William R. Bennett bridge – Ten years of performance monitoring

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ABSTRACT
The five-lane William R. Bennett Bridge across the Okanagan Lake in British Columbia was constructed and opened to traffic in July 2008. The bridge has an elevated-fixed-structure on pile foundations over shallow water and a floating structure on pontoons over deep water. Subsurface soils along the bridge alignment include very soft to soft compressible silts and clays and loose to compact sands. The bridge approach embankments were constructed following preload treatment. Settlement of the approach embankments was estimated using laboratory test data on soils, then updated using analysis of the preload settlement data. Settlement of pile foundations was estimated using consolidation parameters derived from laboratory tests. Post-construction settlement survey was conducted to assess the performance using shallow and deep settlement gauges installed on and within the approach embankments, on the bridge piers and on the abutments. Baseline survey for settlement monitoring was carried out immediately after construction in June 2008 and then annually from June 2011 to June 2018. The survey monitoring data shows satisfactory performance and the measured settlement magnitudes are within the prediction. This paper presents a brief description of the design and construction of the bridge, geotechnical analyses, preload data, settlement prediction and the measured post-construction data.

RÉSUMÉ
Le pont à cinq voies William R. Bennett sur le lac Okanagan, en Colombie-Britannique, a été construit et ouvert à la circulation en juillet 2008. Il est doté d'une structure fixe surélevée reposant sur des fondations sur pilotis sur une eau peu profonde et d'une structure flottante sur un ponton au-dessus d'une eau profonde . Les sols souterrains situés le long de l'alignement du pont comprennent des limons et argiles compressibles très mous à mous et des sables meubles à compacts. Les remblais d’approche du pont ont été construits après traitement de précharge. Le tassement des remblais d'approche a été estimé à l'aide de données d'essais de laboratoire sur les sols, puis mis à jour à l'aide d'une analyse des données de tassement à la précharge. Le tassement des fondations sur pieux a été estimé à l'aide de paramètres de consolidation dérivés d'essais en laboratoire. Une enquête de tassement a été réalisée après la construction pour évaluer la performance à l'aide de jauges de tassement peu profondes et profondes installées sur et dans les remblais d'approche, sur les piliers du pont et sur les culées. La surveillance de la colonisation a été effectuée immédiatement après la construction en juin 2008, puis annuellement de juin 2011 à juin 2018. Les données de surveillance montrent une performance satisfaisante et les amplitudes de règlement mesurées sont conformes aux prévisions. Ce document présente une brève description de la conception et de la construction du pont, des analyses de tassement, des données de précharge, des prévisions de tassement et des données mesurées.

1 INTRODUCTION

The 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 side of the lake (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.

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.

The bridge has an elevated-fixed structure supported on pile foundations over shallow water and a floating structure supported on pontoons over deep water. Total span of the bridge between the west and east abutments is 1060 m. Key components of this bridge crossing include an approximately 190 m long west approach embankment; the west abutment and five piers over a length of 275 m; the west transition span of 52 m; an approximately 730 m long floating section supported on nine pontoons; an approximately 50 m long east transition span; the east abutment; and an approximately 300 m 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. Details of the geotechnical analyses, design and construction are provided in Uthayakumar and Naesgaard (2011).

The presence of very soft to soft compressible soils underlying the west approach embankment required preload treatment and the use of expanded polystyrene (EPS) light-weight fill. The east approach embankment was constructed entirely using mineral earth fill following preload treatment.

Settlement of the approach embankments and pile foundations was estimated using consolidation parameters derived from laboratory tests on undisturbed soil samples. Post-construction settlement prediction of the approach embankments was later updated using the analysis of the measured preload settlement data.

Figure 1. Location of the William R. Bennett bridge

Post-construction settlement monitoring was carried out using survey of monuments installed on top of the pavement along the approach embankments and at the top of pile caps at the piers and abutments.

Figure 2. Photograph showing the bridge crossing

The west abutment and the five piers are supported on driven, 914 mm diameter steel pipe piles with embedment depths of 30 m to 50 m. The east abutment is supported on driven 610 mm diameter steel pipe piles with an embedment depth of 43 m to 45 m. 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.

2 GEOLOGY AND SUBSURFACE CONDITIONS

The Okanagan Lake occupies a relatively narrow elongated north-south tending fjord-like trench (Nasmith, 1962). Near the centre of the bridge alignment the lake is approximately 45 m deep, elsewhere, water depths are up to 238 m deep.

Eyles et al, (1990) indicates bedrock at approximately 200 m 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 20 m, firm to stiff low plastic silt with layers of sand and low to high plastic clay between 20 m and 30 m depth, compact to dense silty sand with inter-layered silt to approximately 45 m depth followed by dense silty sand. Record of a typical CPTu hole completed at the west approach embankment is shown in Figure 3.

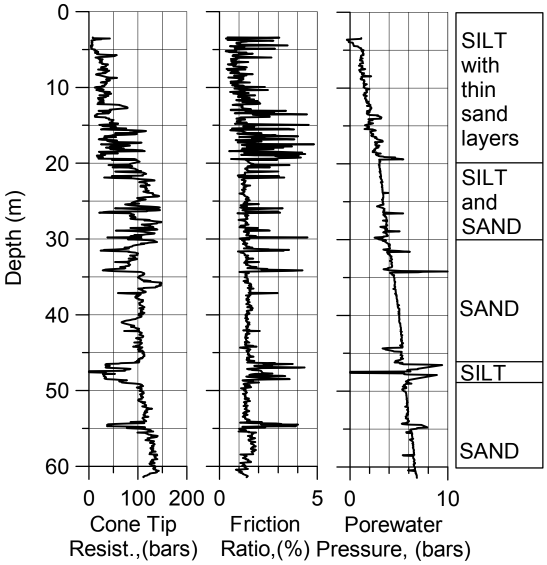


Figure 3. Typical CPTu data from west side of lake

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 7 kPa to 60 kPa within 20 m depth.

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 45 m 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%. Record of a typical CPTu hole completed at the east side of the lake shore is provided in Figure 4.

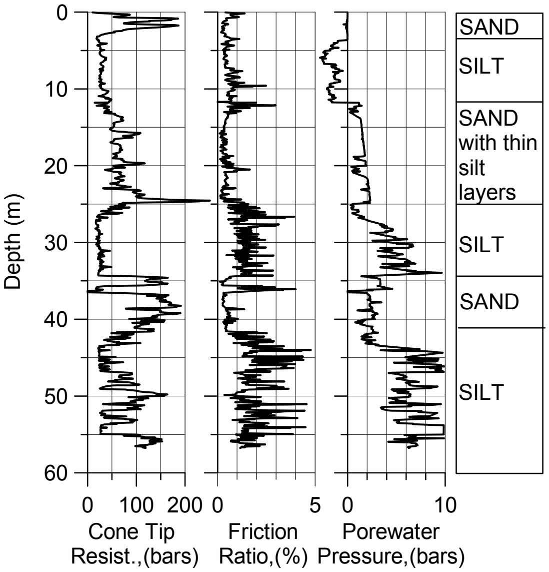


Figure 4. Typical CPTu data from east side of lake

3 WEST APPROACH EMBANKMENT

The west approach embankment is approximately 190 m 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 35 m on the western limit to 25 m at the abutment. Earth fill and various light-weight fill options with preloading were considered. For the final design, a combination of earth and EPS fill was chosen.

The west approach embankment includes the following:

- An earth fill embankment from the lake bottom to Elevation 343 m, maximum thickness of approximately 5 m at the west abutment. All elevations given in this paper are referenced to the geodetic datum.
- EPS fill above Elevation 343 m, thickness varying from 0.6 m to 6.6 m.
- A 0.25 mm (10 mil) thick polyethylene sheet cover over the EPS.
- A minimum 1 m thick granular fill cover above the EPS on side slopes.
- A 1.2 m thick cover over the top of the EPS embankment consisting of 0.2 m thick sand fill and, 1 m thick gravel and asphalt pavement structure.

3.1 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 2.9 m above bottom elevation of the EPS fill. Note that the design includes a combined 1.2 m thick granular fill and pavement structure over the EPS. Therefore, the net surcharge, prior to any settlement is 1.7 m of earth fill. Figure 5 shows a cross section of the embankment, near the west abutment.

Construction of the embankment as preload, consisting of sand and gravel fill, commenced in June 2005, and completed in October 2005 when the fill elevation reached 345 m. During this construction period, the measured surface settlement was between 400 mm and 700 mm. Subsequently, an additional 1 m thick earth fill was placed during the second week of December 2005, bringing the top of the preload to an elevation of 346 m. 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.

Instrumentation to monitor the response of preload included pneumatic piezometers, settlement gauges (SG) with their base plates at the bottom of the fill and deep settlement gauges (DSG) with 13 to 17 monitoring points at various depths below the bottom of the original ground surface.

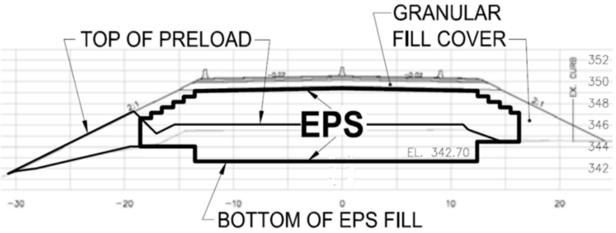


Figure 5. Cross section of embankment near west abutment

3.2 Settlement Analysis of 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 from the site. The parameters are given below:

- Coefficient of compression, C_c - 0.63 for the first 7.5 m depth below the lake bottom, 0.7 between 10.5 m and 17. 5 m depth, and 0.3 for the deeper layers.

- Coefficient of re-compression C_r - 0.17 for the first two layers noted above and 0.08 for the deeper layers.
- Coefficient of secondary compression, C_α – 0.02 for the first two layers noted above and was ignored for the deeper layers.

Using the above parameters post-construction settlement, after preload treatment was estimated to be in the order of 215 mm at the end of 40 years. (i.e.: this magnitude excludes the settlement which would occur during preload treatment).

3.3 Preload Settlement of West Approach Embankment

Figure 6 shows the measured preload 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. 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. The "secondary" part of the settlement curve shows long-term settlement of the west approach embankment at a rate of 160 mm per log cycle (i.e.: 160 mm of settlement for each ten-fold increase in time). With construction completion in approximately 1000 days following fill placement, post-construction settlement at the end of the 30-year concession period was predicted to be about 175 mm near the west abutment. An additional settlement of about 30 mm 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, 215 mm and 205 mm respectively.

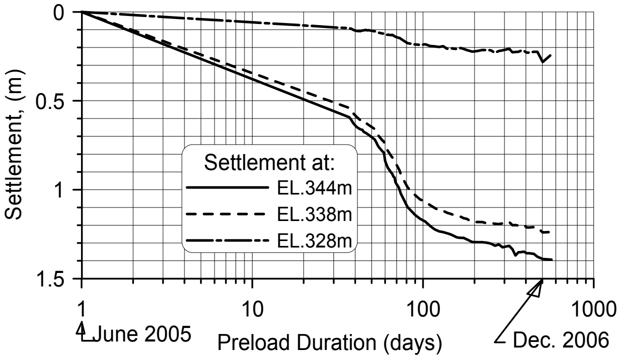


Figure 6. Measured preload settlement near west abutment

3.4 Compression of EPS Fill

The specified minimum requirements of the EPS fill are:

- minimum density of 21 kg/m^3 measured in accordance with ASTM D1622.
- minimum compressive strength of 115 kPa at 5% strain (ASTM D1621, Method A).
- minimum modulus of elasticity of 4750 kPa (ASTM D1621).
- minimum flexural strength of 276 kPa (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. Figure 7 shows the variation of measured modulus of elasticity with density. For comparison, test data from Negussey, (2007) is also shown.

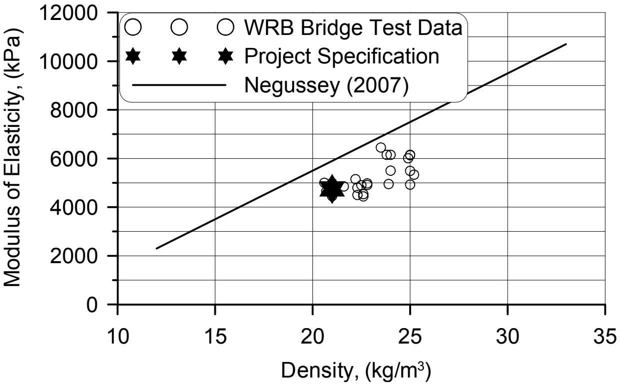
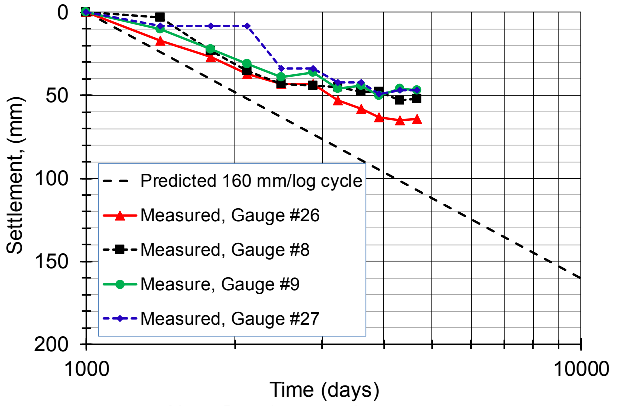


Figure 7. EPS modulus of elasticity

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

3.5 Post-Construction Settlement of West Approach Embankment

Measured post-construction settlement over a 10-year period, from 2008 to 2018 is summarized in Figure 8.



Note: Day 1000 --> June 05, 2008

Figure 8. Post-construction settlement of the west approach embankment

Post-construction settlement was calculated as the difference between the first survey elevation, taken on

June 05, 2008 and that taken in subsequent years. In the analysis and presentation of the data, time was counted from the start of the approach embankment construction in 2005. The time of the first post-construction monitoring in June 2008 corresponds to 1000 days from the start of embankment construction.

Figure 8 shows that the measured settlement magnitudes are less than the predicted rate of 160 mm per log cycle. The measured data show settlement rate of 75 mm to 100 mm per log cycle, and the rate seems to be decreasing with time.

4 EAST APPROACH EMBANKMENT

The east approach embankment is approximately 350 m in length and rises from the existing grade to a maximum of about 7 m above the 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 100 m long segment of the embankment, starting from the east abutment was constructed as preload in May to June 2005. The preload consisting sand and gravel fill, included a surcharge of 2.2 m above the design road elevation. The preload was removed in mid-November 2006 at which time the measured settlement varied between 200 mm and 335 mm.

Figure 9 shows the measured preload settlement data from deep settlement gauges at two different elevations near the east abutment. It may be noted that the primary consolidation was essentially completed 30 days after preload construction.

The “secondary” part of the settlement curve shows a long-term settlement rate of 120 mm per log cycle at elevation 342 m. With construction completion in approximately 1000 days following fill placement, post-construction settlement at the end of the 30-year concession period was predicted to be about 130 mm near the west abutment. An additional settlement of about 25 mm is predicted at the end of 40 years.

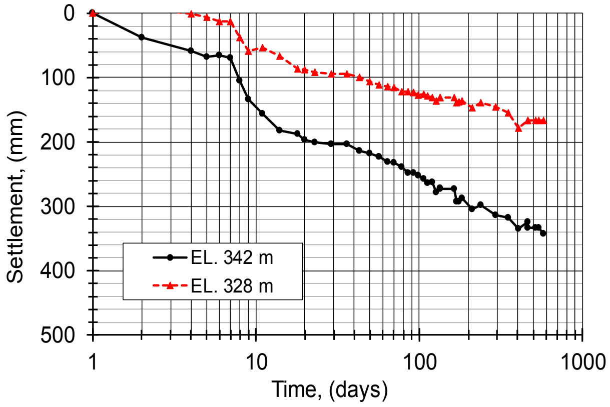


Figure 9. Measured preload settlement near east abutment

4.1 Post-Construction Settlement

Post-construction settlement of the east approach embankment measured over a 10-year period, from 2008 to 2018 is shown in Figure 10. It can be seen that the measured settlement rate for the first year, from 2008 to 2009 is similar to the predicted rate of 120 mm per log cycle. However, the rate of settlement has decreased considerably since 2009 to approximately 50 mm per log cycle, and seems to be decreasing further with time, similar to the trend observed for the west approach embankment.

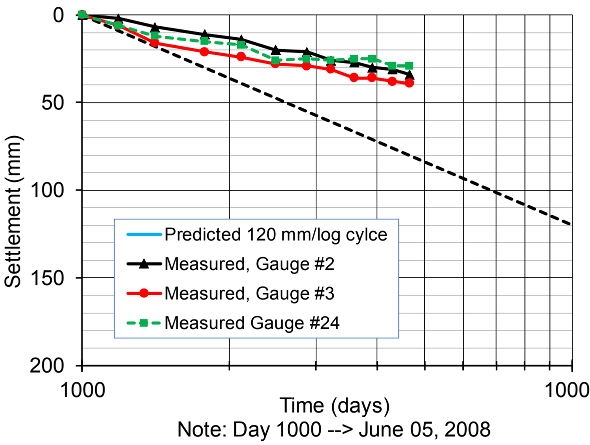


Figure 10. Post-construction settlement of the west approach embankment

5 PILE FOUNDATIONS

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.

5.1 Pile Load Test Program - Pre-Design Phase

A pile load test program was completed prior to the final design and the details are given in Naesgaard et al (2006). The steel pipe piles were 610 mm in diameter with a 12.7 mm thick wall.

Five piles, one central-axial load test pile and four reaction piles, were driven to 45 m depth below the lake bottom. The central pile and two of the reaction piles were driven with closed end plates 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 18 m 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 3,600 kN to 4,000 kN;
- Vibratory driving of the upper 18 m length did not affect the axial capacity;
- Capacities derived from CAPWAP analysis of the PDA test data and that from the static axial load tests were similar;
- Calculated capacities from empirical methods can vary widely;
- Capacities derived using six different methods varied from 4300 kN (LCPC method, Bustamante and Gianeselli, 1982) to 9100 kN (European Method, DeRuiter and Beringen, 1979);
- Residual loads from pile installation can have a significant effect on load distribution along the length of the piles and back-calculation of design parameters from static pile load tests.
- Back-calculated effective stress parameters β and N_t were 0.09 and 22 respectively when residual stresses are considered.
- If the development of residual stresses is ignored, the back-calculated β and N_t were 0.15 and 15 respectively.

The results were used to calibrate the soil parameters for the final design of the 610 mm and 914 mm diameter driven steel pipe piles with various embedment depths. The LCPC method was used for the calculation of pile capacities for the final design.

5.2 Pile Load Test Program - Construction Phase

For the final design of the west side piers and the west abutment 914 mm diameter steel pipe piles with a 12.7 mm 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 610 mm diameter steel pipe piles with a 9.5 mm thick wall. The piles were driven with a closed end to tip elevations varying from 298.8 m to 300.7 m.

PDA tests were carried out on two piles at each of the pier and the abutments to confirm design capacities. Table 1 presents a summary of the pile embedment depths and mobilized capacity during PDA testing. The unfactored ultimate demand on the pile is also shown in Table 1. The PDA tests were carried out using a 5443 kg 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.

5.3 Pile Foundation Settlement

Predicted settlement of the foundations was 50 mm to 100 mm. Construction of the bridge girders commenced in March 2007. Preload settlement data in Figure 6 indicates that the pile foundation settlement could have completed within 6 months of dead load application, before the start of post-construction monitoring in June 2008.

Measured long-term settlement of the pile foundations between June 2008 and July 2018 varies from 0 to 7 mm. Table 2 summarizes the measured settlement in 2018, relative to that in 2008.

Table 1. Summary of mobilized capacity during PDA tests

Location	Embedment Depth Below Mudline, (m)	PDA Capacity, (kN)	Unfactored Ultimate Demand/pile, (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

Table 2. Measured settlement of structure foundations

Location	Settlement in 2018, ten years after construction (mm)
West Abutment	3, 6
Pier 1	1, 0
Pier 2	0, 0
Pier 3	7, 4
Pier 4	0, 0
Transition Pier	0, 0
East Abutment	0, 0

6 DISCUSSION

The instrumentation data collected from the site indicates primary consolidation settlement of the subsurface soils has been completed during the preload treatment phase. It has been assumed that the measured post-construction settlement is due to secondary consolidation settlement.

Secondary consolidation settlement is directly proportional to the logarithmic of time, time after completion of primary consolidation (Lambe and Whitman, 1969). However, the post-construction settlement data obtained from this site over the first 10-year period indicates settlement rate decreasing with log-time. This implies either one of the following causes:

1. The measured data from the settlement gauges (shallow and deep gauges) and the piezometers did not capture the response of soils deeper than the deep gauges. It is probable that primary consolidation was continuing within the deeper soils. However, the strong response of the soils at shallower depths during the preload treatment phase (i.e.: large settlement) masked the response of the deeper soils which were undergoing smaller magnitudes of primary consolidation settlement due to smaller loading than that of the near surface soils.

The post-construction settlement data in Figure 8 seems to indicate a change in slope at about 3000 days, supporting an alternative interpretation that the end of primary consolidation was about 3000 days, or

2. the assumption that secondary consolidation settlement is directly proportional to the logarithmic of time may not be applicable for the soils at this site. The assumption that secondary consolidation settlement being directly proportional to the logarithmic of time has been developed based on laboratory element test data and has been extrapolated to field conditions with multi-layered soils deposited over hundreds to thousands of years. It is a convenient assumption for practicing engineers to predict long term settlement rates for projects which require post-construction settlement predictions over many years - 40, 50 and/or 75 years. The data from this site shows this method of settlement prediction results in safe but somewhat conservative predictions.

The pile foundations supporting the bridge piers were constructed in a circular arrangement and are inclined outwards at 1H:6V. This pile group arrangement resulted in significantly smaller loading on the subsurface soils than those under the approach embankments. Measurements show post-construction settlement of the pile foundations supporting the piers is negligible to less than 10 mm. Unfortunately, no settlement measurements are available for the first two-years, between 2006 and 2008, since the construction of the pile foundations. It is assumed major part of the settlement of pile foundation has been completed within this two-year period.

7 CONCLUSIONS

This paper presents a brief summary of the design and construction of the William R. Bennett Bridge across the Okanagan Lake in Kelowna, B.C. Also, the paper describes settlement analyses of the bridge approach embankments and foundations, preload settlement data, settlement prediction and measured post-construction data. 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 measured 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.

Predicted long-term settlement was 120 mm and 160 mm per log cycle for the east and west approach embankments respectively. However, the measured rate of post-construction settlement over the first 10-year period was found to be about 50 mm per log cycle for the east abutment and 75 mm to 100 mm per log cycle for the west abutment, about 40% to 60% of the prediction.

The bridge piers and abutments supported on driven steel pipe pile foundations have undergone post-construction settlement in the range of 0 to 7 mm over the first 10 years. These measurements indicate well-designed pile foundations for axial loading would perform satisfactorily with regards to long-term settlement.

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