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Performance of a tied-back concrete pile wall for slope stabilization near Peace River, Alberta

Performance d'un mur de soutènement de pieux en béton armé avec des niveaux ancrages multiples pour la stabilisation d'une pente près de Peace River, Alberta

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ABSTRACT: The Peace River region in Northern Alberta is well known for slope failures affecting highways in the valleys. Along Highway 2:60, immediately east of the Town of Peace River, a tied-back tangent drilled reinforced concrete pile wall was constructed adjacent to an existing cast-in-place concrete cantilever pile wall in response to road settlement and tension cracks caused by slope movement. Due to the variable depth to the slip surface of the landslide, the wall was divided into six sections with different combinations of cast in place concrete piles and levels of soil anchors. The aim of this paper is to select one section of this wall to study its performance to date. The section selected had the deepest slip surface, most levels of anchors, and the greatest noted road settlement. The design is reviewed at a high level to determine expected loads, then the available instrumentation data, including Shape Accel Arrays, Strain Gauges, and Vibrating Wire Load Cells, are used to estimate performance of the wall since construction. The comparison of these data with the expected values estimated during design shows that the wall section has experienced some bending deformation, but that it has not exceeded any allowable loads or movements.

RÉSUMÉ : La région de la rivière la Paix, située dans le nord-ouest de l'Alberta, est renommée pour les nombreux glissements de terrain qui sillonnent les vallées de ses rivières. Le long de l'autoroute 2:60, situé à l'est de la ville de Peace River, un mur de soutènement, muni de tirants d'ancrage, composé de pieux de béton armé forés tangents a été construit à proximité d'un muret en porte-à-faux existant composé de pieux de béton armé, afin de stabiliser le remblai routier suite à l'apparition fissures de tension et du tassement de la chaussée occasionnés par le mouvement d'un glissement de terrain. Vu de la profondeur variable de la surface de rupture, le mur a été construit avec six sections critiques ayant différentes combinaisons de pieux et d'ancrages. L'objectif de cet article est d'étudier la section critique comportant la surface de rupture la plus profonde. Les critères de base de la conception sont abordées sommairement dans un premier temps afin de déterminer les contraintes de chargement admissibles et la performance de la paroi est ensuite abordée par l'analyse des données recueillies de plusieurs instruments, dont une chaine de capteur ShapeAccelArray, des jauges de contrainte et des cellules de charge à corde vibrante. Cette analyse a démontré que même si cette section du mur a subi une certaine déflexion latérale, qu'elle n'a cependant pas excéder le seuil de mouvement tolérable envisagé pour l'ensemble des charges admissibles prévue à cet endroit.

KEYWORDS: tied-back pile wall, slope stability, geotechnical instrumentation.

1 INTRODUCTION

The Peace River area in Alberta is well known for slope instabilities. The focus of this study is a landslide that was affecting a section of Highway 2:60 on the East Hill section that descends the north valley slope of the North Heart River, east of the Town of Peace River. At this location, the highway was constructed on a sidehill alignment over ancient landslide terrain. Movement was observed along a section of valley spanning from the highway surface level along a 160 m drop in the slope down to the North Heart River floor. Along the downslope edge of the highway embankment, the depth to the slip surface varied from zero to 18 m. A tied-back tangent pile wall was constructed to stabilize the landslide.

Instrumentation was used to aid with design and to provide information on the performance of the structure. This paper serves to evaluate the performance of one section of the wall using instrument readings obtained from the onset of construction over period of three years after construction.

2 BACKGROUND INFORMATION

Highway 2:60 at this location (kilometer 34) is a three-lane highway including an eastbound climbing lane with an Average Annual Daily Traffic (AADT) value of 4,580 for 2019 (Alberta Transportation, 2019). The highway was originally constructed in 1956 through an ancient landslide upslope of the North Heart River. Construction of the initial highway embankment activated movement along the western flank of the ancient landslide causing the highway to drop several meters. In response to this movement, the highway was re-aligned further into the hillside. In 1998, a cast-in-place cantilever concrete pile wall was constructed downslope of the highway at approximately kilometer 33.86 in response to retrogression of an interior slide block toward the highway (Figure 1). In 2010, slope inclinometers (SIs) were installed upslope and downslope from the wall to monitor the wall performance. Following the construction of this initial wall, downslope creep movement continued, removing passive resistance on the downslope side of the pile wall, and the slide area expanded beyond the west end of the wall into the highway embankment.

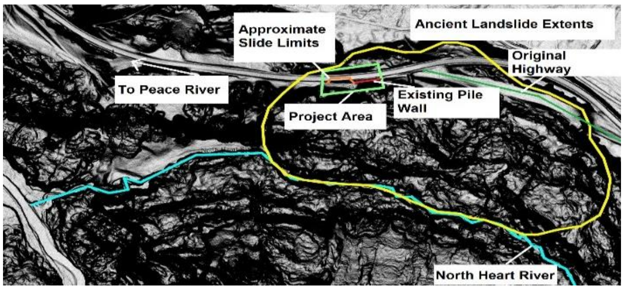

Figure 1. General site details.

Three of the five SIs installed in 2010 were sheared off within two years of installation. SIs installed in 2013 and 2014, west of this wall to investigate new highway dips and cracks, experienced lateral deformation rates of up to 87 mm per year. Construction of a new pile wall was initiated in 2016 which consisted of a multi-section cast-in-place (CIP) concrete tangent pile wall immediately west of the existing wall. The new wall consisted of six design sections with different configurations of piles and anchors depending on the depth to slip surface. The focus of this study is on design Section 4 which is the section with the deepest slip surface, the most rows of anchors, and two rows of concrete piles.

3 DESIGN AND CONSTRUCTION

Although many pile walls have been constructed in the Peace River area, this project was unique considering the depth to the failure surface, at 18 m below ground surface, and the resulting high loads that would act on the wall if the soil on the downhill side of the wall moved away down the valley leaving the wall face of the wall unsupported above the failure surface.

Figure 2 shows a stratigraphic cross-section through Section 4. The subsurface conditions consisted of clayey colluvium extending down to the slip surface of the landslide, underlain by stiff to very stiff high plastic clay over very stiff to very hard clay till. The shear load that the wall would need to resist was initially determined based on a two-dimensional limit equilibrium slope stability analysis using a minimum factor of safety of 1.3. The resulting load was then converted to an equivalent lateral earth pressure with a triangular distribution with 265 kPa of pressure acting above the slip surface on the upslope side of the wall. The earth pressure load would be resisted by the passive earth pressure in front (downslope) of the pile below the slip surface based on an ultimate resistance (q_u) of 1,600 kPa in the high plastic clay, and 2,500 kPa in the clay till.

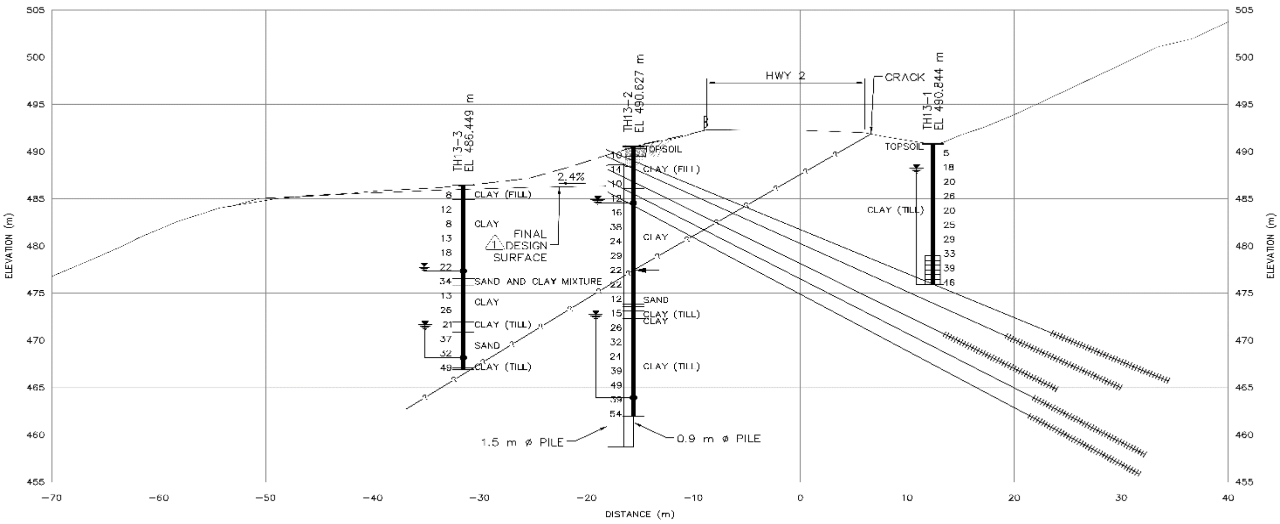

Figure 2. Cross-section through wall showing soil stratigraphy and anchor design.

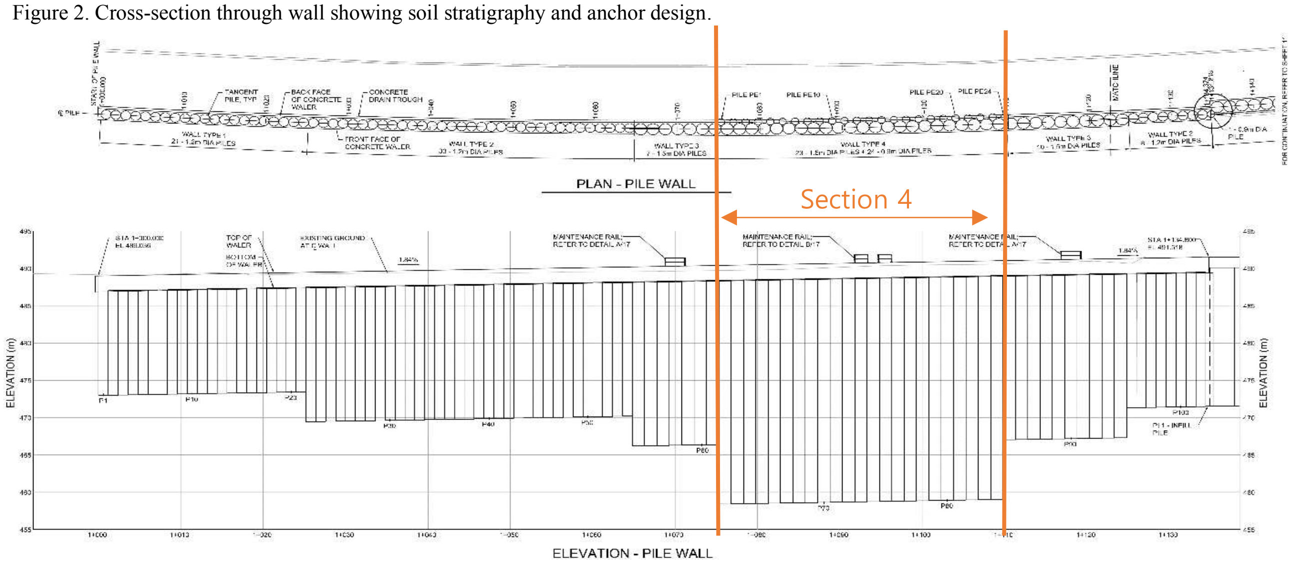

Figure 3. Elevation and plan view of pile wall showing location of design Section 4.

The structural analysis performed for the design of the steel reinforcement in concrete piles used springs to simulate the soil resistance based on a horizontal subgrade modulus, k_{s1} of 110 MN/m^3 per meter width of pile for the high plastic clay, and 170 MN/m^3 for the clay till. These values were adjusted for limits states design and pile spacing using appropriate resistance and spacing factors. The soil springs were modelled as non-linear springs, with a linear elastic zone up to the lateral soil bearing capacity, dropping to 50% of the bearing capacity in the plastic zone.

The piles were generally modelled using both uncracked and cracked section moduli for the service and ultimate limit state, respectively. The piles were modelled using an initial cracked section modulus value, and then the model was iterated to compare the moment-curvature relationship with the corresponding section modulus.

Ground anchors were structurally modelled as linear springs, with their stiffness calculated based on the free stress anchor length, cross-sectional area, and steel modulus of elasticity. Initial jacking (lock-off) forces were assumed and then iterated in each wall type to balance initial loading with the final acceptable reaction configurations; these initial forces affected the deflected shape of the wall in the initial and final load configurations. The anchor reactions were compared with the bond zone friction resistance values to confirm that anchor slip would not occur.

Structural models were iterated until a number of factors were determined to be acceptable, including ground anchor configurations and loads, demand and capacity of the piles, and total wall deflection for Serviceability Limit State (SLS) criteria. Pile capacities were reduced at wall anchor locations to account for loss of reinforcing where the piles were cored through for ground anchor installation. The waler was modelled horizontally, with ground anchor loads applied sequentially to determine demands during construction.

As shown on Figure 3, Section 4 of the wall was designed with twenty-three 1,500 mm diameter CIP concrete piles drilled to 29.9 m depth and capped with a 2.2 m high concrete waler. Each pile was reinforced with twenty-eight 35M vertical steel reinforcing bars. Cast into the spaces between the back of the piles was a row of 914 mm diameter CIP concrete piles drilled to 29.4 m depth. These were reinforced with twenty 30M vertical steel bars.

Five rows of pressure grouted double corrosion protected ground anchors comprised 32 mm diameter high strength threaded steel stressing bars were installed. The anchors were installed with a 200 mm diameter grouted bond zone of 12 m in length and with free stress zones long enough to place the bond zones below the slip surface of the landslide and to provide the anchors with sufficient elasticity to accommodate wall movement under various load scenarios.

Ultimate Limit State (ULS) factored design loads for the anchors varied from 337 kN at the upper waler anchor up to 425 kN at the lowest pile anchor. Similarly, SLS design loads varied from 337 kN at the upper waler anchor to 175 kN at the lowest pile anchor.

SLS loads were optimized to limit the horizontal deflection of the pile heads to less than 50 mm. It was anticipated that as the downslope soil in front of the wall were excavated, the pile wall would flex towards the excavation. When each row of anchors was prestressed to the design lock-off load, this would result in pulling the wall partially back in the upslope direction. The anchors installed in the waler were locked off at loads higher than the design SLS load under the expectation that as the soils upslope of the wall reached equilibrium, these anchors would relax over time.

4 INSTRUMENTATION

During casting of pile P74, which is located at roughly the center of Section 4, 28 Vibrating Wire Strain Gauges were attached to the front (downslope) and back (upslope) sides of the steel reinforcement cage between depths of 0.2 m and 28.2 m along the pile. A Shape Accel Array (SAA) was also installed in pile P74 to measure deflections, and Vibrating Wire (VW) Load Cells were installed at the location of the outer anchor lock-off plate for each of the five anchors on P74.

The instruments were read through all the stages of construction and bi-annually since completion.

4.1 *Readings*

The following sections outline the findings from the instruments read during and after construction of the new pile wall. Plots and data from the SIs and SAA were reviewed to note deflections in the slope and the pile wall, strain gauges were used to observe deformation trends as well as bending moments along the pile, and load cell data were used to compare anchor loads to those anticipated during design.

4.2 *SAA/SI Data*

Prior to construction of the retaining wall, this site had eleven SIs that were read bi-annually from 2010 to 2015. Between September 2010 and September 2012, three of the five SIs installed in May of 2010 had sheared off at depths between 6.1 m and 12.2 m with rates of deflection preceding the shearing between 50 and 54 mm per year. In December 2014, the maximum rate of deflection in the 2014 SIs was 87 mm per year between 3.7 and 9.8 m depth.

The plot in Figure 4 shows the cumulative deflections from August 2013 to December 2015 in SI13-2 which was installed in 2013 adjacent to the highway where the slumping and cracking was most notable. This location later coincided with the wall Section 4. The plot clearly shows significant deflections at approximately 17 to 18 m depth, prior to wall construction.

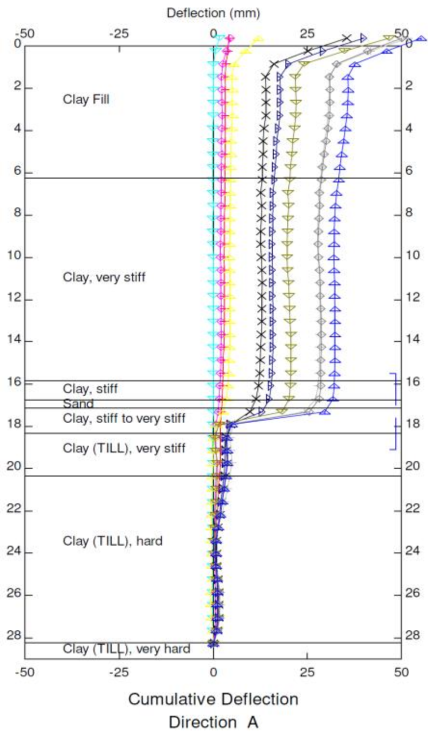

Figure 4. Cumulative SI plot for SI13-2 prior to construction.

Figure 5 shows the change in displacement over time at SI14-4 prior to, during, and after construction. SI14-04 is located downslope of the pile wall, which had experienced rates of displacement of up to 87 mm per year prior to construction of the wall. The displacement is shown to increase steadily until June 2017 which corresponds to the time of anchor installation and lock-offs, level off until September 2019, then increase again but at a lower rate between September 2019 and October 2020.

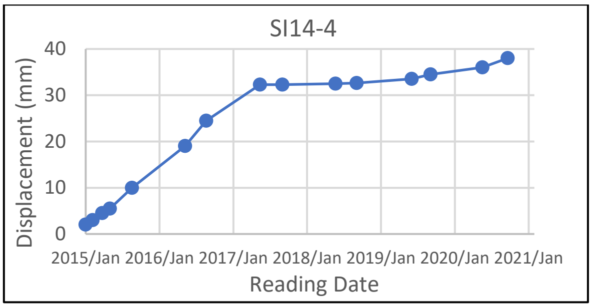

Figure 5. Displacement over time at SI14-4.

Based on the SI data, the slope movement downslope of the wall appears to have slowed significantly which suggests that the pile wall has reduced the driving force on the portion of the slide mass downslope of the wall. Some creep is still expected to occur downslope of the wall; however, the wall is expected to protect and retain the highway, which was the main concern for this project. SIs are still in place downslope of the wall to continue monitoring any creep movements or new accelerations.

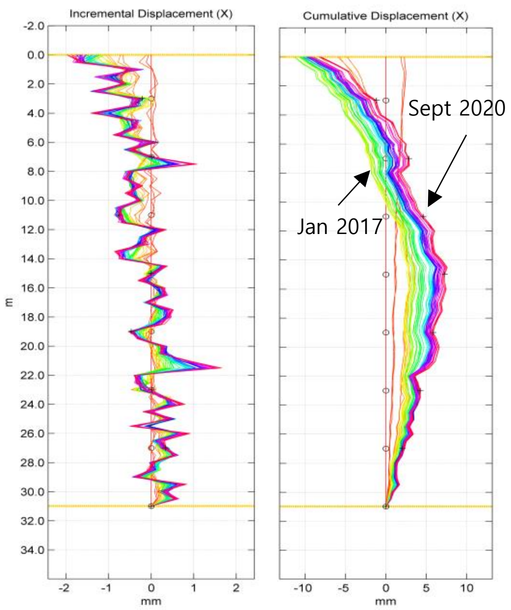

Figure 6. Cumulative and Incremental deflection plots from P74 SAA from January 2017 to September 2020.

The plots in Figure 6 show the cumulative and incremental displacement in the SAA installed in pile P74, from January 2017 during construction (green) to the latest reading in September 2020 (red). The zero depth on the plot corresponds to the interface between the top of pile and bottom of waler. By the end of construction, the top of the pile had been pulled 12 mm toward the highway in response to locking off the anchors in the waler, and the pile had deflected up to 4 mm in the downslope direction at 16 m below the top of pile (approx. slip surface depth). Over the period after construction leading up to 2020 some redistribution of the loading had occurred reducing the pile head deflection to 7 mm toward the highway and increasing the deflection at the slip surface to 7 mm in the downslope direction. These deflections are well under the serviceability limit of 50 mm of pile head deflection.

4.3 *Microstrain and Bending Moments*

Strain gauges placed between 0.2 m and 28.2 m below the pile head in pile P74 were read using a datalogger between January 2017 after pile construction and excavation to the top of the piles, and September 2020. VW frequency (Hz) readings obtained were converted into units of microstrain (με) using a conversion factor provided by the instrument supplier (RST Instruments. 2019).

Bending strains were analyzed and compared with the SAA reading plots from the same date for April 10th, 2017, after the upper pile row of anchors were locked off (Figure 7), for August 29, nearing the end of construction (Figure 8), and September 1, 2020, the latest reading (Figure 9).

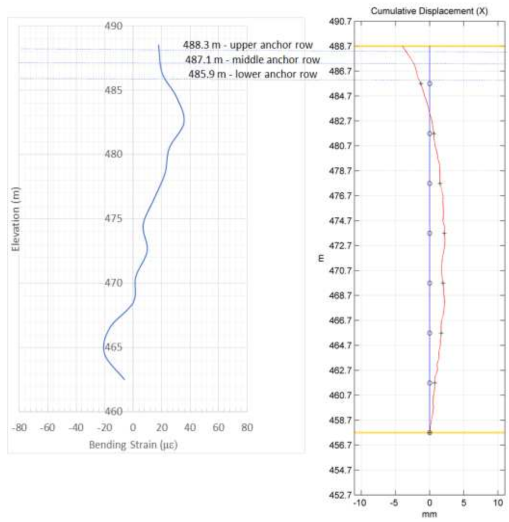

Figure 7. Bending strains and SAA movement reading for April 10, 2017 – lock off of upper row of anchors.

Figure 8 plots a similar shape to Figure 7 except with higher magnitudes in both compression and tension. At approximately 486 m elevation, the strain gauge was no longer functioning but there appears to be a peak in tensile strain before a rapid transition to compression. This phenomenon is also shown in the SAA movement plot as a possible inflection point in the curve of the pile.

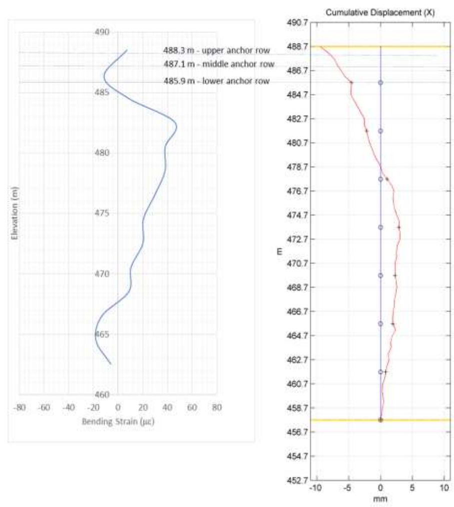

Figure 8. Bending strain and SAA plot on August 29, 2017 – end of construction.

Finally, Figure 9 shows the bending strain and SAA movement as of September 1, 2020, three years post-construction. The SAA movement plot shows multiple inflection points along the pile, and cumulative displacements had increased from 3 mm to 7 mm. The bending strain plot shows similar shape to Figures 7 and 8 but with higher compressive microstrains. With maximum bending strains at approximately 60 με, the sudden increase in magnitude is not a cause for alarm. Based on the overall strain readings, it is unlikely that cracking in the pile concrete has yet initiated.

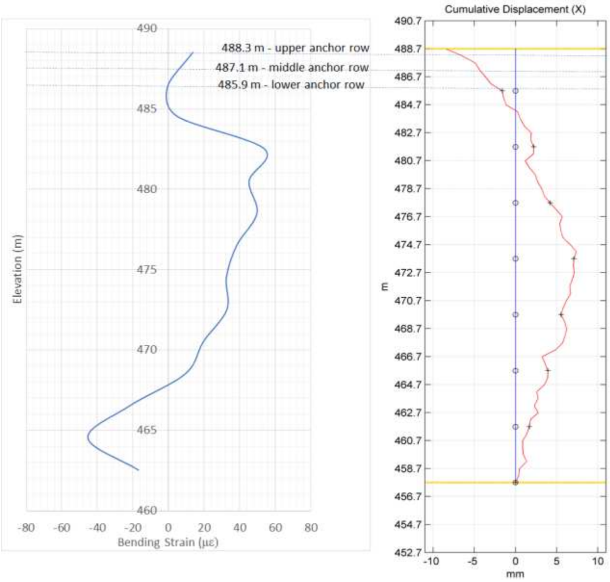

Figure 9. Bending strain and SAA plot on September 1, 2020 – 3 years post-construction.

The strain gauge data was used to determine whether the pile concrete had cracked during service. Since the strains did not appear to exceed the threshold for cracking, it could be assumed that the structural behavior of the pile remained in the linear elastic range with uncracked behavior. It is also worth noting that the reinforcement has not yet even come close to the specified yield strain of 2,500 με. Given this assumption for structural behavior, the strain gauge data was used to compute the bending moments along the length of pile P74 from April 10, 2017 when the middle bench was being excavated to the latest readings on September 1, 2020. The moment at each strain gauge elevation was calculated as:

$$M = \frac{EI\Delta\varepsilon}{y} \tag{1}$$

where EI is the flexural stiffness of the transformed concrete section, Δε is the change in the flexural component of the strain from the unstressed state, and y is the distance to the neutral axis of the pile.

The cracking moment, M_{cr}, was calculated using the software RESPONSE 2000 by transforming the concrete and steel composite section into an equivalent concrete section and calculating the moment it can sustain in pure bending at initial cracking:

$$M_{cr} = \frac{f_t I}{y_t} \tag{2}$$

where f_t is the flexural rupturing strength calculated using CSA S6-19 (2020) (MPa), I is the second moment of inertia (m^4) of the transformed concrete section and y_t is the distance from the outside of the pile to the neutral axis (Clark & Richards, 2006).

Figure 10 shows the bending moments along pile P74 calculated using Equation 1 for April 10, 2017, August 29, 2017, and September 1, 2020. The cracking moment was calculated using Equation 2 and varies along the height of the pile, based on the changes in the longitudinal reinforcement in the concrete cross section.

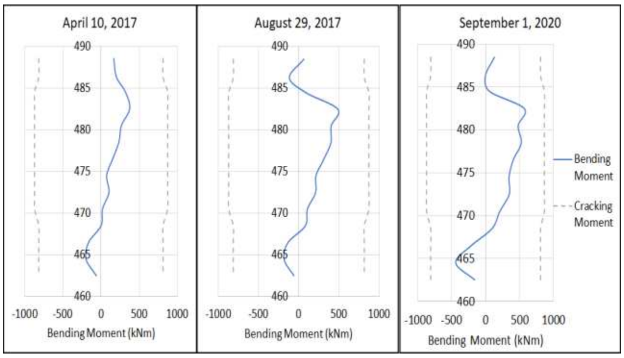

Figure 10. Bending moments calculated at pile P74.

The calculated bending moments and pile deformation exhibit patterns consistent with what is expected. At the beginning, large initial moments developed in the upper portion of the pile as the uphill soil resisted movement of the pile as the anchors were stressed. The double curvature that occurs at the top of the pile arises as the lower pile anchor rows are stressed; this is seen from the April to August bending moment diagrams in Figure 10. Over time, the moment at the top portion of the pile transitions to single curvature as the soil loading increases on the uphill side of the wall and is expected intensify as the supporting soil on the downhill side of the wall is lost and the lower anchor rows in the top of the pile pick up additional load; this is seen in the change in the bending moment from August 2017 to September 2020 in Figure 10. As expected, the bending moment is inverted in the lower portion of the pile due to restraint from the soil – this remains the case under ultimate limit state conditions as shown in the structural model output in Figure 11 below. Figure 11 shows the bending moment distribution superimposed on the structural model of the pile at the ultimate limit state. Currently, the low magnitude of the moments in the pile, strain readings, and minimal deformation (compared to the ultimate limit state) not only suggest that the pile has yet to begin recruiting a minute fraction of its ultimate structural capacity, but that it hasn't yet begun to crack. However, the trends observed in the changing bending moment and deformation profiles suggests that the internal demands on the structure are developing as expected as the soil loading increases on the pile.

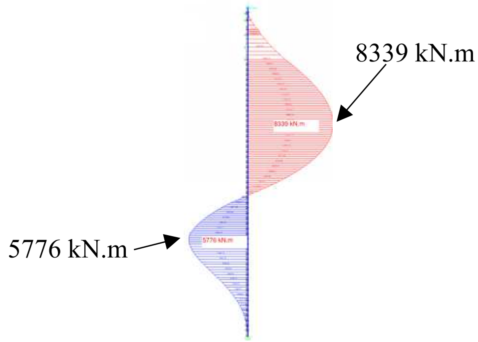

Figure 11. Bending moments from structural model at Ultimate Limit State.

4.4 *Anchor Loading*

As discussed in section 3 of this paper, loading conditions were determined for the ultimate limit state and the serviceability state.

Table 1 shows the design loads, lock-off loads, and measured loads at the end of construction and in September 2020. These readings show that, as expected, the upper anchors with higher lock-off loads have slackened over time, and those with lighter lock-off loads have started to pick up more load over time.

Table 1. Anchor loads in kN (Oct 2017 is end of construction).

Anchor Row (Anchor#)	ULS Factored Design Load	SLS Design Load	Lock-off Load	Load Oct 2017	Load Sept 2020
Waler Up (G80WU)	303	246	331	307.6	295.6
Waler Low (G80WL)	368	293	337	313.3	307.6
Pile Upper (G134PU)	391	288	229	199.0	200.4
Pile Middle (G167PM)	409	302	175	157.1	161.6
Pile Lower (G190PL)	425	306	105	105.7	120.3

Figures 12 and 13 show plots of the waler anchors and the lower pile anchor, respectively. The waler anchors show a gradual decrease in load from the lock-off loads with seasonal fluctuations related to air temperatures. The lower pile anchor, which is buried, shows an increasing trend with small jumps in load as the surrounding soils freeze. The decrease in load of the waler anchors and the increase in load of the lower pile anchor are in line with the expected response, and no further loss of load has occurred since the end of construction.

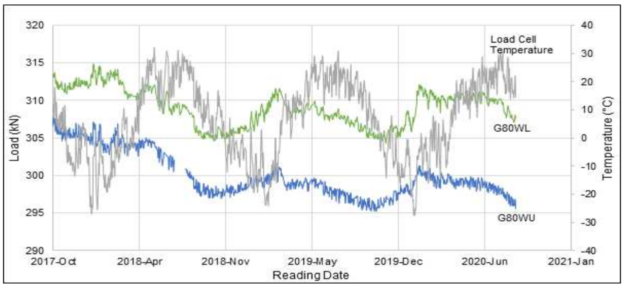

Figure 12. Load cell at the waler anchors compared to temperature.

Figure 13. Load cell at the lower pile anchor compared to temperature.

5 CONCLUSIONS

Based on the analyses conducted for this study, the tied-back cast-in-place concrete tangent pile wall appears to be behaving in line with expectations during design. SI plots after completion of the wall show a significantly decreased rate of slope movement, and the SAA embedded in pile P74 shows minimal deflections in the order of 1 to 2 mm.

Strain gauge data does not show any concerning increases in deformation and bending moments along the pile show that soil loading is increasing on the pile but that deformations have not even begun to approach those determined under serviceability or ultimate limit states conditions. Finally, load cell data shows that anchors are supporting wall loads as expected, and loads are fairly steady far below the design load values.

This type of pile wall has been widely used for landslide mitigation in the Peace River area and this study has shown that it is a suitable repair option for deep seated slope failures up to 18 m below ground. Water bearing sand layers did pose complications during construction; however, with appropriate workarounds, these complications did not hinder the performance of the wall.

6 ACKNOWLEDGEMENTS

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