Geotechnical design for the William R. Bennett Bridge, Kelowna, British Columbia

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ABSTRACT
The new five-lane William R. Bennett Bridge across the Okanagan Lake in British Columbia was constructed and opened to traffic in May 2008. Subsurface soils along the bridge alignment include very soft to soft silts and clays and loose to compact sands. Key components of this new crossing include the light-weight fill west approach embankment; the west abutment and five piers supporting the fixed section of the bridge; a transition span; the floating section of the bridge supported on pontoons; the east transition span; the east abutment; and, the east approach embankment. The west abutment, the five piers and the east abutment are supported on driven, 610mm and 914mm diameter steel pipe piles with embedment depths of 30m to 50m. This paper presents an overview of the subsurface soil conditions, geotechnical analyses, test results and post-construction settlement.

RÉSUMÉ
Le nouveau cinq voies pont William R. Bennett à travers le lac Okanagan, en Colombie-Britannique a été construit et ouvert à la circulation mai 2008. Sols sous-sol le long du tracé du pont comprennent très doux au limons et argiles mous et lâches de sables compacts. Les éléments clés de ce nouveau passage inclure le léger remblai approche ouest, la culée ouest et cinq piliers supportant la partie fixe du pont; une période de transition, la section du pont flottant pris en charge sur les pontons, la durée de transition est, les culée est, et le remblai approche est. La culée ouest, les cinq piliers et la culée est sont pris en charge sur entraîné, 610mm et 914mm de diamètre pieux tubulaires en acier avec des profondeurs d'ancrage de 30m à 50m. Cet article présente un aperçu des conditions du sous-sol, des analyses géotechniques, les résultats des tests et de règlement post-construction.

1 INTRODUCTION

The new five-lane William R. Bennett Bridge across the Okanagan Lake in British Columbia was constructed and opened to traffic in May 2008. This bridge links the City of Kelowna on the east to Westbank on the west (Figure 1). The previous three-lane Okanagan Lake Bridge, which was in service for 50 years and was located to the immediate south of the new bridge was demolished following the opening of the new bridge. The new bridge was designed and constructed under a Design-Build Finance and Operate contract.

Figure 1. Location of the William R. Bennett bridge

Subsurface soils along the bridge alignment include very soft to soft silts and clays and loose to compact sands. These soil units are of lacustrine origin, and are near normally-consolidated to slightly over-consolidated.

Design of a fixed bridge with deep foundations was considered, however, the cost of a fixed structure was determined to be high due to the water depth and the presence of very deep compressible and soft soils. A design which included an elevated-fixed structure supported on pile foundations over shallow water and a floating structure supported on pontoons over deep water was chosen.

Total span of the bridge between the west and east abutments is 1060m. Key components of this new crossing include an approximately 190m long west approach embankment; the west abutment and five piers over a length of 275m; the west transition span of 52m; an approximately 730m long floating section supported on nine pontoons; an approximately 50m long east transition span; the east abutment; and, an approximately 300m long east approach embankment. Figure 2 presents a photograph showing the west approach embankment, the elevated bridge structure and a portion of the floating structure.

The presence of very soft to soft compressible soils underlying the west approach embankment required preload treatment and the use of light-weight fill. A number of light-weight fill materials, including cellular concrete, pumice, hog fuel (wood waste) and expanded polystyrene (EPS) were considered. EPS was chosen following analysis of the options.

The east approach embankment was constructed entirely using mineral earth fill as the subsurface soils are not as compressible as those on the west side. Post-construction settlement of the approach embankments was calculated using both laboratory test and preload settlement data.

Figure 2. Photograph showing the bridge crossing

The west abutment, the five piers and the east abutment are supported on driven, 610mm and 914mm diameter steel pipe piles with embedment depths of 30m to 50m. A pile load test program, consisting of static axial, lateral, and dynamic (pile driving analyzer, PDA) components was completed prior to the final design (Naesgaard et al, 2006). In addition, a number of PDA tests were carried out during construction.

Dynamic numerical analyses were carried out, with multiple earthquake records of varying return periods, to study the seismic response of the lake-bottom sediments and embankments.

2 GEOLOGY AND SUBSURFACE CONDITIONS

The Okanagan Lake occupies a relatively narrow elongated north-south tending fjord-like trench. Near the centre of the bridge alignment the lake is approximately 45m deep, elsewhere, water depths are up to 238m deep.

Eyles et al, (1990) indicates bedrock at approximately 200m depth near the bridge location and the absence of a thick till sequence over the bedrock. Bedrock is presently exposed on the western bank of the lake just south of the bridge location. The eastern shore is a creek delta deposit and has a relatively flat relief.

The soils above the bedrock surface are glaciolacustrine or lacustrine deposits. In the vicinity of the eastern side of the bridge the lacustrine sediments are inter-layered and overlain by deltaic silty sand and sand layers deposited by the Kelowna Creek.

2.1 Subsurface Soil Conditions

Subsurface exploration program included boreholes with Standard Penetration Tests, split-barrel and Shelby tube sampling, electric Cone Penetration Tests (CPTu), auger hole drilling and seismic shear wave velocity measurements. Laboratory tests included determination of natural moisture content, Atterberg Limit tests, grain size analyses, one dimensional consolidation tests, and monotonic and cyclic simple shear tests.

Soil conditions on the west side of the lake include soft to firm low plastic silt with sand layers and occasional layers of organic matter to an approximate depth of 20m, firm to stiff low plastic silt with layers of sand and low to high plastic clay between 20m and 30m depth, compact to dense silty sand with inter-layered silt to approximately 45m depth followed by dense silty sand.

Natural moisture content of the silts and clays varied from 23% to 74%. Liquid limit and plastic limits varied from 23% to 65% and 17% to 44% respectively. Undrained shear strength of the silt was obtained from laboratory vane shear tests, simple shear tests and from in-situ CPTu tests. The shear strength was found to vary considerably, from 7kPa to 60kPa within 20m depth. Log of a typical CPTu hole completed at the west approach embankment is shown in Figure 3.

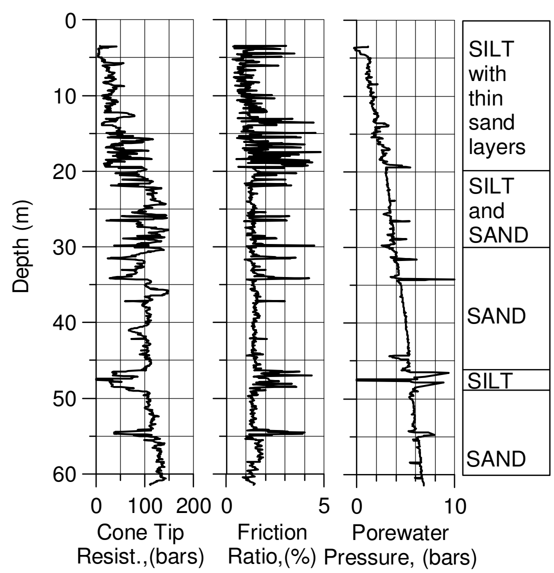

Figure 3. Typical CPTu data from west side of lake

Soil conditions on the east side of the lake include inter-layered loose to compact sand and soft to firm silt and clay with organic matter to more than 45m depth. Natural moisture content of the silts and clays varied from 22% to 94%, liquid limit varied from 35% to 87% and plastic limit varied from 24% to 38%. Log of a typical CPTu hole completed at the east side of the lake shore is provided in Figure 4.

The design High Water Level of the lake was 342.5m, Low Water Level at 341.3m and Mean Water Level at 341.9m. All elevations given in this paper are referenced to the geodetic datum.

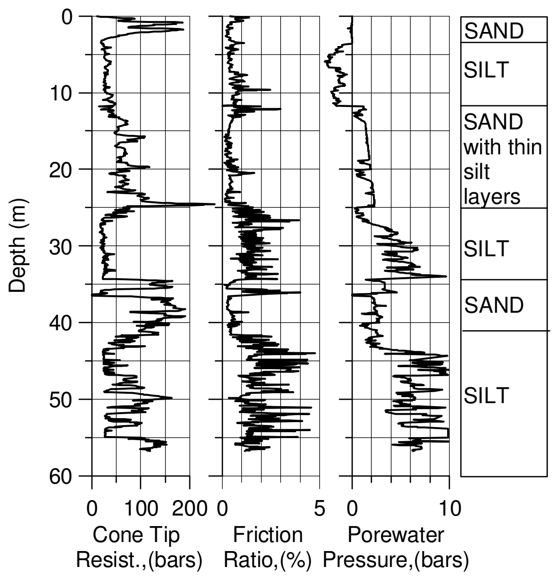

Figure 4. Typical CPTu data from east side of lake

3 SEISMIC DESIGN PARAMETERS

Seismic design criteria, specified by the BC Ministry of Transportation, required that the bridge should remain open to emergency vehicles immediately after a 475-year return period earthquake; and should not collapse or suffer sudden loss of load carrying capacity from a 1000-year return period event.

Design response spectra for the 475-year and 1000-year return period earthquake motions at outcropping “firm ground” are shown in Figure 5.

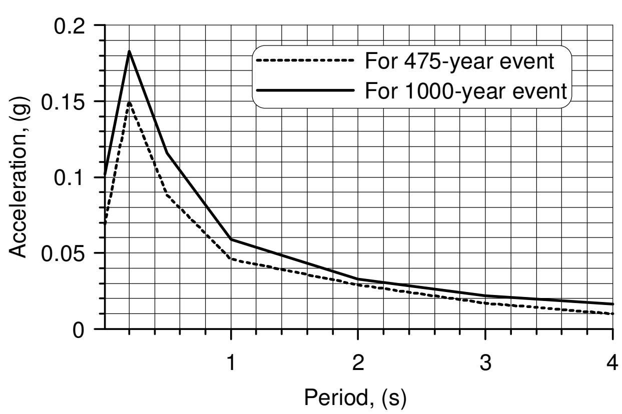

Figure 5. Design response spectra at outcropping “firm ground”

Firm ground is considered as very dense soils or soft bedrock with a shear wave velocity of 360m/s to 750m/s as defined in the Canadian Highway Bridge Design Code, CAN/CSA-S6-00 (Design Code for this project). As noted previously, no “firm ground” was encountered in the deepest drill hole completed at the bridge site, 90m below the lake bottom at elevation 205m. To obtain near-surface response spectrum for structural design, site-specific seismic ground response analyses were completed.

3.1 Seismic Ground Response Analysis

Site specific ground response analyses and seismic deformation analyses were carried out using commercially available computer programs SHAKE (Idriss and Sun 1992) and FLAC (Itasca 2005) respectively. The purpose of the SHAKE analysis is to obtain near-surface response spectra for structural design, to calculate the extent of subsoil liquefaction, and to develop in-profile acceleration time histories for use in seismic ground deformation analysis that were carried out using FLAC.

Nine earthquake records were chosen for the analyses, four for the 475-year event and five for the 1000-year event. The records were modified using the computer program SYNTH (Naumoski 1985) such that the resulting record’s response spectrum matches that shown in Figure 5. The modified records were used in SHAKE analysis as outcropping firm ground input motions.

As the depth to the firm ground was not known, a sensitivity analysis with various depths to firm ground was carried out. Generally, shallower firm ground depths resulted in higher (more severe) response. From the sensitivity analysis a firm ground depth of 135m was chosen for design.

3.2 Liquefaction Assessment

Liquefaction susceptibility of the subsurface soils was assessed using the procedures given in Youd et al (2001). Detailed discussion on liquefaction assessment method and results are beyond the scope of this paper. In summary, the assessment for the 475-year event indicates liquefaction is unlikely, except for thin zones in the upper 4m depth in the vicinity of Pier No. 3 (i.e., factor of safety less than 1.2). Similarly, a factor of safety less than 1.0 was obtained for the 1000-year event in the vicinity of Pier No. 3, in thin zones within 4m to 5m depth from the lake bottom. Also, liquefaction was predicted for the 1000-year event in the vicinity of the east abutment for approximately 6m depth. The consequences of liquefaction on bridge foundations, embankments and pontoon anchors were assessed and addressed in the design, but are not included in this paper due to space limitation.

4 WEST APPROACH EMBANKMENT

The west approach embankment is approximately 190m long and increases in height from the existing road on the western limit to approximately 12m above the lake bottom at the west abutment. The width of the embankment varies from approximately 35m on the western limit to 25m at the abutment. Earth fill and various light-weight fill options with preloading were considered. An embankment made entirely of earth fill required a very large footprint with side slopes of 4H:1V to 6H:1V due to stability concerns and a preload duration of 2 years or more. For the final design, a combination of earth and EPS fill were chosen.

Detailed slope stability, settlement and seismic deformation analyses with and without EPS fill were completed. An EPS fill embankment with vertical concrete panel walls on the sides and a concrete capping layer at the top was initially considered. The concrete panel walls were designed to be supported on shallow strip footings. However, for the final design the vertical walls were eliminated and replaced with side slopes (Figure 6). Following detailed settlement, slope stability and seismic deformation analysis, the following design was adopted:

- An earth fill embankment from the lake bottom to Elevation 343m, maximum thickness of approximately 5m at the west abutment;
- EPS fill above Elevation 343m, thickness varying from 0.6m to 6.6m;
- A 10mil polyethylene sheet cover over the EPS;
- A minimum 1m thick granular fill cover above the EPS on side slopes;
- A 1.2m thick cover over the top of the EPS embankment consisting of 0.2m thick sand fill and 1m thick gravel and asphalt pavement structure.

4.1 Preload Treatment

To reduce post-construction settlement, the earth fill portion of the embankment was constructed approximately 2.5 years in advance of final construction. The embankment footprint was preloaded with a surcharge of approximately 2.9m above bottom elevation of the EPS fill. Note that the design includes a combined 1.2m thick granular fill and pavement structure over the EPS. Therefore the net surcharge, prior to any settlement is 1.7m of earth fill. Figure 6 shows a cross section of the embankment, near the west abutment.

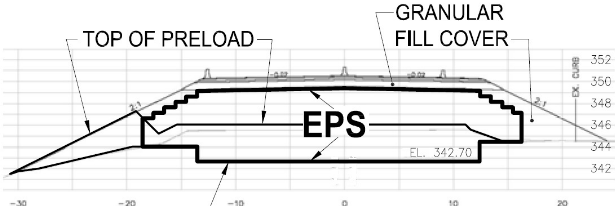

Figure 6. Cross section of embankment near west abutment

Construction of the earth fill embankment, as preload, commenced in June 2005. The fill was placed in thin lifts with a time interval between the lifts. The required time interval between successive lifts was estimated by monitoring pore water pressure dissipation in the underlying clays and silts using pneumatic piezometers installed prior to fill placement. In addition, settlement and horizontal movement of the ground were monitored using two sets of deep settlement gauge arrays, ten surface settlement plates and two slope inclinometers. In general, 1m thick lifts were used for placement of fill below the lake surface and 0.5m lifts for fill above the lake water level. The rate of fill placement was at one lift per week.

Construction of the preload was completed in October 2005 when the fill elevation reached 345m. During this construction period, the measured surface settlement was between 400mm and 700mm. Subsequently, an additional 1m thick earth fill was placed during the second week of December 2005, bringing the top of the preload to an elevation of 346m. In the second week of December, 2006 the preload fill above the design bottom elevation of the EPS was removed and taken off-site, approximately 500 days after the start of preload construction.

4.2 Settlement of the West Approach Embankment

Settlement analysis was completed prior to the construction of the embankment and the results were compared to the monitored settlement during preloading. Soil parameters for this analysis were derived from laboratory test data, including 1-D consolidation tests on “undisturbed” samples obtained form the site. In addition, a back analysis was completed using measured settlement of the existing Okanagan Lake Bridge west approach embankment. The derived parameters are:

- Coefficient of compression, C_c - 0.63 for the first 7.5m depth below the lake bottom, 0.7 between 10.5m and 17. 5m depth, and 0.3 for the deeper thin layers;
- Coefficient of re-compression C_r - 0.17 for the first two layers noted above and 0.08 for the deeper thin layers;
- Coefficient of secondary compression, C_α – 0.02 for the first two layers noted above and ignored for the deeper thin layers.

Using the above parameters post-construction settlement was predicted to be in the order of 215mm at the end of 40 years. Figure 7 shows the measured settlement data from deep settlement gauges at three different elevations near the west abutment. Settlement data from the deep and surface settlement gages indicates that the primary consolidation was essentially completed 150 days after preload construction.

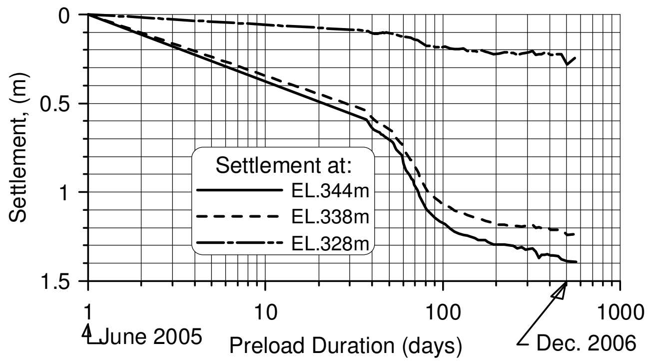

Figure 7. Measured Preload settlement near west abutment

End of primary consolidation settlement was defined using the "log-time method", as the time at which a change in the slope of the tangent to the settlement curve occurs. By extending the "secondary" part of the settlement curve, post-construction settlement at the end of the 30-year concession period is predicted to be in the range of 175mm to 225mm near the west abutment. An additional settlement of about 25mm is predicted at the end of 40 years. It may be noted that the predicted post-construction settlement using the laboratory test data and that from extrapolation of the preload settlement data are in close agreement.

The elastic compression of the EPS fill, using the specified elastic modulus of 4750kPa and a thickness of 6.6m was calculated as: 42mm under the dead load of the granular fill and pavement structure above the EPS and an additional 22mm under an assumed vertical stress 16kPa from traffic loading. The 42mm elastic compression would have occurred during construction. Further discussions on EPS fill and test data are provided later.

4.3 Post-Construction Settlement

Post-construction settlement over a 2-year period, from 2008 to 2010 is summarized in Table 1.

Table 1. Summary of post-construction settlement

Location	Measured settlement on:		
	June 2008	July 2009	July 2010
Top of embankment near west abutment	0	17mm	27mm
Bottom of EPS Fill near west abutment	0	8mm	12mm
Top of embankment near east abutment	0	12mm	15mm

Measured settlement of the pile supported piers is provided later. It can be noted from Table 1 that the difference in measured settlement at the top and bottom of the 6.6m thick EPS fill near the west abutment is 9mm to 15mm.

4.4 Tests on Expanded Polystyrene (EPS) Fill

The specified minimum requirements of the EPS fill are:

- minimum density of 21kg/m^3 measured in accordance with ASTM D1622;
- minimum compressive strength of 115kPa at 5% strain (ASTM D1621, Method A);
- minimum modulus of elasticity of 4750kPa (ASTM D1621);
- minimum flexural strength of 276kPa (ASTM C203);
- Maximum water absorption of 4% by volume (ASTM D2842).

In addition to the supplier's quality control tests to meet or exceed the above specifications, a series of tests were completed. Figures 8 and 9 show the variation of measured compressive strength and modulus of elasticity respectively with density.

For comparison, test data from Negussey, (2007) is shown in Figures 8 and 9. The compressive strength increases linearly with the density of EPS, and in agreement with the test data obtained by Negussey, (2007). The modulus of elasticity values are lower than those provided in Negussey, (2007) and do not appear to show a clear trend of linear increase with density. It is recognized that the number of test data points and the range of density are not sufficient to establish a clear trend.

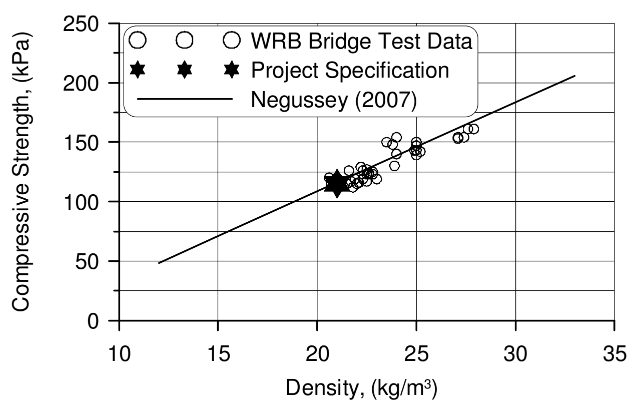

Figure 8. EPS compressive strength at 5% strain

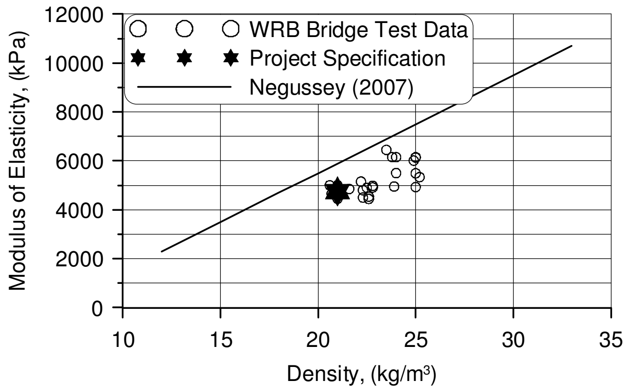

Figure 9. EPS modulus of elasticity

The ASTM Standards for compression strength and modulus specifies that the cross sections area of EPS test specimens should be between 2580mm^2 and 2320 mm^2 with a height of minimum 25.4mm. Accordingly, 50mm x 50mm x 50mm specimens were used for the tests. All test specimens were prepared by cutting using a hot-wire. The test results can be significantly affected by the preparation method of such small specimens.

The study by Negussey (2007) shows that elastic modulus measured using small specimens as per ASTM can be much smaller than that from tests on larger blocks of EPS or as measured in the field. The study also shows that EPS density is a good index parameter which can be related to other design parameters such as strength and modulus. The test data shown in Figure 8 confirms the findings of Negussey (2007).

To further confirm the design assumptions, density of randomly selected EPS blocks was checked in the field. About 10% of the EPS blocks delivered over the duration of construction was used for this purpose. A calibrated scale was used for the measurement of weight and the dimensions were obtained using a tape measure. The EPS blocks had length of 2.5m to 4.9m, width of 1.2m and depth of 0.6m to 1.2m. The calculated density, from measurement of 470 blocks, is graphically shown in Figure 10. The blocks with lower than the specified density of 21kg/m^3 were rejected and taken off-site.

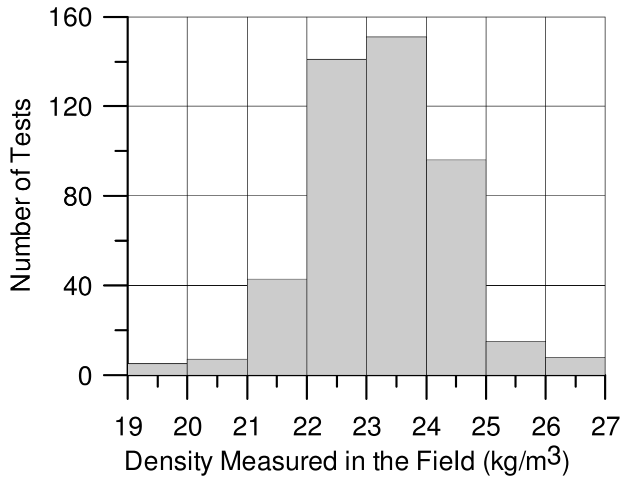

Figure 10. Density of EPS blocks measured in the field

4.5 Construction of EPS Fill Embankment

The EPS fill was placed on a 100mm thick layer of clean sand overlying the embankment fill. Block joints at a given elevation were offset a minimum 0.6m relative to blocks placed in adjacent rows. The long dimension of the blocks was rotated 90° with each successive lift to avoid continuous vertical joints.

A few blocks were cut in the field using a hot-wire to suite the embankment geometry. The sides and top of the EPS were covered with black 10mil polyethylene sheets. A minimum 1m thick granular earthfill cover was placed on top of the EPS along the side slopes.

A 200mm thick layer of clean sand was placed on top of the poly covered EPS fill over the top of the embankment. This sand layer was compacted to 90% of the maximum Standard Proctor Dry Density (SPDD). Then a 600mm thick layer Select Granular Sub-base course was placed in 150mm thick lifts and compacted to 95% SPDD, followed by a 300mm thick well graded base course was placed in 150mm thick lifts and compacted to 100% SPDD. Finally, construction of 100mm thick asphalt was completed.

Two photographs showing construction of the EPS embankment are given in Figure 11.

Figure 11. Photographs showing placement of EPS fill

4.6 Stability of the EPS Fill Embankment

Stability analysis of the embankment was carried out using soil parameters obtained from laboratory tests, in-situ CPTu and vane shear tests. The laboratory tests include monotonic and cyclic simple shear tests and vane shear tests. The CPTu data indicates that the shear strength ratio S_u/p' of the soft to firm silty layer below the lake bottom could be highly variable - from 0.15 to more than 0.5, where S_u is the undrained shear strength and p' is the vertical effective stress at a given depth.

Historical data from construction files of the Okanagan Lake Bridge shows that a failure of the west approach embankment occurred during construction in 1957. A back analysis of this failure showed an S_u/p' of 0.21 for the lake bottom silts and clays if the friction angle of the embankment rock fill is 45 degrees.

For the analysis of the new EPS embankment, the mass of the EPS and the overlying earth fill was modelled as surcharge pressure at the bottom elevation of EPS. The specified unit weight of the EPS is 0.21kN/m^3, however, a unit weight of 1.0kN/m^3 was used in the analysis to account for possible water absorption by EPS. Also, a 30kPa vertical stress was included to account for the 1.2m thick granular fill and pavement structure above the EPS fill.

A factor of safety (FS) higher than 1.5 against slope failure for long-term drained condition was calculated using a friction angle of 21 degrees for the lake bottom sediments and 45 degrees for the embankment rock fill. Analysis with S_u/p' of 0.21 for the lake sediments, for short-term undrained conditions, showed a lower FS of 1.2.

The seismic ground response analyses described previously indicated that the near surface Peak Ground Acceleration (PGA) of 0.06g to 0.08g for the 475-year and 0.07g to 0.09g for the 1000-year return period design events. Seismic stability analyses showed a minimum factor of safety of 1.1 when the seismic coefficient is 0.1g, indicating negligible movement for the two design seismic events, which have PGAs less than 0.1g.

Seismic ground deformation analyses, using the computer program FLAC indicated seismic deformation of 25mm and 55mm for the two seismic events, 475 and 1000-year year return periods respectively.

5 EAST APPROACH EMBANKMENT

The east approach embankment is approximately 350m in length and rises from the existing grade to a maximum of about 7m above surrounding grade at the east abutment. The subsurface soils along this embankment, inter-layered sands, silty sands and silts, were found to be less compressible compared to those found on the west side of the lake.

An approximately 100m long segment of the embankment, starting from the east abutment was constructed as preload in early-June 2005. The preload included a surcharge of 2.2m above design road elevation. The preload was removed in mid-November, 2006 at which time the measured settlement varied between 200mm and 335mm.

The rest of the east approach embankment was constructed in mid-December, 2006. The preload included a surcharge of 1m of earth fill for embankment 3m above the surrounding grade. The surcharge was removed after four months. The measured settlement at the end of four months varied from negligible amount to 100mm, the higher settlement corresponding to a fill height of 3m to 4m above the surrounding grade.

6 FOUNDATION DESIGN

As the near surface soils are very soft to soft and compressible, shallow foundations were not considered for supporting the bridge abutments and piers. Pile foundations, including driven pre-cast concrete and steel pipe piles were considered and for final design driven steel pipe piles were chosen. The effects of downdrag were considered in the selection of steel pipe piles for the final design.

Axial capacity for 406, 610, 914, 1219, and 1524mm diameter driven steel pipe piles were calculated using the LCPC method, utilizing the CPT data (Bustamante and Gianeselli 1982). A pile load test program was completed earlier and the results were utilized to calibrate the calculated capacities.

6.1 Pile Load Test Program – Pre-Design Phase

A pile load test was carried out in March to April 1999, on the west side of the lake shore, near the location of Pier No. 3. This test program was completed prior to the award of the design-build contract. Details of the pile load test program are given in Naesgaard et al (2006). The piles were 610mm in diameter with a 12.7mm thick wall steel pipes.

Five piles, one central-axial load test pile and four reaction piles, were driven to 45m depth below the lake bottom. The central pile and two reaction piles were driven closed ended and the other two piles were driven open ended. The central (closed end) pile, one each of closed and open ended reaction piles were driven with a vibratory hammer for the first 18m of penetration. The remaining length of these three piles and the two remaining piles were driven with an impact hammer. The piles were instrumented with strain gauges, tell-tales and slope inclinometers with monitoring during driving and load testing. In addition to the static axial and lateral load testing, Pile Driving Analyzer (PDA) tests were completed at different stages, spaced over several weeks.

The pile load test program indicated the following:

- Open and close ended piles showed similar axial capacity, in the range of 3,600kN to 4,000kN;
- Vibratory driving of the upper 18m length did not affect the axial capacity;
- PDA testing with CAPWAP analysis and static axial load tests showed similar axial capacity;
- Calculated capacities from empirical methods can vary widely;
- Residual loads from pile installation can have a significant effect on load distribution along the length of the piles.

6.2 Pile Load Test Program – Construction Phase

For the final design of the west side piers and the west abutment 914mm diameter steel pipe piles with a 12.7mm thick wall were selected. Each of the west side pier and the west abutment were designed to be supported on twelve piles. The piles were driven with open end, initially with a vibratory hammer and then with an impact hammer.

The east abutment is designed to be supported on thirteen 610mm diameter steel pipe piles with a 9.5mm thick wall. The piles were driven with a closed end to tip elevations varying from 298.8m to 300.7m.

PDA tests were carried out on two piles at each of the piers and the abutments to confirm design capacities. Table 2 presents a summary of the pile embedment depths and mobilized capacity during PDA testing. The PDA tests were carried out using a 5443kg drop hammer.

As the calculated pile capacities using theoretical methods varied widely, the pile load test program, including the static load test and the PDA tests provided greater confidence of the design.

Table 2. Summary of mobilized capacity during PDA tests

Location	Embedment Depth below mudline, (m)	PDA Capacity (kN)	Design Ultimate Capacity (kN)
West Abutment	31 to 32	3,500	3,150
Pier 2	37 to 42	5,400 to 5,800	5,250
Pier 3	45 to 50	5,800 to 6,500	5,250
Pier 4	51 to 52	6,900 to 7,100	6,200
Transition Pier	51 to 52	6,300 to 6,700	5,600
East Abutment	43 to 45	3,900 to 4,000	3,000

6.3 Pile Foundation Settlement

Predicted settlement of the foundations, including that during construction is 50mm to 100mm. Preload settlement data in Figure 7 indicates that the pile foundation settlement would have been completed within 6 months of dead load application. Construction of the bridge girders commenced in March 2007.

Settlement of pier foundations was monitored starting from June 2008. Measured settlement between June 2008 and July 2010 varies between 0 and 6mm.

7 CONCLUSIONS

This paper summarizes the results of the geotechnical investigation and analysis completed for the design and construction of the new William R. Bennett Bridge across the Okanagan Lake in Kelowna, B.C. Presence of soft compressible soils required use of light-weight EPS fill and preload treatment for bridge approach embankments, and use of pile foundations to support the elevated bridge structure. The importance of quality control tests on EPS fill and site-specific pile load tests was demonstrated.

Test results indicate the compressive strength of EPS increases linearly with the density.

Preload settlement data was utilized for the prediction of post-construction settlement of the embankments. The predicted post-construction settlement using two different analysis methods, one using laboratory test data and the other using extrapolation of preload settlement data are in reasonable agreement.

Pile Driving Analyzer (PDA) tests together with the previously completed static load tests were utilized for pile design.

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