Analysis of Field Performance of Landslide Repair Using a Pile Wall

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ABSTRACT
A historically slow moving embankment fill landslide on Secondary Highway 682:02 accelerated to failure in the spring of 2007. The landslide was translational with the failure plane located between 4.5 and 6.5 m depth within native lacustrine clay. The stabilization repair method selected was a pile wall that consisted of 67 cast-in-place reinforced concrete piles spaced at 1.5 m on centre.

Field performance of the pile wall after completion of construction was verified using two slope inclinometers that were embedded in two of the concrete piles. Post-construction deformations were within the range suggested during the design. Analysis of the construction deformations using FLAC provided insight into the selection of material properties, especially soil stiffness moduli, for the future design of pile walls used for landslide stabilization.

RÉSUMÉ
Un glissement de terrain historiquement lent du remblai de l'autoroute secondaire 682:02 a accéléré jusqu'à rupture au printemps 2007. Le glissement de terrain a été translationnelle avec l'échec plane située entre 4,5 et 6,5 m de profondeur dans des argiles lacustres. La méthode de stabilisation choisie était une paroi de 67 pieux en béton armé moulés sur place et espacés de 1.5 m au centre.

La performance du mur de pieux après construction a été surveillée au moyen d'inclinomètres installés dans deux des pieux de béton. Les déformations après construction sont demeurées sous la limite suggérée par le concepteur. L'analyse des déformations, faites à l'aide du logiciel FLAC, a éclairé le choix des propriétés des matériaux, particulièrement le module de rigidité des sols, pour la conception future de parois de pieux utilisées pour la stabilisation de glissement de terrain.

1 INTRODUCTION

A historically slow moving embankment fill landslide on Highway 682:02 accelerated to failure in the spring of 2007. This site is located in the northeast part of Alberta, approximately 10 km west of Fairview, Alberta, in an area of landslide terrain associated with an adjacent deep valley. This site was designated the PH4 site within the Peace-High Level region and was actively being monitored by Alberta Transportation (AT) under the Geohazard Risk Management Program. The highway has an Average Annual Daily Traffic (AADT) count of 460 (2008). A traffic closure at the site would result in a detour of approximately 60 km for local rural residents.

2 SITE CONDITIONS

The landslide site encompassed an 11 m high embankment that spans a small ephemeral creek valley. Part of the highway grade line was constructed using cut and fill methods. The landslide was approximately 90 m wide at the head and extended to the centerline of the highway as shown in Photo 1. A pronounced toe push feature was evident near the toe of the embankment slope, which indicated day-lighting of the failure plane. The embankment fill slopes varied from 11° to 17°. A site plan showing the extent of the landslide and borehole/instrument locations is presented on Figure 1.

Photo 1. Aerial view of Hines Creek (PH4) Landslide

A drilling investigation was conducted in July 2007 that included 6 boreholes with pneumatic piezometers and slope inclinometers installed in some of the boreholes. The stratigraphy encountered at the site consisted of 2.0 to 3.5 m of high plastic clay fill overlying high plastic native lacustrine clay. The clay fill was most likely borrowed from the nearby backslope excavation. Review of the information gathered from the drilling program and the instrument monitoring indicated that the landslide was translational in nature with the failure plane located between 4.5 and 6.5 m depth in native lacustrine clay.
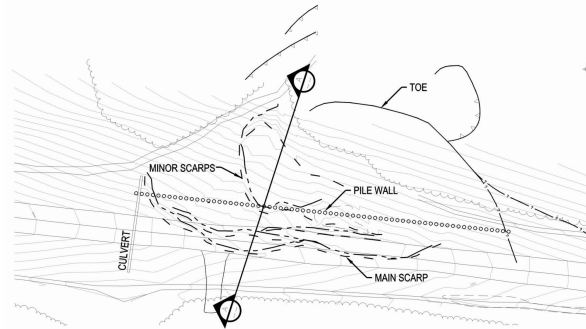

Figure 1. Site Plan

The cause of the landslide was determined to be due to the destabilizing influence of the embankment fill at the site and moderately adverse groundwater conditions. Seepage was present at various locations on the embankment slope, including approximately 2.0 m below the highway grade line. The drainage ditch immediately south of the landslide area was poorly graded with ponded water present after rainfall events. A section of the embankment slope and estimated failure plane is presented in Figure 2.
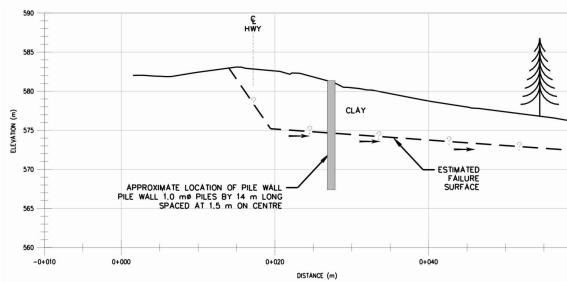

Figure 2. Landslide Cross Section

3   DESIGN AND ANALYSIS

A variety of slope stabilization options were considered, but a pile wall located near the crest of the slope was selected as the preferred solution for this site as it did not significantly alter the state of stresses within the overall valley slope. This was an important consideration due to the site being located in an area of widespread active and historic landslide activity. The design included an option to install tie-back anchors in the future if slope deformations down slope of the pile wall unloaded passive resistance from the face of the pile wall. Additional improvements such as improved basal drainage for the reconstructed highway embankment and improved back slope ditch grading were also included in the design to address some of the surface water and shallow seepage issues.

The detailed design of pile wall was undertaken using finite difference modeling of the soil-pile wall structure interaction using the commercially available software FLAC (Itasca, 2005). The soil properties used in the design analysis are presented in Table 1. The shear strength properties were the most clearly defined properties utilized in the design. The selection of soil moduli was based on the results of multi-stage triaxial testing as well as engineering judgment and experience with other pile wall design and construction projects. Review of the borehole logs, samples and laboratory test results indicated that there was no appreciable difference in the index, strength or stress-strain properties of the lacustrine clay above and below the basal failure plane. The soils along the basal failure plane were pre-sheared and at residual strength due to the magnitude of deformations that had occurred.

Table 1. Materials Properties Used in Design

Material	Unit Weight (kN/m3)	Friction Angle (°)	Cohesion (kPa)	Young's Modulus (MPa)	Poisson's Ratio
Slide mass	18	22°	5	25	0.4
Failure plane	18	13°	0	12.5	0.4
Intact soil beneath slide	18	22°	5	25	0.4

The concrete piles were assumed to be cracked for the purposes of the analysis with a composite modulus of 21.4 GPa. Additionally, as preliminary design of the pile wall involved spacing the piles with a column of soil left in between each pile, the stiffness and strength properties of the pile in the model were proportionally reduced in accordance with a pseudo-3D approach used in FLAC modelling.

The slide was back analyzed using Slope/W and FLAC to confirm soil parameters and site conditions immediately before and after the failure. The modeled slide mass developed a minor scarp during the initial back-analysis using a strain-softening constitutive model in FLAC which generally replicated the location of a minor scarp and degree of deformations experienced on site.

The design of the repairs for the subject landslide was conducted to achieve a factor of safety of 1.3 as per AT guidelines for such work. A Strength Reduction Factor (SRF) approach was adopted during the modeling phase in which the shear strength parameters are reduced by a factor of 1.3 to approximate a factor of safety of 1.3. An SRF of 1.3 was adopted as a "Reasonable Worst Case" scenario and an SRF of 1.1 was adopted as a "Best Case Scenario". The "Most Likely Case" scenario was deemed to be somewhere between these two bracketing values.

The SRF factor was not applied to the moduli selected for the soil units.

The post construction deformations of the pile heads predicted during the design phase varied between 7 and 93 mm for the SRF values of 1.1 and 1.3, respectively. These were judged to be sufficient to minimize the potential for cracking of the reconstructed asphalt surface.

There was uncertainty associated with the long term provision and degree of passive soil support that could be relied upon. There was also uncertainty regarding the selection of appropriate soil moduli. These scenarios were investigated by modeling the staged removal of the soil in front of the wall and by varying soil moduli.

The final design of the pile wall included 67 cast-in-place reinforced concrete piles, 14 m in length, 1.0 m in diameter and spaced at 1.5 m on centre. The area of the steel within the piles was 2.2% of the total pile cross-sectional area. The layout of the pile wall is presented in Figure 1. A contingency plan is in place in the event that embankment fill and asphalt road surface deformations exceed AT tolerances. This contingency includes installation of tie-back anchors to further restrain the wall movements.

4   CONSTRUCTION

Construction started in December 2007. An access road and level pad area were built across the embankment at the pile wall location. The timing of construction was primarily dictated by the urgency associated with completing the repair work. There was some benefit to cold weather conditions as it allowed for ground freezing to improve equipment trafficability on site. Pile installation was started in mid-January and was completed in 10 days. A photograph of the pile wall construction is presented in Photo 2. The ambient air temperature varied between -25 and -40° C throughout piling construction. Pile drilling near the formerly wet area near the zone of initial movement encountered a buried drainage pipe, indicative of historic drainage efforts.

Reconstruction of the upper 2 m of road with granular fill and perforated pipe sub-drainage and asphalt surfacing was completed in May 2008. Approximately 1.0 m of asphalt was removed during road reconstruction, indicating the extent of historic movement prior to 2007.

Photo 2. Pile Wall Construction (late January 2008)

5   FIELD PERFORMANCE

Two slope inclinometers (SI) were fastened to the rebar cage and cast into Piles 20 and 44 to permit measurement of pile wall deflection after installation. The SIs installed in piles 20 and 44 are located approximately 1/3 and 2/3 along the length of the pile wall.

Slope inclinometer readings (on April 29, 2009) indicating the deflection observed in the pile wall since completion of construction are illustrated in Figure 3. The observed deflections were within the bracketed range of 7 to 93 mm predicted during the design phase. The majority of the deflections occurred prior to early summer 2008 when the final grading and paving was complete. There has been limited post-construction deformation or cracking of the embankment slope and repaired road surface above the pile wall.

The groundwater conditions recorded since construction of the pile wall are a close match to those used during the design phase.

6   BACK ANALYSIS

This project presented a valuable opportunity to review the field performance of an instrumented pile wall and to verify the soil properties used in the design. Three back analysis cases were conducted as follows:

- Case 1 & 2: Back analysis using the design model based on Mohr-Coulomb failure criterion with a linear elasto-plastic behaviour using secant moduli corresponding to peak and residual strength values. The shear strength was not modified for this phase due to a high confidence level in their validity. The back analysis was carried out in two steps:
  - Step 1: back calculating the operating SRF based on field performance (Case 1); and,
  - Step 2: back calculating the secant stiffness of the soil units above, along and below the failure plane (Case 2).
- Case 3: Back analysis using hyperbolic non-linear model similar to Duncan-Chang (1970) approach

with Mohr-Coulomb failure criterion. This phase of additional analysis was initiated to further investigate the variation of stiffness of the soil units as the slope failure progresses on the observed basal failure plane. The hyperbolic model parameters were obtained from triaxial and direct shear testing data.

The model parameters and results of the back analysis are presented in Table 2. Figure 3 shows a comparison between pile wall SI readings (April 29, 2009) and deformations back analyzed predicted by various methods described above.

Table 2. Results of Back Analysis

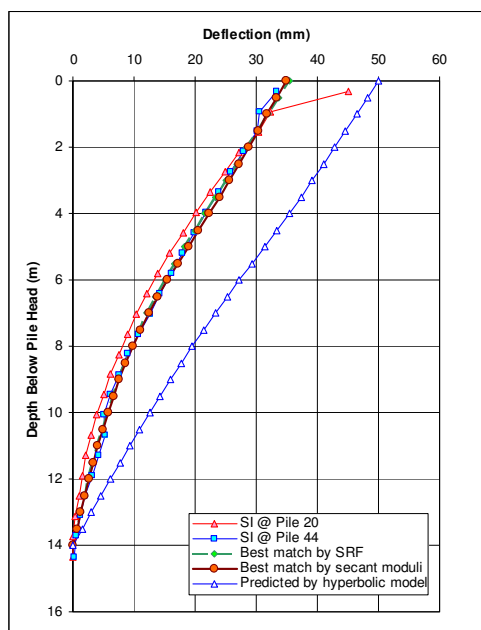| Case	SRF	Slide Mass Modulus (MPa)	Failure Plane Modulus (MPa)	Below Failure Plane Modulus (MPa)
1	1.2	25	12.5	25
2	1.0	15	2	30
3	1.0	38/0.85/0.6*	8.4/0.70/0.6*	38/0.85/0.58*
*Note: Parameters given for hyperbolic model are the initial (tangent) Young's modulus, failure ratio and power exponent, respectively. (see Duncan and Chang, 1970)				

Figure 3. Comparison between pile wall SI readings (April 29, 2009) and back analyzed deformations

7   DISCUSSION

The primary purpose of this numerical exercise was to establish a set of input parameters that provided the best match with the observed deformations as well as to obtain a realistic range of future deformations. Compared to the initial design parameters, the back calculated SRF values were within the range established. The back calculated secant moduli were lower for the disturbed

soils within the slide mass and in the basal failure plane and higher for the soils below the basal failure plane.

The use of the non-linear hyperbolic constitutive model in lieu of a linear elasto-plastic model would better replicate the softening behaviour of the soil mass above, along and below the basal failure plane. The predicted deformations estimated using the non-linear hyperbolic model presented herein are greater than the values observed to date. While they may be considered conservative, they are still within the range of deformations established in the design phase. Future performance of the pile wall will help in assessing the degree of conservatism in this approach and assist in defining when tie back anchors may be required. The analysis will also provide a useful model for design of tie back anchors.

The operating stiffness of the soil is a function of state of shear stresses and strains that the soil experiences. Variability in the soil stiffness across a site is dependant on a variety of factors which includes seepage zones and natural variability of soil deposits. In this case, the varied degree of deformation experienced by the slide mass and basal failure plane soils is reflected in the respective back analysed lower moduli values in comparison to the apparently stable soils below the basal failure plane. Laboratory testing on a limited number of small samples may not provide adequate insight on the variability of soil stiffness for such a situation.

Another useful approach to assess in-situ soil stiffness for similar modeling purposes is to conduct down-hole or surface shear wave testing in SI casing or surface shear wave testing to assess the small strain shear modulus of the soil profile. This would allow assessment of the variability of the bulk stiffness with depth which could provide greater accuracy in selecting soil stiffness for deformation analyses using hyperbolic or strain softening models.

Laboratory tests on small samples were useful in providing estimates of strength and stiffness properties; however some degree of judgment based on previous experience is required to extrapolate soil stiffness from laboratory scale testing to a macro scale such as the subject site. Use of down hole or surface shear wave testing during the investigation phase may provide a means to assess bulk soil stiffness to supplement traditional laboratory testing in the design of landslide repairs using pile walls.

ACKNOWLEDGEMENTS

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