Case history of pile supported highway embankments on very soft and compressible soils

(Uthaya) M. Uthayakumar
Stantec Consulting Ltd., Burnaby, BC, Canada
Michael Oliver
BC Ministry of Transportation and Infrastructure, Victoria, BC, Canada

ABSTRACT
This paper describes the design and construction of pile supported highway embankments on very soft and compressible soils. This case history is from the South Fraser Perimeter Road project, located in Delta and Surrey, British Columbia. The subsurface conditions along the road alignment included highly compressible soils, consisting of peat, very soft silts and clays, thickness varying from less than 10 m to more than 100 m. Groundwater table was located at or within a few metres of the ground surface. Soft soil treatment methods included preloading, use of light-weight fill and use of driven piles to support the embankments. Design of the pile supported embankments included selection of pile type, method of installation, axial capacity, load transfer mechanism from the embankments to the piles to the deep soil strata and seismic response. The design work included simple limit-equilibrium analyses to the more advanced soil-structure interaction analyses. Instrumentations to monitor the performance of the embankments included settlement gauges, piezometers and slope inclinometers. The instrumentation data was used to confirm or modify the final design recommendations, preload treatment duration and the predicted post-construction settlement. Details of the design, construction, instrumentation, monitoring and analysis of the monitoring data are presented.

RÉSUMÉ
Cet article décrit la conception et la construction de remblais de routes supportées par des pieux sur des sols très mous et compressibles. Cette histoire de cas provient du projet South Fraser Perimeter Road, situé à Delta et à Surrey, en Colombie-Britannique. Les conditions de subsurface le long du tracé de la route comprenaient des sols très compressibles, constitués de tourbe, de limons et d'argiles très mous, d'une épaisseur variant de moins de 10 m à plus de 100 m. La nappe phréatique était située à quelques mètres de la surface du sol. Les méthodes de traitement du sol souple comprenaient le préchargement, l'utilisation d'un remblai léger et l'utilisation de pieux battus pour soutenir les remblais. La conception des remblais soutenus par les pieux incluait la sélection du type de pieux, la méthode d'installation, la capacité axiale, le mécanisme de transfert de charge des remblais aux pieux jusqu'aux couches profondes du sol et la réponse sismique. Le travail de conception incluait de simples analyses d'équilibre limite aux analyses d'interaction sol-structure plus avancées. Les instruments de surveillance de la performance des remblais comprenaient des jauges de sédimentation, des piézomètres et des inclinomètres de pente. Les données d'instrumentation ont été utilisées pour confirmer ou modifier les recommandations de conception finales, la durée du traitement de précharge et le tassement prévu après la construction. Les détails de la conception, de la construction, de l'instrumentation, de la surveillance et de l'analyse des données de.

1 INTRODUCTION

This paper presents a case history on the design, construction and monitoring of highway embankments on soft and compressible soils. This case history is from the South Fraser Perimeter Road (SFPR) project, located in Delta and Surrey, British Columbia.

The SFPR project includes the design, construction, finance and operation of approximately 40 km of a new 80 km/hr four-lane highway extending from Deltaport Way in Delta to 168 Street in Surrey, British Columbia. Figure 1 shows the location of the project by the red lines. The duration of the design and construction phase of the project was from 2009 to 2013.

The project was delivered through a Public-Private Partnership program. The SFPR project also included design and construction of four interchanges, about 30 overpass structures and the associated approach embankments. As topography varied significantly across and along the road alignment, several kilometers of retaining walls were required to retain the road embankment fills within the property limits.

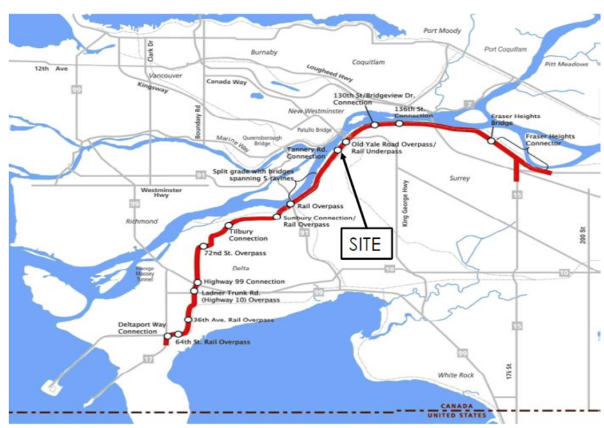


Figure 1. Location of the project site

The subsurface conditions along the road alignment included highly compressible soils, consisting of peat, very soft silts and clays, thickness varying from less than 10 m to more than 100 m. Groundwater table was located at or within a few metres of the ground surface.

Techniques used to minimize post-construction settlement of the road embankments included preload treatment, excavation of compressible soils and replacement with granular fill, use of light-weight fills and constructing the embankments on pile foundations.

Figure 1 shows the location of a site, known as Tannery Interchange where pile supported embankments were constructed. Driven timber piles, 15 m in length were used. After driving, the piles were cut-off below the water table and then piezometers were installed, followed by the construction of a 2 m thick gravel layer reinforced with geogrids. Settlement gauges and slope inclinometer casings were then installed and the embankments were constructed.

The embankments with no pile support were constructed and preloaded with 2 to 3 m thick surcharge. Surcharge is defined as a specified extra thickness of earthfill placed above the design road elevation. Thickness of the surcharge was designed based on the subsurface soil conditions, height of the permanent embankment, available time for preload treatment, stability and bearing capacity considerations, predicted settlement during preload treatment and post-construction settlement tolerance. River sand was used as fill for both, permanent embankment and surcharge. After the preload treatment, the sand fill was removed to the underside elevation of the pavement gravel, followed by construction of the pavement, consisting of sub-base and base courses, and an asphalt layer.

Instrumentations to monitor the performance of the preload treatment included settlement gauges, piezometers and slope inclinometers. The instrumentation data was used to confirm or modify the final design recommendations, treatment duration and the predicted post-construction settlement.

2 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

The surficial geology of the region where the site is located has been mapped by Armstrong and Hicock (1980). Their Surficial Geology Map No. 1484A describes the subsurface soils at this site as "*bog, swamp and shallow lake deposits: lowland peat up to 14 m thick, in part overlying Fraser River sediments*".

The geotechnical exploration program to assess the subsurface soil conditions consisted of electric Cone Penetration Test (CPTu) holes, measurement of shear wave velocity of the soil layers, boreholes with Standard Penetration Tests, measurement of field vane shear strengths, auger holes and test pits.

2.1 Subsurface soil and groundwater conditions

The generalized subsurface soil profile at Site 1 consists of the following units in the order of increasing depth:

- Fill consisting sand and gravel within the footprint of previous roadways, 0.9 m to 3.8 m thick. Wood waste (hogfuel), 0 to 1.4 m thick was found outside the previous roadways. Ground surface elevation at the time of borehole exploration work varied from 1.5 m to 3.5 m.
- Very soft to firm amorphous to fibrous peat, 1.7 m to 4 m thick. Natural moisture content of the peat varied from 300 to 800%. Bottom of the peat layer was about 3 to 5 m below the ground surface, at an approximate elevation of -1.5 to -3.5 m.
- Very soft to soft to firm organic SILT, clayey SILT to silty CLAY, 7.5 m to 10.5 m thick. Natural moisture content of this soil unit varied from 30 to 90%, plastic limit varied from 30 to 50%, liquid limit from 40 to 120% and plasticity index varied from 12 to 55. Bottom of this soil unit was about 13 m to 15 m below the ground surface, at an approximate elevation of -12 m.
- Compact to dense SAND with thin silt interbeds. Thickness varied from 13 m to 21.5 m. Bottom of this sand layer was about 35 m below the ground surface, at an approximate elevation of -32 m.
- Interlayered clayey silt to silty clay, thickness greater than 15 m. This soil unit extended below the bottom of the 50 m deep boreholes.

Elevation of the groundwater table was 0.9 m to 1.8 m and the ground elevation at the site prior to any fill placement was about 1 m to 1.5 m. All elevations presented in this paper are referenced to the geodetic datum.

The results of a typical CPTu completed at the site are shown in Figure 2.

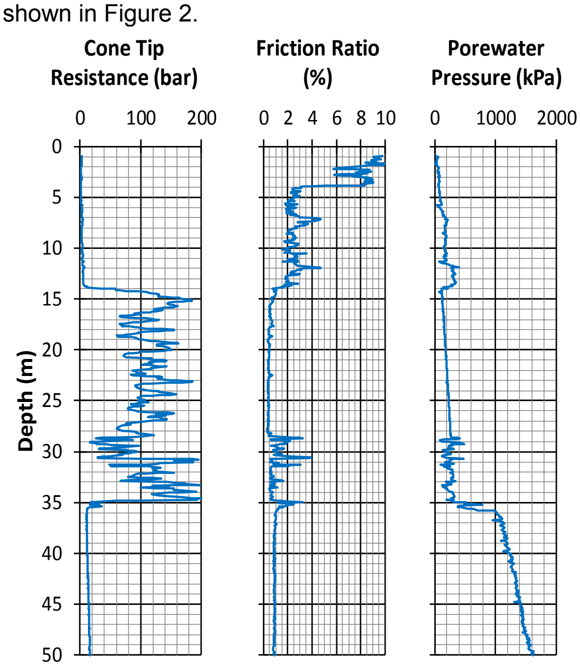


Figure 2. Results of a typical cone penetration test at the site.

3 THE INTERCHANGE AND THE EMBANKMENTS

The Tannery Interchange includes two overpass structures (one over the SFPR and the other over two railway lines) and seven approach embankments. Figure 3 shows an aerial view of the site. The main SFPR embankment is denoted as L2000 in Figure 3 and the approach embankments are denoted as L730, L750N, L750S, L760, L770 and L780. The embankment between the two overpass structures is referred to as L750. The L750N, L750, L760 and L770 embankments are retained by MSE walls. Wick drain installation at 1.5 m triangular spacing to an elevation of -11 m was completed for all embankments except for the L2000.

Foundations for the two overpass structures consist of driven steel pipe piles. Design details of the overpass foundations and the MSE walls are outside the scope of this paper.

Figure 3. Aerial view of the site showing the embankments and overpass structures (Modified from Bing Aerial Imagery)

3.1 L2000 embankment - SFPR

The design road elevation of the L2000 embankment varies from 3.3 m to 4.2 m. The design included preload treatment with a surcharge for this embankment. Thickness of the surcharge was 2 to 3 m (i.e.: thickness of fill above the design road elevation).

3.2 L760 embankment – north on-ramp

The L760 embankment includes the following two sections:

- a 230 m long section in which the design road elevation increases from 3.8 to 10.5 m. This section was preloaded with surcharge. Thickness of the surcharge was 2 m;
- a 50 m long section where the road elevation increases from 10.5 to 13.5 m. This section was designed to be supported on timber piles. No surcharge was required for this section.

The embankment was designed to be retained by MSE walls on both sides along the longitudinal direction. The design included stabilization piles driven to the immediate north of the embankment, driven prior to the construction of embankment and MSE walls. The stabilization piles are 16 m long untreated timber piles, driven in three rows at 1.5 m equilateral triangular spacing. In addition, a toe berm consisting of sand fill, 1 to 4 m wide was constructed to elevation 4 m. The purpose of the stabilization berms and piles are to increase the factor of safety against slope failure and to minimize the magnitude of lateral spreading as discussed later.

3.3 L770 embankment – north off-ramp

Similar to the L760, the L770 embankment includes the following two sections:

- a 240 long section in which the design road elevation increases from 3.8 to 9.2 m. This section was preloaded with surcharge. Thickness of the surcharge was 2 m;
- a 75 m long section where the road elevation increases from 9.2 to 13.5 m. This section was designed to be supported on timber piles. No surcharge was required for this section.

A single row of 16 m long untreated timber piles was driven to the immediate north of the embankment to improve stability.

3.4 L750N, L750 and L750S embankments – Tannery Road

Tannery Road consists of two overpass structures, one over the SFPR and the other over the existing two sets of railway tracks. The L750N, located north of the railway tracks is approximately 180 m long and its elevation increases from 2 to 11 m. The first 100 m length of the embankment, elevation from 2 to 6.5 m was preloaded with 2 m thick surcharge. The last section of L750N, 80 m in length and elevation from 6.5 m to 11 m was designed to be supported on timber piles. No surcharge was required for this section supported on piles.

The L750 embankment between the two overpass structures is approximately 30 m long and the road elevation is approximately 12 m. The section of the embankment is supported on timber piles and no surcharge is required.

The L750S embankment is approximately 340 m long with side slopes at 2H:1V. Elevation of the first 200 m long section increases from 2 to 4.6 m and for the last 140 m length the elevation increases from 4.6 to 11 m. No treatment of the subgrade was carried out for the first 200 m length as there was an existing road, which could not be closed for any treatment work. Preload treatment with 2 m thick sand surcharge was attempted for the last 140 m long section, but the surcharge had to be removed prior to completion of primary consolidation due to schedule constraints. The L750S is not supported on piles.

3.5 L730 and L780 embankments – south ramps

The design road elevation varies from 3.6 to 10 m with side slopes at 2H:1V. Preload treatment with 2 m thick sand surcharge was attempted, but the surcharge was removed prior to completion of primary consolidation due to schedule constraints. The two embankments are not supported on piles.

4 GEOTECHNICAL DESIGN

Geotechnical design considerations of the embankments included the following:

- Project schedule;
- Stability of embankments during construction;
- Stability of the permanent embankments after construction;
- Post-construction long-term settlement and;
- Performance during and after the design seismic events.

A number of options including treatment of subgrade, embankment fill types and structures were considered during preliminary design. Only the final design option used for construction are discussed in this paper.

4.1 Seismic design

The seismic design criteria used for this project was performance based and are similar to the provisions of the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (The project was completed in 2013, before the release of CAN/CSA-S6-14). The structures and the approach embankments described in this paper were designed to a criterion similar to the "Major Route Structures", described in CAN/CSA-S6-14.

Seismic ground motion parameters and design earthquake records were derived using the 4^{th} Generation Seismic Hazard Model developed by the Natural Resources Canada. The Cascadia subduction zone earthquake motions were also included in the design and analyses.

Liquefaction is predicted to occur within the compact sandy layers, located about 15 m depth below the natural ground surface. Peat and clayey SILT within the top 15 m are not susceptible to liquefaction. The compacted embankment fills and soils above groundwater table are also not considered to be susceptible to liquefaction. The effects of liquefaction were addressed in the design of embankments and foundations so as to meet the project design criteria.

4.2 Settlement and stability analyses

Details of settlement analysis are described in Uthayakumar and Oliver (2017) and are not repeated here. Preliminary analysis showed the embankments constructed using sand fill and with no pile support could settle more than 2 m. However, slope stability analyses of the north embankments (L750N, L750, L760 and L770) over 6 m high showed factor of safety (FOS) less than 1.0 under static loading conditions for the soil conditions near the proposed structures. Options such as toe berms, flatter slopes or toe stabilization piles did not improve the conditions considering the limitation of available space. The options of light-weight fill embankments and pile supported embankments were evaluated and timber pile supported embankments were selected for final design.

The area south of L2000 was used as a stockpile site of sand fill for several months, resulting in an improved subgrade. Slope stability analysis of the embankments L730, L750S and L780, with side slopes of 2H:1V showed satisfactory results, minimum FOS of 1.5 under static loading conditions and meeting the seismic deformation tolerances.

4.3 Pile supported embankments

As noted in the previous subsection, the L750 and parts of the L750N, L760 and L770 embankments were designed to be supported on piles. The purpose of the piles is to improve the slope stability and to reduce the post-construction settlement within acceptable limits of the design criteria. The options of driven precast concrete and timber piles with and without pile caps were analyzed. Untreated, unpealed timber piles with no pile cap were selected for the final design. Douglas Fir and Hemlock timber piles were considered in the design and Hemlock piles were selected for production.

Design of the timber pile supported embankments were carried out using the procedures of Filz et al (2012). Details of the design are summarized as follows:

- The piles were designed to be driven at a square grid pattern at 1.4 m spacing and to a tip elevation of -12 to -15 m, 1 to 4 m embedded into the dense sand layer.;
- The top of the piles was cut off at elevation 0, below the water table;
- Piles were minimum 330 mm in diameter at cut-off elevation and 180 mm diameter at the tip;
- Unfactored ultimate axial resistance of the timber piles below the embankment is 575 to 775 kN;
- Vertical Stress from a 10 m high embankment is estimated to be 180 to 200 kPa, which results in axial loading less than 0.5 of the capacity of piles;
- The minimum required thickness of the "load transfer platform (LTP)" to transfer the embankment load to the piles with no pile cap was estimated using Filz et al (2012) as 1.6 m.
- A 2 m thick LTP, from elevation 0 to 2 m was selected for the design. The LTP is a gravel layer consisting of 75 mm minus well-graded granular fill and geogrid reinforcement. The tensile load on the reinforcement layer was estimated using Filz et al (2012) as 280 kN/m;
- The design was with no pile cap but with geogrid reinforcement within the LTP. One layer of biaxial geogrid and four layers of uniaxial geogrid in each of two orthogonal directions were selected. The geogrid layers are continuous in the horizontal

direction and are at 200 mm vertical spacing. The allowable tensile load on the biaxial and the uniaxial geogrid is 17.5 and 75 kN/m respectively.

- With the above design, the calculated settlement of the timber piled embankment, after completion of construction, is in the range of 75 to 100 mm.
- Slope stability analysis of the timber piled embankment show factors of safety more than 1.5 and 1.1 for static and seismic loading conditions respectively.

4.4 Soil-structure interaction and seismic deformation analysis

Soil-structure interaction analysis of the pile supported embankments included analysis of axial and lateral loading response of the piles and deformation analysis of the entire system of embankment, LTP, geogrids, piles and the subgrade. Both, static and seismic loading conditions were analysed. The computer software FLAC (ITASCA, 2008) was used for the soil-structure deformation analysis. This analysis is referred to as "FLAC analysis" in the proceeding sections. The dynamic (seismic) analysis in FLAC were carried out by incorporating the "total-stress" sub-routine "UBCTOT", version 7f, developed by Beaty and Byrne (2008).

The FLAC analysis was completed for a transverse section of the L750N embankment. One side of the embankment is retained by a vertical MSE wall and the other side by a reinforced soil slope, inclined at 70 degrees to the horizontal direction. The MSE walls and the reinforced soil slope are reinforced with uniaxial geogrids with an ultimate tensile strength of 70 to 114 kN/m.

The FLAC analyses included the following steps:

1. Model ground surface and groundwater table. Solve and allow the model to come to equilibrium under gravity;
2. Model the excavation to elevation zero;
3. Model the timber piles after being driven;
4. Model the installation of the LTP with geogrid reinforcement. Simulate the construction sequence by adding fills in 0.5 m increment, adding geogrids and solving before adding the next lift of fill and geogrid;
5. Model the construction of the MSE walls in steps. Simulate the construction sequence by adding fills in 0.5 m increment, adding geogrids and solving before adding the next lift of fill and geogrid;
6. Allow the model to come to equilibrium under gravity immediately after each of the above noted items 1 to 3 and after each step in items 4 and 5. Monitor the deformation and stresses;
7. With the stress and strain states known under gravity (static) loading condition, start the dynamic loading (earthquake shaking) by applying the input motion at the base of the soil model. The dynamic analysis is performed in time domain. Deformation and forces within the model during the shaking are monitored and saved in files.

Soil parameters used in the analyses were derived from in-situ tests and laboratory tests on representative soil samples. The soil parameters are summarized in Table 1.

The timber piles were modelled as "pile elements" and the geogrids were modelled as "strip elements" as defined in FLAC. The facing of the MSE walls were modelled using cohesion C, linearly increasing from 50 to 100 kPa from the top to the bottom of the wall. Structural parameters used in the model for the pile and strip elements are provided in Tables 2 and 3 respectively.

Table 1. Soil parameters used in the analysis

Soil Description	Density (kg/m^3)	Shear Modulus, G, (MPa)	Friction Angle (degree)	Cohesion, C (kPa)
Pavement gravels	2000	10	40	0
MSE wall backfill	1850	13 to 39[a]	35	0
LTP Gravel Fill	2000	43 to 47	40	0
Silty sand fill	1900	15 to 25	33	0
Peat	1100	3	0	25
Silt	1750	5 to 9	0	20 to 33
Sand	1850	20 to 23	35	0

[a] G = 13MPa immediately below the pavement gravels and then linearly increases to 39MPa at the bottom of the embankment.

Table 2. Structural parameters used for pile elements

Pile Diameter (mm)	Density (kg/m^3)	Elastic Modulus (GPa)	Pile Spacing (m)
330	900	7	1.4

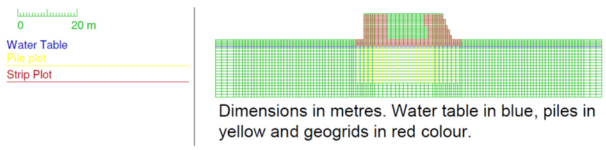
Table 3. Structural parameters used for strip elements

Parameter	Uniaxial	Uniaxial	Uniaxial	Biaxial
Tensile ultimate strength (kN/m)	70	114	175	30
Tensile yield strength (kN/m)	63	103	158	27
Nominal thickness, (mm)	0.338	0.544	0.910	0.461
Density, (kg/m^3)	950	950	950	900
Elastic modulus (GPa)[b]	2.14	2.28	2.03	1.12
Initial apparent interface friction coefficient, f_o	0.8	0.8	0.8	1.0
Minimum apparent interface friction coefficient, f_1	0.55	0.55	0.88	0.8
Transition confining stress, (kPa)	60	60	70	40
Tensile failure strain limit, (%)	11	11	11	15

[b] Calculated using load at 2% strain, unit width and nominal thickness

The pile-soil interface parameters for pile shaft, as defined in FLAC manual were derived by calibrating the axial and lateral loading response of a timber pile. The parameters are cs_sstiff = 18.8 MPa, cs_nstiff = 0.6 MPa, cs_scoh = 47 kN/m and cs_ncoh = 15 kN/m. Similarly, the pile toe parameters are estimated as cs_sstiff = 188 MPa, cs_nstiff = 2.69 MPa, cs_scoh = 5640 kN/m and cs_nfric = 35.

Figure 4 shows the model used in the FLAC analysis. The model is 130 m wide and 29 m high, consists of 5040 elements and includes a 9 m high embankment retained by a MSE wall and a reinforced soils slope, the LTP and the piles.



Dimensions in metres. Water table in blue, piles in yellow and geogrids in red colour.

Figure 4. FLAC analysis model

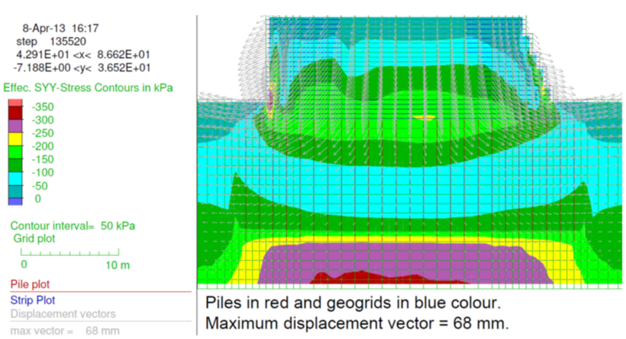
Boundary conditions for pre-earthquake static analysis: The base of the model was fixed against horizontal and vertical movements. The lateral boundaries were fixed against horizontal movement but vertical movement was allowed for pre-earthquake static analysis.

Boundary conditions for dynamic (seismic) analysis: For the dynamic (seismic) analysis the base of the model was fixed against vertical movement and horizontal earthquake motion was applied at the base. The lateral boundaries were set to "free field boundaries" to simulate propagation of seismic waves horizontally to infinite extent (ITASCA, 2008).

The stress-strain model within the "UBCTOT" routine simulates propagation of earthquake motion.

The FLAC model was set-up to provide time histories of acceleration, velocity, displacement, stress and strain at various nodes and elements of the model. The results of the analyses include the above noted time histories for each input earthquake motion and the state of the model (deformation, forces, stresses and strains) at the end of shaking.

Figure 5 shows the vertical effective stress contours and displacement vectors for static loading conditions (i.e.: prior to the application of the design earthquake motion).



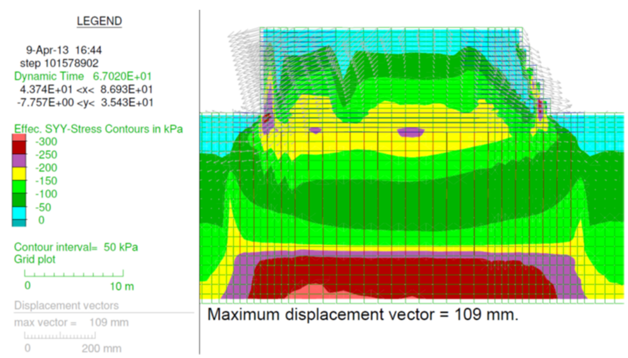
Piles in red and geogrids in blue colour.
Maximum displacement vector = 68 mm.

Figure 5. Vertical effective stress contours and displacement vectors, static loading

Dimensions and displacement vectors in Figure 5 are in metres and stresses in kilo Pascal units. Piles and geogrids are shown in red and blue colors respectively. Note the reduction in vertical effective stress within the subsurface soils of the piled area under the embankment.

The displacement vectors in Figure 5 show a maximum of 68 mm, at the mid-height of the vertical MSE wall. The maximum deformation of the reinforced soil slope is 40 mm, located at the bottom of the wall. Maximum values of the axial strain within geogrid reinforcement is 0.9% for the vertical MSE wall and 0.3% for the reinforced soil slope. Maximum horizontal displacement at the top of the outer timber piles (the first and last piles) are 47 mm on the side of the vertical MSE wall and 42 mm on the side of the reinforced soil slope.

Figure 6 shows same information as that of Figure 5, but at the end of the earthquake shaking, the design motion for 475-year return period event. The maximum displacement at the end of shaking is 109 mm. The mode of displacement is different for the two sides, rotation about the toe of the vertical MSE wall and rotation about the foundation subgrade under the reinforced soil slope. Table 4 provides the maximum vertical and horizontal deformations at the top of the embankment from seismic loading.



Maximum displacement vector = 109 mm.

Figure 6. Vertical effective stress contours and displacement vectors at the end of earthquake shaking

Table 4. Summary of maximum deformation at the top of embankment in millimeter units

Earthquake record	MSE Wall		Reinforced Soil Slope		10 m back from MSE wall	
	H	V	H	V	H	V
OLYNS475	91	28	14	24	62	54
LPEW475	108	29	5	23	76	56
SFNC475	76	23	36	21	46	49

H and V – Horizontal and Vertical deformations respectively.

5 EMBANKMENT CONSTRUCTION

Construction of the embankment included the following steps:

- Excavation to elevation zero for pile supported embankments and pumping of groundwater using sumps and pumps;

- Driving timber piles and cutting-off at elevation zero. Tip of the piles were at elevation -12 m to -14 m. The diameter of the piles were a minimum 180 mm at the tip and 330 mm at cut-off. The piles were driven using an impact hammer with a rated energy of 20 kJ;
- Installation of piezometers as described in the following subsection;
- Construction of the LTP within the timber piled areas;
- Installation of settlement gauges and slope inclinometer casings as described in the following subsection;
- Construction of the embankments, MSE walls and reinforced soil slopes, and sloped embankments (outside the timber piled areas);
- Placement of the surcharge for embankments outside the timber piled areas.

5.1 Instrumentation

Instrumentations used for monitoring the performance of the embankments included the following:

1. Pneumatic Piezometers: Push-In Pneumatic Wellpoint Piezometers with tubing attached to 32 mm diameter steel riser pipes were installed within the clayey, silty soils. The tip of the piezometers was located at elevations between -4 and -6 m. Piezometers were installed along the centreline of the embankment at the centroid of the four surrounding timber piles. Piezometers were also installed at the centreline of the embankments with no timber piles.
2. Slope Inclinometers: Inclinometer casings with a nominal 70 mm outer diameter were installed to a tip elevation of -16 m at the distance of 1.5 m from the toe of the MSE wall or reinforced soil slope.
3. Settlement Gauges: Each settlement gauge consisted of a plywood sheet base (600 mm x 600 mm x 38 mm thick) attached to 1.5 m long, 38 mm diameter, steel riser pipes and couplers. The riser pipes were attached to the plywood sheet by a 150 mm square (or 150 mm diameter round) x 12 mm thick steel flange that was bolted to the plywood base. The base of the settlement gauges (i.e.: the bottom of the plywood base) were located at elevation 3 m (1 m above the top of the LTP). Outside the timber piled areas the base of the settlement gauges was located at the bottom of the embankment fill. All settlement gauges were installed along the centreline of the embankment at 20 to 50 m horizontal spacing.

5.2 Instrument monitoring frequency

Readings of the instrumentations were taken once every week during construction. Upon completion of embankment and surcharge placement the instrument readings were taken at the following frequency:

- at the end of each week for a total of four weeks, then;
- readings at the end of sixth and eighth weeks and then;
- monthly readings starting from the 12th week.

6 ANALYSIS OF INSTRUMENT MONITORING DATA

6.1 Settlement and piezometer monitoring data

Typical responses from a set of instruments for embankments supported on piles and those not on piles are presented and discussed in this section. Figure 7 shows the response of a set of gauges located within the pile supported embankment L770. Figure 8 shows data of a settlement gauge located within the L780 embankment (with no pile support). It may be noted that the two embankments are of similar height and are on similar subgrade but have significantly different settlement magnitudes, rates and patterns.

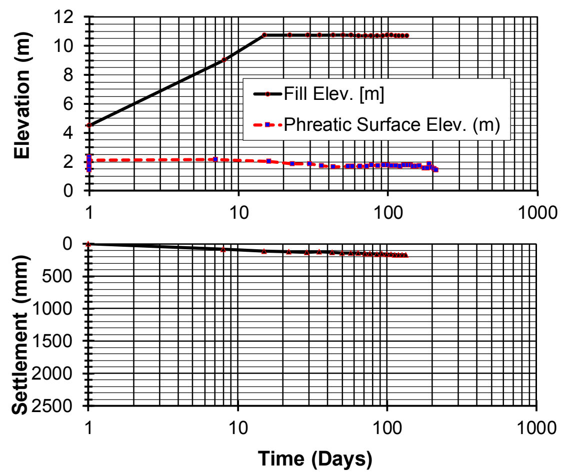


Figure 7. Settlement and piezometer monitoring data of a pile supported embankment

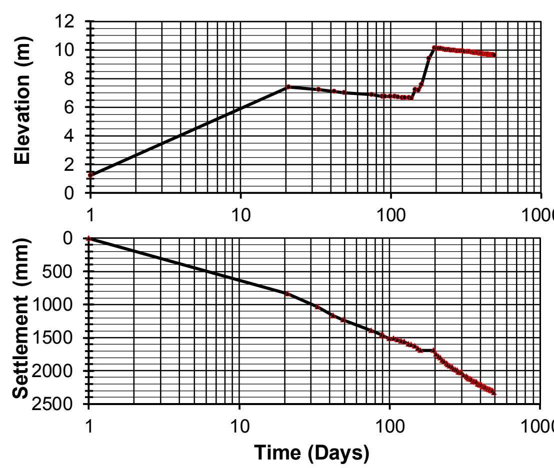


Figure 8. Settlement monitoring data of an embankment with no pile support

The contractor was able to construct the embankments on piles rapidly, within two weeks to the final elevation. The embankments with no pile support on the other hand required about two months to raise the fill elevation from 6.5 to 10 m (i.e.: two months was required to place 3.5 m of fill without causing slope failure). The pile supported embankment settled about 170 mm, about 100 mm of the settlement during fill placement and construction, and the remaining 70 mm after construction. No significant increase in settlement is noted after 100 days of the start of filling. The embankment with no pile support however, has settled more than 2300 mm. The rate of settlement has not slowed down after 500 days. The data obtained from other settlement gauges within the embankments showed similar response.

The settlement pattern from the gauges on piled embankments indicates elastic response. The phreatic surface remained nearly the same (between elevation 1.5 and 2.0 m for the gauge data shown in Figure 7) for the piezometers located below the piled embankments. This response confirms the design assumptions – that the embankment loading is transferred to the piles and from the piles to the deep incompressible soil layers. Negligible loading on the shallow compressible soils is inferred as no porewater pressure increase or consolidation settlement is noted.

6.2 Post-construction settlement prediction

Post-construction settlement of the embankments was derived using preload settlement monitoring data. Linear-extrapolation of the secondary portion of preload settlement-log(time) was used for this purpose. Details of the method are described in Uthayakumar and Oliver (2017). The calculated post-construction settlement, starting from 2014 is provided in Table 5. “Other Embankments” as defined in Table 5 were preloaded with surcharge but with no pile support. Preloading was continued until confirmation of the end of primary consolidation settlement.

Road construction of the L730, L750S and L780 embankments were completed before the end of primary consolidation settlement due to conflict in project schedule and with the acknowledgement that future maintenance work would be required.

Table 5. Predicted post-construction settlement of embankments after Year 2014

Time	2 years	5 years	10 years	20 years
Piled Embankments	25 mm	50 mm	75 mm	100 mm
Other Embankments	65 mm	125 mm	225 mm	325 mm

6.3 Photographs of construction

Figure 9 shows a photograph taken during installation of timber piles for a part of the L760 embankment. Construction of the LTP may be seen on the middle-left side of the photograph. Steel pipe pile foundations for the south abutment of overpass #8290 may be seen at the end of the rows of timber piles.

Figure 10 shows the partially completed L760 embankment and the south abutment of overpass #8290. The reinforced soil slope, inclined at 70 degrees and the vertical MSE wall of the L760 embankment can also be seen.

Figure 9. Photograph showing installation of the timber piles

Figure 10. Photograph showing the partially completed L760 embankment

7 CONCLUSIONS

Design, construction and monitoring of embankments supported on driven timber pile foundations are described in this paper. The subsurface soils consisted of highly compressible soils, including peat up to 4 m thick, followed by very soft silts and clays up to 10.5 m thick, followed by sands up to 21.5 m thick, followed by interlayered silty clay and clayey silt to more than 50 m depth below the original ground surface. Groundwater table was located at or within a few metres of the ground surface.

Analysis showed embankments constructed using sand fill and with no pile support could settle more than 2 m. However, slope stability analyses showed factor of safety less than 1.0 under static loading conditions for four of the embankments. Stability analysis of earthfill embankments within a former stockpile site, without pile support showed satisfactory results.

Untreated, unpealed timber piles with no pile cap were selected for four of the embankments. The piles were designed and driven at a square grid pattern at 1.4 m horizontal spacing and to a tip elevation of -12 to -15 m, 1 to 4 m embedded into the dense sand layer. The top of the piles was cut off at elevation 0, below the water table. The piles were minimum 330 mm in diameter at cut-off elevation and 180 mm diameter at the tip. A 2 m thick load transfer platform was constructed to transfer the embankment load to the piles. Simplified limit equilibrium analyses to detailed soil-structure interaction analyses using the computer software FLAC (ITASCA 2008) were utilized for the design.

With the above design, the calculated settlement of the timber piled embankment, after completion of construction, was in the range of 75 to 100 mm. The measured settlement after completion of construction was 70 mm. No significant increase in settlement is noted after 100 days of the start of filling. The embankment with no pile support however, has settled more than 2300 mm. The rate of settlement has not slowed down after 500 days.

Long-term post-construction settlement of the embankments was estimated using settlement measurements taken during construction. The estimated post-construction settlement of the pile supported embankments at the end of 20 years is 100 mm. Post-construction settlement of 325 mm at the end of 20 years is estimated for embankments treated with preload until the end of primary consolidation, but with no pile support.

ACKNOWLEDGEMENTS

The project was completed under a design-build-finance-operate contract between Fraser Transportation Group (FTG) and the BC Ministry of Transportation and Infrastructure (BC MoTI). The lead engineering firm for the project was Stantec Consulting Ltd., (civil, structural, environmental, electrical and project management). The geotechnical engineering work described in this paper was completed when the first author was with Trow Associates Inc. The authors thank the BC MoTI (Geoff Freer), FTG (Ramon Fiuza) and Steve Fleck (Stantec). Also, the authors thank Dr. Michael Beaty (Beaty Engineering, LLC) for providing the “UBCTOT” subroutine used in FLAC analysis.

REFERENCES

Armstrong, J. E. and Hicock, S.R. 1980. Surficial Geology, New Westminster, West of Sixth Meridian, British Columbia, Map 1484A. Geological Survey of Canada. 601 Booth Street, Ottawa, ON K1A 0E8.

Beaty, M.H. and Byrne, P.M. (2008). Liquefaction and deformation analyses using a total stress approach. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 134, No. 8, pp.1059 - 1072.

CAN/CSA-S6-14 (2014). Canadian highway bridge design code. Published by Canadian Standards Association. 5060 Spectrum Way, Suite 100, Mississauga, ON, L4W 5N6.

Filz, G., Sloan, J., McGuire, M.P., Collin, J. and Smith, M. 2012. Column-supported embankments: Settlement and load transfer. Geotechnical Special Publication No. 226, published by ASCE, pp. 74 – 77.

ITASCA (2008). Fast Lagrangian Analysis of Continua - FLAC, Version 6, Itasca Consulting Group Inc., Minneapolis.

Uthayakumar, M. and Oliver, M. Preload Treatment for Highway Embankments on Soft Soils – A Case History. Proceedings of the Canadian Geotechnical Conference, October 1 - 4, 2017, Ottawa, Ontario.