Eureka River Landslide, Hwy 726:02 Near Worsley, Alberta: Design and Construction of Remedial Measures

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Challenges from North to South
Des défis du Nord au Sud

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ABSTRACT
A number of landslides are affecting Highway 726 through the Eureka River valley in northwest Alberta, Canada. One of these sites required immediate attention. The site investigation revealed a deep-seated landslide in a weak, high plastic clay layer that toed out into the River 15 m below the highway. Remediation in 2012/2013 consisted of two lines of concrete piles constructed downslope of the highway spanning the landslide. The upper pile wall was tied back with two rows of grouted anchors. The lower, cantilever design pile wall was installed which acts to maintain lateral soil pressure against the upper wall. Other measures, such as EPS lightweight fill, were implemented in order to further reduce driving forces on the upper wall. This paper summarizes the geotechnical investigation, design, and repair measures, including instrumentation monitoring results at the remediated site.

RÉSUMÉ
L'autoroute 726 traversant la vallée de la rivière Eureka est affectée par plusieurs glissements de terrain. Une étude entreprise sur un site sensible a mis en évidence un glissement de terrain profond situé dans une couche d'argile molle à plasticité élevée, terminant dans l'emprise du cours d'eau à une élévation d'environ 15 m en dessous du niveau de la chaussée. En 2012/2013, deux murs de soutènement (pieux tangents en béton), furent construits en aval de la chaussée. Deux niveaux d'ancrages furent ajoutés au premier mur (supérieur). Le second mur (inférieur), en porte-à-faux, fut construit plus bas le long du talus afin de maintenir une pression latérale sur le mur supérieur. Des mesures additionnelles de protection, dont un remblai léger composé de blocs de polystyrène expansé, furent aussi adoptées afin de réduire les forces de poussée exercées sur le mur supérieur. Cet article présente les éléments pertinents de la reconnaissance et de la conception des techniques de mitigation, ainsi que les données de l'instrumentation du site.

1 INTRODUCTION

Highway 726:02 is a paved two-lane highway located about 700 km northwest of Edmonton, Alberta. The highway is aligned in a north-south direction, and crosses the Eureka River about 7 km south of Worsley. The Eureka River Valley at this location has a maximum depth of 40 m. In 1988, the highway was upgraded and shifted to its current alignment, and a new arch culvert crossing was constructed. The highway alignment is skewed to the river, and has a slight sidehill arrangement with fills near the river and cuts further uphill.

2 HISTORICAL DEVELOPMENT OF LANDSLIDES

A shaded relief plan showing approximate locations of the landslide sites at this highway location is presented in Figure 1. Since construction in 1988, six landslides have affected the highway to varying extents along the river crossing slopes.

North of the river, slope movements were first observed in 2001 as minor pavement cracks about 100 m north of the crossing (Site 3), outside of a river meander. Conditions steadily worsened over the years, and a 110 m long scarp crack extended along the highway. Site 3 occurs where the highway is about 15 m above river level. Since 2007, two more landslide sites have developed about 200 m and 350 m north of the river.

Due to the number of landslides and expected high highway repair costs, a functional planning study was undertaken in 2010/2011 to determine if an alternate alignment was feasible to avoid the landslide issues. The study was predominantly based on LiDAR data, and estimated a cost of $19 million (2011 dollars) to undertake the most feasible realignment option. The timeline for implementation of a new alignment was expected to be many years. Due to the severity of the Site 3 landslide, it was considered prudent to repair it, and authorization was granted in 2011 to remediate it.

3 GENERAL DESCRIPTION OF STUDY AREA

3.1 Surface Conditions

As shown on Figure 1, the highway alignment traverses the Eureka River over a 108 m long by 11 m wide by 6 m high concrete arch culvert. About 100 m downstream of the culvert outlet, the river makes a 90 degree turn towards the highway, before resuming its northwest alignment.

Available aerial photographs from 1951, 1978, and 1983, and LiDAR Digital Elevation Models were used to describe the surface expressions and progression of the landslide developments within the site area. The uplands region is fairly flat with some rolling areas south of the valley crossing. Slope instability is pervasive within all the stream valleys in the area. Indications of extensive historic landslide activity are common in the vicinity of the existing crossing. Landslides in the area are indicated by hummocky terrain, the presence of multiple steep scarps, extensive gullies within the slope, irregularity in the shape and texture of the slope surface, tilting trees, and significant toe bulging. The early photos indicate that there were some areas that were not well drained in the area of the existing crossing (which probably contained organics/peat), but that with time they drained and showed associated evidence of erosion. Vegetation consists predominantly of heavy shrubs and trees from the river's edge to the uplands, except for grassed areas existing along the ditches and immediately downslope of the highway which had been previously cleared of vegetation.

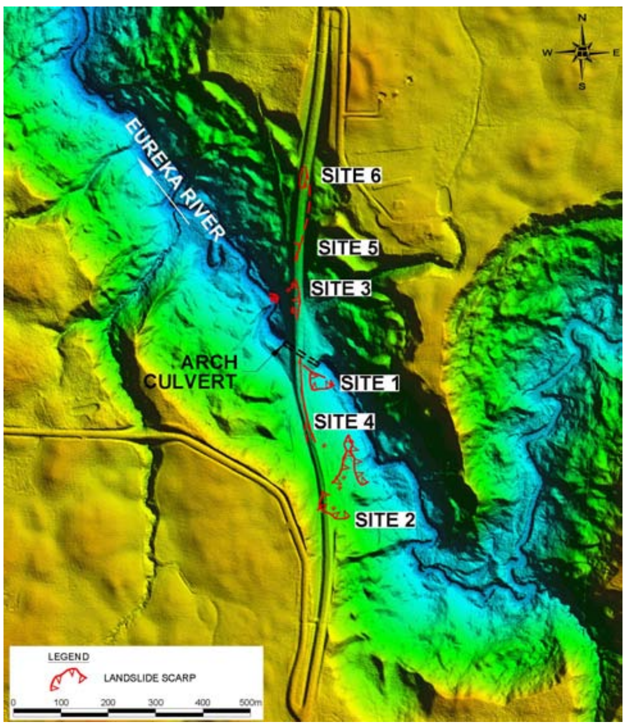

Figure 1. Shaded relief plan showing landslide locations

3.2 Geology

A brief discussion of the area geological setting was prepared based on published geological reports.

The Bedrock Topography of Alberta Map (Pawlowicz and Fenton, 1995) indicates the Eureka River at this general site location is located in a valley north of the Shaftesbury Channel Thalweg, and is not located within a pre-glacial valley. There is also less than 15 m of drift overlying bedrock in this area. The bedrock (EUB, 1999) is Upper Cretaceous Dunvegan formation deposits composed of fine-grained feldsparthic sandstone with hard calcareous beds, laminated siltstone, and grey silty shale, which are deltaic to marine in origin. The surficial geology of the area (Atkinson and Paulen, 2009) is indicated to consist of poor to well sorted deposits of lacustrine clay/silt/sand, overlying glacial till, with mixed glacial/bedrock slump/colluvium located mainly along the flanks of the valleys.

4 SITE 3 LANDSLIDE AND REMEDIAL MEASURES

4.1 Landslide Description

At Slide Site 3, the highway is about 15 m higher than the river on an embankment inclined at between 3.5H:1V to 5.5H:1V. The valley slope above the highway is 30 m high and inclined between 7H:1V to 9H:1V. Figure 2 shows the Slide Site 3 area details.

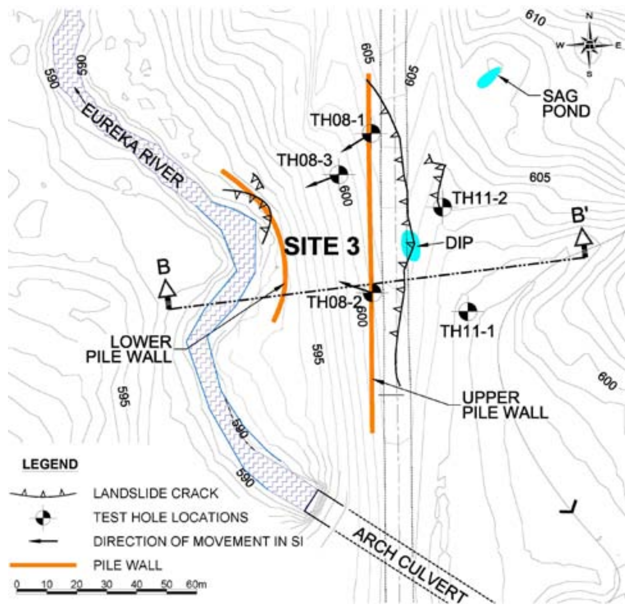

Figure 2. Slide Site 3 Layout

The site was monitored through Alberta Transportation's (TRANS) Geo-hazard Risk Management Program (GMRP) between 2001 to 2011, and the landslide features included steadily degrading conditions. The landslide headscarp originated on the east (upslope) highway shoulder, where it had created water ponding in a 500 mm deep dip/settlement, and then extended diagonally across the highway in both directions, affecting a 100 m length of pavement. The landslide toed out in the river, a lateral distance of about 60 m from the highway. Observations of pavement distress were noted to be more severe north of the dip, containing an open crack up to 300 mm wide and 400 mm deep, having a 75 mm vertical differential drop. Other slide features included multiple cracks on the paved highway surface extending further south of the main scarp crack, and a few slide blocks located downslope (west) of the highway above the river. A secondary headscarp was also noted in the east ditch, as well as steep river banks and toppled riverside blocks.

Repairs at this site were initially limited to infrequent pavement patching and crack sealing. Between 2009 to 2011, the scarp crack began to re-appear through the fresh patches within weeks of placement. Using the probability and consequence scale of geo-hazard risk associated with the GRMP this landslide site was assigned a risk level rating of 84 in 2011, indicative of the urgent need to implement remedial measures to avoid potential closure of the highway. By comparison, the risk level rating was 65 in 2007.

4.2 Subsurface Conditions

In 2008, three test holes, complete with slope inclinometers and dual pneumatic piezometers, were drilled at this site along the downslope side of the highway. In 2011, two test holes complete with dual vibrating wire piezometers in each were drilled east (upslope) of the highway.

The geotechnical investigations indicated that the sub-surface conditions generally consisted of up to 6 m of highly plastic clay fill, overlying native highly plastic clay that extended to depths between 9 to 18 m below surface. The consistency of the native clay varied from firm to very stiff. Very stiff to hard clay till was encountered underlying the clay, and extended to depths of at least 20 to 25 m below surface. The interface between the lacustrine clay and glacial clay till corresponds to about elevations 586 m to 589 m. The clay till surface sloped downwards from east to west towards the river, and from south to north along the highway. The ground water levels within the landslide area were 7 to 12 m below ground surface. Figure 3 shows a simplified stratigraphic cross-section of the slope through the landslide area at cross-section B-B'.

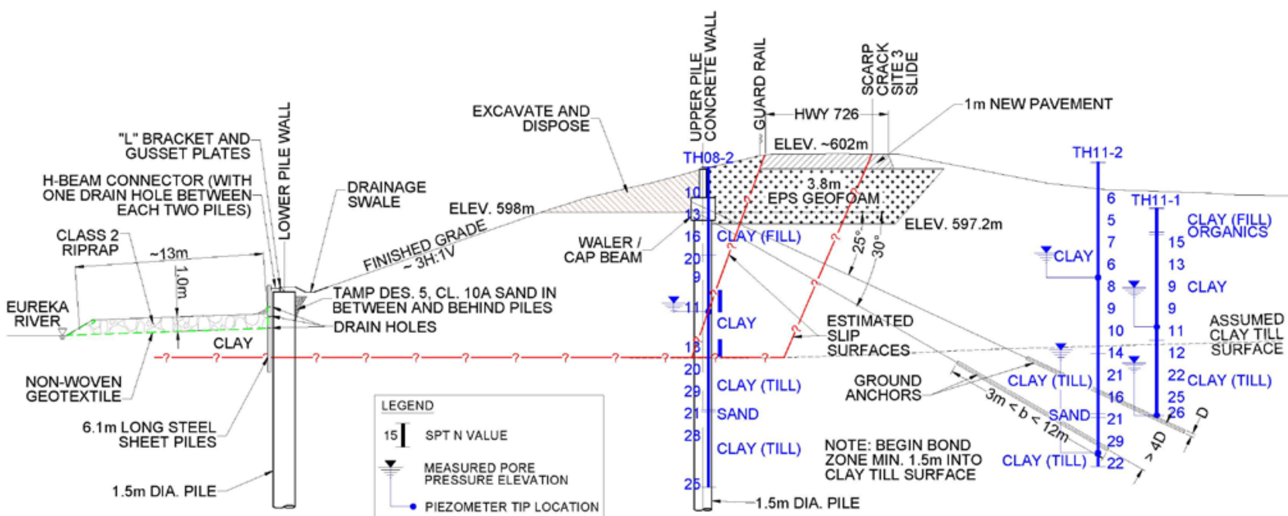

Figure 3. Stratigraphic Cross Section B-B' Perpendicular to Highway

4.3 Landslide Mechanism

The slope inclinometers (SI) showed that there were two distinct slip surfaces present at each SI location, with the lower one about 3 m below the upper slip surface, and located about 8 to 13 m below the highway surface (varying from elevations 589 m to 597 m). The slip surfaces were located entirely within the high plastic lacustrine clay formation, and extending as deep as, and moving along the surface of the clay till near the center of the landslide. This indicated that the clay/clay till interface appeared to form the base of the landslide slip surface, which was later considered an important design feature. The orientation of the resultant in all three inclinometers indicated that movement was directed towards the outside bend of the river, as shown in Figure 2. It was therefore assumed that the slip surface followed the surface of the clay till until it met the base of the River.

There was visual evidence to suggest that progressive erosion over time along the outside bend of the river was contributing to the instability by removing toe resistance, resulting in a loss of lateral support downslope of the highway location. It was inferred that there were likely a series of retrogressive slide block failures radiating outwards from the outward bend of the river, with the initial ones beginning nearer the river, and the furthest and most recent currently affecting the highway, (the slide scarp features observed on site are shown on Figures 2 and 3).

The initial assessment of the landslide mechanism were shallow rotational slip surface intersecting the slide planes identified by the inclinometers. However the overall geometry, the elevations of the multiple slide scarps, and the deep, near vertical slide scarp observed on site through the pavement near the center of the slide, appeared to favour a slide mechanism consisting of translational slide blocks having steep entry angles and sliding along a common, deep seated slide plane below the highway. This latter slide mechanism was kinematically consistent with the site observations, and was a more conservative interpretation, and was used in the mitigation analysis and design.

4.4 Design and Remedial Measures

4.4.1 General

The design of a 120 m long tied-back cast-in-place reinforced concrete pile wall to retain the upper portion of the landslide mass was completed in late 2011. Figure 2 shows the layout of the upper pile wall. The centerline was offset a distance of about 8.5 m downslope of the existing highway centerline (or 4 m downslope of the top of paved shoulder edge). A cast-in-place concrete waler/cap beam was required along the top of the piles to provide additional restraint to the pile wall movement and to tie-off the anchors. A cast-in-place concrete retaining wall was constructed on top of the waler/cap beam to retain the highway.

Due to the change in soil stratigraphy across the site, three cross-sections perpendicular to the highway were used to set up separate stability analyses, so as to economically design the pile walls. Section A at the north end was 20 m long, where the slide plane was 8.5 m below surface at the upper pile wall location. Section B was 50 m long at the south end, where the slide plane was 13 m below surface at the upper pile wall location. Section B+2m was 50 m long about midway between Sections A and B, and reflects the most critical conditions, where the slide plane is 15 m below surface at the pile wall. The slide plane was assumed to follow the clay till surface on a fairly flat lying horizontal inclination below the highway down to the base of the Eureka River.

A 60 m long lower cantilever pile wall was designed adjacent to the river approximately along elevation contour 592.5 (shown on Figure 2), where it would perform effectively as a cantilever wall based on the assumed failure plane elevation. The main purpose of this lower wall was to prevent soil from moving downslope and away from the upper pile wall. This was required to maintain higher resistive forces against the upper line of piles which would reduce the net resultant driving forces at the upper pile wall location). It would also reduce erosion and resulting soil loss caused by the river.

In order to further reduce and permanently maintain lower driving forces acting on the upper pile wall, additional measures were undertaken. Lightweight Expanded Polystyrene (EPS) “Geofoam” fill was used as soil replacement beneath the reconstructed pavement surface on the upslope side of the upper pile wall. Soil excavation to different depths for each of the three wall sections and on each side of the pile wall, varying from 4 m to 5.5 m, was also implemented.

4.4.2 Upper Tied-Back Pile Wall and Resulting Pile Configuration Details

The upper pile wall was designed in three separate sections (with the boundaries as described previously). The pile wall was also designed to accommodate both short term and long term loading conditions.

The short term loading conditions were targeted to a factor of safety at least 15% higher than the existing conditions without any force on the piles. The short term loading conditions were checked based on the assumption that excavation must first be completed on both sides of the pile wall to offload the slide area, and then installing the piles, but prior to installing and stressing the anchors at all sections.

The long term loading conditions were based on the construction sequence after the tie backs had been installed and locked off onto the waler/cap beam at all three design sections, and the appropriate materials (lightweight, common and pavement fills) had been backfilled against the upslope side of the waler/cap beam and retaining wall. It was also assumed that lateral resistance was provided by the slide mass in contact with the downslope side of the piles below the respective excavation depths at each design section. An additional recommendation for the long term loading conditions design was to limit pile head deflections to 50 mm. For this condition, a tied-back pile wall was required to resist a net un-factored load of 400 KN/m for all three design sections, plus the equivalent traffic load, in order to achieve a minimum long term factor of safety increase of 30%. This 400 KN/m was converted to an equivalent, design pressure distribution acting on the piles over the lengths above the slip surface at each design section. The un-factored loading pressures, resistance pressures, and design parameters for the long term loading conditions at the upper pile wall for the most critical (central) design section B+2 are shown on loading schematic Figure 4.

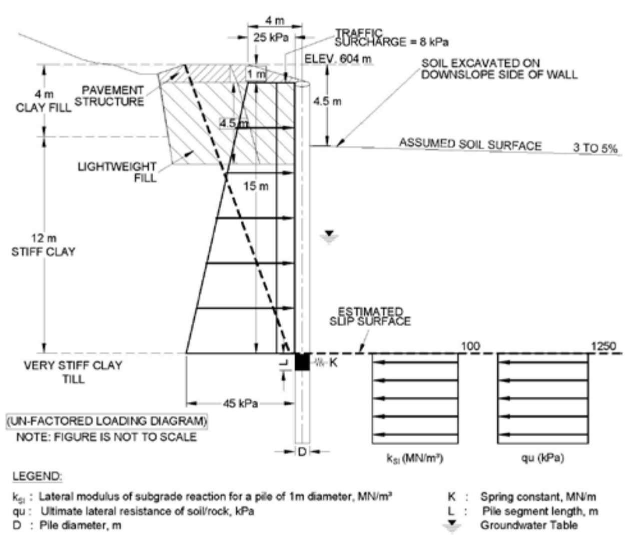

Figure 4. Lateral Soil Model for Central Section of Upper Pile Wall

The net resultant unfactored force of 400 KN/m was first determined for each of the three design sections by: a) isolating the soil mass from the pile wall to obtain a total force; then b) subtracting the calculated force obtained using the active earth pressure from the soil acting on the back side of the wall below the appropriate downslope excavation depths, reduced by 20%. This was deemed conservative enough to alleviate the need to allow for future subsidence or settlement of the landslide mass away from the piles.

Each of the three design sections were then checked for an additional scenario assuming a force embedded within the soil mass (i.e. not isolated), to achieve a minimum long term factor of safety increase of 50%. This was found to be more critical at Design Section A, and the design was adjusted at this location to include additional excavation/lightweight fill replacement while maintaining the same force for overall pile wall design consistency.

The upper pile wall included the construction of a total of 33, 1.5 m diameter, and 23 to 27 m deep reinforced concrete piles with a centre-to-centre spacing of 3.75 m (i.e. 2.5 times the pile diameter), and a reinforced concrete waler along the tops of the piles.

4.4.3 Tie-Back Anchor Details

Anchor design input for pullout resistance stipulated that the anchor bond zone must be located completely within the very stiff clay till stratum beginning a minimum of 1.5 m below its surface, The recommended adhesion values for post grouted anchors in this clay till was 65 kPa ultimate, or a ULS factored value of 40 kPa (using a geotechnical resistance factor of 0.6 which assumes that every anchor is proof tested). Anchor design details are shown on Figure 3.

A total of 198 post-grouted ground anchors were installed along the upper wall, six per pile in two rows. The inclinations of the anchors were staggered to reduce the risk of installing them closer than the minimum 4 diameter recommended spacing between consecutive anchor installations in the same row. The anchors were inclined at between 23 to 27 degrees from the horizontal for the upper row of anchors, and from 28 to 32 degrees for the lower row. The anchors were 200 mm in diameter to maintain practicality for drilling and post-grouting. The anchor design loads were 300 kN and the specified lock off loads were 240 kN (= 80% of design). The anchors had free lengths ranging from 14 m to 44 m, and 12.2 m bond lengths in the clay till. The wide variance in anchor lengths was due to the dip of the clay till surface, as noted in the geotechnical investigation and verified during pile and anchor installation. Figure 5 shows the documented clay till surface elevations at the site. The structural element consisted of 36 mm diameter, Grade 1035 MPa, DYWIDAG threaded Double Corrosion Protection bars.

Prior to the installation of the production anchors, two pre-production anchors were installed and tested to confirm the design loads (using larger 46 mm diameter steel to accommodate the extra load). Proof testing (to 1.33 times the design load) was undertaken for all production anchors, and performance tests were carried out on eight production anchors to confirm the creep and strength characteristics of the anchorage system. After the proof and performance loading tests were completed, the anchors were locked off as per a prescribed lock-off sequence to maintain uniform loading on the wall. The anchors were locked off at a target about 5% higher than the specified 80% of design load, as anchor loads are known to dissipate with time due to the non-simultaneous locking of anchors and re-distribution of loads along the wall. Lift-off tests were carried out after locking off the anchors to confirm load losses due to seating effects.

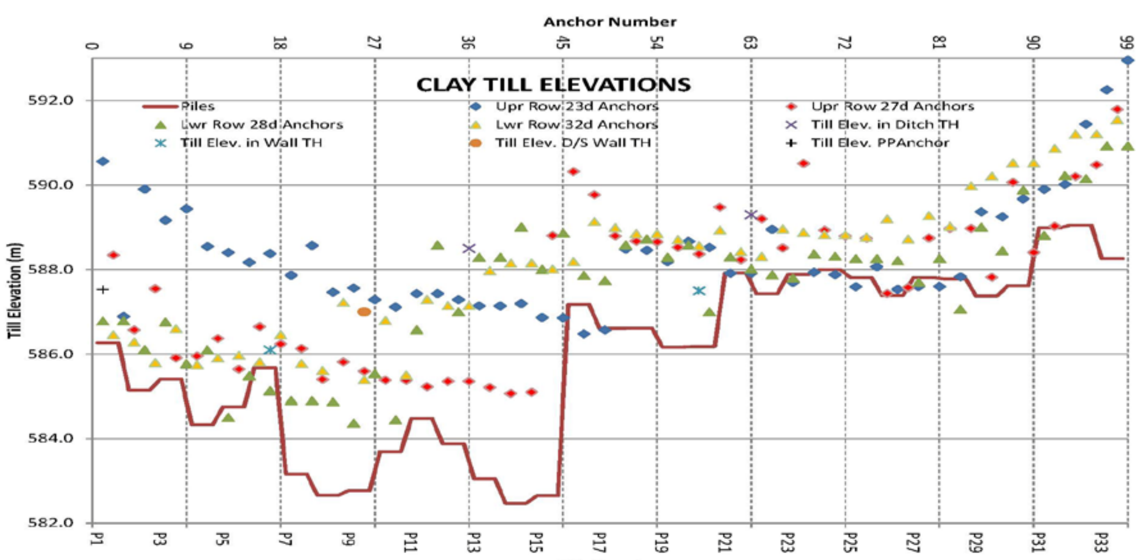

Figure 5. Clay Till and Anchor Bond Zone Elevations along Upper Wall

Work was suspended over the months of February to May, 2013, which are the most severe winter months and during spring thaw. It was recommended that some load be installed on the portions of the waler wall that was already constructed (the northern and central sections), in order to prevent potential slide induced movements from displacing the wall prior to construction completion. Intermediate design checks were performed by the structural sub-consultant, to determine what safe loads could be applied against the waler wall to limit wall deflections as per design, without having the lightweight fill and surrounding backfill providing resistance against the upslope side of the wall (since none of it had been constructed to this point). It was determined that it would be prudent to apply a 240 KN lock-off load per anchor on a maximum of 2 of the 6 anchors surrounding each pile (and both must be on the bottom row), where deflections of up to 29 mm might occur for this stage of loading. If this 29 mm was added to 11 mm of deflection after construction completion of the wall), it would result in about 40 mm of total deflection, which was less than the 50 mm deflection allowed for in the design. Therefore, in February, 2013, every lower row anchor on each side of each of the piles (ie. neglecting the center anchor on the lower row and all 3 anchors on the upper row of each pile) in the north and central sections were temporarily locked off with an approximate 240 KN load, until final anchor stressing took place.

4.4.4 Grade Beam Cap (Waler) Details and Connections

The piles from all three wall sections along Pile Wall 1 were completed with a reinforced cast-in-place concrete grade beam cap (waler), 2.325 m deep and 0.89 m wide. Steel tubing sleeves (trumpets) that were monolithically connected to the anchor bearing plates, were placed through the grade beam over the installed anchor threaded bars to maintain the permanent anchor free stressing length through the waler grade beam. Figure 3 shows schematic details.

The waler grade beam was structurally connected to an overlying concrete retaining wall that was 0.4 m wide and varied in height from 2.63 m to 0.15 m at the ends. A concrete drainage swale was installed adjacent to the top of the retaining wall to drain surface water via a subdrain that outletted near the river. A 1.1 m high handrail was then installed along the top of the retaining wall to provide fall protection.

4.4.5 Lower Cantilever Pile Wall Details

The forces acting on the lower pile wall were limited to a maximum of 220 KN/m in order to avoid incorporating tie-backs. Using this magnitude of resisting force, a slope stability analysis was undertaken to achieve a minimum factor of safety increase of 30%. It was found that excavation below the upper pile wall was also required in order to meet this target factor of safety on the lower wall.

In order to form the new design surface, the minimum excavation depths as outlined for the same three design sections at the upper pile wall were then sloped down towards the river in the order of 3% to 5% until daylighting on the existing slope surface. It was recommended that the piles for the lower, cantilever wall be designed to limit the pile head deflections to 100 mm.

The lower pile wall included the construction of a total of 16, 1.5 m diameter, and 20 m deep reinforced concrete piles with a centre-to-centre spacing of 4.0 m (i.e. 2.7 times the pile diameter).

During the spring of 2013, it was noticed that erosion and associated soil loss was occurring between the upper portion of the piles of the lower pile wall. Although the design was adequate based on arching theory and allowance for up to 4.5 m of slumped soil away from the downslope side of this wall, this soil loss would affect the functionality of the wall. Therefore, a design modification was implemented that basically involved two key concepts: 1) protecting the pile wall with a continuous curtain of sheet piling over the affected area, and 2) removing some of the slumped soil in front of (on the river side) of the wall and protecting it against river erosion with riprap. Details of some of these lower pile wall design modifications are shown on Figure 3.

The continuous sheet piling was 6 m high and 6 mm thick, driven along the downslope face of the lower pile wall over the northern 45 m length of the wall. The sheet piles were welded to an HP10x33 and 16 mm thick H-Beam that straddled the top outside edge of the piles, which was in turn welded to brackets that were connected to the top face of the piles. The slumped soil was removed to a depth of 2.5 m below the top of the piles and replaced with Class 2 riprap which was 1 m thick that extended from the sheet piling to the river's edge. Weep holes were drilled through the sheet piling and covered with non-woven geotextile to manage pore water pressure dissipation. Voids upslope of the wall were filled with tamped sand and capped with clay that formed a drainage swale.

4.4.6 New Embankment Construction & Lightweight Fill Placement

Subsequent to construction of the waler grade beam, the remaining minor excavation and grading upslope of the upper wall was completed. Two subdrains were installed along the entire length of the wall, and extended through the base of the waler and outletted near the river, to collect and drain accumulated seepage water upslope of the wall.

A 200 mm thick bedding sand layer was then placed over the excavated clay subgrade and subdrains. Lightweight fill (LWF) consisting of EPS Geofoam blocks were then systematically installed in 5 consecutive layers until design grade was reached near the top of the retaining wall. Each layer of LWF was surrounded by compacted sand, which was in turn confined with compacted clay on the 3 sides away from the upper wall, to confine and lock the LWF in place prior to commencement of the next overlying layer. A compacted clay cap about 0.7 m thick was installed overtop the completed uppermost LWF layer to protect the LWF from possible future hydrocarbon spills.

The original paved highway was then restored by placement of a new pavement structure consisting of 380 mm of granular base course and 130 mm of asphalt concrete pavement.

4.4.7 Instrumentation Monitoring Details

A total of six slope inclinometers (SI's), were installed to measure the lateral deflections of the pile wall, and to assess the effectiveness of the remedial measures, and confirm design assumptions. In the lower pile wall, SI's were installed in piles P3, P9, and P14, and were labelled as SI12-P3L, -P9L, and -P14L respectively. In the upper pile wall, the SI's were installed in piles P9, P17, and P26, and labelled as SI12-P9U, -17U, and -P26U respectively. Vibrating wire load cells were also installed in six of the anchors for long term monitoring of the anchor loads.

It should be noted that the SI reading initialization dates varied with the pile construction schedule, and on the upper wall were performed prior to any backfilling or anchor stressing activities had taken place (ie. they were read below the top of piles). Later, the SI's were extended up through the waler concrete when it was poured, and then through the backfill placed behind the retaining wall when it was constructed over the waler. Interim anchor stressing on the northern 2/3 of the upper pile wall (and subsequent lockoff of 2/3 of the anchors along the bottom row only) was completed on Feb. 6, 2013, prior to winter shutdown (and prior to backfill placement behind the wall). The south section of the waler was not poured until June 26, 2013, which precluded final anchor stressing and lock-offs which were completed on Aug. 27, 2013. SI readings were taken in the fall of 2013 (after all of the anchors had been permanently locked off for about a month), and again in both the spring and fall of 2014.

The readings in the upper wall SI's generally indicated that, prior to stressing the anchors, pile movements were slightly toward the river in the downslope (positive) direction. However, upon stressing, the piles and waler were initially pulled into the hillside in the negative direction, by between 12 mm to 17 mm at the top of pile, and by up to 31 mm at the top of the waler. However the movements have since reversed and are trending downslope as the slide pushes against the pile wall, with the latest (fall) 2014 readings showing downslope movements between 0 and 11 mm/year over the length of the piles. For illustration purposes, the incremental and cumulative deflections versus depth plots in SI12-P17U (the centre inclinometer in the upper wall) are presented in Figure 6.

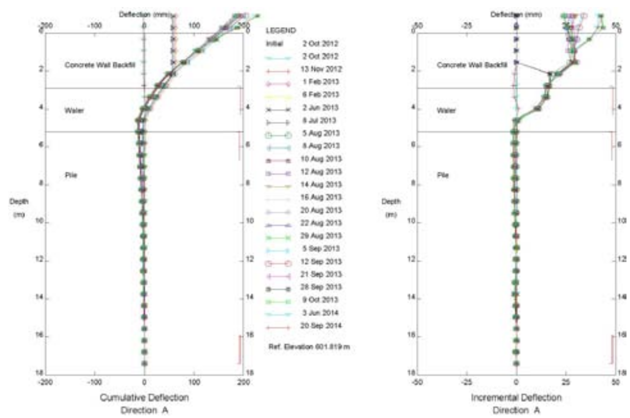

Figure 6. SI12-P17U (Cumulative and Incremental Deflections vs Depth Plots)

At the lower pile wall, the fall, 2014 readings (about 2 years after installation), have indicated total movements of the pile heads between 2 mm to 3 mm downwards toward the river in the 3 SI's. These deflections correspond to rates between about 2 to 8 mm/year over the length of the piles since spring, 2014.

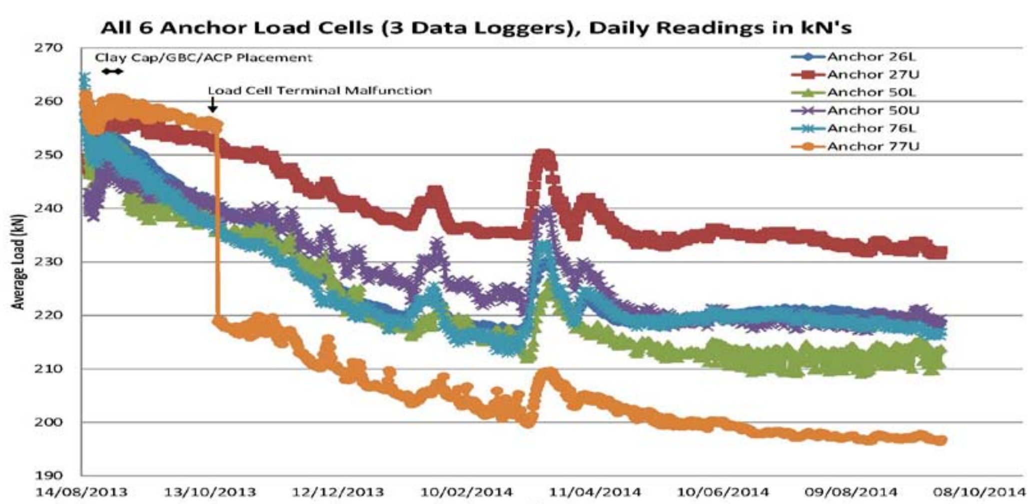

Figure 7. Variations of Anchor Loads with Time

Figure 7 shows the variation of the six anchor loads (VC1759 to VC1764 inclusive) with time for the upper pile wall. Three dataloggers were set up to record load readings every 12 hours starting in August, 2013. As mentioned previously, the anchors were targeted to be locked off about 5% higher than the design lock-off loads (ranging between 245 to 255 kN), anticipating future load dissipation. Initially the load cells for the three anchors along the lower row reflected loads in the target range, and then decreased by about 4 to 6% after 1 month. Comparatively, the load cells for the three anchors along the upper row were more variable, and initially increased by about 5 to 10 KN (~2 to 4%) in response to compaction of the highway pavement structure, and thereafter dropped off. In the fall of 2014, five of the anchor loads ranged between 213 to 220 KN, which is a drop of 10 to 15%, however anchor VC1764 read 235 KN, a drop of 6%. All six of the anchors exhibited a general trend of load fluctuations between about 3 to 5 KN (~2%) due to daytime and nighttime temperature changes.

The reduction in the anchor loads can be attributed to the non-simultaneous locking of anchors and re-distribution of loads along the wall. It is of interest to note that the decline in anchor loads was not accompanied by any significant ground movements and therefore the performance was considered acceptable as long as the current and future load losses do not reduce the anchor loads below that required for maintaining the stability of the retaining structure. So far, the pile deflections indicate that the pile walls and LWF are effective in stabilizing the landslide.

5 SUMMARY AND CONCLUSIONS

This paper provided a summary of a landslide repair project along Highway 726:02 in Northwest Alberta.

The main points from this paper are as follows:

- A number of landslides are currently affecting the length of Highway 726:02 traversing the Eureka River Valley, some of which have developed since construction in 1988. A functional planning study was undertaken in 2010/2011, and the most feasible long term option was a highway realignment utilizing the same culvert crossing. However, due to the time required to design and implement the realignment, the severity of Site 3 conditions necessitated urgent remediation.
- The landslide at Site 3 affected an approximate 110 m length of highway, which required frequent on-going maintenance.
- The cause of the landslide affecting the highway at Site 3 was erosion around an outside bend of the river. This resulted in a series of retrogressive, and translational slide blocks radiating outwards from the river, with the furthest affecting the highway. The slide surface was contained within highly plastic lacustrine clay, based along the interface with underlying glacial clay till near the center of the slide mass at a depth of 15 m below the highway.
- In 2012, a 120 m long tied-back concrete pile wall was implemented along the downslope edge of the highway as the primary measure to remediate the landslide. To offset the significantly large resultant driving forces acting on this pile wall, additional design measures were required, which consisted of: a 60 m long cantilever pile wall near the river to retain soil in place against the downslope side of the upper wall; excavation of material on both sides of the upper pile wall; and reinstatement of the highway fill uphill of the upper wall using lightweight EPS Geofoam fill as soil replacement.
- Loss of soil between the upper portion of the piles of the lower pile wall necessitated supplementary remediation in 2013 consisting of protecting the wall with a continuous curtain of sheet piling over the affected area, and replacing the slumped soil on the river side of the wall with riprap armour.
- Instrumentation installation consisted of 6 SI's (3 in each pile wall), and 6 load cells installed on selected anchors in the upper pile wall. Monitoring of deflections and loads to date are within expected design criteria.

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