Highway 27:10 – Replacement of Red Deer River Bridge East of Morrin, Alberta: Abutment Stability Pile Wall

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ABSTRACT
Highway 27:10 crosses the Red Deer River west of the town of Morrin, Alberta, via a bridge built in 1959, which required replacement by its owner, Alberta Transportation. Site investigations indicated that the crossing was underlain by a steeply dipping bedrock surface and that the future eastern abutment area featured a 2.5 m thick and weak bentonite layer at a depth of approximately 15 m, overlain by native compressible clays. Stabilization was undertaken using a cast-in-place concrete pile wall located at the toe of the maximum 12 m high abutment fill. Instrumentation of the pile wall and abutment area allowed for the monitoring of slope and pile movements as well as the dissipation of pore pressures prior to commencing the installation of driven steel abutment piles. This work included several design, construction, and quality assurance challenges in a saturated riverbed environment.

RÉSUMÉ
Alberta Transportation a remplacé un pont, érigé en 1959, traversant la rivière Red Deer sur l'autoroute 27:10 à l'ouest de Morrin, Alberta. Les études géotechniques ont dénoté une forte inclinaison de la surface du socle et la présence d'une couche de 2.5 m de bentonite à environ 15 m de profondeur recouvert d'un sol naturel composé d'argiles compressibles à l'emplacement suggéré de la culée Est. Afin de stabiliser un remblai de 12 m de haut, un mur de soutènement composé de pieux de béton coulés en place a été construit à la culée Est. Des instruments érigés dans le mur et à l'emplacement de la culée ont permis de surveiller les déplacements du talus et du mur, ainsi que la baisse des pressions interstitielles avant le battage des pieux d'aciers de la culée. L'emplacement de l'ouvrage dans un lit de rivière saturé occasionna plusieurs défis lors de la conception au niveau de l'assurance de la qualité lors de la construction.

1 INTRODUCTION

Highway 27 is a 151 km long west-east running provincial highway that extends across central Alberta, terminating in the east at an intersection with Highway 56, east of the town of Morrin, Alberta. Approximately 9 km west of Morrin the highway travels over the Red Deer River, which divides Starland and Kneehill Counties. The average annual daily traffic for this portion of the highway is approximately 1,800 vehicles. Figure 1 presents a satellite image of the project site.

The crossing of the north-south running Red Deer River was serviced by a bridge built in 1959. This bridge provided an important link between the two counties, and accommodates truck traffic bypassing the Town of Drumheller, with the next nearest bridge crossings located approximately 22 km to the north (Highway 585) and 28 km to the south (Highway 56/Bridge Street).

The bridge was approximately 178 m long, with a 7.3 m wide roadway, and featured two 61 m long through truss spans with 28 m long rolled beam spans at both ends. The four-span bridge was supported by abutments founded on driven steel H-piles and three concrete river piers founded on driven untreated timber piles. Figure 2 presents a photograph of the structure along the Red Deer River.

Based on assessment of the bridge by its owner, Alberta Transportation (AT), the bridge had neared the end of its service life due to its poor condition, as well as due to horizontal and vertical (from the overhead steel truss) clearance restrictions and was identified for rehabilitation or replacement.

A preliminary engineering assessment of these options was completed in 2008 based on a preliminary geotechnical investigation completed in 2006 that included test holes on each bank and one test hole drilled in the river from a constructed ice bridge.

Based upon the preliminary work, it was recommended to explore the replacement of the bridge with a new structure on an offset parallel alignment, located approximately 20 m to the north of the existing bridge.

Based upon the 2006 geotechnical investigation, two major geotechnical issues were identified that would affect the future development of a new bridge structure:

- The east abutment area is underlain by a weak bentonite layer which was expected to control the global stability of a new east abutment fill.
- The subsurface stratigraphy based on the advancement of test holes contains an intact bedrock contact that dips downward considerably from east to west with competent bedrock not being encountered at the west abutment at a total explored depth of 33 m below existing ground.

Thurber was subsequently retained by the selected prime design consultant, CH2M Hill (now Jacobs Engineering Inc.) in 2012 to provide geotechnical engineering services for the detailed design, tendering, and construction phases of the bridge replacement project.

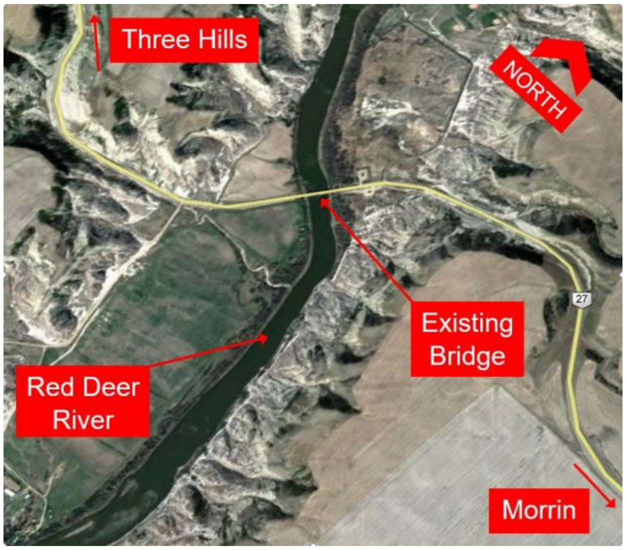

Figure 1. Satellite image showing site location


Figure 2. Existing bridge configuration looking from west to east

An additional geotechnical investigation for the project was completed in 2014 to further characterize the previously identified bentonite seams and steeply dipping bedrock surface. This included two test holes drilled from a constructed ice bridge at the proposed pier locations.

Design support was also provided for the new offset bridge structure which consisted of a three-span 140 m long steel girder bridge with a 12 m wide roadway and an estimated construction cost of CAN$21,300,000.

The new bridge consists of a three-span arrangement supported by two concrete river piers founded on driven steel H-piles, with the center span extending 64 m in length and spans of 38 m on each end between the piers and abutments.

The design also included a cast-in-place concrete pile wall at the toe of the east abutment head-slope in order to stabilize the area due to the additional loading caused by the placement of abutment fills of approximately 10 m in height above native ground over the area with the underlying compressible clay and bentonite layers.

This paper provides a summary of the design, construction, and quality assurance challenges of this wall in a saturated riverbed environment. It also includes a summary of the instrumentation that was installed to monitor the responses of the pile wall and foundation soil and bedrock to the construction of the overlying bridge abutment fill and foundation elements.

2 SITE DESCRIPTION

2.1 General

Two geotechnical investigations, consisting of a total of eight test holes advanced using both auger and coring methods were carried out in 2006 and 2014. These test holes were advanced in both the abutment locations as well as in the river during the winter months from a constructed ice bridge.

The studies characterized the subsurface conditions and provided geotechnical recommendations for the bridge design as well as for other complimentary components of the project. A simplified stratigraphic cross-section with the test hole information along the bridge alignment is presented in Figure 3.

2.2 Surficial and Bedrock Units

Surficial geology maps indicate that the bridge crossing is located on stream and eroded deposits. These generally consist of exposed till and bedrock and slumped material from valley walls in addition to typical alluvial deposits such as silts and clays (Shetsen 1987).

The dominant bedrock in the uplands is the Paskapoo Formation consisting of sandstone and siltstones with some clay shale and coal. However, the Red Deer River valley is eroded through this unit exposing the Kneehills Member (bentonitic shale and white sandstone) of the Edmonton Formation in the valley walls. The Edmonton Formation is composed of sandstone, siltstone, clay shale and coal beds. Underlying the Edmonton Formation is the Bearpaw Formation clay shale (Borneuf 1970).

Based on bedrock topography mapping, the bedrock elevation in the uplands above the Red Deer River valley is approximately 790 m where drift thickness is likely between 15 m and 30 m. The mapping shows that the depth to bedrock should be less than 15 m in the Red Deer River valley and the bedrock could be exposed in many areas (Carlson 1969).

The estimated groundwater elevation is 760 m at top of the valley. Groundwater flow in bedrock aquifers is generally horizontal to the Red Deer River. There is some minor downward flow and valley wall seepage in the overlying drift material (Borneuf 1970).

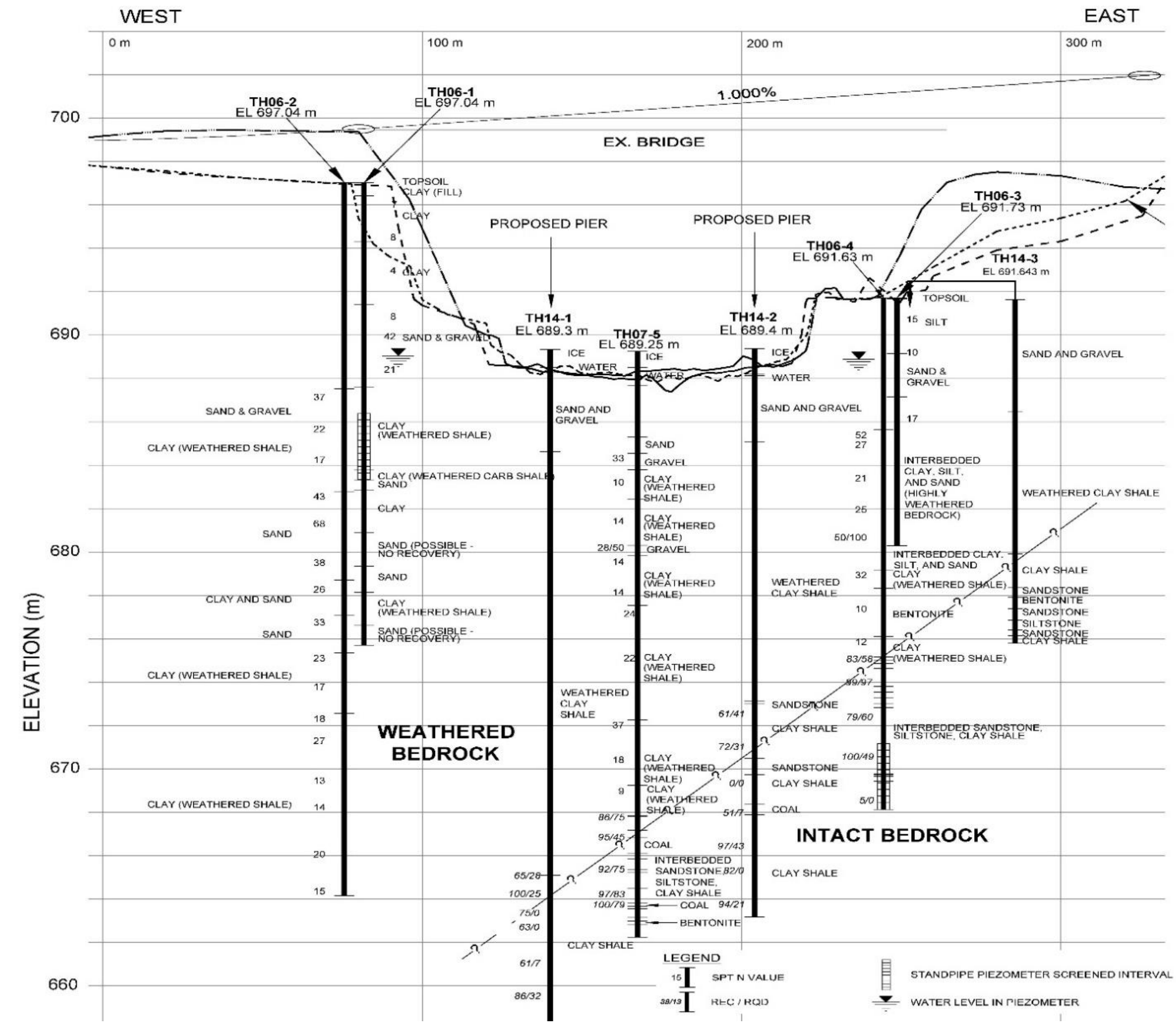

Figure 3. Simplified stratigraphic cross-section along bridge alignment

2.3 Surface Conditions

The Red Deer River is an underfit river located in a valley cut through bedrock. Bedrock outcrops were visible on the sides of the valley. There are two levels of terraces levels adjacent to the existing river. The terrace on the east side is somewhat lower than the terrace on the west side of the river. South of the bridge crossing, the lower terrace is also visible on the west side of the river

The Red Deer River is oriented approximately north-south at the bridge site with flow downstream south towards Drumheller. The bridge site is located on a relatively straight stretch of the channel on alluvial flood plains located within the larger river valley. The river channel is approximately 90 m wide from bank to bank in a valley that is approximately 1 km wide. The river is about 5 m below the surrounding banks and the overall valley is on the order of 120 m deep.

Vegetation on the flood plains is generally limited to agricultural crops. Trees were observed around gullies and closer to the river bank. Vegetation near the river level consisted of shrubs and grasses.

2.4 East Abutment Subsurface Conditions

Based upon the test hole information from 2006 (TH06-3 and TH-06-4), at the east abutment, the upper soil stratigraphy consisted of layered silt, sand and gravel, and clay/weathered clay shale to an elevation of 675.2 m. SPT N values ranged from 10 to 52 and moisture contents were highly variable even within similar material types.

A zone of bentonite was observed in TH06-4 between 13.4 m and 15.6 m below ground surface (678.3 m and 676.1 m elevation). This material was high plastic and had a water content near 50 percent. The SPT N value in this material was 10 indicating a relative consistency of stiff. The result of an Atterberg Limit test indicated very high plasticity: a liquid limit of 384 percent and a plastic limit of 37 percent.

Intact bedrock was encountered at elevation 675.2 m and consisted of interbedded clay shale, sandstone, and siltstone. In rock mechanics terms, the relative strength of the bedrock was very weak to weak and the rock was fresh (no signs of weathering). No distinct coal layers were encountered; however, coal stringers were frequently observed.

An additional boring (TH14-3) was advanced to further characterize the previously encountered bentonite layer in the east abutment area. Sand and gravel were encountered at the surface to elevation 686.5 m where weathered clay shale and bentonitic sandstone was encountered. An unconfined compression test on the weathered bentonitic sandstone measured an undrained shear strength of 357 kPa.

The zone of bentonite was observed between elevation 677.5 m and 677.9 m within the weathered bedrock zone. A grain size analysis conducted on the bentonite showed a silt content of 19.7 percent and a clay content of 80.3 percent. The result of an Atterberg Limit test of the bentonite material indicated very high plasticity: a liquid limit of 407 percent and a plastic limit of 38 percent. A direct shear test on the bentonite material showed peak strength parameters of c'=35 kPa and ϕ'=12 degrees and residual strength parameters of c'=0 kPa and ϕ'=8 degrees. The result of an unconfined compression test showed the bentonite had an undrained shear strength of 95 kPa.

The intact bedrock encountered beneath the bentonite below an elevation of 677.5 m consisted of interbedded sandstone, siltstone, and clay shale to a depth of 15.8 m (elevation 675.8 m).

The groundwater level measured in 2007 in an installed standpipe piezometer in the east abutment area indicated the groundwater elevation was approximately 688.9 m, which mirrors the adjacent river surface elevation.

3 EAST ABUTMENT

3.1 Abutment Stability Design

A plan and profile of the East Abutment area for the new bridge replacement structure is provided in Figures 4 and 5 respectively.

Slope stability and structural analyses were completed for the head-slope of the abutment to develop the final configuration presented in Figures 4 and 5.

The abutment fill itself consists of a maximum fill of approximately 12 m that rises 10 m above the original native ground to a finished elevation of approximately 701.2 m. The fill is composed of a 12 m deep gravel wedge at the face of the head and side-slopes. The remainder of the abutment fill is composed of select engineered fill, typically low to medium plastic clay. The head-slope also contains three layers of geogrid reinforcement in the upper half of the fill that extend back into the abutment fill a maximum length of 40 m from the head-slope face. The geogrid was included to provide reinforcement in a uniaxial direction for the stability of the south side-slope that had to initially be built with a steeper inclination to accommodate a temporary detour.

Considering construction sequencing, the northern side-slope could be completed during the initial installation, but the southern side-slope could only be partially completed due the proximity to the abutment structure and fill for the existing bridge. The south side-slope would thus need to be finally graded following the demolition of the existing bridge. Consideration was also given to the reduced headspace that would be present while compacting the abutment fill beneath the girders and trusses for the existing bridge.

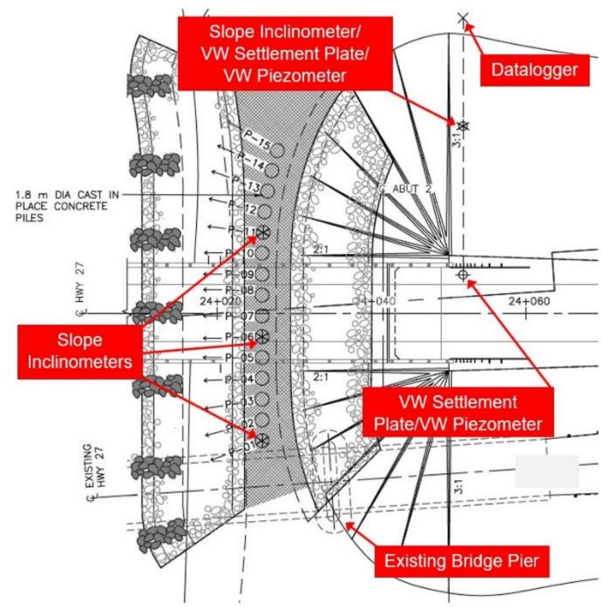

Figure 4. Plan view of east abutment design

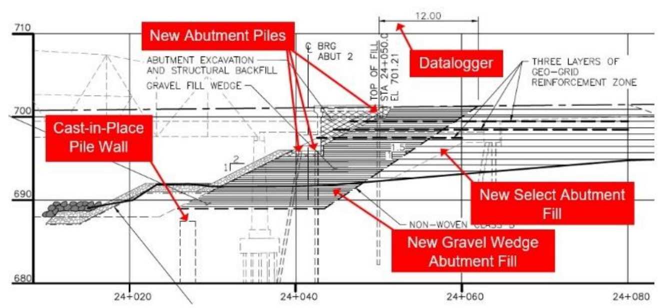

Figure 5. Section view of east abutment design

To provide global slope stability for the head-slope, a subsurface cast-in-place concrete pile wall was included at the toe of the head-slope. The pile wall consisted of 15 piles that were 1.8 m in diameter, 23.8 m long, and spaced 2.7 m from centre to centre. This pile arrangement was optimized from four design options, which include pile sizes varying between 1.5 m to 2.0 m diameters, at spacings varying between 2.4 m to 3.0 m during the preliminary design phase. To reduce the amount of concrete required the cut-off elevation of the pile was to be located at an elevation of 687.8 m, approximately 3.0 to 3.5 m below existing grade. As shown in Figure 4, the pile wall was extended further northward than southward from the centerline of the bridge. This was arranged in this manner because it was expected that the existing bridge pier located immediately southeast of the new abutment fill would provide additional stability with similar effect to a cast-in-place pile and there was insufficient headroom to install new shear piles below the existing bridge.

The piles were designed to deflect a maximum of 60 mm at the pile head in order to engage and resist the horizontal force along the identified bentonite layer induced by the vertical fill loading during the construction of the abutment fill. To monitor this movement three of the piles (P01, P06, and P11) were fitted with slope inclinometer casing along their lengths, which was tied to the inside of the reinforcement bar cages that were inserted into the bored pile hole prior to the installation of concrete.

Each of the 15 piles were also fitted with six polyvinyl chloride (PVC) pipes along their length to allow for future integrity testing using crosshole sonic logging (CSL) techniques, as required by current AT standards. These tubes were also tied to the inside of the reinforcement bar cages.

Following the completion of the pile wall, additional instrumentation was to be installed beneath the abutment fill footprint, as described in Section 3.4. The fill was then required to be constructed at a prescribed rate of 1 vertical m per week to manage pore water pressure generation in the subsurface clay, weathered shale and bentonite, which would possibly contribute to additional head-slope instability. The instruments would then be monitored to observe the rate of consolidation settlement in the native subsurface, the self-weight settlement, and pore pressure dissipation in the abutment fill area before permitting installation of the driven steel H-piles for the abutment seat. A delay of up to six months was planned to allow for the fill to settle and spread prior to pile installation to minimize negative skin friction and lateral deflections on the installed abutment pile foundations.

3.2 Pile Wall Construction

The pile wall was completed between October 12, 2017 and January 22, 2018 prior to placing abutment fill. Figure 6 shows a typical setup of the Soilmec SR-65 piling rig and additional cranes used for casing and reinforcement bar handling and installation. As shown in Figure 6, the pile wall was constructed on the lower terrace at the toe of the future abutment fill, located immediately north of the easternmost existing bridge pier.

The installation of the piles was completed at a slower rate than initially planned due to the variability of the subsurface stratigraphy as well as the difficult conditions related to drilling in saturated conditions next to the Red Deer River. During drilling, sectional steel casing was vibrated into the pile holes to provide a cut-off and seal from groundwater that was seeping into the pile from the water bearing sand and gravel layers. The steel casing was also used to eliminate sloughing into the test holes during auger advancement.

The piles were constructed in an alternating manner generally from north to south by skipping locations so as not to drill a new pile directly next to a newly poured and curing one.

Concrete was installed into the pile holes using tremie methods filling from the bottom up so to ensure complete concrete coverage along the length of the pile, with the steel casing being pulled out of the hole as the concrete level in the pile hole increased.

Full time observation of the pile installation was provided by experienced geotechnical technicians during the duration of the pile installation program and no major concerns were observed or raised with the workmanship of the contractor during this period. The bentonite layer was encountered in all 15 pile holes at a depth of approximately 15.0 m to 15.5 m. It was however observed that the contractor did appear to have some challenges related to installing the piles at a cut-off elevation of 3.0 to 3.5 m below the existing grade. Following the completion of the concrete pouring, the remaining holes were required to be left open to ground surface to provide access for future CSL testing and completion/extension of the SI casings to surface. This configuration led to a challenging to manage work area as well as the introduction of soil slough onto the top of the piles and possibly into the SI casings and CSL tubes.

Figure 6. Typical cast-in-place pile installation setup

3.3 Pile Wall Testing

As the pile wall construction progressed the contractor was required to complete CSL testing of each of the 15 concrete piles. Following the testing of the 15 test piles, the data was reviewed and showed that only two of the 15 piles had results indicating that they were acceptably constructed. A comparison of typical failing and passing CSL signals for Piles 13 and 12, respectively, are shown in Figure 7. Additionally, some of the CSL tubes were not useable by the testing contractor because they had become either damaged or plugged by concrete or sloughed soil.

The failing CSL result signal shown in Figure 7 was typical for 10 of the failed piles. The irregular signal pattern was postulated by the testing subcontractor as possible debonding between the CSL tubes and the surrounding concrete, rather than poor quality concrete.

Besides the possibility of debonding effects in 10 of the piles, two of the piles showed signs of possible defects affecting up to 15% of the cross-sectional area of the pile, and four of the piles showed signs of possible "soft-toe" conditions where the concrete at the toe of the pile may not have properly cured. Four of the piles showed multiple possible defect modes.

Prior to AT commenting on a future course for addressing these deficiencies, the contractor took it upon themselves to perform pile integrity testing (PIT) using acoustic methods on some of the piles. At the time of this initial PIT testing program, AT did not accept PIT testing as a suitable pile testing method.

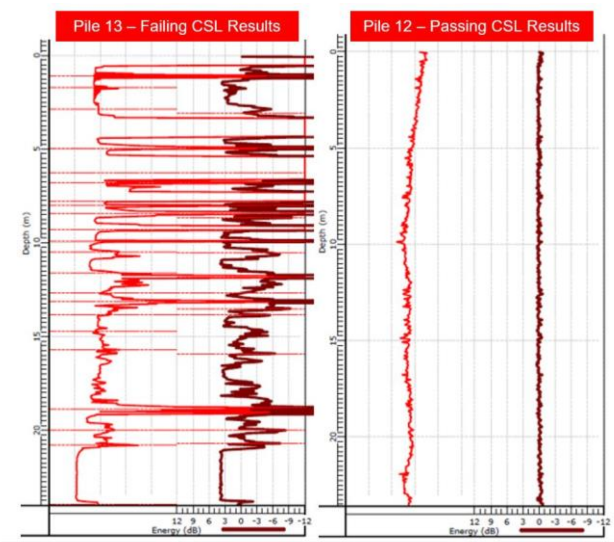

Figure 7. Typical failing and passing CSL result signals

Following the receipt and analysis of the CSL results, the construction records were reviewed for any source of possible defect. As discussed in Section 3.2, there was no identifiable construction anomaly that would lead to consistently defective piles, therefore it was considered possible that there may be an issue with the CSL testing method or configuration.

The contractor was then directed by the prime consultant and the owner to undertake the following additional testing program to ensure the integrity of the concrete piles:

- Complete PIT testing on all of the 15 piles
- Two of the piles should be cored in order to evaluate the condition of the installed concrete. The cores would be evaluated visually and would be further tested for compressive strength and density in a laboratory.
- Complete further CSL testing using reamed and/or flushed existing CSL tubes that had been previously blocked, as well as using the above core holes to advance the CSL probes.

Using this array of additional testing, it was possible to further assess the integrity of each of the 15 piles. Table 1 provides a tabulation of the various identified possible defect modes in each of the piles, as well as the additional testing that was undertaken on each pile.

The coring of the piles was undertaken on P09 and P13 to limited depth. The coring advancement was limited by both the suitability of the contractor-supplied coring rig as well as the difficulty in keeping the core string plumb through the difference in elevation from the existing ground to the cut-off elevation of the piles situated at least 3.0 m below. The coring was truncated in both piles after they encountered the reinforcement bar cage embedded within the pile. The visual inspection of the retrieved cores showed no obvious defects and the density and compressive testing of retrieved cores indicated that the concrete met the project specifications.

PIT testing was completed on all piles and did not reveal any defects based on the analysis of the signals retrieved from that method.

Additional CSL testing on the cored piles (P09 and P13), using the core holes, also indicated passing results for the depths that were cored, which was contrary to the CSL results from the installed CSL tubes for the same piles.

Considering this additional testing, and in accordance with the review of the construction observation records, there was no reason to believe that any of the piles had defects as indicated in the initial CSL testing. It was decided by the owner and the consultant team that it was probable that the CSL tube debonding effect offered by the contractor was plausible.

To further satisfy the owner, the consultant team completed additional analyses to determine the implication of having piles that may have the possibility of the following two types of defects:

- 15% reduction of cross-sectional pile area within the defined bentonite zone (to simulate the presence of possible large defects)
- 50% reduction in soil capacity/modulus of subgrade reaction below an elevation of 666 m (to simulate possible soft toe conditions)

The results of these analyses indicated that any additional deflections that may be generated by these two possible conditions were within the allowable design limits of the cast-in-place piles.

With this further satisfaction that even if the defects and soft toe conditions noted in the initial CSL testing were real, but would not have significant negative effect on the performance of the pile wall, the following pile acceptance instructions were given to the contractor by the owner:

- The totality of the testing evidence supports the CSL tube debonding hypotheses provided by the testing subcontractor.
- The consultant team was satisfied that the contractor had done suitable due diligence to prove out the condition of the piles without causing additional damage to the piles via a large-scale coring program.
- Three of the piles that fully passed CSL testing were approved by the owner.
- The remaining 12 piles would officially be considered "not approved/not disapproved" by the owner.

Table 1. Cast-in-place concrete pile testing results matrix

Pile #	Completion Date	CSL 1 - 12/4/17	CSL 2 - 12/5/17	CSL 3 - 12/6/17	CSL 4 - 12/8/17	PIT 1 - 12/8/17	CSL 5 - 12/27/17	CSL 6 - 1/22/18	CSL 7 - 2/5/18	PIT 2 - 3/15/18	CSL 8 - 4/2/18	PIT 3 - 4/2/18	Visual Inspection - 4/3/18	PIT 4 - 4/5/18	PIT 5 - 4/10/18	PIT 6 - 4/16/18	Concrete Testing - 4/27/18
P01	11/18/17		DB												OK		
P02	1/15/18							ST							OK		
P03	11/23/17		DB												OK		
P04	1/22/18								DB/ST	OK							
P05	11/15/17		DB											OK			
P06	1/9/18							DE/ST				OK					
P07	11/13/17		DB											OK			
P08	12/22/17							DB				OK					
P09	11/6/17	DB									OK*		OK*				OK**
P10	12/12/17						DE/ST			OK							
P11	11/23/17	DB														OK	
P12	12/1/17			OK		OK											
P13	10/28/17	DB			DB					OK	OK*		OK*			OK	OK**
P14	12/15/17						OK										
P15	10/24/17	DB			DB	OK											

Legend

OK	Passed CSL Testing	OK**	Passed Compressive Strength and Density Testing
OK*	Passed CSL Testing to Limited Depth	DB	Debonding Identified in CSL Testing
OK	Passed PIT Testing	DE	Defect Identified in CSL Testing
OK*	Passed Visual Inspection of Retrieved Concrete Cores	ST	Soft Toe Identified in CSL Testing

With the project facing tight timelines to continue with embankment constructions and the fill loading/settlement waiting period, the contractor was permitted to proceed if they accepted the risk of the piles being defective within a specified warranty period which was to be further negotiated between the contractor and the owner. The alternative would be for the contractor to core the remaining 12 piles to full length, which they elected not to complete.

3.4 Instrumentation and Monitoring During Fill Operations

Following the completion and conditional acceptance of the pile wall construction, abutment fill operations by the contractor could proceed.

Prior to fill loading, additional instrumentation was installed beneath the footprint of the new east abutment fill. Referring to Figure 4, this additional instrumentation consisted of the following:

- An additional SI casing located beneath the future north side-slope and oriented towards the north to monitor any horizontal subsurface movements of the side-slope embankment area
- Four vibrating wire piezometers to monitor the increased pore pressure generated by abutment fill loading – two located beneath the future north side-slope at different depths as grouted into the installed SI casing (P1 and P2) and two located beneath the future full abutment fill height as installed in a drilled test hole attached to an inserted PVC pipe (P3 and P4)
- Two vibrating wire settlement plate systems to monitor the consolidation settlement of the native ground under embankment fill loading – one installed beneath the future north side-slope (P5) and one installed beneath the full height of the future abutment fill (P6)
- A solar powered and remote accessible data logger mounted on a deep grouted pole to the north of the north side-slope footprint in a zone that was outside of the influence of the abutment fill loading.

At this time the SI instruments located within the pile wall were also extended and backfilled to ground surface. This initialization was completed with some difficulty due to the exposure of the SI casing between the pile cut-off elevation and the ground surface elevation. The SI casings in this area had been damaged and they needed to be re-excavated down to the pile cut-off elevation for repair. In addition, the casings needed to be flushed as sediment had sloughed into the SI casing from the excavation and/or river flooding.

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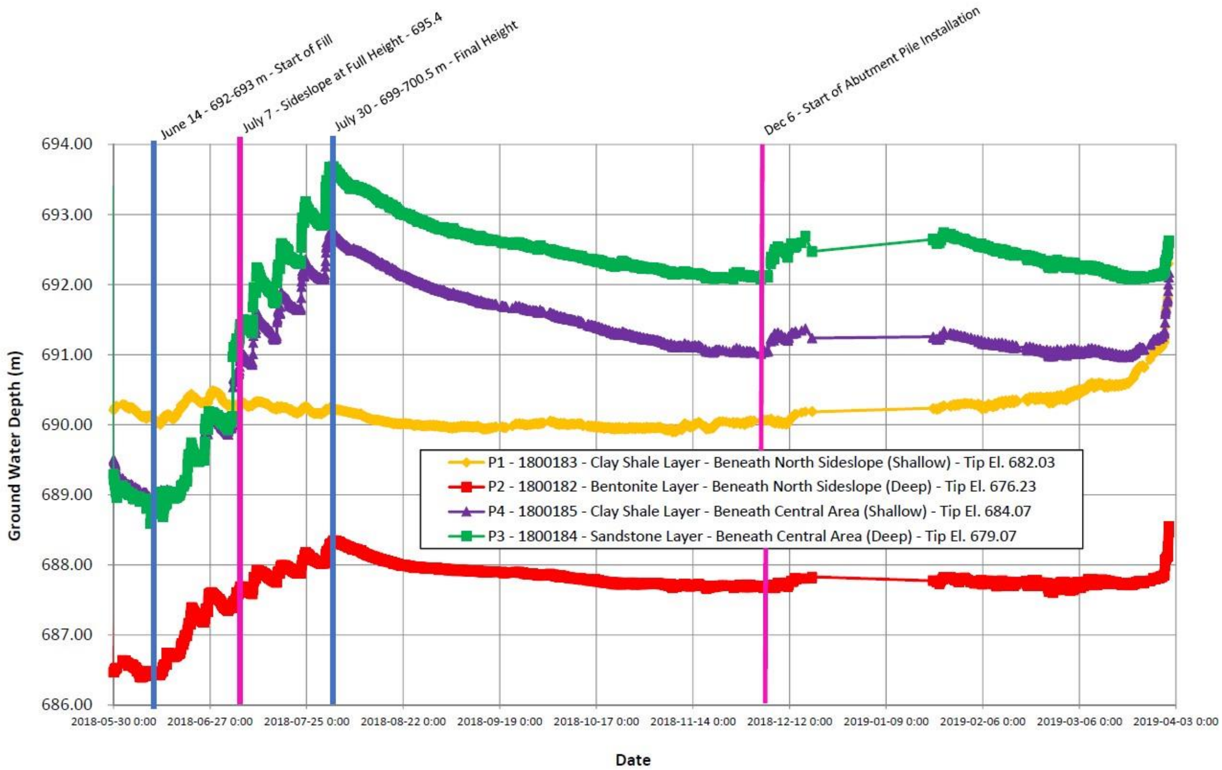

Figure 8. Vibrating wire piezometer response beneath east abutment fill footprint

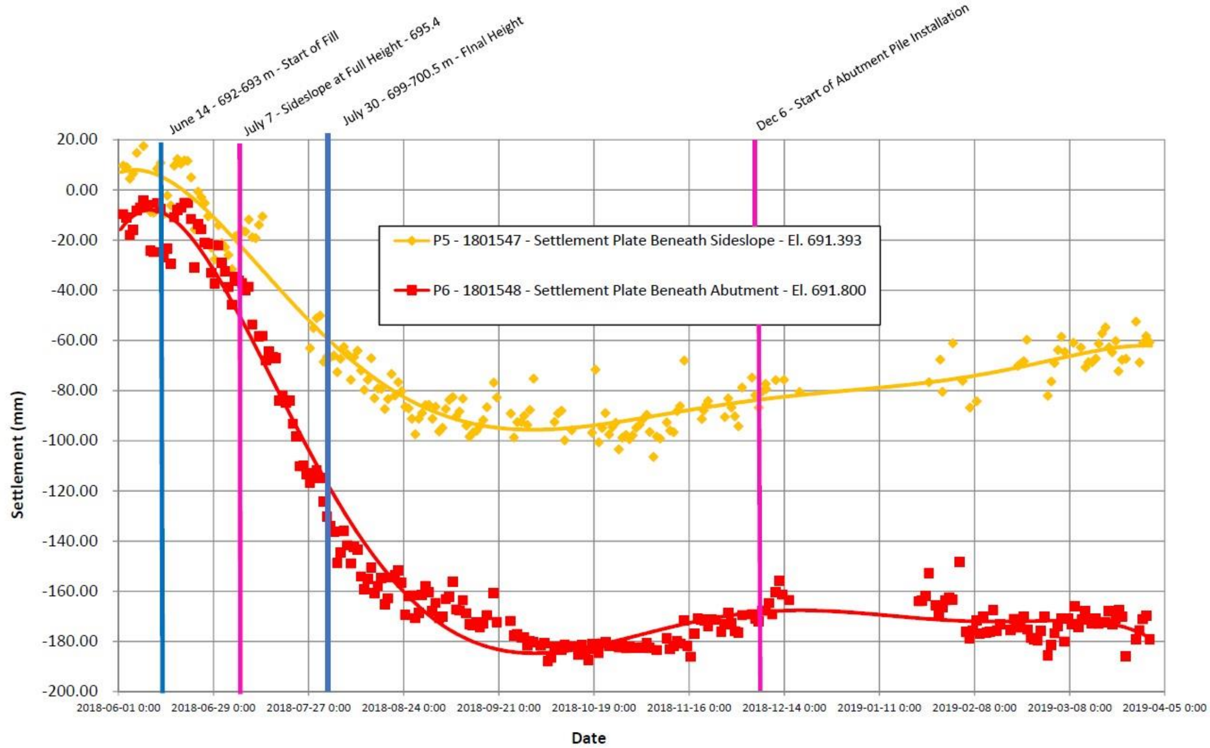

Figure 9. Vibrating wire settlement plate response beneath east abutment fill footprint

Following the instrument installation and excavation, fill operations commenced on June 14, 2018 at the maximum rate of 1 vertical m per week. Fill continued from an initial elevation of approximately 691 m to a full-height elevation of 700.5 m, which was completed by July 30, 2018. During this period, fill over the side-slope instruments were completed on July 7, 2018.

The observed responses of the four vibrating wire piezometers (P1 to P4) and the two vibrating wire settlement plates (P5 and P6) over the loading period and through to the beginning of April 5, 2019 are provided in Figures 8 and 9 respectively.

The response of the vibrating wire piezometers generally shows a step-mode increase corresponding to the incremental 1 m weekly loading intervals over the fill construction period followed by pore pressure dissipation over the following months. It should be noted that until July 1, 2018 both P3 and P4 appear to be hydraulically linked until the instruments experienced an offset (of unknown source) from each other on that date. From that date forward the responses of these two instruments continued to mirror each other but featured this offset, indicating that they most likely remained hydraulically linked.

A maximum water pressure head increase of approximately 4.5 m was experienced in the sandstone bedrock beneath the full abutment height with an increase in pressure head of approximately 4.0 m in the clay shale layer above it. The pore pressure response of the piezometers beneath the north side-slope was less pronounced with a pore pressure increase of approximately 2.0 m in the bentonite layer and an even subtler response observed in the overlying clay shale layer.

As illustrated on Figure 9, the settlement experienced at the interface of the bottom of the abutment fill and the native ground also began to develop during the abutment fill construction period. The points shown on the plot are daily readings with a plotted polynomial trendline. These instruments were observed to have some daily variability in their response, most likely owing to atmospheric conditions (pressure and temperature). The rebound encountered in both instruments around October 2018 coincided with colder temperatures and eventual ground freezing at the site, and it is suggested that this change in seasons slightly affected the behavior in the two settlement plates. This understanding may be supported by the observation that the curves begin to turn slightly downward again near the end of the reading period in April 2019 during spring thawing conditions. The long-term response of the settlement plates is discussed in Section 3.5.

During the fill construction period, biweekly readings were also taken of the SI instruments. The responses of the SI instruments indicated that during the fill period until the beginning of August 2018, the pile wall performed as expected with movements towards the river ranging between 8 mm to 17 mm for each of the three SIs installed in the pile wall. The smallest amount of movement was experienced in P1 where it would be expected that lateral movements would be somewhat restrained by the presence of the existing bridge foundations, with observed movement increasing in the central area (P6) and the maximum amount of movement experienced along the northern flank of the abutment footprint in P11. The SI installed beneath the northern side-slope experienced approximately 9 mm of lateral movement towards the north.

3.5 Abutment Pile Construction

The monitoring of the geotechnical instruments continued for the period following the fill construction to determine if the consolidation settlement of the native ground and the self-weight settlement had continued to a point where the effects of negative skin friction and lateral spreading on the planned abutment piles would be minimized. Without allowing for this consolidation to occur over a significant time period, pile settlements of up to 100 mm had been predicted for piles installed through granular fill, using the neutral plane axis analysis method.

Referring to Figure 9, it was observed that the consolidation settlement of the subsurface had been substantially completed by approximately mid-October 2018, with a total of approximately 180 mm of settlement for the central abutment area and 95 mm for the north side-slope area. This amount of settlement compared very well to the 175 mm of elastic and consolidation settlement that had been predicted during the initial geotechnical design, using standard consolidation theory analyses and Settle3D software.

In addition to the vibrating wire settlement plates, monuments were installed on the surface of the new abutment fill to assess the self-weight settlement. By mid-October the amount of the self-weight settlement on top of the abutment fill at full height was observed to be approximately 32 mm, compared to the elevation measured in that location at the completion of fill construction.

In order to determine if settlement had been virtually completed, these instruments were monitored for another month to ensure that the responses were steady before permitting the contractor to proceed with abutment pile driving. As observed in the instruments, settlement remained virtually unchanged over that time and the top of the abutment fill settled a further 3 mm to a total of 35 mm since fill completion. At this point determination was made to allow the contractor to proceed with abutment construction and that any negative skin friction effects on piles driven through the fill would be negligible.

The vibrating wire piezometers were also monitored during this period to ensure that the pore pressures induced by fill loading continued to dissipate, which occurred in all instruments, as shown in Figure 8. Additional pore pressure was generated during the abutment pile driving, which commenced on December 6, 2018, but this also appeared to dissipate with time over the following months.

It should be noted that there is a visible gap in the data collection in both Figures 8 and 9 from mid-December 2018 to mid-January 2019, which is attributable to the seasonal lack of daylight at the site and the failure of the datalogger to charge using the supplied solar panel. Additionally, the solar panel was discovered in January 2019 to need cleaning and maintenance. Following this maintenance, the datalogger was brought back on-line to continue the monitoring.

An additional pore pressure response can also be seen in the data occurring around the end of March 2019. This can be attributed to a river ice jam at the site around the existing bridge which resulted in an increase in the river level and in turn an increase in the groundwater elevation.

SI readings were also taken monthly during the settlement period. Only relatively small amounts of additional pile deflection were observed in the three stability piles, ranging from 0 mm to 10 mm, resulting in total pile movements between 12 mm and 25 mm, which were below the allowable design deflection of 60 mm. No additional significant movement was observed in the north side-slope SI during this monitoring period.

4 CONCLUSIONS

The following provide a summary of the conclusions and lessons learned from this project.

- The cast-in-place pile wall has proven to be an effective stabilization measure to ensure the stability of the east bridge abutment despite the presence of the subsurface bentonite layers and the increased fill loading.
- The installed array of instrumentation proved to be a successful way of tracking the performance of the pile wall and the abutment fill during construction and for confirming that settlements due to fill placement had completed to a point where negative skin friction effects on later installed piles would be minimal.
- There is a great deal of inherent difficulty and risk with installing cast-in-place concrete piles in conditions below the water table or in a waterside environment. Despite the contractor’s best efforts and full-time observation, much care and attention must be taken in ensuring best practices are followed, and even then, unforeseen difficulties may arise.
- Terminating subsurface cast-in-place piles at a cut-off elevation significantly below the surface should be avoided in future projects that either have a high water table or are located next to a waterway. This configuration led to significant difficulty for the contractor during construction and once anomalies were found with the CSL testing results the contractor was then in a situation where they had to access the piles at the bottom of deep holes for future testing and coring. Additionally, it led to difficulty with protecting the protruding SI casing up to the surface
- CSL tubes should be composed of steel pipe rather than PVC for durability and better bonding purposes. The responsibility of properly protecting and capping the CSL tubes should be solely that of the contractor.
- If the CSL testing yields failing or uncertain results coring of the piles may be required via the use of an approved and experienced rock coring subcontractor with approved and tested equipment.
- The project owners may have very rigid and specified pile testing requirements that cannot always be completely satisfied for numerous reasons and alternative methods may need to be explored when the specified testing does not yield acceptable results but there is no indication to otherwise assume there are defects.
- The keeping of complete construction records should be employed to determine if any construction methodology suggests the piles may be defective and to determine that there is no other plausible source of defect.
- If specified testing does not yield certain results, additional tools may be required, including additional testing and further analysis using conservative assumptions, may need to be used to increase certainty in the as-built condition of the piles.
- Good lines of communication between contractors, sub-contractors, prime consultants, and owners who have experience with installation and testing are crucial to project success and find an acceptable solution to all parties.

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