Slope Stabilization at Mt. Alberta View SE, Calgary, Alberta

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ABSTRACT
The regional pathway on top of the Bow River escarpment in the vicinity of Mt. Alberta View SE, Calgary, Alberta, is an important link within the city-wide regional pathway system. In the summer of 2005, following a period of unusually heavy rainfall and associated flooding, a portion of the pathway was damaged as a result of a slope failure on the 35 m high valley slope. In 2006, a design option of re-grading and relocating the damaged pathway further back to the residential side, in order to off-load the crest area, was developed, approved by stakeholders and subsequently implemented over the winter. The relocated pathway was constructed to the base course grade and left unpaved. A new tension crack developed in the spring of 2007 due to retrogressive slope failure resulting in damage to, and unsafe use of, the reconstructed pathway. The height of back scarp increased with time and was measured from approximately 1.5 m to over 2.5 m by the summer of 2008. Pathway rerouting or detour was not an option. In order to protect the pathway, four slope stabilization options were assessed. The option selected consisted of an anchored concrete caisson wall located along the crest of the affected slope. After construction, the caisson wall was 90 m long and consisted of 70 discrete concrete caissons, 35 permanent ground anchors and a concrete waler system. Piezometers and slope indicators were also installed. The paper briefly describes the project background, geological setting and geotechnical conditions, design and construction activities, and results of anchor testing and instrumentation.

RÉSUMÉ
La piste cyclable qui longe la crête de l'escarpement de la rivière Bow dans la communauté de Mt. Alberta View de Calgary est un segment important du réseau de sentiers pavés de la ville de Calgary, Alberta. Après la période de pluies intenses et d'inondations de l'été 2005, la pente de 35 m de hauteur a glissé et la piste cyclable subit des dommages consédérables. En 2006, les réparations qui consitaient de reculer la piste cyclable furent approuvées par les résidents et furent exécutées. Au printemps 2007, une nouvelle fissure apparue et le glissement de pente se reactiva causant des bris aux réparations. Le glissement continua de bouger et la portion vertical du glissement mesura de 1.5 à 2.5 m de hauteur a la fin de l'été 2008. Parce qu'il n'etait plus possible de changer la piste cyclable d'endroit, plusieurs options de stabilisation des pentes ont été examinées. L'option choisie fut la construction d'un mur de caissons de béton ancrés à la crête de la pente. Le mur de 90 m de long est fait de 70 caissons avec 35 points d'ancrages. Des piezomètres et des indicateurs de pente ont aussi été installés. Cet article décrit le projet, les conditions geologiques et géotechniques, le design, la construction et les résultats d'essais.

1 INTRODUCTION

A part of the City of Calgary (City) regional pathway system, at Mt Alberta View SE, was damaged as a result of a slope failure during and immediately following a period of unusually heavy rainfalls during the month of June and July 2005. The City retained Golder Associates Ltd. (Golder) to investigate causes and mechanisms of failure and to identify options for slope stabilization. At the same time, the site was secured and tension cracks were sealed with bentonite mix.

The report was issued in May 2006 (Golder, 2006) and provided several possible remedial measures including Reduce Surface Water Infiltration, Slope Drainage, Structural Support and Slope Re-profiling. Surface drainage issues were also discussed. Notices were delivered to the property owners adjacent to the slope failure area to request that the downspouts and drainage pipes within private properties be corrected to comply with City drainage bylaws.

A design to re-grade and relocate the damaged pathway further back to the residential side in order to off-load the crest area was completed by Golder in 2006. Public meetings and a number of communications with the affected residents were made to present the design to the residents. Construction started at the end of October 2006 and 80 % work was completed over the winter. Temporary gravel surface was provided for the relocated pathway.

In the spring of 2007 when the contractor resumed the work, a new tension crack developed and was located approximately 0.3 m (at the closest point) in front of the existing gravel pathway. It was anticipated that future retrogressive failure of the slope will continue to encroach upon the relocated pathway. Efforts were made to seal the tension cracks in the spring of 2007; however, the slope continued to move causing to form a vertical backscarp along the 2007 crack. The height of vertical backscarp increased with time and was measured to be approximately 1.5 m to over 2.5 m by the summer of 2008. A temporary chain-link fence was installed along the crest of the failed slope to provide protection to the public.

The spring 2007 storm event made it apparent that the original plan, which intended to restore the damaged pathway to pre-flood condition at minimum cost was not effective. The City's Parks Department confirmed that the regional pathway is an important link in the city-wide regional pathway system and required additional investment. The pathway is a heavily used by local residents and citizens across the city. River valley pathways, such as this section, are the core of the regional pathway system. Furthermore, at this location there is no alternative for a pathway rerouting or detour due to topography, vegetation and adjacent residents. Long term options were explored. Four design options along with cost estimates were developed by Golder(Golder, 2007). The options considered were status quo (do nothing), mechanically stabilized earth (MSE) retaining wall, and two alternative configuration for anchored caisson wall.

Public meetings were held to present the design options and obtain consent from the affected residents. An anchored concrete caisson wall option was chosen. Regular updates were sent to the stakeholders during detailed design stage. Detailed design andproject specifications including project ECO Plan and Restoration Plan were developed by Golder and included in the project tender package by the end of June 2008. The project was tendered (City of Calgary, 2008) at the beginning of July 2008. The contract was awarded on August 14 and construction started on September 8, 2008, with substantial completion by November 2008.

2 GEOLOGICAL SETTING AND GEOTECHNICAL CONDITIONS

The area of study is shown on Figure 1 and is approximately 100 m long. The slope in the vicinity of the affected area is about 30 m to 36 m high and has average slope angles between 20° to 25°.

The geotechnical field investigation program for the site included drilling and sampling of four boreholes to depths ranging from about 20 m to 38 m, below existing ground surface. Bedrock coring was carried out in two of the boreholes. Standpipe piezometers were installed at all borehole locations. A slope inclinometer casing was installed in Borehole BH05-MA-1.

In general, the soils composing the slope in the subject area consist of glacio-lacustrine silt overlying generally stiff to hard silty clay till to clayey silt till. These surficial soils are in turn underlain by bedrock of the Porcupine Hills Formation. This formation comprises a series of interbedded sandstone, siltstone, claystone and calcareous bentonitic shale.

Soil and bedrock exposures observed during site reconnaissances are consistent with this stratigraphy.

Aerial photograph interpretation indicated evidence of previous retrogressive, block-type slope failures in the study area. Relatively steep backscarps and benching as a result of the slope failures was also observed.

The main backscarps in the vicinity of the study area as a result of the 2005 and 2007 slope movements are indicated in Figure 1. The backscarps are approximately 100 m wide with heights of about 1 m to 2.5 m.

Slope inclinometer readings indicate that no shear zones had developed behind the 2007 backscarps (upslope) at the study area. The tip of the slope inclinometer was installed about 38 m below ground surface and top of bedrock was estimated to be about 17 m below ground surface.

Based on the measured groundwater levels, observations of seepage during drilling, and understanding of the geology of the general area, there are two main groundwater regimes within the slope. The first of these is relatively shallow and located primarily within the upper till deposits and is expected to be relatively sensitive to groundwater infiltration due to precipitation, changes in surface drainage, leaking services, etc., and to vary seasonally. A deeper groundwater regime exists within the deeper till deposits and bedrock. This deeper groundwater regime is more

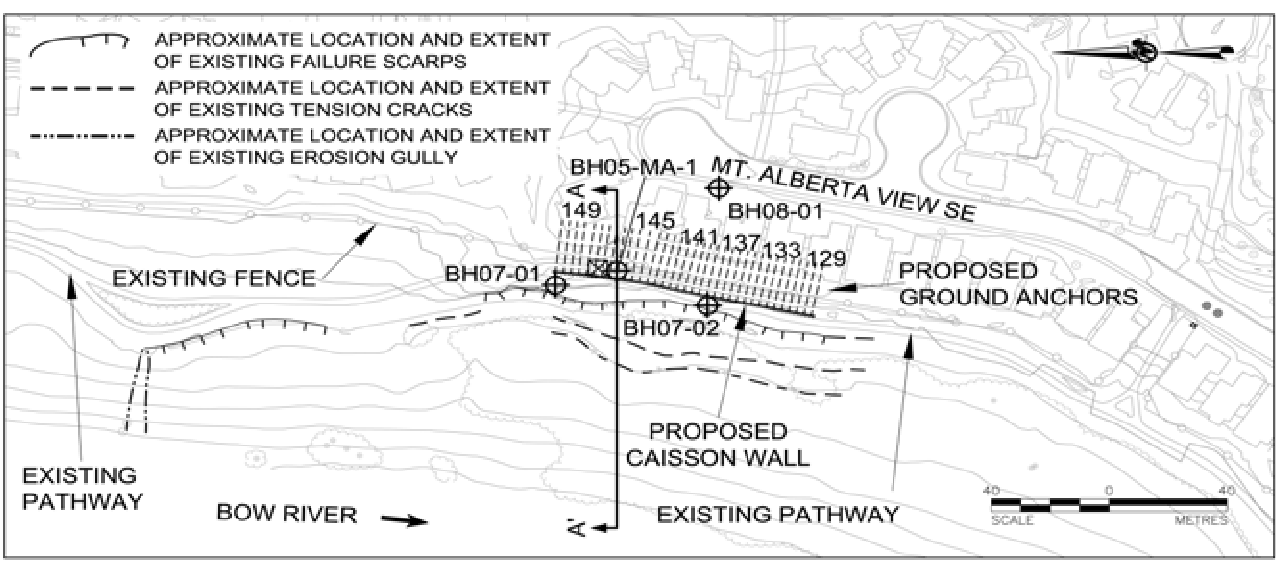

Figure 1. Area of study

regional in nature. Although it may be affected by many of the same aspects that affect groundwater levels within the upper till deposits, it is expected to be less sensitive to local influences, and more responsive to large-scale regional changes in groundwater levels.

Based on the relative difference in groundwater levels, a downward flow (gradient) appears to exist within the slope. This condition is expected in a slope of this type where the fractured bedrock is expected to have an overall higher hydraulic conductivity than that of the overlying clayey soils.

3 SLOPE FAILURE MECHANISM AND SLOPE STABILITY ANALYSIS

3.1 Geological Cross-Section

The geological cross-section used in the slope stability analysis is shown on Figure 2. The slope model comprises a layer of stiff to very stiff clayey till 6.6 m thick, overlying a layer of hard clayey till, which in turn overlies bedrock to the bottom of the slope.

The bedrock surface was modeled to be at approximately Elevation 1,020.3 m at the location of Borehole BH05-MA-1, dropping towards the river valley. The bedrock elevation along the toe of the river valley slope was taken to be at about elevation 1,006.5 m, which is the highest elevation of visible bedrock exposures along the toe at this location.

It was assumed that a weathered, weak zone potentially affected by glaciar shear exists at the bedrock/till interface. This layer was not specifically identified during the drilling investigation, but this failure mechanism is not uncommon in river valleys. Furthermore, the back analysis of the slope failures, the geometry of the on-going failure scarps and the tension cracks suggest that this type of deep-seated failure mechanism may have contributed to the historical retrogressive failures apparent at the site.

3.2 Ground Parameters

Strength parameters for analysis were inferred based on subsurface information obtained during the geotechnical investigation. These parameters have been selected based on direct shear testing, typical correlations with soil type, grain size distributions, plasticity and density, as well as our past experience with similar materials.

The strength properties of the materials were modeled using a Mohr-Coulomb failure criterion. A summary of the soil parameters is provided in Table 1.

Table 1: Parameters used in Stability Analysis

Material Type	Unit Weight (kN/m^3)	Effective Friction Angle ϕ' (º)	Effective Cohesion c' (kPa)
Stiff to very stiff till	20	28	5
Hard till	20	30	5
Weathered Weak Bedrock	20	25	0

3.3 Groundwater Levels

Upper and lower bounds of piezometric levels were used in the stability analyses as shown on Figure 2. The lower bound piezometric level is based on piezometric measurements taken during winter 2005 and evidence of seepage on the face of the slope. This piezometric level is likely not the highest that the slope has experienced in the past or will experience in the future.

The upper bound piezometric level was obtained from back-analysis of previous slope failures. Using the parameters shown in Table 1, the piezometric levels were

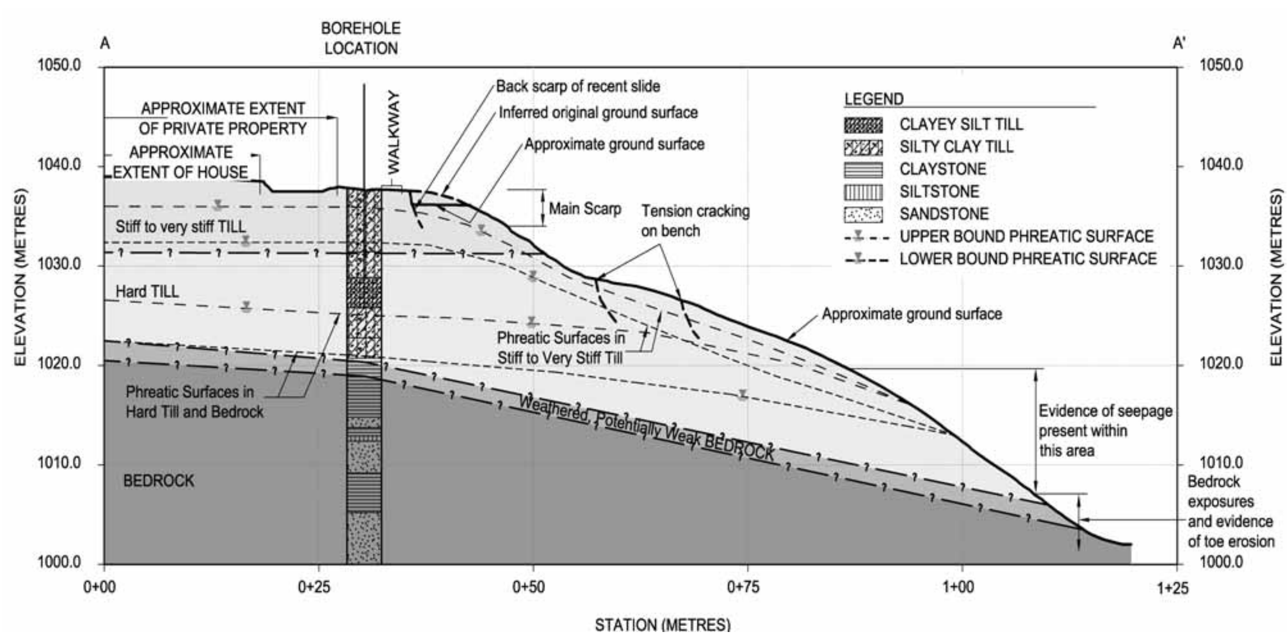

Figure 2. Cross-section used in slope stability analysis

raised with respect to the lower bound piezometric levels until a calculated Factor of Safety of 1.0 was achieved. This corresponds to an increase in piezometric levels within the stiff to very stiff till and within the hard till of approximately 3.5 m and 4 m, respectively, from the levels measured in December 2005. This would represent a limiting condition of a failure.

3.4 Overall Slope Stability

The calculated Factors of Safety for the overall slope are presented in Table 2.

Table 2: Minimum Calculated Factors of Safety for Overall Instability

Piezometric Level	Depth of Groundwater Table at the Location of Borehole BH05-MA-1		Minimum Calculated FOS
	Stiff to very Stiff Till	Hard Till and Bedrock	
Lower Bound	5.6 m	17.1 m	1.1
Upper Bound	2.1 m	13.1 m	1.0

The calculated Factor of Safety against overall slope failure is approximately one using the parameters shown in Table 1.

Based on the results of the analysis, the overall slope would generally be considered marginally stable for lower bound piezometric levels. Furthermore, when the piezometric levels are high (such as during periods of heavy rainfall, extensive surface infiltration etc.) the slope would be likely become unstable. Retrogressive slope failures similar to those which have occurred in the past would therefore be expected to continue in the future.

4 DESIGN OPTIONS

Four design options were identified and evaluated for long-term pathway protection.

A status quo option was included in the assessment of preferred options. However, past observations of the slope retrogression indicated that this option was no longer viable. A long term, reliable protection method was needed because the pathway is heavily used by local residents and by citizens across the city.

An option included a mechanically stabilized earth (MSE) wall along the crest of the slope. The maximum height of the proposed MSE wall was 6 m and included the installation of a cantilevered caisson wall adjacent to the private property line to protect the adjacent private properties during wall construction. This option allowed the centreline of the existing pathway to be maintained at or near its current location from the crest of the slope. However, this option may not prevent future retrogressive soil movements from encroaching upon and undermine the wall and City's pathway.

Another option consisted of constructing an anchored concrete caisson wall along the crest of the existing slope (Figure 3). The caisson wall was proposed to be installed down into competent bedrock beneath potential slope slip surfaces. The caisson wall would consist of 70 drilled concrete caissons, 915 mm in diameter, spaced at 1.3 m centre to centre spacing, reinforced with steel rebar cage. An external support force would be applied to the caisson wall utilizing a waler system and a row of ground anchors to provide lateral resistance to the retained soil behind the caisson wall. The ground anchors would be pressure grouted anchors installed at 2.6 m intervals and 2 m to 3 m below the top of the caisson wall at a 45 degree angle, with a bond zone 7 m long.

To facilitate installation of a row of anchors near the top of the wall, the top of the existing slope would need to be cut-down to approximately 1 m below existing ground to accommodate an 8 m wide proposed temporary access or working platform. The platform provides for construction equipment to operate from the top of the slope in front of the wall. The design accommodates a loss in ground, due to ongoing slope retrogression, of about 6 m in front of the wall. That is, substantial near vertical backscarps can develop at the wall without affecting performance of the wall to provide protection to the pathway. The calculated minimum factor of safety for the overall slope is greater than 2.0 because the caissons are socketted into competent bedrock and failure surfaces would pass below the caisson wall. The caisson wall would be designed to obtain a service life in the order of 50 to 75 years.

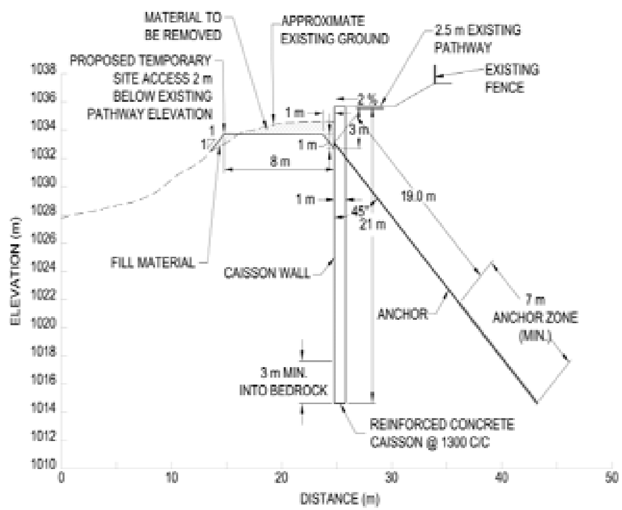

Figure 3. Anchored Concrete Caisson Wall

A fourth option considered was an extension of the anchored caisson wall option that included an additional row of concrete anchorage caissons as part of the overall anchorage system to reduce risk (enhance safety) to the crew and equipment during construction. The installation of ground anchors otherwise would require construction equipment to work on the top of the existing marginally stable slope beyond (downslope) the backscarp. This option included the installation of a row of anchorage caissons located between the caisson wall and private property line. The anchorage caissons would be connected to the caisson wall's waler system utilizing horizontal tendon cables. This method allows for the anchors to be installed largely beyond the crest of the existing slope on unfailed ground. However, it would be more expensive than the other caisson wall option.

5 DETAIL DESIGN CONSIDERATIONS

The anchored caisson wall (Figure 3) was chosen by the City of Calgary and was refined during detailed design. It was assessed that the risk related to working on top of the overall slope beyond (downslope) the backscarp was within acceptable limits.

The waler system consisted of 35 independent structural steel beams. Each beam is connected to two adjacent caissons to provide increased flexibility to the caisson wall.

The row of ground anchors are corrosion protected anchors and with a minimum ultimate strength of 835 kN. Two performance tests on ground anchors were carried out prior to the installation of working ground anchors based on a design load of 285 kN. Each ground anchor was proof tested prior to locking off and lift-off tests were performed after locking off.

The lateral earth pressure distribution used to assess the geotechnical stability of the caisson wall, estimate the anchor design load, design the steel reinforcing for the caisson wall and design the waler system is shown on Figure 4. The proposed earth pressure distribution takes into account a loss in ground in front of the wall of about 6 m to model potential ongoing slope retrogression.

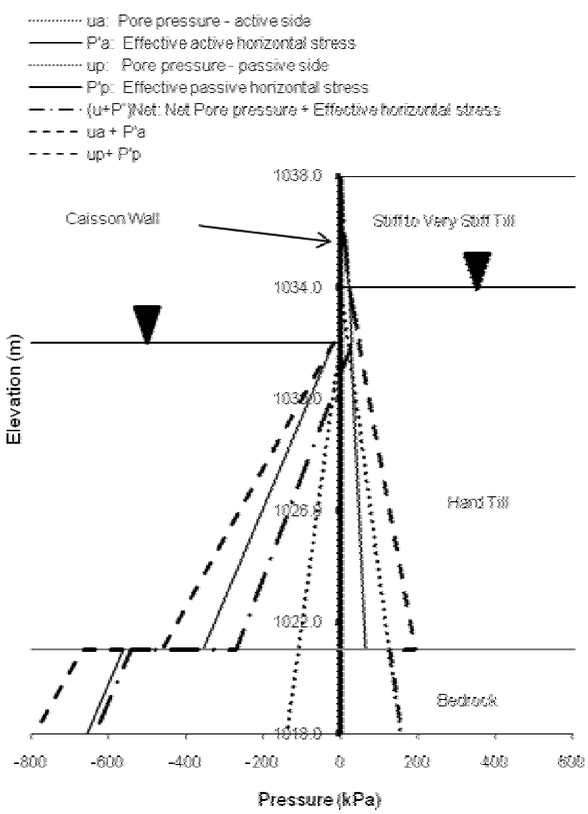

Figure 4. Design Lateral Pressure Diagram

6 CONSTRUCTION ACTIVITIES

Construction activities started in September 2008. Primary activities including caisson wall, waler and ground anchor installation were completed by November 2008. Pathway construction and final grading were completed by July 2009. Construction activities satisfied the Project ECO Plan.

In general, construction activities were conducted according to design drawings and contract specifications with exception of the waler system which was redesign by the Contractor`s engineer to optimize construction costs and improve construction performance. The as-built waler system consisted of 18 independent reinforced concrete beams, each of them connecting four adjacent caissons and supported by two ground anchors.

During caisson installation, a temporary steel liner 11 m to 17 m long was installed, as required, to control seepage and sloughing generally occurring at about 9 m and 16 m deep. The steel rebar cage was erected and installed by a crane set up beside the drill rig. Concrete was poured in two stages using free fall concrete placement method. Figure 5 is an aerial view of the site showing the drill rig and crane used during caisson installation. Figure 6 shows steel liner installation.

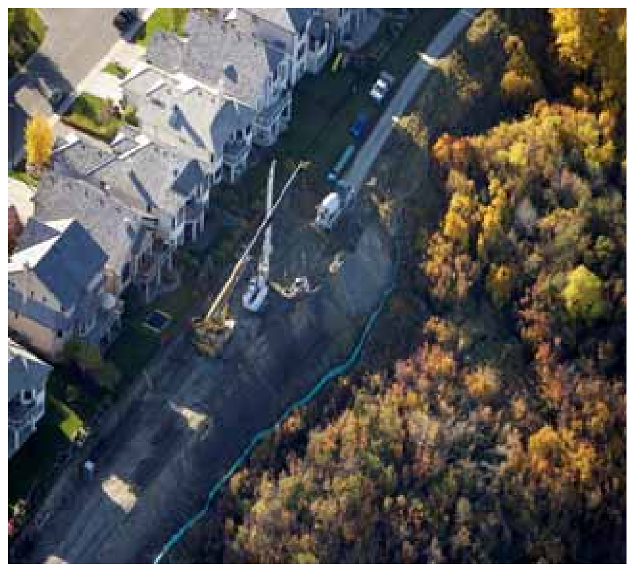

Figure 5. Aerial photograph of the site showing drill rig and crane working during caisson installation

The ground anchors consisted of double corrosion protected (DCP) DYWIDAG bar anchors. A manufacturer's bond breaker was installed within the unbounded zone. The DCP bar anchor minimum ultimate strength is 835 kN and average cross section area is 806 mm^2. In general, the full length of the ground anchor was tremie grouted using non-shrink grout mix with a minimum 28-day compressive Strength of 30 MPa. Provisions for post grouting of the bond zone were incorporated by installing a manufacturer's post grouting tubing into the bond zone. The anchorage cover at the top of the ground anchor consisted of a steel galvanized cover filled with corrosion inhibiting compound to improve corrosion performance of the anchor system. Figure 7 shows ground anchor installation works at the south end of caisson wall.

Figure 6. Steel liner installation to control seepage and sloughing during caisson wall construction

Figure 7. Ground anchor installation at the south end of caisson wall

During drilling for installation of caissons and ground anchors, soil and bedrock conditions were confirmed to be similar to those encountered during the geotechnical investigation.

6.1 Anchor Testing

All ground anchors were proof tested to 1.5 times the design load of 285 kN prior to locking off. Lift-off tests were subsequently performed on selected ground anchors after locking off to confirm that the lock off load satisfied the design load requirements. The proof tests and lift-off tests complied with contract specifications and no post grouting or pressure grouting was required.

Performance testing was carried out on two selected ground anchors. Performance testing was carried out to verify assumptions made during the design stage with regard to the working bond stress sustainable by the ground and to confirm that these bond stresses would be attainable with the installation method and that excessive plastic creep will not occur with time. The performance tests complied with contract specifications and geotechnical requirements.

Performance testing was conducted by incrementally loading the anchor according to the following schedule:

AL = Alignment load =0.1DL
DL = Design load for the anchor = 285 KN
Cycle 1: AL, 0.25DL (hold 10 minutes), AL
Cycle 2: AL, 0.25P, 0.50DL (hold 30 minutes), AL
Cycle 3: AL, 0.25 DL, 0.50 DL, 0.75 DL, (hold 30 minutes), AL
Cycle 4: AL, 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL (hold 45 minutes), AL
Cycle 5: AL, 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL (hold 60 minutes), AL
Cycle 6: AL,0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, 1.50 DL (hold 300 minutes), AL

Preproduction testing or design testing carried out to investigate in advance the performance of the production anchors was not conducted due to limited site access and project costs considerations. An anchor was considered acceptable when the recorded elastic elongation of the tendon exceeded 60 % of the theoretical elastic elongation of the free stressing length but did not exceed 100 % of the theoretical elastic elongation of the free stressing length plus 50 % of the bond length. The criterion was applied to all test cycles of the performance tests. The creep movement at peak load for each test cycle was considered acceptable when not exceeding 2.0 mm over log cycle of time.

The results of one of the two performance tests carried out during the construction stage of the project are shown on Figure 8. The other test yielded similar results. Figure 8a shows a plot of load versus corresponding movement reading for each cycle. Figure 8b indicates that the movement is within the minimum and maximum limits of the acceptance criterion. The observed net movement, as a result of inelastic behaviour, is in all cases less than 2 mm. Figure 8c shows that the creep movement is less than 2 mm over the testing period for all loading cycles.

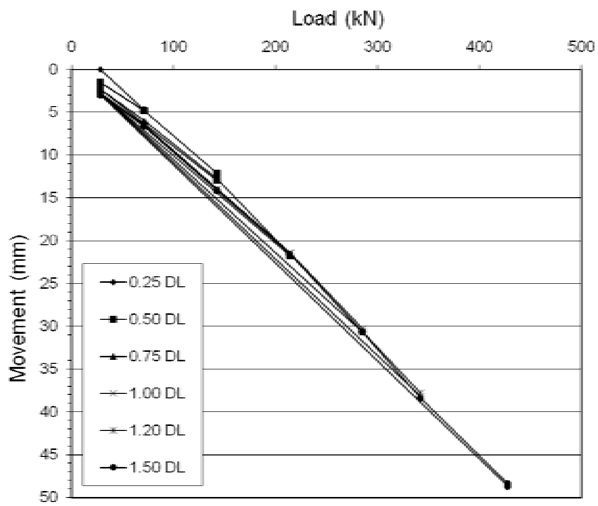

Figure 8a. Load versus movement reading for each cycle of loading

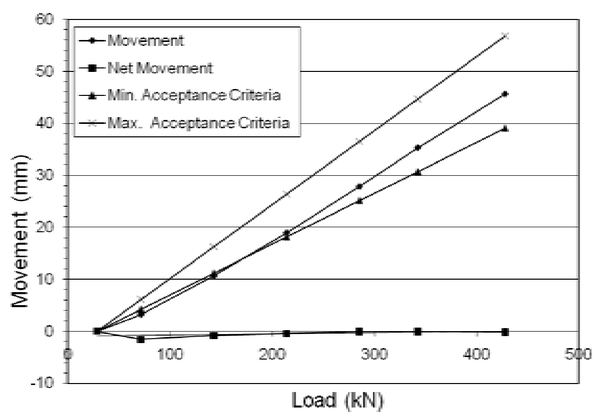

Figure 8b. Graphical analysis comparing ground anchor movement with acceptance criteria

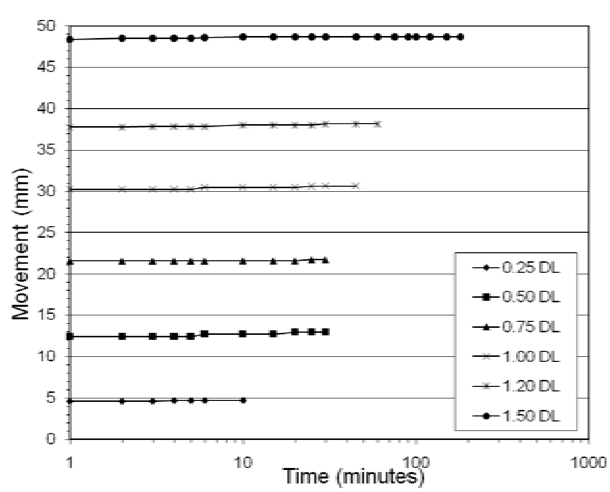

Figure 8c. Movement versus time for each cycle

6.2 Instrumentation

One slope inclinometer casing was installed within the south and north portion of the caisson wall. Both inclinometer casings were installed 23 m below the top of the caisson wall. Base line readings were taken in December 2008. The first set of readings was taken after anchor stressing on January 2009. Slope inclinometer readings corresponding to the south inclinometer casing are shown in Figure 6. The A and B directions indicated are the directions perpendicular and parallel to the caisson wall, respectively. Deflections observed in Figure 9 are considered to be associated with compaction activities behind the caisson wall. Additional readings are planned to be taken after the heavy rainfall season (usually June and July) in 2009 (i.e. prior to preparation of this paper).

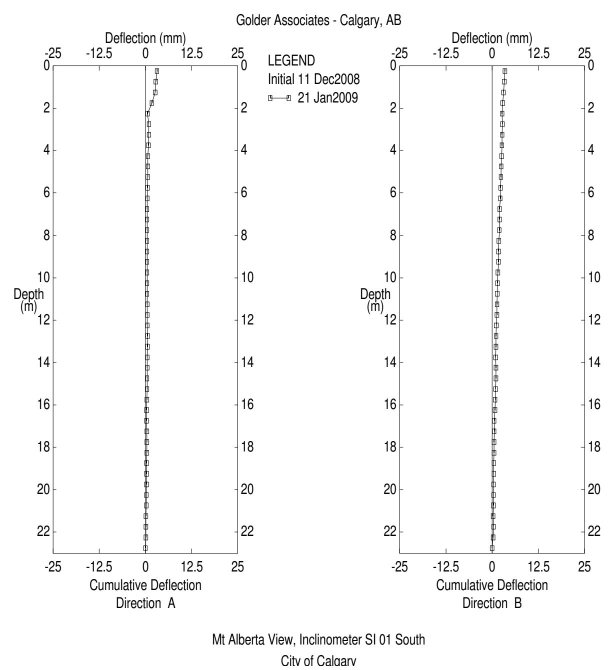

Figure 9. Slope Inclinometer Readings for the Inclinometer Casing within the South Portion of the Caisson Wall

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