Soil-Structure Interaction Response of a Tangent Cast-in-Place Pile Wall in Peace River, Alberta

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ABSTRACT
A cast-in-place tangent pile wall was completed in 2018 to prevent further retrogression of a slide affecting a road embankment located on Highway 986:01, near Peace River, Alberta. The wall consisted of 82 concrete piles supported by three rows of ground anchors. In-place Shape Accel Array inclinometers, strain gauges and load cells were installed on selected piles. In this paper, the soil-structure interaction response of the wall was investigated by back calculation of pile earth pressures, and pile bending moments and shear forces using the instrumentation data and the “integration method” de rigueur in structural analysis. Partial pile hinges derived from the strain gauges were considered in the back calculation and proved to be the determining factor to estimating pile internal forces. The integration method was coded in Mathcad and the results obtained were validated with the commercial structural software S-FRAME. This approach was used to adjust the wall building sequence to the challenges encountered during construction. It can also be used to predict the long-term performance of the wall.

RÉSUMÉ
Un mur de pieux tangents en béton coulé sur place a été construit entre 2017 et 2018 afin de limiter la rétrogression du glissement d’un talus routier situé le long de l’autoroute 986 :01 près de Peace River en Alberta. Le mur était composé de 82 pieux en béton armé et supporté par trois rangées d’ancrages au sol. Des inclinomètres en place de type ShapeArray, des jauges de contrainte et des cellules de charge ont été installés sur trois des pieux. Dans le présent article, la réponse des interactions sol-structure du mur a été étudiée par le rétrocalcul de la poussée des terres ainsi que des moments fléchissants et des forces de cisaillement à l’aide des données obtenues des instruments et de la méthode d’intégration communément utilisée en analyse structurelle. Des déformées partielles découlant des données des jauges de contrainte ont été incluses dans le rétrocalcul et se sont avérées être déterminantes dans l’estimation des forces internes agissant sur le pieu. La méthode d’intégration a été codée dans le logiciel Mathcad et les résultats obtenus validés à l’aide du logiciel d’analyse structurelle S- FRAME. Cette approche a été adoptée durant le projet afin d’évaluer les interactions sol-structure du mur. Elle s’est avérée être un outil pratique d’aide à la décision afin d’ajuster la séquence de construction à la suite de certaines difficultés rencontrées. Elle peut également être utilisée afin de prédire la performance à long terme du mur.

1 INTRODUCTION

Since its construction in the 1980s, Highway 986:01, in the Peace River region of northern Alberta, has been an important transportation corridor for the resource industry, with an average daily traffic count of 840 vpd. The road is affected by erosion and slope stability problems. These problems are notorious since the highway was built on active landslide areas. Alberta Transportation (AT) initiated a geotechnical instrumentation monitoring program in 2000 to assess the movement progression of various at-risk geohazard sites. One site, located on Highway 986:01 about 20 km northeast of the Town of Peace River, was selected in 2014 to undergo extensive landslide mitigation work. At this location, tension cracks opened on the embankment slope and extended diagonally across the roadway, and the pavement abruptly dropped up to 300 mm (Figure 1).

The landslide mitigation work was designed and administered by Thurber Engineering Ltd. (Thurber) on behalf of AT and consisted of stabilizing the road embankment using a cast-in-place concrete tangent pile wall supported by ground anchors.

Figure 1. Looking southwest at cracking and scarp across roadway in 2015.

2 SLOPE FAILURE MECHANISM

Highway 986:01 at the site is located within an historic landslide area on a gentle north-facing hillside slope. The roadway is approximately 30 m above an unnamed creek tributary to the Peace River to the west and was built on a road embankment consisting mainly of high plastic clay fill. In general, the embankment is overlaying high plastic clay colluvium from the previous ancient landslides overlying a high plastic clay glaciolacustrine deposit.

The roadway embankment has experienced ongoing slow movement since construction in the 1980's. In 1991, a slide impacted the embankment and was estimated to be about 100 m wide with a failure surface of up to 14 m deep above stable glaciolacustrine clay based on previously installed slope inclinometers.

The creek at the toe of the slope created erosion that encouraged slope instability. At this location, a repair that relied on dewatering, by pumped wells was installed in 1995. After several years, the pump system fell into disrepair and this solution was no longer effective in stabilizing the slide. Movement rates began to increase and tension cracks were observed during subsequent inspections. For many years the cracks and pavement settlement were managed though selected pavement patching and milling operations. By 2014 the effort required to maintain the highway in a safe condition had become untenable and a more permanent solution was sought.

3 DESIGN AND CONSTRUCTION

The selected road embankment stabilization consisted of building a tied-back cast-in-place concrete tangent pile wall as shown in the typical cross section in Figure 2.

The pile wall was 98 m long and included 82 piles of 1.2 m in diameter and 30 m in vertical length connected by a continuous concrete pile cap. Three rows of ground anchors (230 anchors in total) were used to support the wall. The combined bond and free length of the anchors varied from 29 m to 46 m.

The anchors were installed through the piles at 2 m, 4m and 7.5 m below the top of the pile cap. The anchors used were 32 mm diameter double corrosion protected DYWIDAG anchors. The bond length was 12 m and was installed within the stable glaciolacustrine clay (Figure 2). The anchors were inclined between 27 and 30 degrees from the horizontal.

The wall was designed for a factor of safety (FOS) against slope failure of 1.3 and a total exposed wall of up to 10 m. The maximum total combined anchor load required for these conditions was 1268 kN per 1.2 m of wall (per pile). To ensure this load would be achieved, all the anchors were proof tested to 532 kN. The expected ultimate geotechnical anchor capacity is about 668 kN per anchor.

To estimate the maximum total combined anchor load, a slope stability analysis was conducted assuming that the high plastic clay colluvium was sliding over stable glaciolacustrine clay and had a residual friction angle of 14.5 degrees. Figure 2 shows the inferred slip surface based on the slide backscarp visible in the highway pavement and the upslope highway ditch further south, as well as the previously installed slope inclinometers. The slide geometry was further confirmed by the soil logging during piling activities and an additional geotechnical drilling investigation conducted during construction.

Vertical drains behind and in front of the wall were installed to control the groundwater pressure behind the wall.

After the installation of the anchors, the third anchor row was buried to about 1.5 m below the finished ground surface downslope of the pile wall (or about 6 m below the top of the pile cap) as shown in Figure 2. Since the wall was designed assuming a total exposed wall of up to 10 m, this implies that the finished ground surface downslope the pile wall may drop down an additional 4 m without jeopardizing the geotechnical and structural stability of the wall. This drop in grade was accounted for in the design as mainly caused by long term creek created erosion at the toe of the slope. It was also assumed that by offloading the

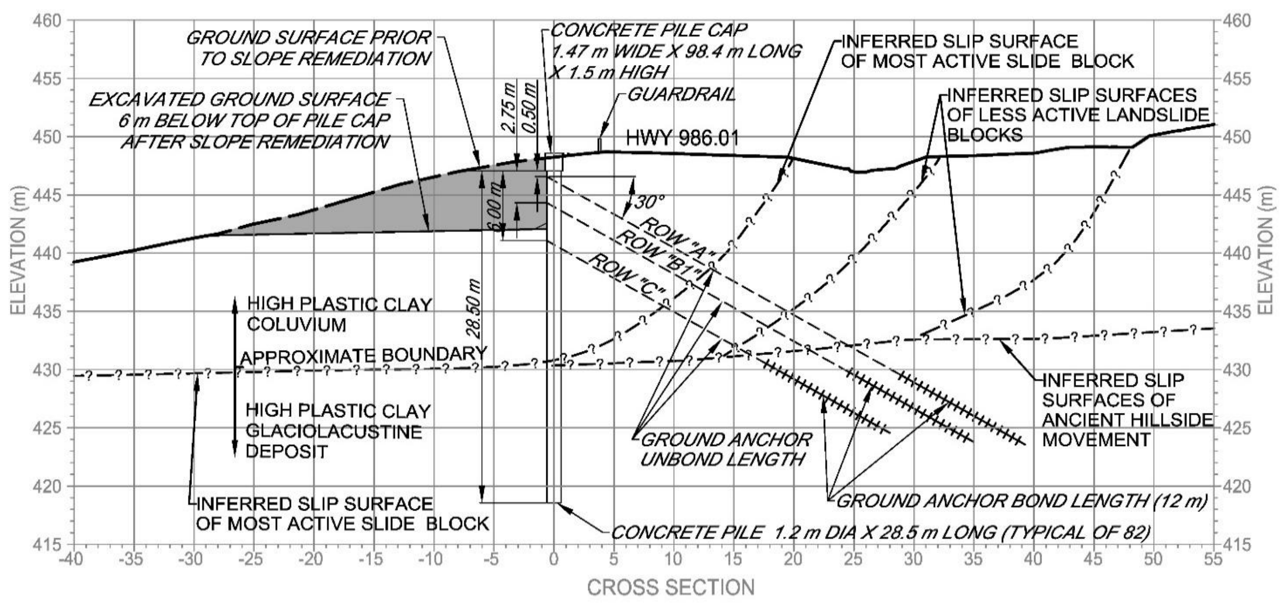

Figure 2. Typical cross section of the tied-back pile wall

Figure 3. Nighttime anchor installation

slope in front of the wall by 6 m, it would contribute to the overall slope stability of the embankment.

The wall building sequence had to be adjusted to face challenges encountered during the installation of the ground anchors which took about 6 months longer than expected, including nighttime installation (see Figure 3). The location and original design load magnitudes of the ground anchors had to be adjusted to accommodate the delay.

The tied-back pile wall construction took place from January 2017 to March 2018. The completed pile wall structure is shown in Figure 4.

To monitor the soil-structure interaction behavior of the wall, an in-place Shape Accel Array (SAA) inclinometer was embedded in three piles located on the east, middle and west portions of the wall. Load cells were installed at the three anchor head locations on each of the three instrumented piles. A string of strain gauges was attached to two vertical rebars of the reinforcing steel cage (one upslope and the other downslope) of the middle wall pile. The strain gauges in each string were at 2 m vertical intervals. The location of the gauge strings was chosen such that either tension or compression strains are measured depending on the shape of the deflected pile at a particular gauge location.

Vibrating wire piezometers (VWPs) and Slope inclinometers (SIs) were installed upslope and down slope from the pile wall to monitor the global slope stability of the wall. This included one SAA installed a few meters down slope of the pile wall.

The cables of the electronic instruments were routed to a central datalogger enclosure equipped with a modem to allow for remote download of the data.

Instrument readings were taken from the installation period (between February 2017 and March 2018) until June 2019. Future readings will be taken as part of a longer-term asset management program AT is currently developing.

Figure 4 Drone view of the completed pile wall

4 INSTRUMENTATION

4.1 General

A comprehensive geotechnical instrumentation program was implemented to monitor the soil-structure interaction of the wall and global geotechnical slope behavior during and after construction.

4.2 Instrumentation Results

Instrument readings for the pile located at the middle of the wall are shown in Figure 5 and Figure 6.

In Figure 5, the first horizontal plot reports daily air temperatures over time from Environment Canada. The remaining horizontal plots present cumulative anchor loads and cumulative pile deflections at each anchor location.

This figure reveals that deflections mainly occurred during construction and that the maximum pile deflection to June 2019 was about 50 mm. The maximum deflection is regarded as small considering the size of the piles (30 m pile length).

The responses of the upper two anchor load cells suggest a gradual load increase during the winter of 2019 followed by a rapid load decrease as the air temperature rises above zero degree Celsius in early spring. The response of the lowest load cell suggests that the anchor load at this elevation is less sensitive to winter. It is our opinion that the load increase in the upper two anchors during winter is due to some combination of anchor head contraction imposed by the freezing temperatures and expansion of freezing soil and ice exerting pressure on the wall. The lowest row of anchors is covered by 1.5 m of fill, and some amount of snow in the winter; and thus, it is mostly protected from temperature changes.

The maximum anchor load recorded was about 330 kN which is well below proof test results (532 kN).

It is anticipated that the loads acting on the upper two rows of anchors will continue to fluctuate due to seasonal changes in groundwater and temperature; however, these

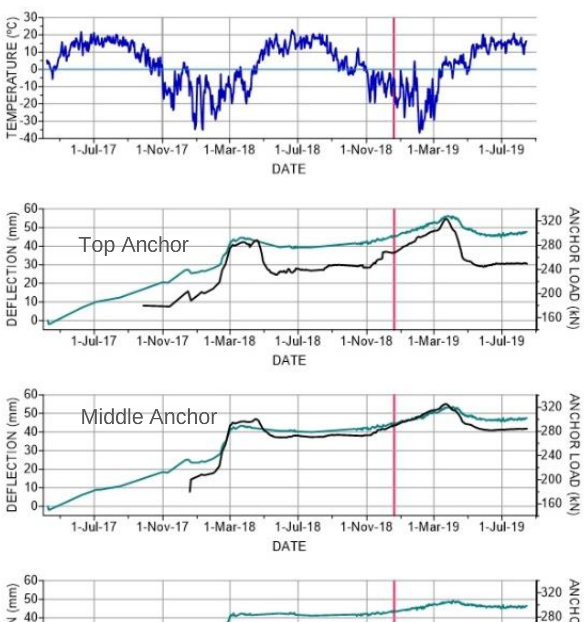

Figure 5. Temperature (in blue), cumulative anchor loads (in black) and cumulative pile deflections (in green) at each anchor location.

load fluctuations are considered small compared to the total load they are subjected to, and well below the expected ultimate geotechnical anchor capacity (668 kN).

Figure 6 presents the pile deflection recorded on December 20, 2018 (red lines on Figure 5) along with the strain gauge results at 2 m vertical intervals. On this date, the loads recorded for the top middle and lower anchors

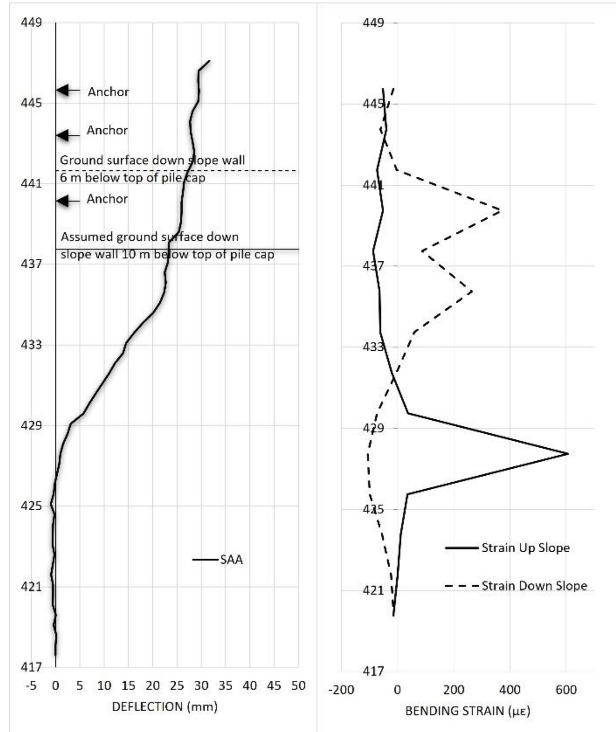

Figure 6. Pile deflections and pile strains at 2 m vertical intervals for December 20, 2018

were 268 kN, 290 kN and 153 kN respectively as shown in Figure 5.

The strain gauge data suggest that possible partial hinges in the pile may have developed at three locations as indicated by the two strain spikes in the gauges facing down slope near the lower anchor at about elevation 438 m, and one strain spike in the gauges facing upslope near elevation 428 m. These partial hinges are likely the result of internal stresses being transferred from the concrete to the reinforcing steel as the pile cracked in response to deflections.

The interpretation of the instrumentation results at this date for the pile located at the middle of the wall is used to illustrate the approach adopted for the project on how to assess the soil-structure interaction behavior of the wall.

5 SOIL-STRUCTURE INTERACTION ANALYSIS

The soil-structure interaction response of the wall was investigated for the instrumentation data on December 20, 2018, for the pile at the middle of the wall using the "integration method" in structural analysis. This method is based on force and moment equilibrium of a differential segment of a beam. From this, shear forces, bending moments, gradient of deflections (i.e., slopes) and deflections can be calculated if the loads acting on the beam and its bending stiffness (EI) are known. The integration method was coded in Mathcad and the results obtained were validated with the commercial structural software S-FRAME. The results of the analysis are shown in Figure 7.

The subroutine coded in Mathcad calculated the pile shear forces, bending moments, slopes, and deflections by inputting the measured anchor loads, and an initial estimated/trial set of earth pressures, based on lateral earth pressure theory. The goal was that after several sets of trial earth pressures, the calculated deflections start to converge towards the measured pile deflections. The estimated earth pressures obtained from this method are the resultant forces on the wall at each interval, i.e., the difference between the mobilized active and passive forces.

The average bending stiffness (EI) that was used for the analysis was 1,736 MN/m^2, which corresponds to the EI of the cracked pile.

Since deflection convergence was not achieved after several trials, the two lower pile partial hinges suggested by the strain gauge data in Figure 6 were included in the Mathcad subroutine as additional boundary conditions to achieve convergence. This addition was the key to match the calculated deflections with the measured deflections as shown by the Mathcad and S- FRAME results in the plot of deflections in Figure 7. If hinges are not included, the calculated deflections (and thus, the associated bending moments and shear forces) could be quite different as shown by the S- FRAME results in the plot of deflections in Figure 7.

The hinges used in the analysis can be seen in the plot of slopes in Figure 7. The partial hinge in the vicinity of the lower anchor was not included in the analysis as its contribution did not substantially impact the results.

The maximum measured deflection for the pile at the middle of the wall was about 30 mm. This value appears to be high when compared to a typical maximum wall deflection of 25 mm. However, when comparing the measured anchors loads and the back calculated internal loads in the piles (bending moments and shear forces) with the actual anchor/pile capacity, it appears that the pile structural future performance is guaranteed.

The bending moment calculated from the strain gauge data on December 20, 2018, corresponding to the lower strain spike of 354 kN.m plotted in Figure 6, shows an excellent agreement with the back calculated bending moment curves using hinges from Mathcad and S-FRAME. The bending moment of 354 kN.m is substantially lower than the 2094 kN.m bending capacity of the pile. It should also be noted that the value of the lower strain spike (tension) was about 600 micro strains which is well below the yielding strain of the steel which is 2000 micro strains. This suggests that this hinge is an elastic and not a plastic hinge.

PYWall software was used to estimate the design pile shear forces, bending moments and deflections for a total exposed wall of 10 m. The analysis was conducted for a trapezoidal earth pressure distribution calculated using a load 1.3 times the active load. The design curves are shown in Figure 7 and are greater than the curves for December 20, 2018, since at this date the ground surface in front of the wall was only 6 m below the top of the wall. It should be noted that at the time this paper was submitted, the ground surface in front of the wall is unchanged.

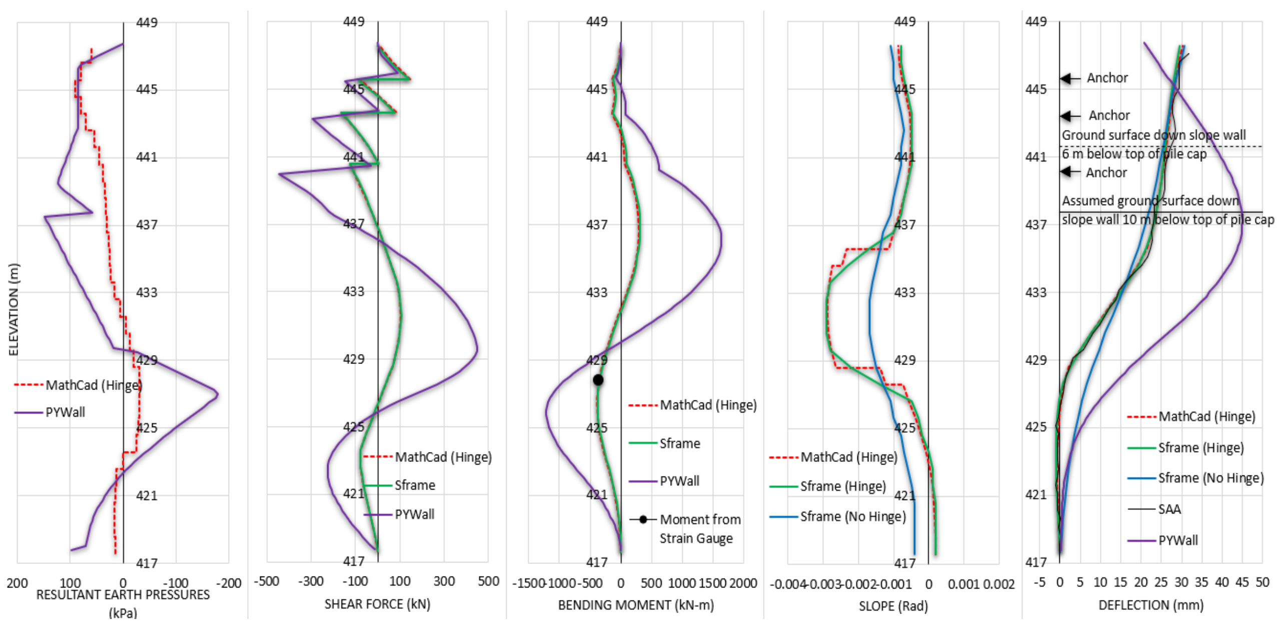

Figure 7. Results of the soil-structure interaction analysis using various methods of analyses for pile at the middle of the wall on December 20, 2018: from left to right, resultant earth pressures, shear forces, bending moments, slopes and deflections.

6 CONCLUSIONS

A site, located on Highway 986:01 north of the Town of Peace River, underwent extensive landslide mitigation work from 2017 to 2018 as tension cracks and drop of pavement developed on the sidehill road embankment. The remediation option consisted of a tangent cast-in-place pile wall supported by three rows of anchors. A geotechnical instrumentation program was implemented to monitor the global stability of the road embankment and the structural behavior of the wall during and after construction. The instrumentation on the wall included SAAs, load cells and strain gauges.

The measured deflections, anchor loads, and strains from the pile located at the middle of the wall were used to estimate the earth pressures, shear forces and bending moments acting on the pile. The analyses were carried out using a soil-structure interaction model developed inhouse coded in Mathcad. Convergence was obtained after measured strain spikes in the pile were modeled as partial hinges in Mathcad. The results of this model were validated by another structural model built in S- FRAME.

These results show that the measurement of concrete pile deflections is not sufficient to fully characterize its structural behavior; but the estimation of shear forces and bending moments are also required.

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