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Challenges from North to South
Des défis du Nord au Sud

Fraser Heights Bridge – Geotechnical Considerations for Top-Down Construction on Difficult Soils

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ABSTRACT
The Fraser Heights Bridge was constructed span-by-span, using a top-down approach, to satisfy the environmental obligations to minimize the impact on an environmentally sensitive wetland. Because of highly variable subsurface conditions, driven pipe piles supporting the bridge extended to almost 45 m depth on the east, but on the west where glacial till-like material was shallow, it was difficult to drive the piles sufficiently deep to obtain the necessary lateral support. Other geotechnical considerations included obstructions (tree trunks, boulders or glacial erratics); artesian pressure; high compressibility and seismic softening of silts, sensitive clays and peat; and, liquefaction and lateral spreading of hydraulically placed sand fills. Another factor related to top-down construction was the minimal time available to address unexpected geotechnical issues such as damaged or obstructed piles.

RÉSUMÉ
Le pont Fraser Heights a été construit section par section, en utilisant une approche de construction de haut en bas. Cette approche visait à répondre aux exigences environnementales au droit d'un milieu humide sensible. Les conditions géotechniques étaient passablement variables le long du trajet et les fondations du pont ont nécessité l'utilisation de pieux atteignant 45m de long. Il était parfois difficile de foncer les pieux jusqu'à la profondeur requise à l'obtention du support latéral minimum dû aux conditions géotechniques variables. Parmi les autres défis rencontrés, on note: des obstructions (tronc d'arbres ou blocs), des venues d'eau artésiennes, des matériaux à compressibilité élevés, des silts remaniés sismiquement, des argiles sensibles et de la tourbe. De plus, des problèmes de liquéfaction et d'affaissement latéraux des remblais de sable placés hydrauliquement ont également été rencontrés. Un autre défi relié à la méthodologie de construction utilisée fut le peu de temps disponible pour solutionner les problèmes géotechniques rencontrés, tels que les pieux endommagés ou obstrués.

1 INTRODUCTION

The Fraser Heights Bridge (FHB) was constructed as part of the Port Mann/Highway 1 (PMH1) Improvement Project which included a 2 km section of the South Fraser Perimeter Road referred to as the Fraser Heights Connector (FHC). The FHC which included the FHB is located in northeast Surrey BC (Figure 1).

Design and construction requirements were detailed in the Design-Build Agreement (DBA) for the project. Two major determinants were: 1) a mandated tight schedule so completion of the FHB would be ahead of the new Port Mann Bridge to provide a non-tolled alternative; and, 2) an obligation that construction had to occur with minimal impact on the environmentally sensitive wetland the FHB crossed. This latter requirement resulted in the bridge being constructed span by span, using a top-down approach.

This paper discusses the geotechnical design and construction of the FHB and the particular considerations that arise from a top-down construction approach. Construction commenced in April 2011 and the bridge opened to traffic in December 2012. The estimated construction value of the bridge was about $25 Million.

2 DESIGN CRITERIA

2.1 Environmental

The environmental obligations to protect the wetland were:

- The footprint of the permanent and temporary works combined must not exceed 27 m^2 (later increased to 45 m^2)
- Permanent and temporary works must not be placed in the watercourses.

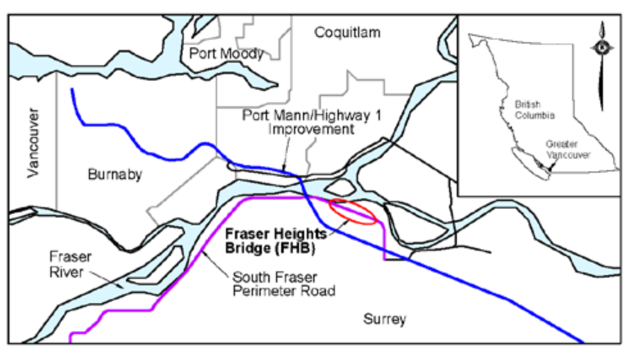

Figure 1. Site location.

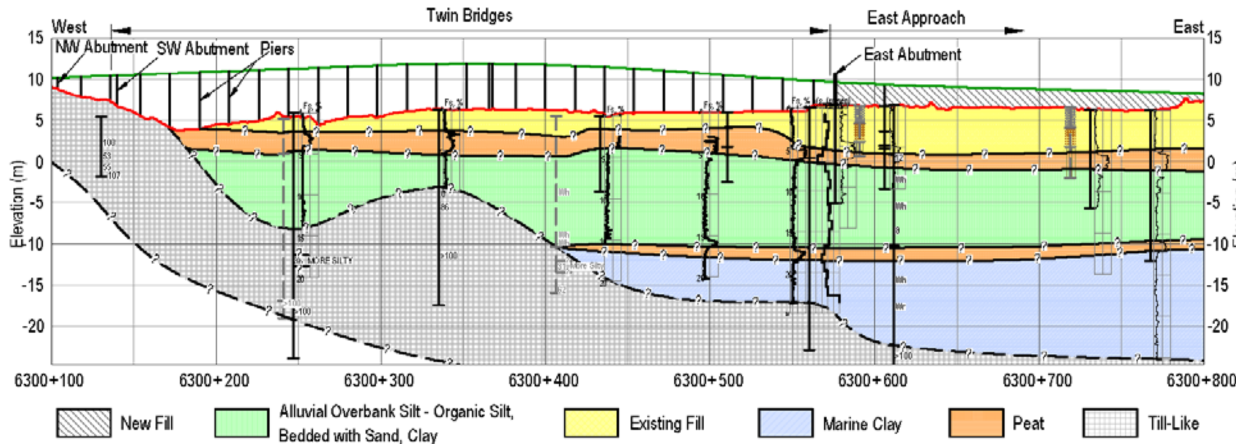

Figure 2. Soil profile.

- Watercourses must not be temporarily or permanently diverted.
- Construction equipment was only allowed on the wetland on temporary trestles or on the finished structure.

2.2 Settlement

The following criteria applied to pavements and embankments over the 5-year Warranty Period:

- Smoothness and cross-slope requirements must be met.
- Ponding and sheeting of water must be prevented.
- Pavement drainage must be maintained.
- Function of culverts and ditches must be preserved.

In addition, total settlements of embankments were not to exceed 300 mm with respect to adjacent grades, but at bridge approach fills, greater settlement was permissible provided that angular distortions of the roadway surface did not exceed 1/200 over the length of the approach fill.

The approach slabs were required to be full-width and 6 m minimum length, and over the 75-year design service life, the maximum differential settlement had to be less than 100 mm.

2.3 Seismic

The FHB was classified as an Economic Sustainability Route structure which meant it was required to satisfy the following post-seismic return-to-service and repairability performance objectives:

- Immediate use by emergency vehicles following the 475-year design earthquake.
- Return to full service after repairs, following the 975-year design earthquake.
- No collapse, with non-repairable damage acceptable, for the 2475-year design earthquake or for the Cascadia Subduction event.

3 SITE CONDITIONS

3.1 Description

The site extended from the Fraser Heights glacial uplands in North Surrey out over a peat wetland in the floodplain of the Fraser River. The Fraser River was about 1 km from the site. The CN Rail Intermodal yard bounded the north side of the site and had been built about 20 years before by pumping dredged river sand onto the peatlands. This hydraulic fill extended partly onto the footprint of the east approach fill and intermittently along the bridge alignment.

3.2 Subsurface Soils

The soil profile varied significantly along the bridge as shown in Figure 2. As a result of the environmental access restrictions onto the wetlands, 6 boreholes spaced at about 80 m intervals were available along the alignment. Particulars of the subsurface profile (Figure 2) were:

- West: Dense glacial materials at shallow depth.
- Central: Variable thin fill over up to 4 m of peat, over 3 m to 10 m of overbank silt, over dense glacial material.
- East: Between 1 m to 5 m of hydraulically placed sand fill, over about 3 m of peat, over about 13 m of overbank silt, over a thin buried amorphous peat, over up to 5 m of sensitive marine clay, over dense glacial material.
- The glacial materials were described as till-like as they tended to be more silty and in places had lower and more variable Standard Penetration Test (SPT) N-values than was normally associated with local glacial till.

3.3 Groundwater

Standing water and flowing watercourses were present, and in addition to seasonal fluctuations related to flood stages in the Fraser River, beaver activity resulted in unexpectedly sudden changes in surface water levels. Excess piezometric heads of up to 12 m above ground surface were measured near the contact between dense glacial material and the overlying clay deposits.

4 BRIDGE STRUCTURE

4.1 Considerations

Careful engineering was required to overcome construction and environmental constraints, approach fill settlement and seismic performance requirements. Construction equipment was not permitted onto the wetland and drainage runoff could only be discharged into the wetland once it had been treated. Surcharging of the east approach fill, and a lack of access to the west abutment, governed the construction start time and access of equipment and materials.

Being part of a competitive Design-Build project, a simple, cost-effective approach that managed construction risks was required. The low-height bridge, about 4 m above the wetland, results in a relatively stiff substructure. However, a long continuous superstructure was preferred to minimize the number of expansion joints, optimize vehicle ride quality, and maximize seismic performance. Given the compressible soils present, piles were necessary and potential plastic hinges needed to be accessible for inspection. Firm ground seismic effects were amplified, and the variability of soil conditions across the site had to be addressed. The key issue was to find and integrate the best solution to meet all of these demands well, without compromising on any mandatory or key requirement.

There was minimal time available to address construction issues as delivery was required one year after start of construction to provide the non-tolled alternative. Two parallel spans had to be complete and operational every 10 days. Every construction component was on critical path.

4.2 Top-Down Construction Approach

The environmental constraints meant that either the use of a work trestle, or that the permanent bridge and/or piles was required to support the construction equipment. The work trestle option was quickly ruled out because of cost, but more significantly, the impact of its wetland footprint would have been unacceptable. Accordingly, the focus shifted to developing a rapid construction design using top-down, span-by-span construction (Figure 3).

4.3 General Arrangement

The FHB consists of two parallel, 472 m and 436 m long, low-height, pile-supported trestles with steel girders and 250 mm full-depth precast concrete panelized decks constructed using top-down methods. Each trestle is 11 m wide and carries two travel lanes plus shoulders. The trestles are separated by a gap of approximately 4 m to allow sunlight through to the wetland habitat below.

The bridge has relatively short 18 m spans which were selected to avoid excessive reach demands on the construction crane. The west end spans were reduced to approximately 14 m length, and the east end span is a special 18 m long simple span which incorporates jacking provisions to compensate for possible long-term settlement of the east approach fills. The approach fills at the abutments were about 4 m high.

As shown in Figure 4, the intermediate support bents consisted of two (per trestle), 762 mm diameter by 19 mm wall thickness, driven, steel pipe piles, with specified yield strength of 310 MPa. The piles were extended above ground and connected to steel box-section cap beams. The piles and their connections were reinforced and filled with concrete. There were a total of 112 piles.

To obtain continuous, joint-free superstructure lengths of up to 250 m between the single, mid-bridge expansion joint and abutments, without applying excessive displacements to the supporting piles under thermal straining, laminated elastomeric bearings were placed beneath the girders. The bearings absorb much of the thermal strains via shear deformation, with the remainder of the strain being accommodated by flexure of the steel pipe piles. The relative distribution of these strains between bearings and piles varies significantly with the extended pile lengths and the variation in soft soil depth above the dense till-like material.

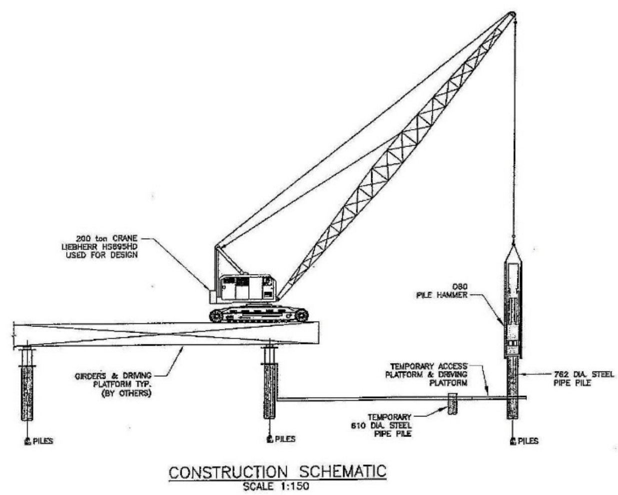

Figure 3. Top-Down construction schematic

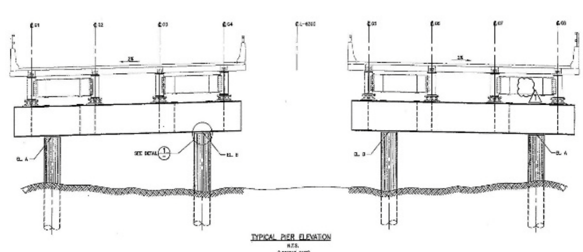

Figure 4. Typical bridge elements

5 SEISMIC DESIGN

5.1 Performance-Based Design

The seismic criteria required a performance-based design approach which in turn requires the use of a displacement-based design and component performance assessment, rather than a force-based method. In the latter method, elastically calculated flexural demands are reduced for ductility, with the levels of damage or other limit states implicitly considered adequate, but unchecked. For a hybrid seismic system (i.e., isolation bearings with potentially yielding piles in extreme events), a displacement-based approach was able to explicitly address the design issues (Kennedy et al., 2013).

5.2 Structural Response Spectra

Site-specific ground response analyses were performed using the computer program SHAKE2000 (Ordonez, 2008) and the ground motions (acceleration-time histories) provided for the project as a part of the DBA. Two site-specific response spectra were provided for use in the structural analyses of the western and eastern sections of the bridge structure. As shown in Figure 5, varying degrees of amplification or de-amplification of response would occur for short/long period ranges because of the difference in depth to firm ground and characteristics of the overlying soft soils within the two sections.

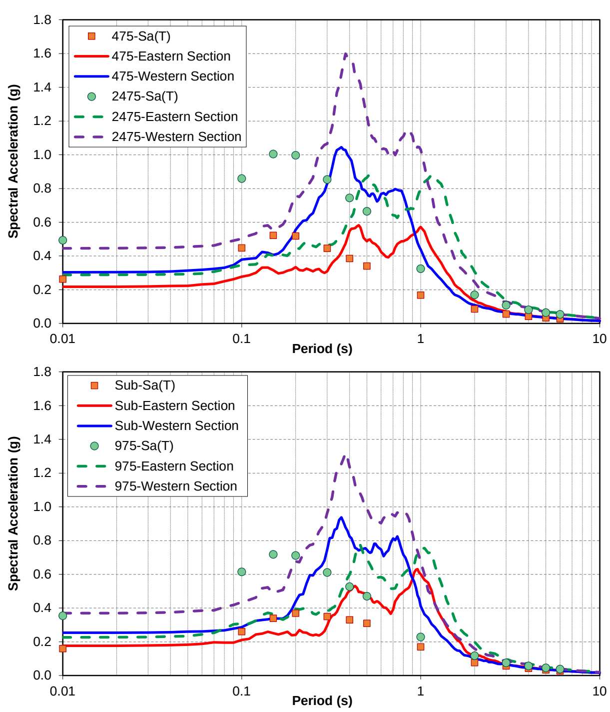

Figure 5. Structural response spectra for the eastern and western sections of the bridge structure

5.3 Assessment of Liquefaction / Cyclic Softening

The Cyclic Stress Ratios obtained from the SHAKE analyses were used to assess the liquefaction susceptibility using the approach recommended in ATC-49, Appendix D, which is based on the 1996 NCEER Workshop (Youd et al., 2001). Cone Penetration Test (CPT) data were used to calculate the Cyclic Resistance Ratios. The liquefaction triggering analyses showed that hydraulic fill layers underlying the east approach would liquefy during the seismic events.

Seismic/cyclic softening of fine-grained materials was evaluated using the methods recommended in the 2007 Task Force Report, including Bray and Sancio (2006). It was found that localized pockets of overbank silt may be prone to seismic softening.

5.4 Lateral Soil Resistance (p-y curves)

Site-specific load-deflection (p-y) curves were developed using the program LPILE v5.0 (Ensoft, 2004). The soil model and respective parameters were chosen based on the materials encountered in the boreholes, the CPT results, published empirical relations, and previous experience in similar soils. The contribution of soil resistance in peat was ignored.

5.5 Bridge Design

Although the structural arrangement was set based on fabrication, erection, thermal articulation and durability factors, it was recognized early that the substructure and bearing design would also provide a robust lateral load resisting system for seismic requirements

Exploiting the seismic isolation characteristics of the elastomeric bearings would also increase the seismic resilience of the bridge system, reduce damage, and allow return to full service more quickly. The use of isolation bearings in series with the extended steel pipe piles on deep, soft soil could lead to excessive flexibility, large seismic displacements, and possibly an inability of extended piles to allow the isolation bearings to strain as intended.

5.6 Post-Elastic Behaviour

The lateral load resisting system was defined as “seismic isolation”. This system carried significant costs in bearing testing and quality control. However, it provided the benefit of shifting the structural period by more than 1.5 seconds, which was necessary to achieve meaningful reductions in inertial demands. Table 1 shows the periods in the first three modes in each direction using the global model developed for use with the 475-year event. Table 2 shows the lateral displacements of the superstructure and the extents of yielding (plastic hinging) in the seismic gap joints and piles for each event. The levels of yielding, strains and damage levels in the pile hinges were determined. The isolation system proved extremely robust and very effective, with only minor plastic behaviour occurring in select piles for severe (975- and 2475-year) events.

Table 1 – Modes and periods of vibration

Mode	Natural Period (s)	
	West	East
Longitudinal	1.70	2.18
Transverse 1	1.49	1.67
Transverse 2	1.33	1.58

Table 2 – Seismic displacements and expected yielding

Event	Max Disp. (mm) Trans/Long		Expected Yielding
	West	East	
475	154/109	165/146	None
975	212/154	227/211	At gaps of several bents
2475	341/253	363/325	At gaps of several bents Minor yielding of several piles

Linear elastic response spectrum analyses were performed for design and proportioning of structural members and bearings. Static non-linear pushover analyses were performed to identify bent system behaviour by including the linear isolation springs, non-linear soil behaviour, and plastic hinging at the seismic gap at the top of the piles, or within the body of the piles below grade. Various sectional behaviours were characterized using confined concrete properties at the gap joint, confined concrete composite columns of the piles below grade, and secant springs of the soils along the piles. A final non-linear time history analysis, building on all of the modeling and properties developed for earlier analyses, was performed for the 975-year earthquake event.

6 GEOTECHNICAL DESIGN

6.1 East Approach

The east approach required particular considerations because improvement of the soft compressible soils was necessary to avoid static/seismic embankment instability and to mitigate long-term settlement; but, at the same time, provide access for top-down bridge construction. A further consideration was that the anticipated settlement and lateral soil displacements during surcharging would impose large and unacceptable lateral loads on the abutment piles.

A complication in designing the ground improvement was that portion of the east approach had been filled with hydraulic fill resulting in a partially preloaded footprint. Figure 6 shows the soil profile and the anticipated settlement and the tendency for increased settlement on the south side of the approach fill.

Various alternatives were considered for constructing the east approach including a piled embankment, polyurethane grouting, lightweight fill, etc., but ultimately after many meetings of the design team, the following innovative design and construction approach was adopted:

- Install wick drains and place surcharge in stages to design height (nominally 2 m above final grade).
- Partially remove the surcharge to allow installation of the piles for the first and second trestles from the approach fill (i.e. not the abutment piles).
- Construct the second and third bridge spans.
- Replace the surcharge to design height.
- Install two simply-supported temporary bridges, designed to accommodate about 1 m of settlement at the east end. These bridges extended from the top of surcharge down onto the completed second span of each bridge and provided access for the crane and trucks delivering material.
- Continue bridge construction westwards towards the west abutment.
- Once arrived at the west abutment, dismantle the crane and relocate it to drive the east abutment piles.
- Remove the temporary bridges and surcharge at the east abutment, and drive the east abutment piles.
- Construct the east span as a rotating span able to allow adjustments by jacking in the event of long-term settlement.
- Complete the east approach fill using expanded polystyrene as shown in Figure 6.

6.2 Foundations

6.2.1 Geotechnical Resistance Factors

Because it was planned to undertake dynamic pile testing prior to construction to confirm pile capacity, a resistance factor of 0.5 was used for static loading and the 475-year seismic event. In addition, based on AASHTO (2007) and ATC-49 (2003), a resistance factor of 1.0 was recommended for the 975- and 2475-year earthquake as well as the Subduction events.

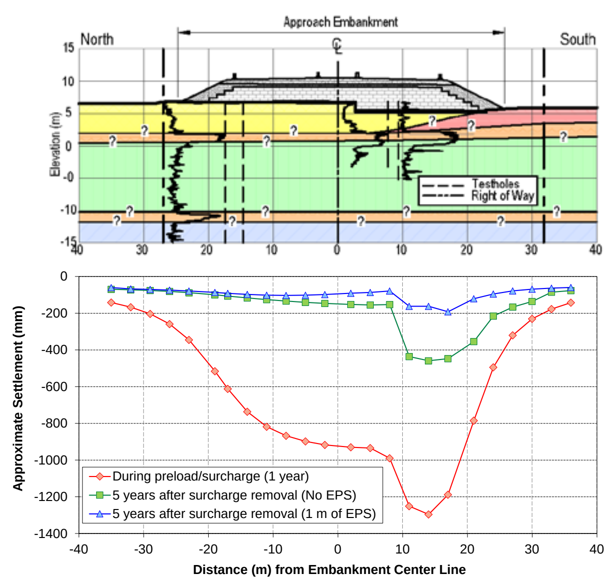

Figure 6. East approach fill with expanded polystyrene lightweight fill, soil profile and predicted settlement.

6.2.2 Pile Details

The calculated maximum service load was 2,100 kN. The maximum factored load was 2,700 kN, which corresponded to the static load case (i.e. design was governed by service conditions). This resulted in a required unfactored ultimate geotechnical capacity of 5,400 kN for each pile.

The upper 11.5 m of each pile was concrete filled, except that for shorter piles, the concrete fill was placed to within 1.5 m of the pile tip. A rebar cage with 2.6% longitudinal reinforcing steel was placed inside the pile to form the cap connection, as the steel pipe was curtailed 50 mm below the soffit of the pile cap. This connection was intended to form a plastic hinge if sufficient seismic displacement occurred in order to capacity-protect the pile cap. The rebar cage penetrated into the pile cap through a bottom flange cut-out.

The nominal spacing of each pair of trestle piles was 7 m; the cut-out was extended 300 mm transversely to accommodate the pile tolerance of 150 mm. This tolerance was found to be reasonable during installation and the maximum deviation was 100 mm. Kennedy et al. (2013) present further details of the capacity-protected pile-to-cap connection.

6.2.3 Pile Axial Capacity

Open-ended pipe piles were selected in order to facilitate penetration into the dense glacial till-like material and to make it easier to maintain tolerances. The Meyerhof (1976) and Beta methods as outlined in CAN/CSA-S6-06, AASHTO (2007), and CFEM (2006) were used to estimate unfactored Ultimate Limit State (ULS) capacities.

Pile penetration into dense glacial till-like material in the order of 3 m to 6 m were anticipated for plugged piles. For unplugged or partially plugged conditions (e.g. Randolph et al., 1991), the required penetration was anticipated to be about 6 m to 8 m. In addition, a minimum embedment depth of 4 m in till-like material was required for the piles near the west end to develop the required fixity to resist lateral loading.

6.2.4 Driveability and Installation Methodology

From the outset, it was recognized that pile installation would be difficult and that there was significant risk of piles encountering obstructions and/or not obtaining sufficient penetration into till-like materials to satisfy lateral loading demands.

In top-down construction, the appropriate selection of the pile installation methodology is crucial because the hammer size and weight determine the crane capacity, which can affect the design of the bridge. Furthermore, if a pile is obstructed or damaged, access onto the wetland to perform remedial work is difficult and expensive. It also requires special environmental permissions and has major schedule impacts because work cannot proceed. The then-available environmental permit prohibited the use of machinery on the wetlands.

The following installation risks were identified:

- Variable depth to dense glacial material: Because of the limited number of boreholes along the alignment, this was a risk with no simple mitigation.
- Obstructions: These included both boulder erratics associated with the glacial materials and also large organic debris and tree trunks in the peat and overbank silt deposits.
- Pile layout: The pile layout was such that additional piles could not be installed without affecting seismic performance of the bridge.
- Artesian pressures: Open-ended pipe piles driven into artesian layers would be difficult to clean out if the artesian layers were not sealed off and/or the piles do not plug.
- Dynamic pile testing: The hammer must be sufficient to mobilize the piles during dynamic testing otherwise the results may not demonstrate that adequate axial capacity has been achieved

Ultimately, based on driveability studies and contractor input it was decided that the pile installation should consist of open-ended piles vibrated to dense material, followed by impact-driving to set using a D80 diesel hammer. The hammer was specifically oversized to handle obstructions and mobilize piles sufficiently during dynamic testing. The crane was supported by a temporary work platform, supported by the permanent substructure. Providing a work platform was more cost-effective than strengthening the complete westbound superstructure.

7 CONSTRUCTION

7.1 General Sequence

The construction was arranged so that the 250 tonne erection crane was supported by a work platform resting on the permanent trestles. The crane travelled along the westbound (northern) trestle which allowed the parallel eastbound trestle to be utilized for access and delivery of construction materials. At each location the crane installed the four piles beneath the next trestle.

Figures 7 and 8 illustrate the arrangement of the two bridges with the erection and pile driving crane working from the construction platform.

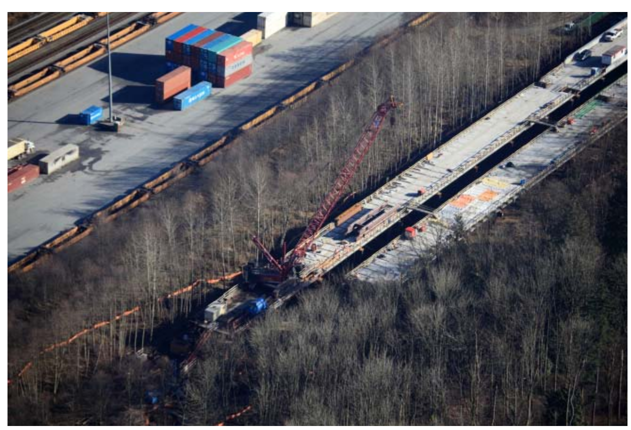

Figure 7. Aerial view of bridges under construction.

Figure 8. Pile driving crane and hammer.

Cleanout proved very time-consuming, and was undertaken using a clamshell bucket as the originally intended method of using vacuum excavation proved ineffective because of the fibrous nature of the peat. Concerns of encountering artesian conditions during pile cleanout did not materialize, likely because the piles penetrated and sealed potential artesian layers.

7.2 Pile Installation

7.2.1 Pile Lengths

The piles were driven through the soft upper soils into the underlying dense glacial till-like deposits. However, there was significant variation in length as shown on Figure 9. The installed pile lengths varied from about 8 m to 45 m.

7.2.2 Obstructed and Damaged Piles

A total of 16 piles stopped on obstructions or were damaged and had to be drilled out, replaced or remediated. The obstructions consisted primarily of large boulders in the till-like deposits but on two occasions shallow refusal occurred on buried logs in the peat and overbank silt deposits.

Shallow boulders near the west abutment outside of the wetland area were removed relatively easily using an excavator. Where deeply buried boulders obstructed pile driving, prevented sufficient pile capacity from being achieved, or damaged the piles, an HP 360x174 pile (referred to as a stinger pile) was driven down the inside of the pipe piles to supplement the bearing capacity.

Five obstructed piles were extracted and after cutting off the damaged ends, were re-driven in conjunction with drilling out. Inspection of the extracted piles indicated that in most cases, the damage took the form of the piles folding in on themselves. This had the effect of making it necessary to extract a pile and cut off the damaged end before undertaking remedial work such as drilling-out or driving stinger piles.

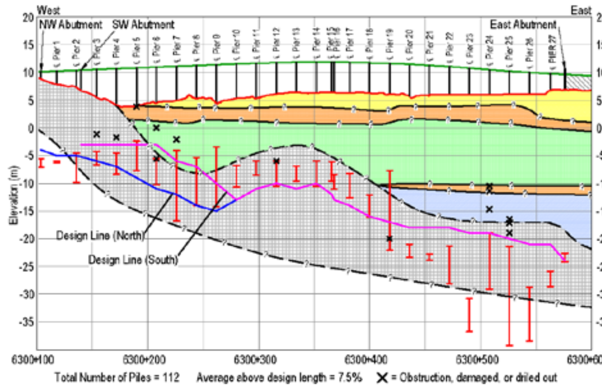

Figure 9 Variations in installed pile lengths.

In many cases plugging did not occur and this was considered to result from "fluffing" up of the soils within the pile during driving, possibly because of the effects of artesian pressures.

7.2.3 PDA Results

Figure 10 summarizes the Pile Driving Analyzer (PDA) test results along the alignment. The PDA capacities were estimated using the Case Pile Wave Analysis Program (CAPWAP).

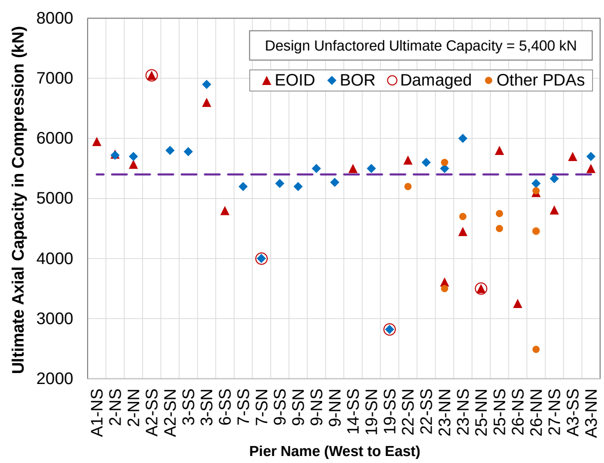

Figure 10. Summary of PDA test results.

Figures 11 and 12 show the variations in the unit shaft and toe resistance obtained from the CAPWAP signal matching process. Figures 13 and 14 show the back-calculated Beta and N_t coefficients. Data plotted in these figures are shown for three groups of piles (where PDA was available): 1) Abutment 1 to Pier 7 (western section), 2) Piers 9 and 14 (central section), and 3) east of Piers 19 to Abutment 3 (eastern section).

These results indicate the following:

- There was a significant variability.
- Unit shaft resistance in till-like soils rarely exceeded 100 kPa; typically ranged from 25 kPa to 75 kPa; and, could be as low as 15 kPa over a significant length of pile embedded in such soils.

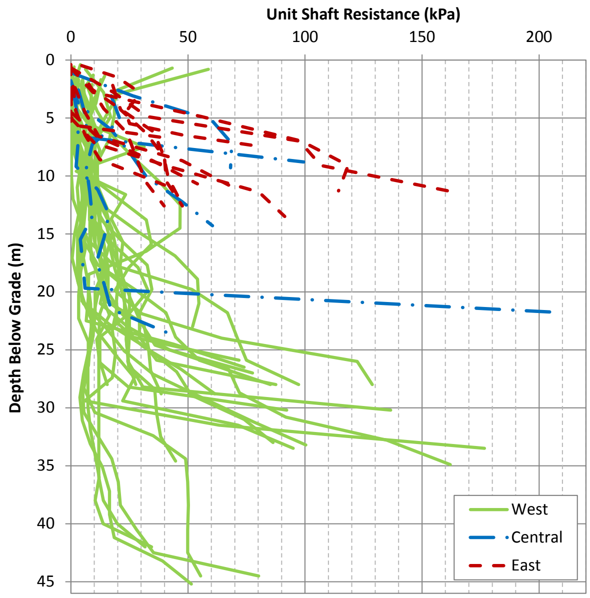

Figure 11. Unit shaft resistance (kPa) from CAPWAP.

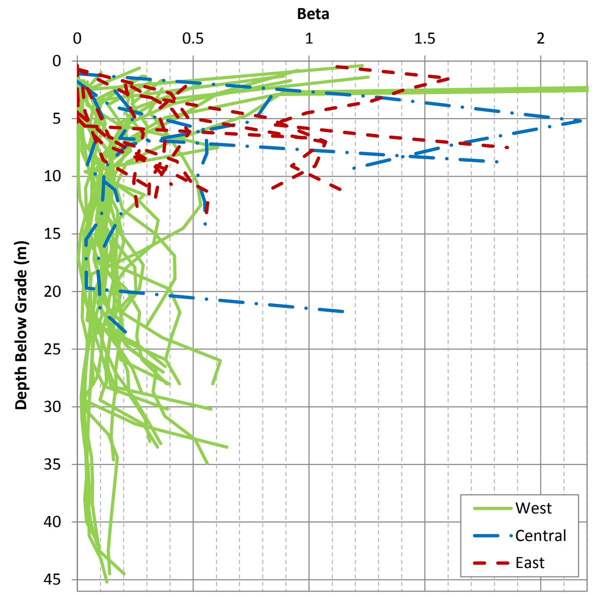

Figure 13. Back-calculated Beta coefficients.

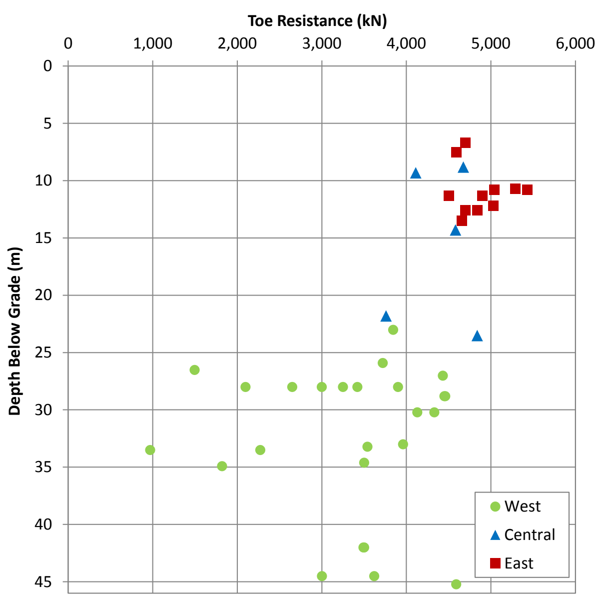

Figure 12. Unit toe resistance (kPa) from CAPWAP.

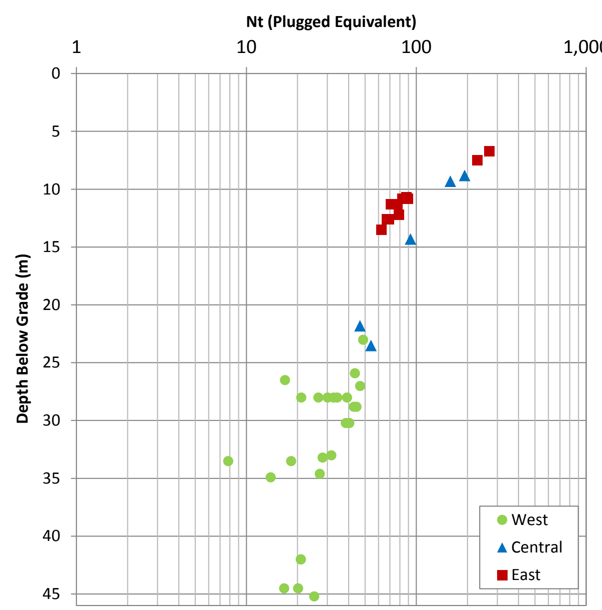

Figure 14. Back-calculated N_t coefficients.

- Unit toe resistance in till-like soils calculated for an equivalent plugged condition (closed-ended pipe area of 0.45 m^2) typically ranged from 10,000 kPa to 12,000 kPa in the western section; and, from 6,000 kPa to 10,000 kPa in the eastern section, with values as low as 2,000 kPa.
- Unit shaft and toe resistance did not have a direct correlation with increasing depth (or vertical effective stress). However, for a given pile, the unit shaft resistance generally increased with depth.
- Back-calculated Beta values for till-like soils typically ranged from 0.3 to 1 in the western section (where till-like soils are at shallow depths); and, from 0.05 to 0.3 in the eastern section (where these soils are at great depths). For comparison, the recommended range in CFEM (2006) for “Dense sand” is 0.8-1.2.
- Back-calculated N_t values for till-like soils typically ranged from 60 to 200 in the western section (where these soils are at shallow depths); and,

from 20 to 50 in the eastern section (where these soils are at great depths). The recommended range in CFEM (2006) for “Dense sand” is 100-120.

8 CONCLUSIONS

The completed structure appears as a simple and functional structure, but this belies the difficult soil conditions, environmental obligations prohibiting access onto the wetland, stringent seismic performance criteria, and mandated tight schedule for bridge completion. The competing requirements were addressed by extensive interaction between the designers and bridge constructors to integrate structural and geotechnical engineering with construction methodology.

Design features provided to address the difficult conditions included:

- Temporary bridges to provide access over the top of the settling approach fill surcharge to deliver materials and equipment to allow construction to proceed.
- Trestle-type piers which consisted of driven pipe piles (two piles per trestle) which extended above ground and connected directly into steel box-section cap beams. This type of cap was used because precast concrete caps would have required a larger cane than was available and would have increased seismic demands.
- Elastomeric isolation bearings which accommodated thermal strains and minimized expensive deck joints while meeting the seismic performance requirements.
- A pre-fabricated superstructure comprising steel girders and full-depth precast concrete deck panels was selected for its rapid installation, light weight, and ability to immediately support construction loads.
- Rotating east end spans with jacking provisions to compensate for long-term settlement of the east approach fill.

Top-down construction requires the following specific geotechnical considerations:

- There is invariably insufficient geotechnical borings because the environmental constraints which necessitated top-down construction typically mean that unimpeded access is not available for comprehensive soil explorations.
- Construction proceeds in a sequential manner with each operation being required to be completed before moving onto the next. Therefore, any time spent solving unexpected geotechnical issues such as obstructed piles, directly impacts schedule.
- There is limited time to undertake dynamic pile testing, and when undertaken, it is typically based on end-of-initial drive conditions. This can result in conservative pile capacities but can be partially offset by developing reliable estimates of likely pile set-up.
- Piles that stop short of the required embedment depth for lateral capacity have much greater cost and schedule impact than piles that drive too deep.

The following are recommended to address the issues:

- Undertake as many geotechnical explorations along the alignment as permitted.
- Undertake a detailed preconstruction pile test program to assess pile installation methodology, pile driveability, and develop pile set-up versus time relationships.
- Develop detailed contingency plans to address likely geotechnical problems such as obstructions and damaged/obstructed piles. This could include staging suitable materials and equipment for repairs onsite, having environmental permits to enter onto the wetlands ready, etc.
- Allow time in the schedule to deal with the unexpected.

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