settlement analysis was carried out using the finite element software Plaxis 2D-2012 available in the public domain. The soil model used in the analysis was verified and calibrated against historical settlement monitoring data collected over a period of 16 years during preloading, construction and post-construction phases of the nearby Grant McConachie highway overpass approach embankment (Leclair et al., 1989). Our soil model overestimated the observed settlement by 10%.

The loading case consisted of filling up to El. 3.0 m for the Templeton Station road extension along the Canada Line with a setback of 3.0 m from the edge of the tracks and filling to El. 5.3 m for the building preload with a setback of 35 m from the tracks. Within the new road alignment, the fill was placed from existing grade to El. 3.0 m with 2H:1V side slopes using Fraser River sand with a unit weight of 18 kN/m^3. Existing grade was at about El. 1.5 m except within a drainage ditch along the Canada Line where it was down to about El. 0.6 m. Preloading was not used prior to construction of the Canada Line. Therefore, Templeton Station road extension was also not preloaded in order to limit potential settlement effect on the Canada Line.

Figure 3 shows the calculated post-construction total settlements along the longitudinal section for durations ranging from 5 to 30 years for the pipeline top of pipe at El. -0.67 m and El. 0.52 m. The longitudinal settlement profiles are fairly uniform considering the distortion between vertical and horizontal scales. The estimated maximum settlement ranges from 375 to 475 mm after 30 years with maximum long-term differential settlement of about 100 mm.

Figure 4 shows the calculated post-construction settlements after removal of the preload for the buildings and adjacent paved areas at El. 1.50 m along the transverse section for durations ranging from 5 to 30 years. The transverse settlement profiles are also uniform and range from 380 to 400 mm after 30 years with maximum differential settlement of about 20 mm.

Figure 5 shows a comparison between calculated post-construction total settlements with time for grade-supported and pile-supported segments of the Canada Line adjacent to the site. Azizian and Robinson (2007) provide details of the Canada Line pile foundation system. The calculated settlement range from 50 to 80 mm after one year to a maximum of 400 to 450 mm after 30 years. Although this range of calculated total settlement appears large, the estimated differential settlement is reasonable. The calculated long-term differential settlement between the In-bound (I/B) and Out-bound (O/B) tracks ranges from 4 mm after one year to a maximum of 11 mm in 30 years. The results indicate potential differential settlement between the grade-supported and pile-supported segments of the Canada Line ranging from 10 mm after one year to a maximum of about 75 mm after 30 years.

4.2 Settlement monitoring

Monitoring points established along the Canada Line were surveyed monthly from January 18, 2013 to April 20, 2015 during the site preparation and building construction period. This was necessary to address concerns about differential settlement between the grade-supported and the pile-supported segments of the rail line.

Precision tilt meters were placed along selected piers to address concerns about rotation of the elevated light rail guide ways.

The survey monitoring was also necessary because of concerns about the effect of temporary dewatering required to install a 12 m deep storm sewer pump station which caused a significant drawdown in the vicinity of the project site.

Figure 6 shows the surveyed settlement versus log time from January 18, 2013 to April 25, 2015 for the monitoring points located along the grade-supported and the pile-supported segments of the Canada Line, respectively. Points labelled 1 to 8 were located from West to East along the grade-supported segments and points labelled 9 to 13 along the pile-supported segments.

The magnitude of rotation measured on the tilt meters was relatively small and did not indicate any cause for concern or remedial action.

In both plots, a marked settlement increase was noted between May and December 2014, which was mainly due to the drawdown caused by temporary dewatering for the deep storm pump installation about 100 m away.

It is interesting to note that this effect is visible for both grade-supported and pile-supported segments. The extrapolation of the observed settlements after dewatering stopped indicated long-term settlement estimates at 10,000 days ranging from 40 to 150 mm.

The profiles of observed and extrapolated settlements shown on Figure 7 indicate that differential settlements along the rail line up to January 16, 2023 (10 years since the start of monitoring) tend to remain within a tolerable limit of 0.1 % provided by the rail line operators. However, it appears that Points No. 3 and 4 within the grade-supported segments may approach this limit at that time.

5 SITE SPECIFIC SEISMIC GROUND RESPONSE ASSESSMENT

One dimensional site specific seismic ground response assessment (SSRA) was carried out using the computer program SHAKE2000 developed by GeoMotions to determine the Spectral Accelerations (S_a) at the ground surface for the site at different periods. The program computes the seismic ground response of a site subjected to vertically propagating shear waves. These waves are propagated from the underlying dense layer (such as bedrock) to the ground surface to simulate earthquake induced shaking. The soil column was assumed to consist of several homogeneous visco-elastic layers of infinite horizontal extent and a half space as the bottom layer.

Each layer is characterized by thickness, mass density, shear wave velocity (V_s), and shear modulus degradation (G/G_0) and damping ratio curves depending on the soil type. Non-linearity of the soil is accounted for by the use of an equivalent linear constitutive model which obtains values of soil shear stiffness and damping compatible with the effective shear strains in each soil layer through an iterative procedure.

As indicated by Athanasopoulos-Zekkos et al. (2013), the use of a one-dimensional soil column with equivalent-linear stress-strain properties in the program SHAKE2000 is considered adequate since the peak ground acceleration of the input motions (PGA input) applied at the base of the model is less than 0.5g.

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The design V_s profile in the upper 27 m was obtained from site-specific V_s measurements conducted in SCPT12-02. Below 27 m depth, the design V_s profile was obtained from two sources: a) published V_s data collected by the Geological Survey of Canada (GSC) in the vicinity of the Arthur Laing Bridge (Borehole FD90-1) and b) estimation of V_s values using the statistical relationship proposed by Hunter et al. (1999). Figures 13 shows one of the soil columns used in the assessment together with relevant input parameters versus depth.

For analysis purposes, the shear wave velocity of the Pleistocene deposits at depth was linearly interpolated between the bottom of the deep normally consolidated marine clayey silt deposit and the base of the model. A constant shear wave velocity of 760 m/s was assigned to the bedrock.

The calculations were completed using available published shear modulus degradation (G/G_0) and damping ratio (φ) curves. For the Holocene deltaic deposits, the average G/G_0 and φ curves by Seed & Idriss (1970) for sand and those by Vucetic and Dobry (1991) dependent on plasticity index (PI) were used for sand/silty sand and clayey silt layers, respectively. For the Pleistocene deposits and underlying bedrock the G/G_0 and φ curves for gravel by Seed et al. (1986) and EPRI (1993) Rock 4 curves (121 to 250 feet) were used, respectively.

The 2007 Task Force Report on geotechnical design guidelines for buildings on liquefiable sites in accordance with NBC 2005 for Greater Vancouver Region provides recommendations for the selection of the plasticity index (PI) values for fine-grained sediments in the Fraser River delta. Based on the thickness of marine fine-grained deposits used in our analyses and the guidelines in the 2007 Task Force Report, an average PI of 15 was selected for the clayey silt below 20 m depth.

The input motions comprised three records with two orthogonal components each from the Chi-Chi, Joshua Tree and Loma Prieta earthquakes, and one record with one component from the San Fernando earthquake (CALTECH B - San Fernando Dam). Prior to the calculations, a frequency wave baseline drift correction was applied to each modified input motion to correct the displacement history.

Figure 8 shows the response spectra for 5% damping at the ground surface for the 2,475-year return period seismic event for the various soil columns and V_s profiles used. For comparison purposes, the response spectra from NBCC 2010 Site Class E, D and C are also shown together with the recommended design envelope.

The spectral acceleration at zero period, considered as the peak ground acceleration (PGA), was found to range from 0.25g to 0.32g with an upper bound average of 0.38g. PGA values within this range were used for soil liquefaction assessment and for calculating seismic earth pressures on basement walls.

6 SOIL LIQUEFACTION ASSESSEMENT AND MITIGATION

Using current simplified methods of soil liquefaction analysis and the PGA values from our site specific seismic ground response analysis, our assessment of recent CPTs indicates that a 2475 year earthquake can trigger soil liquefaction in pockets below the groundwater level to a depth of 20 m. Consequently, sand boils and lateral and vertical displacements can occur at the ground surface.

Table 2 provides estimated post-seismic vertical and lateral soil displacements at the ground surface for the selected PGA values under the design earthquake. Without ground improvement by soil densification, the post-seismic reconsolidation settlement can range from about 200 to 400 mm while lateral displacements can range from about 60 to 230 mm at a setback distance of 50 m from the river bank. The above post-seismic soil displacements can cause estimated differential settlements ranging from 100 to 200 mm over a distance of about 10 m, assuming that differential settlement is equal to half of the total settlement.

The above estimated settlements are calculated at the ground surface for free-field conditions, which means that the effect of building loads is not taken into account. Post-seismic settlement under the buildings can be higher than estimated on Table 2 due to the dynamic effect of gravity and seismic loads of the buildings.

Ground improvement by in-depth densification of the potentially liquefiable soils is typically used to reduce post-seismic vertical and horizontal soil displacements to within limits tolerable for the proposed buildings and the associated infrastructure such as access roads and underground utilities.

Under the design earthquake and without ground improvement, the granular fill layer and the upper clayey silt layer did not provide sufficient resistance against punching of the foundation pads and strip footings into the underlying liquefiable sand. This was due to the limited thickness of compacted granular fill layer above the water table and the low strength and limited thickness of the upper clayey silt layer. Also, the underlying liquefied sand layer had a very low post-seismic residual strength.

Under the design earthquake, a flow slide may occur along the Fraser River bank with lateral displacements of the order of 3 m. However, analysis indicated that the flow slide was limited to within 30 m from the river bank and was not expected to affect the proposed buildings given that the proposed development was set back at least 50 m from the flood protection dike.

At 50 m from the river bank, analysis indicated that estimated lateral displacements would range from about 60 to 230 mm if ground improvement was not carried out. As ground improvement has not been carried out within the flood protection dike area, it was likely that the dike structure would be damaged during the design earthquake. However, the dike embankment is expected to be relocated and upgraded in the future in view of expected rising design flood levels.

To reduce post-seismic displacements, ground improvement of the underlying soil layers that we considered susceptible to liquefaction was recommended within the building and adjacent access areas. In-depth soil densification using vibro-replacement was considered to be the most effective ground improvement method.

Table 3 provides the target CPT tip resistance to prevent soil liquefaction. Vibro-replacement stone columns with 900 mm nominal diameter laid out in a triangular grid pattern at 3 m axis-to-axis horizontal spacing were considered adequate. The depth of soil densification depended on the range of estimated post-seismic settlement that can be tolerated for the proposed buildings. Ground improvement for previous projects on Sea Island has typically been carried out to depths ranging from 12 to 15 m.

For this project, the ground improvement was carried out to a minimum depth of 10 m below top of floor slab design elevation and extended a minimum distance of 8 m beyond the perimeter of buildings. The ground improvement also covered the plaza areas between the buildings and extended 8 m beyond the perimeter of these areas.

The stone columns extended from about El. –7.35 m to the working surface which was set at about El. 2.75 m. Adequate sedimentation control measures were in place during ground improvement work to contain liquid spoils during in-depth densification.

Figure 9 shows a typical plot of static cone penetration tests (CPT) tip resistance Q_t before and after soil densification at the same location. The results of the CPT carried out after soil densification showed that the tip resistance Q_t exceeded the required values for clean sand zones, where the friction ratio R_f was less than 0.7% and fines content was less than 5%. However, in silty sand or sandy silt zones where the friction ratio R_f was higher than 0.7% and fines content was higher than 5%, the CPT tip resistance after soil densification was generally less than the required value. This shortcoming was attributed to the limitation of the vibro-replacement soil densification method in fine-grained soil layers.

In an attempt to evaluate the effect of this limitation on the site response to seismic loading after soil densification, the post-seismic settlement was calculated for the peak ground acceleration relevant to this site. These calculations used the semi-empirical integrated approach proposed by Zhang, Robertson & Brachman (2002) based on CPT data for level ground sites.

It should be noted that this method is based on limited data available from only two sites that have experienced soil liquefaction during major earthquakes. A comparison between calculated post-seismic settlements and actual measurements after earthquakes indicated that this method is not accurate and generally predicts settlements with an error of about 20 to 50%. Due to the limited historical data base, larger deviations than indicated above between predicted and actual post-seismic settlement performance should be expected.

The post-seismic settlements for the design peak ground acceleration (PGA) of 0.24g were calculated at each test hole completed after densification. Table 4 provides of summary of estimated post-seismic settlement at the ground surface under a design earthquake of moment magnitude M of 7.0 and PGA of 0.24g. The results indicated that the calculated post-seismic settlements would range from 90 to 180 mm. As this was a level ground site, horizontal soil displacements were expected to be relatively small (less than 150 mm) within the densified areas. However, lateral soil spreading could still occur within non-densified areas along the river bank or open drainage channels.

Figure 10 shows a typical plot of SPT blow counts corrected to 60% hammer energy efficiency before and after soil densification at the same location. Generally, after soil densification, the SPT indicated higher blow counts within clean sand layers but sometimes a decrease in blow counts within silty sand layers.

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Table 1 Generalized subsoil profile

Deposit	Depth (m)	Consistency or Density
Sand (fill)	0 - 1.5	Compact to dense. Uniform gradation.
Clayey silt (crust)	1.5 - 3.0	Stiff, compressible.
Silty sand, sandy silt	3.0 - 5.0	Loose. Interbedded.
Sand	5.0 - 20	Loose to compact. Sandy silt seams.
Clay, silty clay, silty sand, sandy	20 - 30	Soft, compressible. Interbedded.

Table 2 Calculated post-seismic displacements at the ground surface without soil densification

Location	Estimated Displacement (mm)			
	PGA = 0.24g		PGA = 0.35g	
	Vertical	Lateral	Vertical	Lateral
CPT12-01	216	160	332	186
CPT12-02	211	61	313	70
CPT12-03	316	163	403	227

Table 3 Target static cone penetration tip resistance Q_c (MPa) for in-depth soil densification

Depth (m)	Clean Sand Fines less than 5% or Friction ratio R_f less than 0.5%	Silty Sand Fines between 5 and 15% or Friction ratio R_f between 0.5% and 1.5%
1.0	5.6	3.3
2.0	6.3	3.8
3.0	7.2	4.5
4.0	8.3	5.2
5.0	9.3	5.8
6.0	10.1	6.4
7.0	10.8	6.8
8.0	11.5	7.2
9.0	12.2	7.6
10.0	12.7	8.0

Table 4 Post-seismic ground surface settlement calculated from CPTs before and after soil densification

CPT Before Soil Densification	CPT After Soil Densification	Settlement Before Soil Densification (mm)	Settlement After Soil Densification (mm)
CPT12-01	CPT13-08	216	93
CPT12-02	CPT13-06	211	112
CPT13-10	CPT13-14	185	126
CPT13-11	CPT13-15	246	144
CPT13-03	CPT13-13	171	156
CPT13-04	CPT13-12	214	184

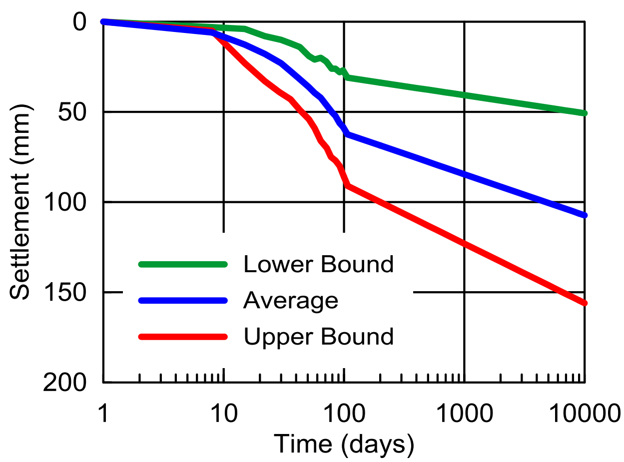

Figure 1 Typical preload observations and extrapolations

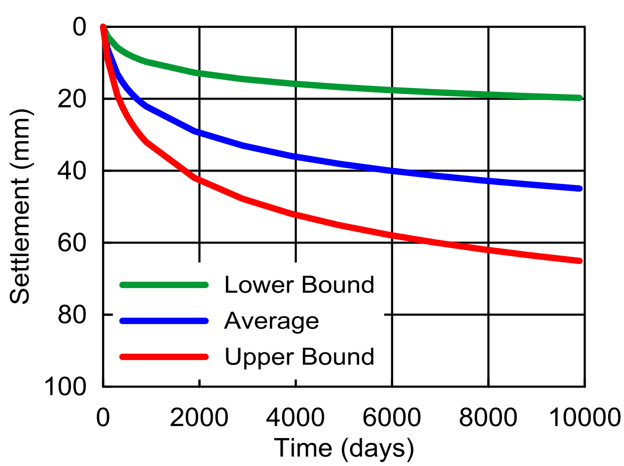

Figure 2 Estimated settlement after preloading excluding deep seated component

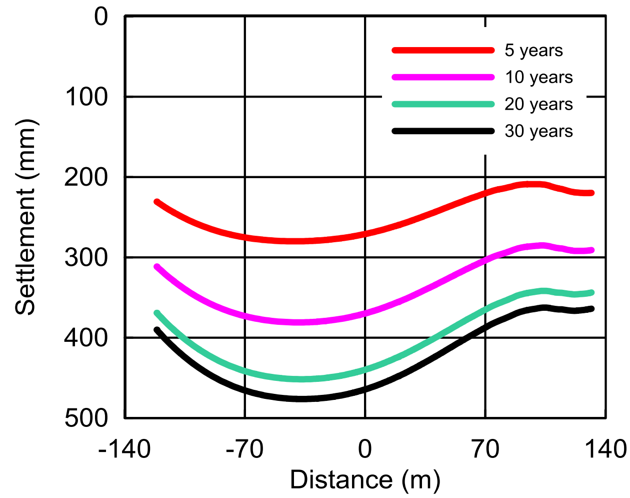

Figure 3 Calculated longitudinal settlement profiles along relocated jet fuel pipeline adjacent to the site

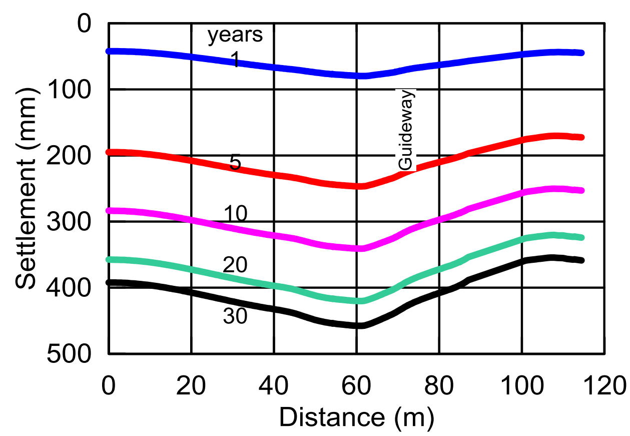

Figure 4 Calculated transverse settlement profiles across Canada Line adjacent to the site

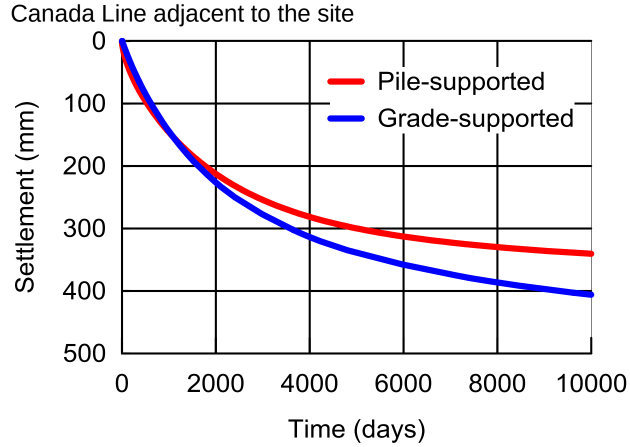

Figure 5 Calculated post-construction settlement versus time for pile-supported and grade-supported segments of the Canada Line adjacent to the site

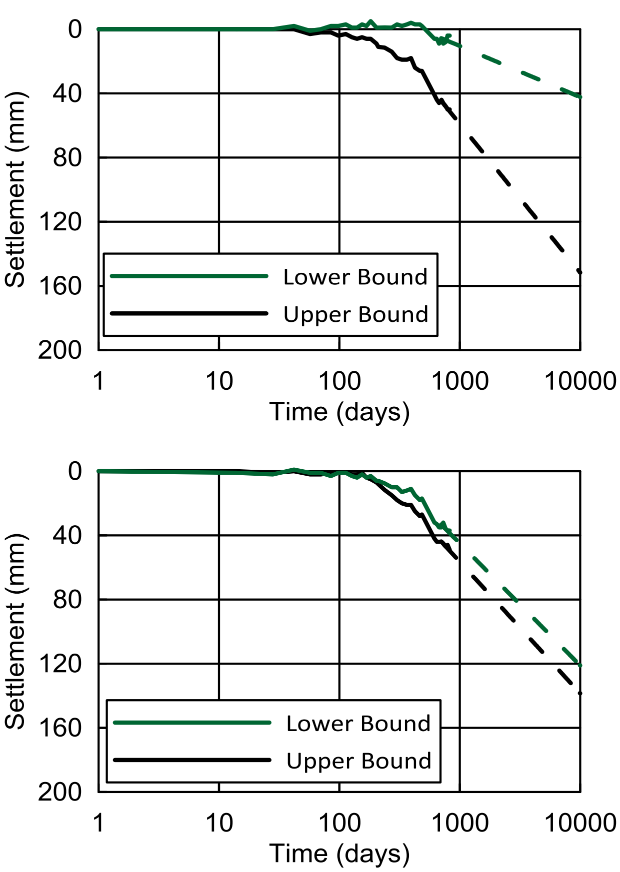

Figure 6 Measured and extrapolated settlement versus log time on the Canada Line for grade-supported segments (top) and pile-supported segments (bottom)

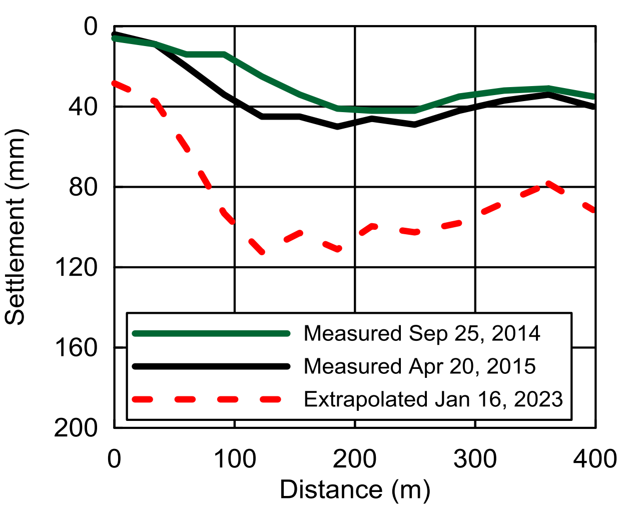

Figure 7 Measured and extrapolated settlement profiles along the Canada Line adjacent to the site

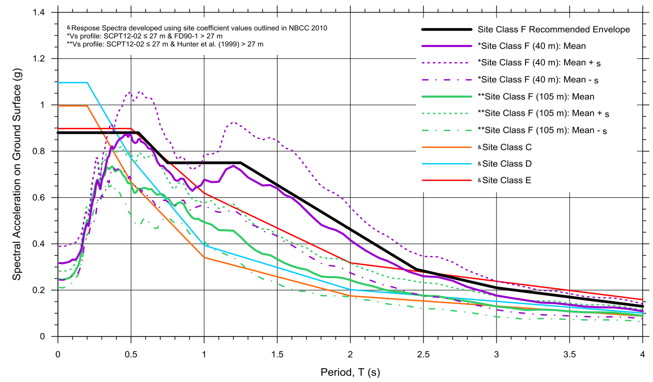

Figure 8 Spectral acceleration for 5% damping ratio versus period

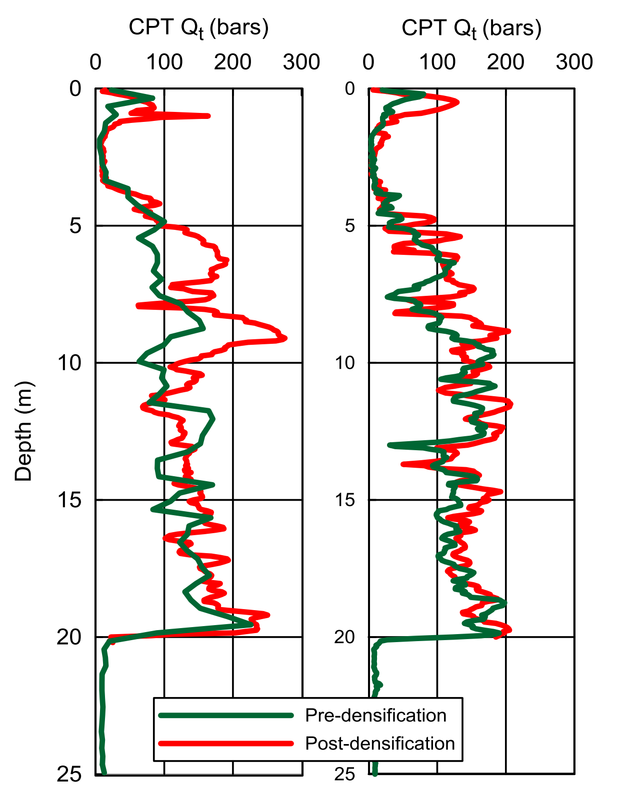

Figure 9 Tip Resistance Q_t versus depth from CPTs before and after in-depth soil densification

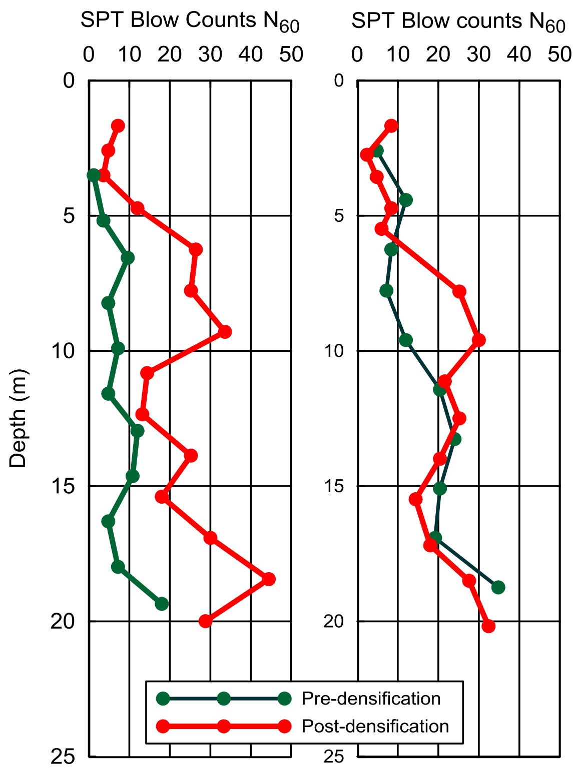

Figure 10 Energy corrected blow counts N_{60} versus depth from SPT before and after in-depth soil densification