Geotechnical instrumentation of a tangent cast-in-place pile wall in Peace River, Alberta

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ABSTRACT
This paper presents the results of a geotechnical instrumentation program for a tangent cast-in-place pile wall located on a sidehill road embankment on Highway 986:01, near Peace River, Alberta. The wall was constructed in 2016 to prevent further retrogressive failure of the south slope of the embankment. The pile wall is 142 m long and included three rows of ground anchors. The instrumentation program included in-place Shape Accel Array inclinometers, inclinometers casings, load cells and vibrating wire piezometers. Measured wall deflections and anchor loads were used to back calculate earth pressures imposed on the wall and pile bending moments during and after construction. The back calculation was carried out using a soil-structure interaction analysis implemented in Mathcad and validated in S-Frame. Additional analyses were completed using the finite-element software SIGMA/W.

RÉSUMÉ
Cet article présente les résultats du programme d'instrumentation géotechnique d'un mur de pieux tangents en béton coulé sur place situé sur l'autoroute 986 :01 près de Peace River, Alberta. Le mur, complété en 2016, a été construit pour prévenir la progression du mouvement de pente du remblai routier. Mesurant 142 m de long, il a nécessité l'installation de trois rangées d'ancrages. Le programme d'instrumentation inclut des inclinomètres de type Shape Accel Arrays, des cellules de charge, des inclinomètres conventionnels et des piézomètres. Les déflexions et les charges mesurées au mur ont été utilisées pour le rétrocalcul de la poussée des terres et des moments de flexion sur les pieux pendant et après construction. Une analyse de l'interaction entre le sol et la structure a été implémentée dans Mathcad et validée dans SFrame. Des analyses supplémentaires ont été complétées à l'aide du logiciel d'éléments finis SIGMA/W.

1 INTRODUCTION

On June 27, 2012, the south slope of the sidehill road embankment on Highway 986:01, 3.3 km west from the Weberville Bridge, near Peace River, failed (Figure 1). The failure resulted on a sudden drop in the pavement on the east-bound lane of about 100 mm deep and 33 m long. The embankment was about 21.5 m high with side-slopes in the order of 3H:1V. A 900 mm CSP culvert traversed under the embankment.

By June 25, 2013, the damage had affected 89 m of the east-bound lane, cracks were starting to develop further north into the west-bound lane and some bulging and backscarps were observed on the south slope of the embankment. On September 4, 2013, the pavement on east-bound lane had dropped a total of about 1500 mm. As a preliminary measure, an emergency temporary highway detour was constructed to the north of the slide area between October and November of 2013. The detour was 340 m long.

Thurber Engineering Ltd. (Thurber) was retained by Alberta Transportation (AT) and proposed several options to remediate the embankment. The selected alternative consisted of realigning the highway to the north and protecting it from future slope retrogressions with a cast-in-place tangent pile wall supported with ground anchors. It was realized that the remaining embankment slope south of the wall would continue to slide, and the design was required to consider this potential movement.

The final design was in the tendering process when the slope damage retrogressed further north on July 23, 2015, affecting the existing detour built in 2013. The drop in the pavement had grown to about 6.5 m deep and 130 m long. Bulging and backscarps were observed throughout the embankment south slope (Figure 2).

Figure 1. Drop in the pavement on the east-bound lane of the embankment on Highway 986:01 in June 2012

AT decided to cancel the ongoing tendering process and to redesign and retender to account for the new slide retrogression. The redesign included the construction of a second emergency detour further north of the initial detour. The construction of the redesign began on January 14, 2016. The cast-in-place piles were completed between January 29 and March 10, 2016. The ground anchors were installed between April 8 and June 26, 2016. The highway realignment was finished on July 18, 2017.

Figure 2. Further retrogressed south slope in July 2015, showing a 6.5 m drop in the pavement.

2 GEOTECHNICAL CONDITIONS

Thurber carried out a field geotechnical drilling investigation in July 2013. Four test holes (TH13-1 to TH13-4) were advanced to depths ranging between 15 m and 30 m below the embankment surface. Standard Penetration Tests (SPT) were carried out and grab samples were taken from auger flights for laboratory testing analyses. The testhole locations are shown on Figure 3. Three of the test holes (TH13-2, TH13-3 and TH13-4) were completed with slope inclinometer (SI) casings. Two pneumatic piezometers were attached to each SI at depths selected based on observed soil and seepage conditions during drilling. TH13-1 was completed with a PVC standpipe. The laboratory testing program included visual soil classification, determination of water content, Atterberg limits and grain-size distribution on selected samples.

The subsurface conditions encountered in the test holes generally consisted of clay fill, overlying colluvial clay, overlying glaciolacustrine clay. The clays were high plastic. A thin sand layer was found in TH13-2 within the colluvium. The glaciolacustrine clay appeared to be over consolidated and it may have been subjected to preshearing during quaternary glaciation. A review of the geological conditions at the site indicates that bedrock may be present about 95 m below the embankment road. The Peace River is approximately 750 m south.

The SIs were substantially sheared just after installation. The existing CSP culvert traversing under the embankment was sheared at about 69 m from the culvert outlet. This was consistent with the location of the slip surface interpreted from the SIs (Figure 3).

The groundwater table interpreted from the pneumatic piezometers is plotted on Figure 3. It shows that the groundwater table was locally high in the vicinity of the culvert breakage.

3 SLOPE FAILURE MECHANISM

The SPT data suggests that the road embankment is sitting on colluvial clay, and that the embankment fill and the colluvium are softer compared to the stiffer (SPT blow count ≥ 15) and deeper glaciolacustrine clay. Based on the SI information and culvert shear locations, the failure surface appears to be located mainly at the interface between the softer and stiffer ground.

Assessment of the bare earth LiDAR data available for the site (prior to embankment failure, Figure 4) suggests that the road embankment may have been constructed on an area where ancient landslide retrogression has occurred. This is noted by the ancient slope backscarps located north and south of the embankment. A large deep-seated ancient slope failure is also apparent just west of the site as shown on the LiDAR image. However, this ancient failure does not represent a current potential risk to the road embankment.

Note that active deep-seated failure surfaces within the glaciolacustrine clay due to potential preexisting sheared zones were not confirmed by the geotechnical investigation, but their existences were not ruled out.
Another important point to highlight is that the embankment slope failure appears to have been accelerated by the ingress of water from the culvert breakage.

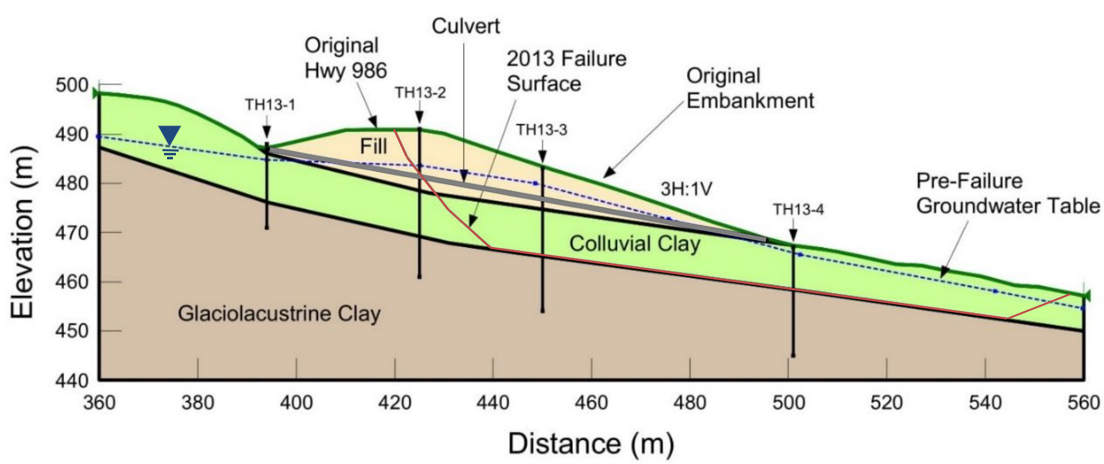

Figure 3. Test hole locations and subsurface conditions prior to the September 2013 event

4 SLOPE FAILURE BACK-ANALYSIS

A slope stability back-analysis was carried out to estimate pre-failure strength parameters of the clay fill and colluvial clay. The analysis used the ground conditions and groundwater levels measured during the field investigation prior to the September 2013 event (Figure 3), as well as knowledge of the failed slope configuration. Because the actual slip surface did not involve the glaciolacustrine clays, the strength parameters of this material were estimated primarily from correlations with laboratory index properties and from experience. A summary of the main design soil parameters is presented on Table 1.

Table 1. Design soil parameters

Material Type	Unit Weight (kN/m^3)	Eff. Friction Angle (°)	Effective Cohesion (kPa)
Existing fill	18	23	5
Colluvial clay	18	18	5
Glaciolacustrine clay	20	20	10

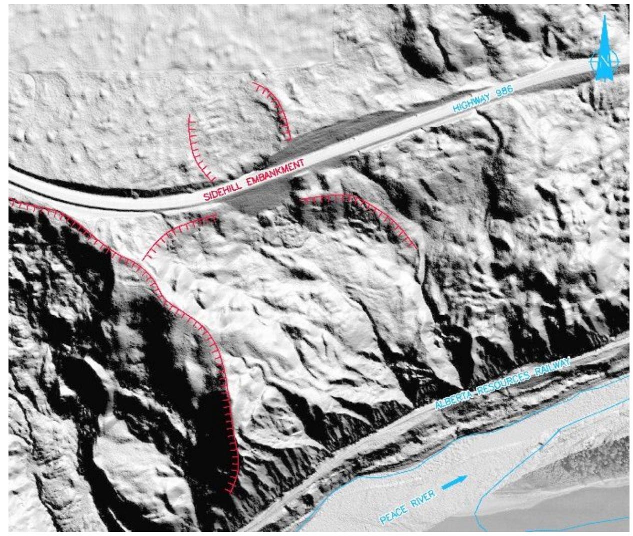

Figure 4. Bare earth LiDAR data prior to the sidehill embankment failure including location of ancient slope backscarps

5 DESIGN AND CONSTRUCTION CONSIDERATIONS

The final realigned highway configuration was about 15 m north from its original location to account for the several recent landslide retrogressions affecting the embankment south slope (Figure 4). A second temporary detour was required north of the initial one to allow for through traffic during construction.

To protect the new highway alignment from future slide retrogressions, a pile wall supported by three rows of ground anchors was built along the south edge of the new highway. The design section is presented in Figure 5. The pile wall is 142 m long and includes a concrete pile cap 2.1 m tall and 118 tangent cast-in-place concrete piles. The piles are 27.8 m in length and 1.2 m in diameter.

The wall was supported by a total of 344 ground anchors installed at 1 m, 5 m and 9 m below the top of the wall. The upper row of anchors was installed through the pile cap. The lower two rows were installed through the piles. The anchors are 32 mm double corrosion protected DYWIDAG anchors. The anchors range from 27 m to 37 m long. The bond length is 12 m and is within the glaciolacustrine clay. The anchors are inclined between 27 and 30 degrees from the horizontal.

The anchor design load (DL) was 230 kN. The ability to meet DL requirements was confirmed in the field by two pre-production anchor tests (one on each side of the pile wall). These tests sustained a load twice the DL (i.e., 460 kN). The target factor of safety (FOS) for slope stabilization was FOS = 1.3.

Vertical drains 500 mm in diameter and 6 m long were installed just upslope of the wall every second pile to reduce the water pressure on the wall. The existing CSP culvert was filled with cement grout.

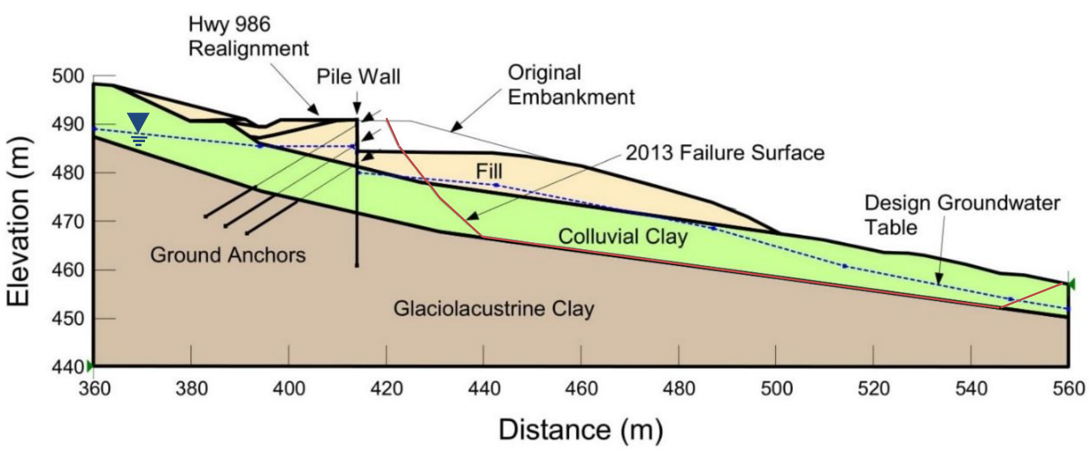

Figure 5. Design section showing the pile wall, ground anchors, and realigned Highway 986:01

The area immediately south of the pile wall was initially excavated to 10.5 m (Figure 6) to allow the installation of the lower row of anchors. Then, this area was backfilled to just below the middle row of ground anchors (Figure 7). After excavation and backfilling, the south slope was regraded down to the toe of the embankment to provide positive drainage away from the wall. The backscarps from previous slope retrogressions were also backfilled.

A few months after construction, the lower portion of the regraded slope, which was about 30 m south of the wall, started to move. However, current readings suggest that the rate of movement is decreasing. This slope movement was anticipated in design by disregarding the passive contribution of the upper 4.5 m of the regraded south slope.

Figure 6. Initial excavation of the area immediately south of the pile wall to a depth of 10.5 m below the pile cap to allow for anchor installation.

Figure 7. Pile wall completed and south slope of the embankment backfilled to final grade just below the middle row of ground anchors.

6 INSTRUMENTATION

6.1 General

Several instruments were installed during construction within the pile wall and the embankment south slope.

An in-place 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 instrumented piles. Vibrating wire piezometers (VWPs) were installed just upslope these piles. The instrument cables were carried to a central datalogger enclosure.

The SAAs were used to monitor the magnitude and rate of the wall deflections. The load cells were employed to compare actual anchor loads to the preproduction testing loads. The VWPs behind the wall were used to estimate the actual water load on the wall.

A set of VWPs and SIs were installed at 6 m, 35 m and 80 m downslope the pile wall. The SIs reached a depth well below the tip of the wall piles. These instruments were installed mainly to measure groundwater pressures and ground displacements in order to assess global stability and particularly, to monitor the location of the slip plane that is still active south of the pile wall.

6.2 Instrumentation Results

Figure 8 shows the instrumentation readings for the pile located at the middle of the wall. The readings span from the installation period (between March and July 2016) until March 8, 2019. The instrumented piles on the east, and west portions of the wall display similar behavior.

The vertical plot on the left-hand side presents the pile deflections on July 17, 2016 (just shortly after wall construction and backfilling of the south slope), November 30, 2018, and March 8, 2019. This plot reveals that deflections mainly occurred during construction and that the maximum pile deflection to date is about 13 mm. The maximum deflection is regarded as small for this type of wall. The left-hand side plot also shows the groundwater head elevation versus time from the three VWPs installed behind the pile. The VWP readings suggest that there may be a shallow and deeper groundwater regimes. The shallow regime is shown by the response of the upper two VWPs. The deeper regime behavior is captured by the response of the lowest VWP. The shallow regime appears to be impacting mainly the road embankment and appears to respond rapidly to spring snowmelt events such as the ones that occurred in May 2017 and 2018.

The first horizontal plot on the right-hand side reports daily air temperatures over time from Environment Canada. The remaining horizontal plots present the anchor loads and cumulative pile deflections over time at each anchor elevation.

The responses of the upper two anchor load cells suggest a gradual load increase during the winter followed by a rapid load decrease as the air temperature rises above zero degree Celsius in early spring. Note that these loads increased again shortly after due to the rapid rise of the shallow groundwater associated with spring snowmelt. The response of the lowest load cell suggests that the anchor load at this elevation is neither sensitive to winter nor spring snowmelt. This may be because the lowest row of anchors is covered by 4.5 m of fill, thus protecting them from temperature changes. Also, spring snowmelt was found to impact only the shallow groundwater which in turn only acts on the upper two rows of anchors.

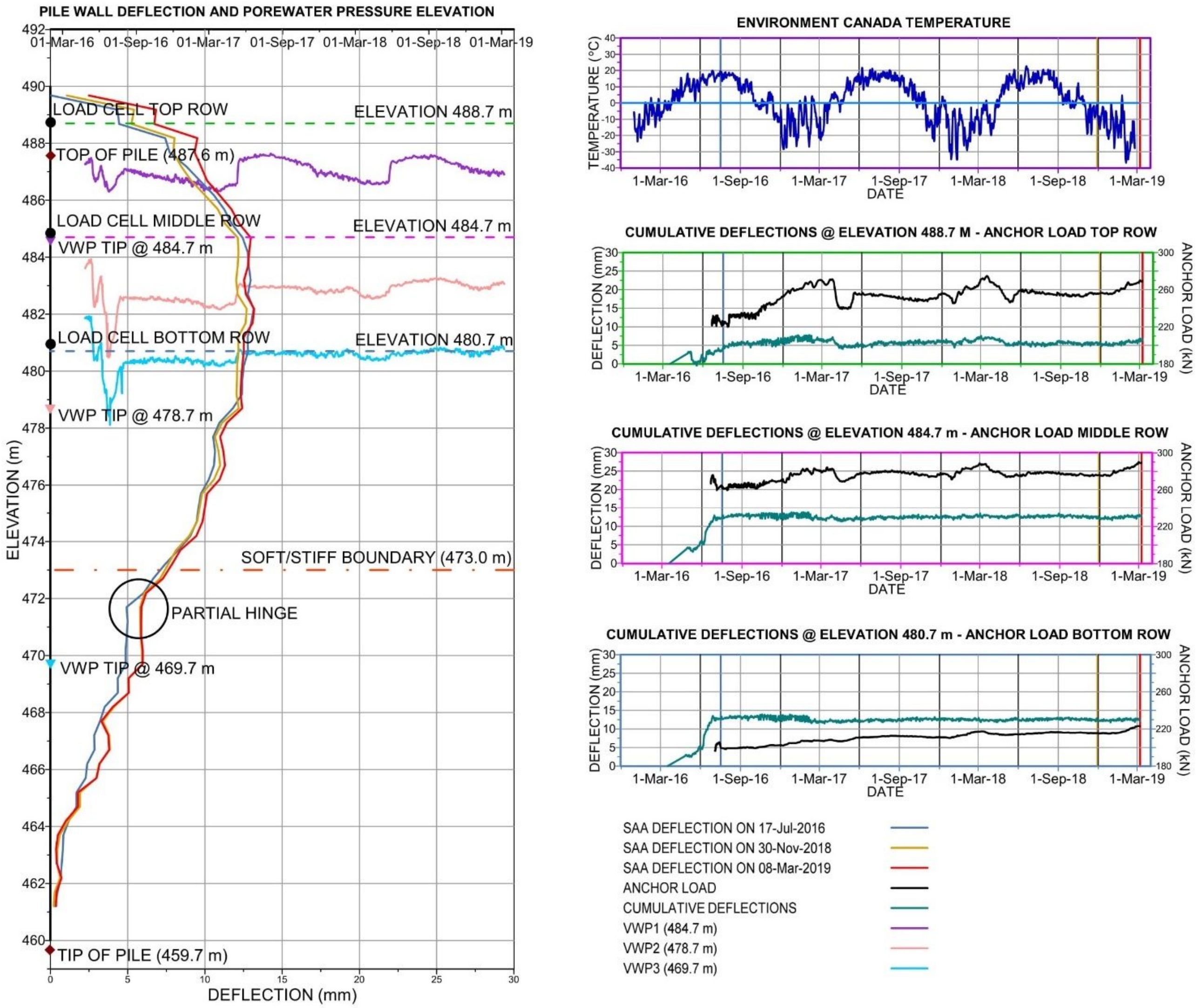

Figure 8. Instrumentation results. Left-hand side plot: pile deflections at different selected dates, VWP tip locations, and water head elevations with time. Right-hand side plots: temperature, deflections at anchor head locations and anchor loads with time.

The maximum anchor load recorded was about 290 kN which is well below preproduction test results (460 kN). Furthermore, since the bond length of the anchors is well within the glaciolacustrine clay (therefore, controlled by the deeper groundwater regime), the capacity of the anchors is neither very sensitive to winter nor spring snowmelt.

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 load fluctuations are considered small compared to the total load they are subjected to, and well below ultimate capacity.

The pile deflections at the anchor elevations (as shown on Figure 8, horizontal plots on the right-hand side) also reveal that these are not very sensitive to winter or spring snowmelt.

The SIs at 35 m and 80 m south of the wall sheared a few months after installation at a depth consistent with the retrogressing slip surface identified during the initial geotechnical investigation. This also correlates to the movement of the lower portion of the embankment slope that was observed to be moving about 30 m south of the wall. It is expected that this movement retrogresses over time towards the pile wall. However, as mentioned in the previous section, this retrogression was anticipated in design by disregarding the passive contribution of the upper 4.5 m of the regraded south slope.

The SI located 6 m south from the pile wall is still functional and suggests that no new slip planes are developing underneath the pile wall.

7 SOIL-STRUCTURE INTERACTION ANALYSIS

The instrumentation results discussed above provide an important understanding of wall performance and global slope stability. However, they do not give assurance whether the piles are structurally sound, or their structural capacity is being reached. To get an insight into the structural behavior of the piles, soil-structure interaction analyses were carried out using various numerical tools: Mathcad, S-Frame and SIGMA/W.

Mathcad was selected because it allows users to develop and implement subroutines (or codes) directly into the software (Mathcad, 2013). Unlike a blackbox computer software, subroutines in Mathcad can be formulated and tweaked by the user to solve simple engineering problems, like the interaction between a single pile and the surrounding soil.

The subroutine developed in Mathcad for the project was based on the integration method in structural analysis, in which the internal forces acting on the pile can be determined by integrating the differential relationships (i.e., equations) that exist between lateral loads, shear forces, bending moments, deflections and gradient of deflections (i.e., slopes). These relationships are based on differential force and moment equilibrium and are associated with the bending stiffness (EI) of the pile. The method assumes that the constants of integration (boundary conditions) are known.

More specifically, the methodology consists of dividing the pile into one-meter length intervals, and inputting the known boundary conditions, like the measured anchor loads. It then integrates sequentially over the one-meter intervals from bottom to top, like a Winkler type of analysis. An initial estimated/trial set of lateral loads (i.e., earth pressures) are assumed, and pile deflections are calculated and compared to the field measured values. Several iterations are required until a specific set of earth pressures are achieved such that field and calculated pile deflections matched closely, within a specified tolerance. The estimated earth pressures obtained from this method are actually 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 pile in the analysis was 1,500 MN/m^2. Figure 9 presents the results of the Mathcad analysis in terms of resultant earth pressures, shear forces, bending moments, slopes and lateral deflections. Figure 9e, in particular, compares the Mathcad deflections with the measured deflections as of July 17, 2016. An excellent agreement is obtained between these two. A partial hinge had to be added between two one-meter segments at elevation 472 m in order to better match the measured deflections. Evidence for the partial hinge can be found in the measured pile deflections reported in Figure 8 (note circled area on the left-hand side plot, at elevation 472 m). This partial hinge is most likely the result of internal stresses being transferred from the concrete to the reinforcing steel as the pile cracked in response to deflections. Other less significant hinges developed in the upper portion of the piles, likely induced by the anchor forces. These hinges were also added to the Mathcad model to improve the matching between measured and calculated deflections. They can be seen in the slope plot (Figure 9d) as discontinuities in the curve.

The back calculated shear forces and bending moments in Figure 9b and c, respectively, are lower than the shear resistance of 617 kN and bending moment resistance of 1395 kN·m and give an assurance that the pile wall is structurally sound for the design loads.

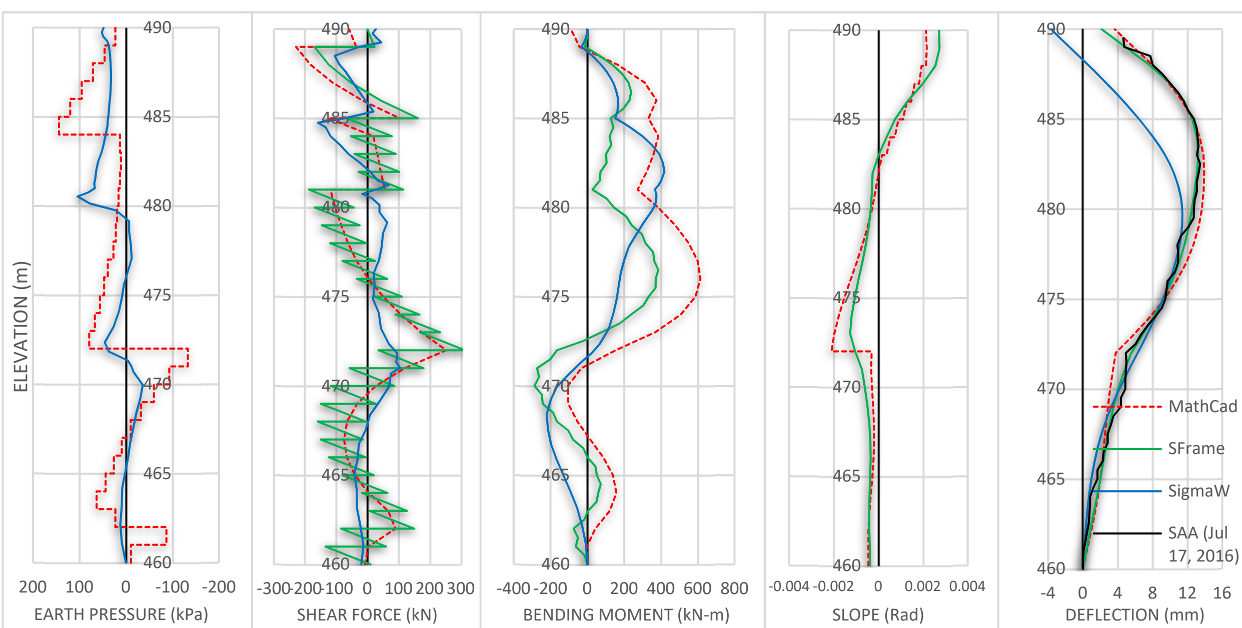

Figure 9. Results of the soil-structure interaction analysis using various methods of analyses: (a) resultant earth pressures, (b) shear forces, (c) bending moments, (d) slopes and (e) deflections along the pile.

To improve the confidence in the results presented above and to validate the subroutine implemented in Mathcad, a structural model was also built in S-Frame (v2017.2.3), a commercial structural engineering program developed by the Canadian-based company S-Frame Software Inc. S-Frame is a very powerful software generally used for the analysis of complex structures (e.g. skyscrapers, trusses, arches, roofs, silos) and loading conditions (S-Frame, 2018). However, this software can also be used to analyze a single pile-soil interaction problem. In this case, the pile was modelled as a beam element and the soil as springs. The software also allows for (total or partial) plastic hinges to be added to the structural element.

For the pile wall case analyzed here, the same loading and pile bending stiffness inputted in Mathcad were used in S-Frame. Partial hinges were also added to the pile element. The results of this analysis are presented in Figure 9, in which a direct comparison between Mathcad and S-Frame can be made. In general, the results of the two programs are very similar and consistent.

To gain further insight into the mechanics of the pile wall-soil interaction problem, and to assess the influence of pore pressures, groundwater flow and time on the response of the pile-soil system, additional analyses were conducted using the finite-element based-program SIGMA/W (v2007), developed by the Canadian-based company GeoSlope International Ltd. The numerical model used a geometry similar to that shown in Figure 5, and considered the various stages of construction, as follows:

1) Pile installation, and soil excavation from elevation 490 m (top of pile) to 489 m.
2) Installation of the first row of ground anchors at elevation 489 m.
3) Excavation to elevation 484.5 m.
4) Installation of the second row of ground anchors at elevation 485 m.
5) Excavation to elevation 480.5 m
6) Installation of the third row of ground anchors at elevation 481 m.
7) Backfill to elevation 484.5 m.

Figure 10 shows a closeout of the model geometry at the final stage of construction, after end of backfilling. The mesh, which comprises quads and triangular elements with secondary nodes, is refined at and near the embankment and anchor areas to improve calculations.

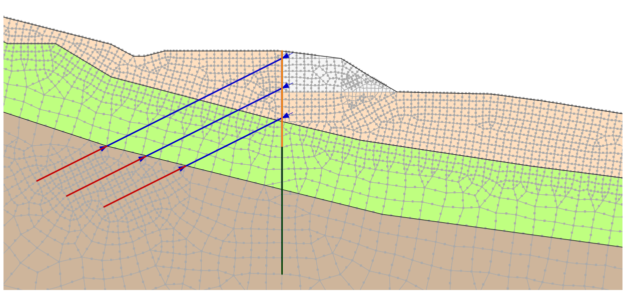

Figure 10. Closeout of the model geometry at the final stage of construction, as implemented in SIGMA/W.

The SIGMA/W model was setup as a coupled stress-pore water pressure (PWP) problem, and thus accounted for the transient nature of stresses, deformations and pore pressures (GeoSlope, 2010). The pile was modelled as a beam element with interface elements to represent the contact between pile and soil. The anchors were modelled as bar elements. The soil layers were modelled as elastic-plastic Mohr-Coulomb materials with the effective strength parameters presented on Table 1.

The results of the finite-element analysis are presented in Figure 9. While pile deflections are relatively comparable among the three programs, shear forces and bending moments are not. On average, the internal forces estimated from SIGMA/W are [higher] [lower] than those calculated from Mathcad and S-Frame. Note that the addition of plastic hinges in beam elements is not supported by SIGMA/W. An attempt was made to introduce weaker (less stiff) sections in the pile element; however, this strategy did not improve the results noticeably.

8 CONCLUSIONS

A sidehill road embankment on Highway 986:01, near Peace River, Alberta, failed and disrupted an important transportation corridor. The selected remediation option consisted of a tangent cast-in-place pile wall with three rows of ground anchors. A geotechnical instrumentation program was developed to monitor the pile wall deflections, anchor loads, as well as movement and pore pressures in the surrounding soils during and after construction. The instruments installed on site included in-place SAAs, SIs, load cells and VWPs. The instrumentation program proved to be a very useful management and decision-making tool, particularly during construction. Thurber assessed and interpreted the instrumentation data on a regular basis, and kept the contractor informed of the pile wall response throughout the stages of construction, as the various components of the wall were being installed. Field data is still being recorded to-date to assess the long-term performance of the wall and to monitor movements of an ancient landslide affecting the global stability of this section of Highway 986:01.

The instrumentation data also proved very useful to evaluate the design during and after construction, and to give an assurance of the structural soundness of the piles. The measured wall deflections and anchor loads were used as input in numerical models to estimate the earth pressures, shear forces and bending moments acting on the wall. Initial analyses were carried out using a soil-structure interaction model developed inhouse and implemented in Mathcad. The results of this model were compared (and validated) by the results of another structural model built in S-Frame. Additional analyses were completed using the finite-element software SIGMA/W to gain further insight into the mechanics of the pile wall-soil interaction problem, and to assess the influence of pore pressures, groundwater flow and time on the response of the pile-soil system. The modelling exercise also showed that it is important to account for the existence of plastic hinges in the pile elements. The addition of hinges not only helped better match the measured pile deflections, but also improve the interpretation of shear forces and bending moments.

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