Baptiste Creek Bridge Landslide, HWY 2:42 Near Athabasca, Alberta: Design and Construction of Remedial Measures

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ABSTRACT
In 2017, a deep-seated rotational landslide impacted the integrity of the bridge foundation and pavement structure at the Baptiste Creek crossing on Highway 2 in North-Central Alberta. An innovative repair methodology was utilized to mitigate the landslide movement and restore the functionality of the bridge. The repair included the installation of a combination of cantilever and tie-back sheet pile walls on the abutment slopes. Light-weight cellular concrete was also placed behind the north abutment wall to backfill the highway embankment and further offload the landside mass. Large diameter casings were installed around the impacted integral-abutment piles to accommodate thermal movements of the bridge superstructure. A slope inclinometer, vibrating wire piezometers, load cells, and strain gauges were installed to assess the effectiveness of the repair measures. This paper summarizes the geotechnical investigation results, remedial design and construction challenges, and instrumentation monitoring results at the repaired site.

RÉSUMÉ
En 2017, un glissement de terrain en rotation profond a eu un impact sur l'intégrité de la fondation du pont et de la structure de la chaussée au franchissement du ruisseau Baptiste sur la route 2 dans le centre-nord de l'Alberta. Une méthodologie de réparation innovante a été utilisée pour atténuer le mouvement des glissements de terrain et restaurer la fonctionnalité du pont. La réparation comprenait l'installation d'une combinaison de murs de palplanches en porte-à-faux et d'ancrage sur les pentes des culées. Du béton cellulaire léger a également été placé derrière le mur de la culée nord pour remblayer le remblai de l'autoroute et décharger davantage la masse du côté terre. Des tubages de grand diamètre ont été installés autour des pieux à culée intégrale impactés pour s'adapter aux mouvements thermiques de la superstructure du pont. Un inclinomètre de pente, des piézomètres à corde vibrante, des cellules de charge et des jauges de contrainte ont été installés pour évaluer l'efficacité des mesures de réparation. Cet article résume les résultats de l'étude géotechnique, les défis de conception et de construction de réparation, et les résultats de la surveillance de l'instrumentation sur le site réparé.

1 INTRODUCTION

According to Thurber Engineering Ltd. (2018), a landslide occurred in 2017 within the northwest approach fill of Alberta Transportation's Bridge File BF7055 located on Highway 2 at the Baptiste Creek crossing, in North-Central Alberta. The site location is presented in Figure 1. The movement resulted in a void below the southbound highway shoulder, loss of soil from behind and below the northwest abutment wingwall, exposure and rotation of two exterior integral-abutment pile casings, and landslide debris constricting the creek channel. The main features of the landslide are presented in Figure 2.

The landslide remedial measures included installation of a cantilever sheet pile wall along the north bank of the creek (within the head slope of the bridge), and a tied-back sheet pile wall parallel to the highway alignment to accommodate the 6H:1V slope flattening of the northwest approach fill. Light weight cellular concrete was also placed behind the north abutment wall to backfill the highway embankment and further offload the landslide mass. To retrofit the integral-abutment piles that had been impacted by the landslide and had lost the flexibility to accommodate thermal movement of the bridge superstructure, large diameter steel casings were installed around the impacted piles in order to provide enough void space for thermal movements.

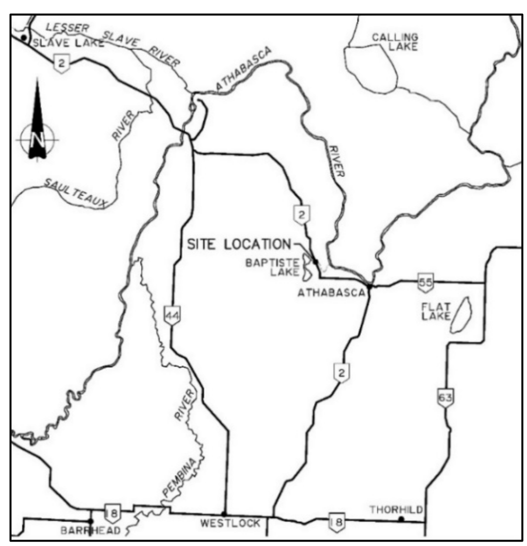

Figure 1. Site Location Map

Figure 2. Landslide Features (2017 and 2018)

The construction activities for the stabilization of the landslide were carried out between March and October of 2019 and the cost of the repairs was approximately $3 million. This paper summarizes geotechnical investigation, design and construction aspects of the project, and a brief discussion of the instrumentation monitoring results.

2 SITE DESCRIPTION

2.1 Background

The existing Baptiste River bridge consists of a single span (47 m long) steel plate girder (2.6 m deep) superstructure supported on integral abutments (Geometrix Group Engineering Ltd., 2018). Each abutment consists of 1.45 m wide, and 1.6 m deep concrete seat supported on ten, HP310 x 132 piles spaced at 1.2 m. As presented in Figure 3, the upper 4.5 m of each of the integral-abutment piles is isolated from the soil and centered within a 610 mm diameter x 10 mm thick steel pipe to accommodate thermal expansion and contraction of the bridge superstructure. Because the substructure and superstructure of the bridge move together, the need for bearings and expansion joints is avoided.

The existing bridge was constructed in 2008 to replace an older three-span bridge structure that was built in the 1970s. The old bridge abutment and pier piles were cut off and left in place during the construction of the current bridge as presented in Figure 3.

As part of the construction of the current bridge, the creek channel was realigned and the 6 m high approach fill head slopes were reconstructed at 2H:1V and 3H:1V on the north and south side of the creek, respectively. The approach fill side slopes were constructed at approximately 3H:1V on both sides of the creek. Approximately 6 m of fill was placed on the west side of the bridge to the north of the creek alignment to accommodate the construction of the 2008 bridge.

Available records indicate that an instability occurred within the south head slope during the construction of the old bridge in the 1970s. The repairs at the time consisted of flattening the head slope from 2H:1V to 3H:1V, resulting in an increase of the bridge span by 10 m. In 1979, an instability also occurred to the east of the north abutment outside the bridge location. The repairs for this second instability consisted of slope flattening, along with the construction of a toe buttress and finger drains.

2.2 Landslide and Emergency Response

In September 2017, Alberta Transportation personnel noticed a lateral movement of the northwest side slopes of the bridge approach embankment, and the formation of voids adjacent to the taper of the existing drain trough and below the northwest corner of the abutment seat. The highway maintenance team poured a Class C concrete into the voids in an attempt to enhance the situation. However, a major slope movement occurred abruptly within the northwest approach embankment in October 2017. The sudden movement resulted in the formation of a gap below the wing wall and the abutment seat, and a complete failure of the drain trough.

An emergency callout inspection was completed by Thurber in November 2017 and geotechnical instruments, consisting of three slope inclinometers, complete with nested vibrating wire piezometers, were installed within the northwest approach fill in January 2018. The failure of the northwest approach embankment continued over the winter months, and an accelerated movement occurred in the spring of 2018. The landslide created a large void below the highway surface and exposed the underside of the wing wall and abutment seat. In addition, the movement exposed and rotated two of the exterior integral-abutment pile casings, restricting the lateral movements of the piles during thermal expansion of the bridge superstructure.

There were concerns regarding the potential for sudden loss of the highway southbound lane; distress of the bridge abutment and supports; and further constriction of the creek channel, which could result in flooding of private lands located upstream of the bridge location.

A temporary stabilization measure was implemented by the highway maintenance crew in July 2018, and included installation of a soldier pile retaining wall with timber lagging parallel to the southbound shoulder and below the north abutment seat; placing granular fill to backfill the gap between the wing wall, abutment seat and the temporary retaining wall; filling the voids below the roadway and behind the wingwall with expanded foam, constructing a concrete curb along the edge of the highway southbound lane to divert runoff away from the landslide area, and placing a half corrugated steel pile (CSP) down-drain at the end of the curb to convey the runoff to the bottom of the highway side slope.

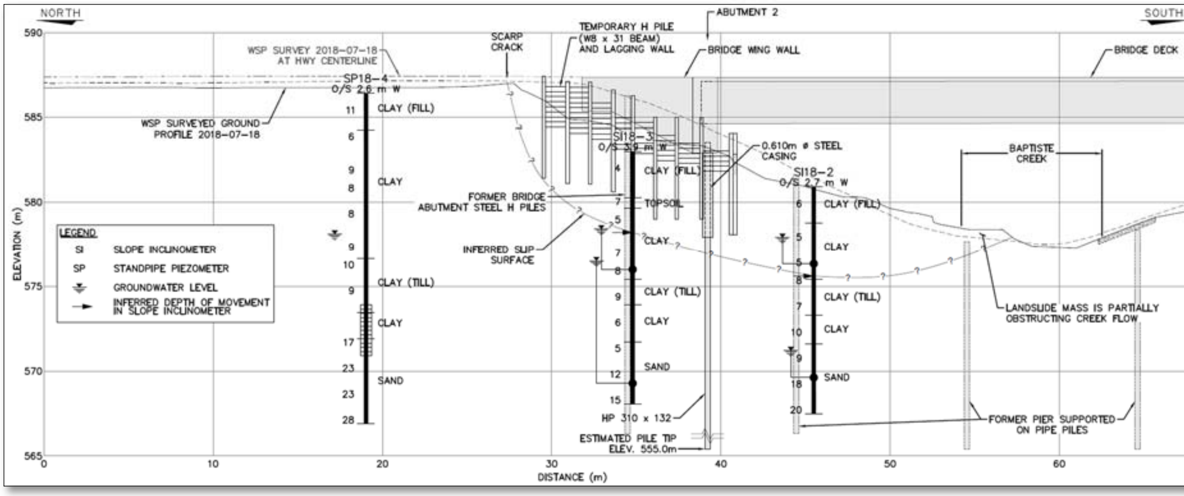

Figure 3. Stratigraphic Cross-section of the Northwest Approach Embankment, with Inferred Failure Surface and Temporary Stabilization Measures.

Figure 3 shows a stratigraphic cross-section of the northwest approach embankment and the temporary soldier pile retaining wall. Geotechnical investigation results, and design and construction of the permanent landslide repair are discussed in the following sections.

3 GEOTECHNICAL INVESTIGATION

3.1 Soil Conditions

The soil conditions encountered in the test hole locations consisted in descending order of clay fill, clay, clay till, and sand, as shown on Figure 3. The clay fill was typically grey to brown, with occasional sand and silt lenses and trace gravel and oxides. SPT 'N' values in the clay fill typically ranged from 4 to 11 blows per 300 mm penetration, indicating a firm to stiff consistency. The natural moisture content of the clay fill typically varied from about 17 to 31 percent. One Atterberg Limits test conducted in the clay fill indicated high plasticity.

The clay and clay till were typically brown to grey with occasional sand and oxides. SPT 'N' values in the clay and clay till typically ranged from 5 to 10 blows per 300 mm penetration, indicating a firm to stiff consistency. The natural moisture content of the clay and clay till typically varied from about 23 to 39 percent. Four Atterberg Limits tests conducted in the clay and clay till indicated medium to high plasticity. Unconfined compressive strength tests conducted in the clay in TH18-2 and TH18-3 indicated undrained shear strength values of 65.6 kPa and 19.2 kPa, respectively.

Sand was encountered below the clay in all test holes. The sand was typically grey, fine grained, becoming medium to coarse grained with depth with occasional gravel. SPT 'N' values in the sand typically ranged from 5 to 28 blows per 300 mm penetration, indicating a loose to compact relative density. The natural moisture content of the sand typically varied from about 15 to 19 percent. Three grain size analyses conducted in the sand indicated 1 to 3 percent gravel, 89 to 94 percent sand and 5 to 9 percent fines (silt and clay).

3.2 Groundwater

The groundwater level in the vibrating wire piezometer installed in the gravel at the TH18-1 location was 4.0 m below ground surface. The groundwater levels in the vibrating wire piezometers installed in the clay ranged from 3.0 m in TH18-2 to 4.6 m in TH18-3. Groundwater levels in the vibrating wire and standpipe piezometers installed in the sand ranged from 3.6 m in TH18-1 to 9.7 m in TH18-2.

3.3 Slope Inclinometers

Three slope inclinometers were installed in test holes TH18-1, TH18-2, and TH18-3 drilled within the landslide area. The slope inclinometers were sheared off a few months after installation at an approximate depth of 5 m. The maximum recorded rates of movement in the three slope inclinometers ranged from 45 to 60 mm per year.

3.4 Failure Mechanism

The inferred slip surface is presented on Figure 3, and the planar extent of the landslide is shown on Figure 4. The results of the geotechnical investigation indicate that a deep-seated failure occurred at this site. The placement of steeply inclined fill (transitioning from 2H:1V at the head slope to 3H:1V at the side slopes), and the presence of weak high plastic clay are likely the main causes for the slope movement. Lack of proper bank erosion protection at the toe of the slope may have also contributed to the instability at this site.

4 PERMANENT REMEDIAL MEASURES

4.1 Landslide Repair Requirements and Challenges

Following installation of the temporary stabilization measures and the completion of the geotechnical investigation, the geotechnical and structural design aspects of the permanent landslide remediation were completed by Thurber Engineering Ltd. and Geometrix Group Engineering Ltd., respectively.

As shown on Figure 4 and described by Thurber Engineering Ltd. (2020), the geotechnical landslide repair methodology and sequence included: a) Installation of a 45 m long, 14 m deep cantilever sheet pile wall (Pile Wall ‘A’) parallel to the creek alignment at the base of the north head slope to retain the landslide mass and permit excavation of landslide debris within the creek channel, b) Completion of creek channel restoration work, c) Installation of a 40 m long, 19 m deep tied-back sheet pile wall with three rows of anchors (Pile Wall ‘B’) parallel to the southbound highway and west of the wing wall to retain the highway embankment fill and accommodate slope flattening, d) Flattening of the northwest embankment side slope to 6H:1V, and e) Completion of the highway and bridge restoration work.

On the structural side, a hydraulic jacking system was originally proposed to rectify the distorted casings surrounding the exterior integral-abutment piles on the northwest head slope. However, further inspection during construction indicated that, Class C concrete, which had been initially placed by the highway maintenance crew, had migrated to fill the impacted pile casings. Migration of concrete into the pile casings complicated the repair methodology as discussed in the following sections. The abutment wall fill was removed to inspect abutment wall, and jackhammer concrete within and around impacted pile casings. The highway restoration works included: (a) Placement of light weight cellular concrete behind the abutment wall to reduce lateral loads on the wall and the tied-back sheet pile retaining wall system, and (b) reconstruction of the bridge approach slab and Asphalt Concrete Pavement (ACP).

Proposed instream work included excavation of landslide debris, and rebuilding the north bank using gravel along with bank armouring to reduce the potential of future erosion. Excavation of the landslide debris (up to 3 m in depth) and placement of gravel were completed in small panels after the installation of Pile Wall ‘A’ to maintain the overall stability of the slope and prevent excessive movement of the pile wall.

4.2 Design of Sheet Pile Walls

Sheet pile walls were chosen after considering other alternatives such as cast-in-place concrete pile walls and King Piles. The cast-in-place pile wall option was discarded due to the potential risks of sloughing pile holes in the water bearing soil conditions present at this site. The king piles were not readily available in the Canadian market within the required timeline for this project. Hence, sheet pile walls, consisting of AZ 38-700N sections, were installed to stabilize the landslide movement and retain the highway embankment. As-built cross sections of the instrumented Sheet Pile ‘B’ is shown in Figure 5.

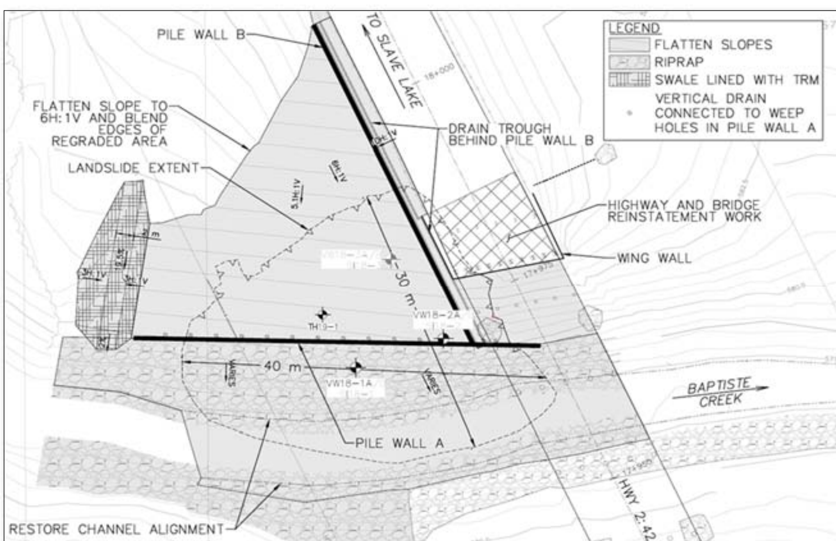

Figure 4. Site Plan Showing Permanent Repair Measures

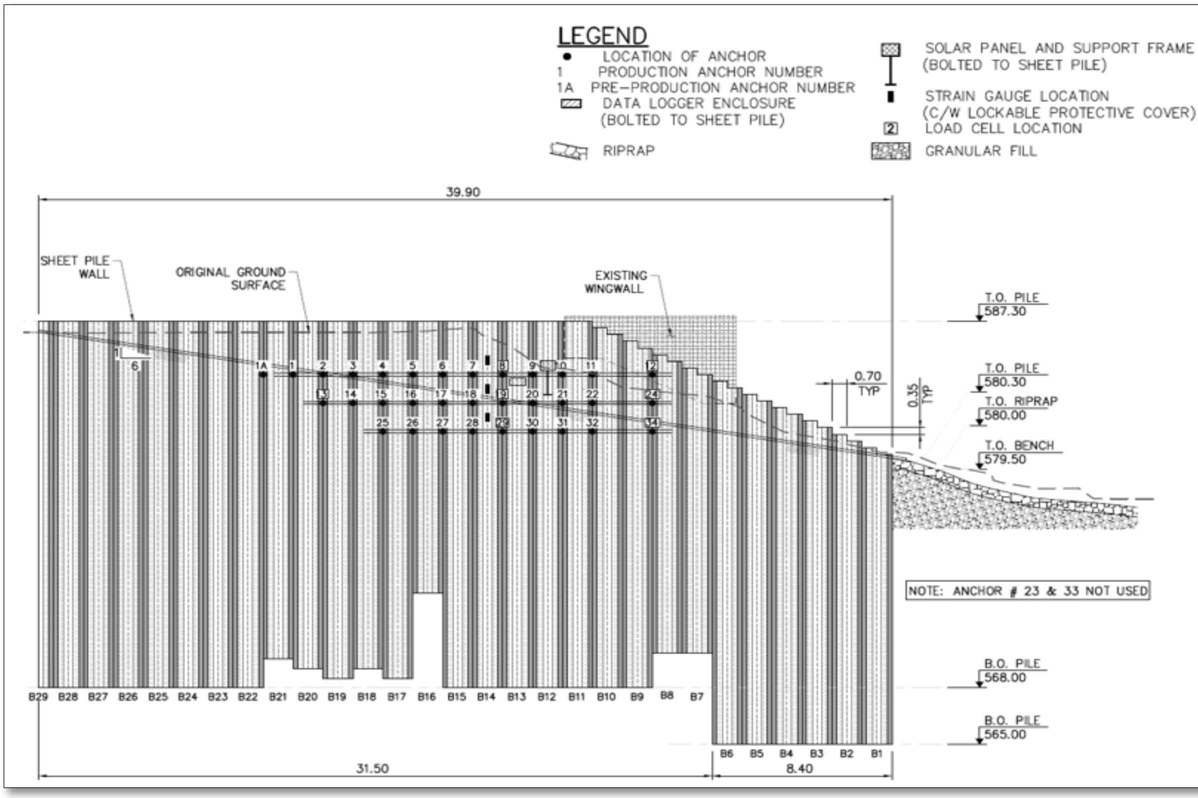

Figure 5. Cross Section of Sheet Pile Wall 'B'

4.3 Installation of Sheet Pile Walls

Prior to driving the sheet piles, a guide frame was set on the ground to help maintain the alignment of the sheet piles. The guide frame consisted of a welded H-beam running along the pile wall alignment and stakes to maintain the frame's position. The sheet piles were installed using a J&M Model 44-50 hydraulic vibro-hammer for the initial drive. Sheet piles were checked for plumbness and alignment during driving to ensure the sheet piles remained within the specified tolerance. After the initial drive with the vibro-hammer, a Pileco D25-32 diesel hammer was used to drive the sheet piles to their required depths. (See Figures 6a, 6b, and 6c).

The contractor proposed a smaller hammer (APE Model 6-2 hydraulic hammer) mounted on a forklift to install a small section of Pile Wall 'A' under the bridge. However, even with the smaller hammer, enough headroom was not available to install the piles under the bridge along the design alignment of Pile Wall 'A'. To overcome the limited headroom issue, the contractor and the design team agreed to (a) shift the eastern 6 m of the Pile Wall 'A' approximately 1 m further to the south, and (b) overlap about 1 m of pile wall at the shifted location. The headroom under the bridge was approximately 3 m at the design pile wall alignment and about 3.5 m at the as-built alignment. Because of the low headroom clearance below the bridge, these five sheet piles had to be installed in 3 m long sections. After each section was fully driven, the next section was spliced onto the top of the sheet pile. While driving the final sheet pile outside of the low headroom clearance zone, an abandoned steel pipe pile from the former bridge was struck and the contractor was unable to advance the pile to the target depth. The contractor had to remove the old pile prior to completing the installation of Pile Wall 'A'.

While driving Sheet Pile Wall 'B' with a vibro-hammer, it was noticed that sheet pile section B15 was tilting out of alignment (Figure 5b). As piling continued, it was observed that sheet piles B14 and B16 were also starting to drift out of plumb. The contractor chose to drill a 200 mm diameter test hole around the sheet pile wall to determine if there was an obstruction that was causing the sheet pile to deflect during driving; however, the test hole did not reveal any obstruction. After a joint survey and discussion about the misaligned piles between the consultant and the contractor, it was agreed to continue driving the piles and monitor them to see if the misalignment would become worse.

It is possible that the out-of-plumbness on these piles was due to the difficult installation conditions presented by the sloping ground along this portion of the pile wall, located between the creek and the crest of the approach embankment.

Figure 6a. Guide Beam Installation

Figure 6b. Installation of Sheet Pile Wall ‘A’

Figure 6c. Installation of Sheet Pile Wall ‘B’

As a result of the misalignment, eight piles of Pile Wall ‘B’ were driven short. These piles were also up to 3 percent out of plumb in the south direction and were offset by up to 125 mm in the west direction. Several options to mitigate the out of tolerance piles were discussed. An analysis conducted after the piles were installed determined that the as-built condition of Pile Wall ‘B’ was acceptable, and it was decided to accept the piles as constructed. The misalignment of the sheet piles presented challenges for the installation of the steel walers. This issue was resolved through using multiple waler segments at each level for some of the anchor rows to accommodate the misalignment of the sheet piles.

4.3.1 Installation of Pile Wall ‘B’ Anchors

Due to the expected sandy soil and shallow groundwater levels, the original design for Pile Wall ‘B’ specified the use of steel helical anchors to avoid the potential for sloughing in the anchor holes. However, the contractor submitted a design change request to replace helical anchors with drilled and grouted anchors at no additional cost to Alberta Transportation. On this basis, the design change was accepted by the design team. The revised anchor design used 26 mm diameter Double Corrosion Protection (DCP) steel threaded anchors. The anchors were designed with a bond zone of 12 m length, with free-stressing lengths of 10 m, 8 m and 6 m for the upper, middle and lower rows, respectively. The design drilled hole diameter for the anchors was 200 mm. The anchors were to be tremie grouted from the bottom of the hole, and post-grouting tubes and valves were included on each anchor to allow for post-grouting of the anchors after the initial installation.

Prior to the start of the production anchor installation, the contractor installed a pre-production test anchor (PPA) to confirm the soil capacity for the design anchors. The PPA consisted of a 32 mm bare steel thread bar with a PVC debonding sheath in the free-stress zone. The PPA had a design bond length of 12 m, a design free-stress length of 10 m and a borehole diameter of 200 mm. The results of the PPA test showed that the anchor exhibited satisfactory elastic and creep displacement behavior under cyclic loading cycles up to 2.5 design load. Based on the satisfactory results of the PPA test, the contractor proceeded with the installation of the production anchors. A total of 32 production anchors were installed (12 anchors in the first row, 11 anchors in the second row and 9 anchors in the third row), as shown on Figure 5b. The anchors were installed using a Comacchio MC 22 micro-drilling rig equipped with solid stem augers to drill the 200 mm anchor holes. Prior to drilling, 750 mm x 750 mm openings were cut in the flanges of the sheet piles at the anchor locations. The contractor assembled the anchor bar sections on the ground and attached the necessary centralizers, bond-breakers, tremie grouting and post grouting lines. The assembled DCP anchors were inserted into the open drill holes and tremie-grouted from the bottom up. After installation, the contractor welded stitch plates around the anchors to cover the holes in the sheet pile, which were sealed using a spray foam.

The installation sequence of the production anchors was carried out from top to bottom as follows: 1) Excavate to the first row of anchors, 2) Cut out sheet pile flanges concurrently with the installation of the anchors, 3) Install first row of anchors and waler, 4) Proof load all anchors, 5) Lock off each of the approved anchors and undertake lift off testing upon stressing completion to confirm the transferred load to the anchors, 6) Repeat the above steps for the lower rows of anchors.

During the first row of anchor installations near the bridge structure, multiple obstructions were encountered due to the abandoned former bridge piles and to the temporary shoring piles that had been installed as part of the highway reinstatement work described in Sections 4.1 and 4.4. These obstructions did not allow for suitable anchor lengths to be achieved at three of the design anchor locations. The contractor proposed to extend the waler and relocate one of the anchors, while requesting to delete the other two anchors, and to change the horizontal skews of two other anchors from 15 degrees to 18 degrees. The proposed installation scheme was reviewed and accepted by the design team, with the condition that the four anchors adjacent to the skipped anchors be locked off at 210 kN instead of the original design load of 140 kN, to compensate for the loss of two of the design anchors.

No significant difficulties were encountered during the installation of the second row of anchors. However, on the third row of anchors, groundwater seepage was encountered at all anchor locations, except for anchors 28 and 30 (Figure 5b). The groundwater seepage was typically encountered between 15 to 18 m along the length of the drilled anchor holes. The contractor later had difficulty inserting the bars in four of the anchor holes on the third row, and the free length was reduced from 6 m to lengths ranging between 3.6 m to 5.5 m.

Before the third-row anchors were proof tested, groundwater was noted to be seeping out at the surface of anchors 13, 25, 27 and 32. Thurber asked the contractor to conduct additional lift off testing for these anchors after the proof/performance testing was completed, to confirm if the anchors had lost any load. The results of the lift off testing showed that these anchors had not lost any appreciable load. To avoid water accumulation and erosion from the groundwater seepage, a subdrain system (consisting of a perforated 150 mm PVC pipe surrounded by 300 mm of filter gravel and non-woven geotextile) was installed at the base of Pile Wall 'B', daylighting into the riprap zone.

4.3.2 Steel Walers and Supports

The anchor loads were transferred to Sheet Pile Wall 'B' by three welded steel walers (C310x45 channel sections) affixed to the sheet piles. The walers were installed upon completion of each successive row of grouted anchors. Before placing the waler onto the supports for the first and second rows of anchors, it was found that the pre-assembled walers would not fit onto the sheet piles due to the misalignment of the sheet piles that occurred during pile driving. To better field-fit the walers, the contractor requested that the upper two walers be installed in sections (see Figure 7). Thurber and Geometrix reviewed the Contractor's proposed change and allowed them to proceed with the revised waler configuration.

Unlike the upper two rows, the contractor was able to install the bottom waler in one section, as per the original design.

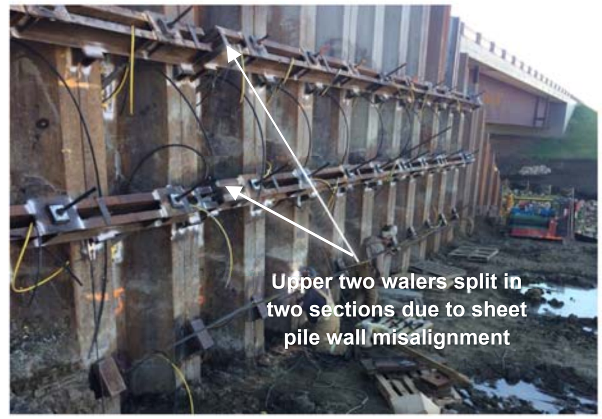

Figure 7. Waler Installations on Pile Wall 'B'

4.4 Highway and Bridge Reinstatement Works

The scope of the permanent highway and bridge reinstatement works included temporary shoring, replacement of the granular/clay fill behind the north abutment with light weight cellular concrete , removal of concrete residual that had migrated into the northwest integral-abutment pile casings, retrofitting pile casings around the northwest integral-abutment piles so that they can flex freely in response to superstructure thermal movements, filling any voids formed below the highway surface and behind the abutment and northwest wing wall with cellular concrete, construction of a subdrain pipe and drainage blankets behind the abutment wall, and reconstruction of the road surface (including sleeper slab, approach slab, waterproofing over approach slab, and the pavement structure).

The temporary shoring used to stage the highway excavation and backfill work consisted of a soldier pile and timber lagging wall system installed along the roadway center line behind the north abutment. After construction, the piles and timber lagging were left in place, but the system was cut off at least 1 m below the final grade. Following the temporary shoring installation, the contractor proceeded to remove and dispose of the materials within the limits of excavations including soil, asphalt, sleeper slab and approach slab.

The upper portions of the two-northwest integral-abutment piles impacted by the landslide were exposed to facilitate their repair. Upon exposing the impacted piles, it was evident that Class C concrete (previously placed to temporarily backfill the voids below the highway and abutment seat) had inadvertently flowed into the casing of the first exterior integral-abutment pile. Therefore, the originally proposed scheme to use a jacking system to re-center the existing casings, had to be adjusted as follows: 1) Cutting the upper 750 mm of the two exterior pile casings; 2) Hand chipping the exposed concrete around the exterior abutment H-piles 3) Hydrovacing a larger excavation around the remaining casings; 4) Installing (and subsequently removing) a temporary shoring system below the bridge; 5) Installing a new 760 mm diameter casing around the existing pile casings in the upper 3 m below the abutment seat (See Figures 8 and 9); 6) Sealing the gap between the underside of the abutment seat and the top of the casings with spray foam; 7) Backfilling the excavation around the new casings with 3 MPa fillcrete.

At the first exterior abutment pile, the bottom 3 m of the original 610 mm diameter pile casing remained filled with Class C concrete, which would result in a stiffer response to thermal movements. However, detailed structural analysis by Geometrix indicated the adjusted system would provide sufficient structural capacity and the required flexibility for accommodating future thermal movements of the bridge superstructure.

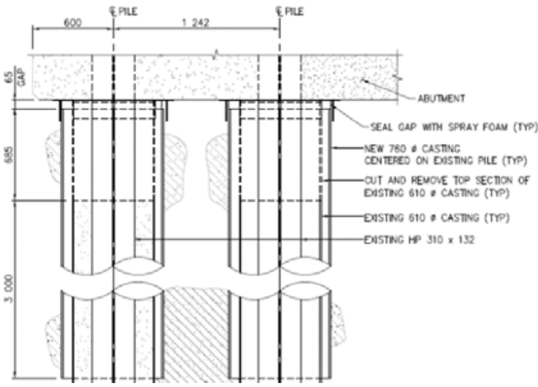

Figure 8. Cross Section of Impacted Piles Retrofit

Figure 9. Installing Casings Around Impacted Piles

Figure 10. Placement of Cellular Concrete

Backfilling and restoration of the highway surface was conducted by installing a subdrain pipe and drainage blankets, followed by supply and placement of light weight cellular concrete (CMEF-475) by Cematrix (Figure 10), supply and placement of granular base course, and re-construction of the sleeper slab, approach slab, and Asphalt Concrete Pavement (ACP).

Figures 11a and 11b show the site condition after completing all permanent repairs required on this project.

Figure 11a. Site Conditions During Final Inspection

Figure 11b. Site Conditions One Year After Construction

5 INSTRUMENTATION MONITORING RESULTS

As part of the construction, a series of new geotechnical instruments were installed to monitor the performance of the remedial measures. These included one slope inclinometer with two nested vibrating wire piezometers installed above the cantilever Sheet Pile Wall 'A', seven vibrating wire load cells to measure changes in anchor loads of the tie-back grouted anchors on Sheet Pile Wall 'B', and three vibrating wire strain gauges welded to the outer face of Sheet Pile Wall 'B' to monitor bending stresses in the wall.

Based on monitoring results up to the Fall of 2020, slope inclinometer SI19-1 has shown no discernible movement since initialization (Figure 12). Plots for the SI seem to indicate possible settlement of the casing around 12 m depth, which could correspond to a potential loss of grout at this depth during installation.

The load cells installed in Pile Wall 'B' (Figure 13) all showed reductions in anchor loads after installation ranging between 4 and 20 percent. However, the readings have been relatively stable between 2019 and 2020 and no signs of significant wall movement were noted after construction completion.

The observation of reductions in anchor loads after initial lock offs is consistent with our experience in similar pile wall projects. Seating and friction losses and load re-distribution between supports are typically the main reasons for load reduction. Anchors 12 and 24, which have the least depth of cover behind the wall, exhibit a well-defined correlation between the anchor load and ambient temperature. A temporary increase of 20 kN in anchor load occurs during the winter season due to frost effect. However, the anchor load during winter season is still below the design load, and hence it is not a concern at present time.

The upper, middle, and lower strain gauges installed on Pile Wall ‘B’ showed decreases in total micro-strain of 24.66 µƐ, 15.41 µƐ and 3.92 µƐ, respectively, since the spring of 2020 readings. This indicates a change of less than 5 MPa of bending stress in the sheet pile wall which is negligible compared to the yield strength of the steel. The observed bending stresses are likely a result of ice lensing and frost heaving in the soil during the cold winter months.

Additional monitoring during the winter months is recommended to confirm the stress and strain trends observed to date.

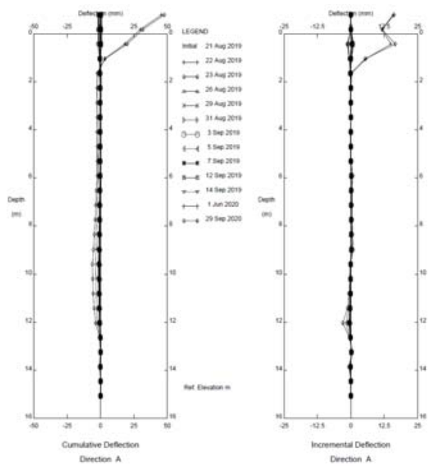

Figure 12. Slope Inclinometer Data, Pile Wall ‘A’

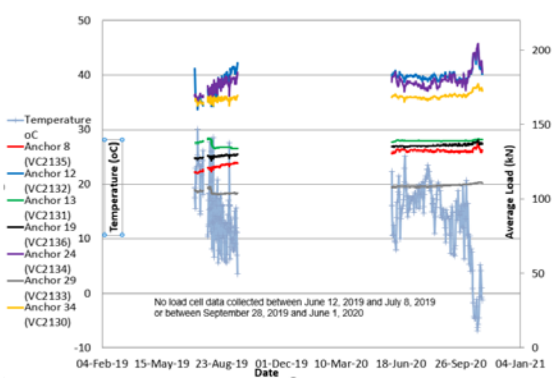

Figure 13. Anchor Load Cell Data, Pile Wall ‘B’

6 CONCLUSIONS

The following conclusions and lessons learned were derived from this project:

a. At this site, the the placement of steeply inclined clay embankment fill (i.e., transitioning from 2H:1V at the head slope to 3H:1V at the side slopes) as well as the presence of weak high plastic native clay soils were likely the main contributing factors to the landslide movement impacting the structural integrity of the Baptiste Creek bridge.

b. A thorough geotechnical investigation should be conducted for bridge replacement/rehabilitation projects regardless of the project size to avoid expensive repair measures in response to slope movements.

c. Integral abutment bridges rely on backfill behind the abutment for proper functioning. Any partial loss of the fill will lead to un-balanced loading between the two abutments and cause shifting and twisting of the bridge. This type of structure should be avoided in areas prone to slope instability unless additional stabilization measures are considered in the design.

d. The permanent design and construction measures described in this paper have proven to be effective in stabilizing the landslide movement.

e. Instrumentation monitoring has been valuable to assess the grouted anchor loads and steel sheet pile wall movements. Continued monitoring of existing instrumentation would be beneficial to assess the long-term performance of the steel sheet pile walls.

f. Multiple challenges were encountered during construction including early sheet pile driving refusal, misalignment of sheet pile walls, piling under low head room, and inability to rectify original integral-abutment pile casings. The success of the project and ability to resolve unforeseen issues was due to effective communication between the design and construction teams and timely response from the owner.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Thurber Engineering Ltd. May 29, 2020. *NC090: Baptiste Creek Bridge Slope Stabilization Construction Summary Report*., Project No. 23051, Edmonton, AB, Canada.

Geometrix Group Engineering Ltd. June 28, 2018. *Preliminary Damage Assessment Report for Landslide at Baptiste Creek Bridge on Highway 2 (BF07055)*., Project No. 18-0610, Edmonton, AB, Canada.