STABILIZATION OF A HIGHWAY EMBANKMENT FILL OVER AN ARCH CULVERT USING STONE COLUMNS

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ABSTRACT
This paper describes the slope failures that occurred in the valley and road embankment slopes during the replacement of two large diameter SPCSP culverts with a 6 m high by 10 m wide precast concrete arch culvert. The road embankment is 24 m high and crosses Hamlin Creek in North Central Alberta. The embankment fill and its foundations consisted of high plastic clay. Several slides occurred on the marginally stable valley slopes during construction, as a result of temporary stockpiling of the excavated embankment fill on the valley slopes. When the embankment fill was within 2 m of finished highway grade a large slide occurred on the embankment west side slope above the culvert. The repair option chosen to stabilize the highway embankment consisted of removal of the slide material above the culvert crown and installation of stone columns on either side of the culvert to reinforce the fill zone. Movements continued in the fill after completion of the highway; however the rates of movements have been slowing down with time. In addition, a deep seated movement was detected below the culvert base originated from slope movement in the northwest slope. The paper describes the results of the geotechnical investigation, slide remediation with stone columns, stability evaluation and performance observations since completion of the repair.

RÉSUMÉ
Cet article présente le développement de glissements de terrain le long des vallées et d'un remblai routier pendant le remplacement de deux buses métalliques de grand diamètre avec une buse de béton préfabriqué de 6m de haut et 10m de large. Le remblai routier a une hauteur de 24 m et traverse Hamlin Creek, situé au centre nord de l'Alberta. Le remblai et ses fondations consistent en argile de haute plasticité. Plusieurs glissements de terrain se produisirent sur les pentes marginalement stables de la vallée pendant la construction du remblai à cause de l'entreposage provisoire de sol provenant de l'excavation des remblais pendant le remplacement des buses. Lorsque le remblai était à à peu près 2m plus bas que le niveau final de construction un grand glissement de son talus ouest se produisit juste au dessus de l'arche de béton préfabriqué. Le choix de la méthode de réparation pour stabiliser le remblai consista en l'excavation des matériaux du glissement situés au dessus de la buse de béton et la construction de colonnes en pierre des deux cotés de la buse pour renforcer le remblai. Les mouvements du sol ont continué à l'intérieur du remblai après la construction de la route mais le taux du déplacement a ralenti avec le temps. De plus un glissement de terrain profond dans la fondation au-dessous de la base de la buse en béton fut détecté après la construction. Ce mouvement profond voit son origine au glissement de terrain de la pente nord ouest de la vallée. L'article décrit les résultats de l'investigation géotechnique, la réparation des glissements à l'aide de colonnes de pierre, l'évaluation de la stabilité des talus et les observations de performance depuis la fin des réparations.

1. INTRODUCTION

This paper describes the slope failures that occurred in development of the valley slopes and embankment fill during the replacement of two large diameter SPCSP culverts under a 24 metre high roadway embankment across Hamelin Creek in North Central Alberta. The replacement culvert consisted of a 6 m high by 10 m wide precast concrete arch founded on a 1 m thick reinforced concrete base slab. The embankment fill and its foundations consisted of high plastic clay.

Construction of the new culvert was scheduled to be completed over the fall 2002 - winter 2003 to allow the new culvert to be operational by spring breakup.

Due to the constricted access across the valley and relatively steep road grades, the Contractor elected to undertake the highway excavation and culvert installation in a staged manner in order to accommodate the detour traffic within the existing highway embankment. An important aspect of the contract was that traffic for the public had to be maintained throughout the construction.

Slope failures occurred in the valley slopes during temporary stockpiling operations. These were dealt with during construction by removal of the temporary stockpiles, or in the case of the southeast slope, by filling across the valley to provide a toe buttress.

The last stage of construction involved rebuilding of the west (upstream) half of the embankment and was completed during spring 2003. When the embankment was within 2 m of final grade, a large failure occurred in the west embankment fill slope above the culvert inlet. A combination of excavation and replacement above the culvert and installation of stone columns on each side of the culvert was used to stabilize the fill and allow completion of the highway while maintaining traffic.

Ongoing movements have been monitored in the valley slopes and also within the west embankment fill. A crack was found within the culvert when the fill was nearing completion. This is believed to be related to deep seated sliding in the northwest slope that is seated below the creek level.

A program of slope and structure monitoring was implemented in early 2004 to determine the rate of ongoing movements. This information will be used to assess the need and method of remediation for the culvert structure.

The paper describes the slide history, results of the geotechnical investigation, slide remediation, stability evaluation and performance observations since completion of the repair. A companion paper provides a detail site history and also provides recommendations for future culvert investigations.

2. SITE CONDITIONS

Hamelin Creek is located in Northwestern Alberta approximately 60 km north west of the Town of Spirit River. A site plan showing the existing and replacement culverts is provided in Figure 1.

The Hamelin Creek valley is approximately 600 m wide from crest to crest and about 60 m deep. Highway 725 crosses the valley on an earth embankment up to 24 m high, with side slopes inclined at an average of 3H:1V.

Two existing large diameter CSP culverts conveyed flow from west to east under the highway embankment. The river flows are relatively high during spring breakup and debris carried in the flow was a problem for the existing culverts. The culverts were to be replaced with a 6 m high by 10 m wide precast concrete arch founded on a 1 m thick reinforced concrete base slab.

A geotechnical investigation undertaken during the design phase identified that the culvert was underlain by relatively competent stiff high plastic clay. The history of slope instability of the valley slopes was not identified during design and no special measures were specified for dealing with the potentially unstable slopes. At the time of the investigation no information was available regarding the type of culvert replacement, method of replacement or detour strategy.

3. HISTORY OF SLOPE INSTABILITIES DURING CONSTRUCTION

The first stage of construction during fall 2002 was to excavate the downstream half of the embankment, and install that portion of the culvert while maintaining a detour road over the upstream slope of the highway embankment.

The excavated material was temporarily stockpiled up to 10 m high on the southeast valley slope, and a deep seated slope failure occurred within the valley slopes soon after stockpiling in October 2002. The southeast slope was stabilized by removing a part of the stockpile and temporarily filling across the creek to provide a toe buttress to the slope. The low fall flows were handled through a temporary steel culvert pipe under the toe berm fill.

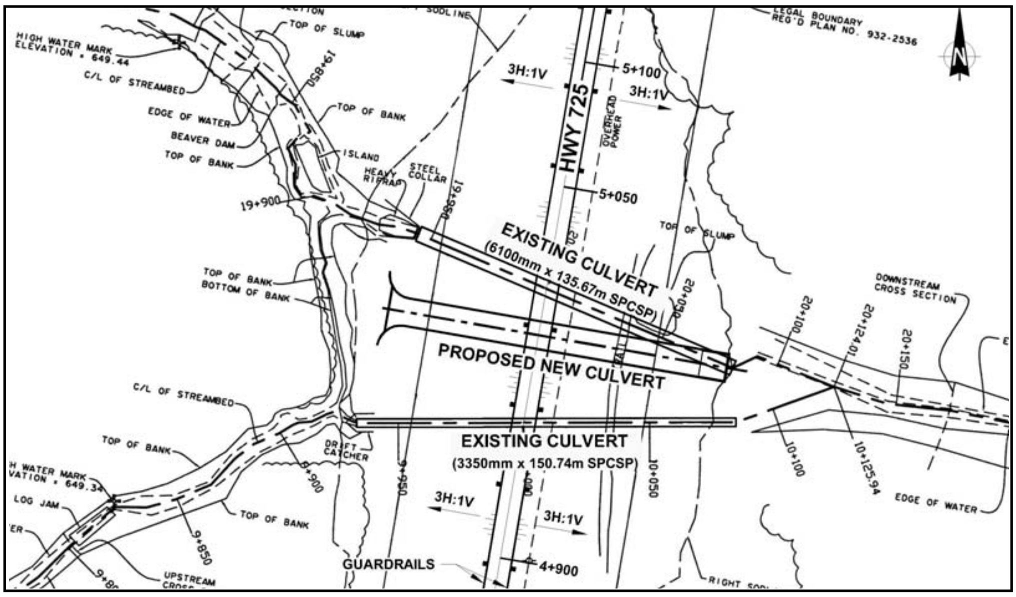

Figure 1. Site plan showing the alignment of existing and replacement culverts.

Once the downstream end of the culvert was constructed in late 2002, the detour road was relocated over the completed culvert to allow the Stage 2 (upstream half) culvert construction.

Excavation of the upstream half of the embankment commenced in January 2003. Once again, the excavated fill was temporarily stockpiled within the valley on the northwest valley slope, to avoid off-site hauling. A deep seated slope failure occurred in the northwest valley slope in February 2003. The slide scarp also extended into the existing embankment fill.

The slope was partially stabilized by removal of a portion of the stockpile and hauling off-site to a temporary stockpile. The slope movements subsided after removal of the stockpile, although the Contractor noted that the toe of the north slope continued to move inwards at a slow rate towards the culvert base that was under construction.

4. EMBANKMENT FAILURE

Backfilling of the second stage of culvert fill was undertaken during winter 2003 so that the culvert could handle the relatively large spring flows. The northwest valley slope was still creeping at the time that the second stage culvert backfilling started.

Backfilling proceeded initially at a rate of about 0.5 m per day, but increased to a rate of up to 0.8 m/day, and finally about 1.5 m per day for the last few metres of fill that were placed. The fill was hauled from a temporary stockpile about 1 km north of the culvert site. Efforts were made to haul unfrozen fill, however the construction was on a dayshift basis only, thus frozen fill, ice and snow became trapped in the fill during the fill placement, as found in the subsequent geotechnical investigation described in Section 5.

When the embankment fill was approximately 2 m below finished highway grade a large slide occurred on the newly constructed embankment side slope above the culvert inlet in late May 2003. The side slope was constructed at a slope angle of 3H:1V and accommodated the detour road near the top of the slope. The back slope above the detour road to the hwy 725 base grade elevation was constructed at a slope angle of 1.5H:1V.

The cracking was first noticed on Saturday (May 24, 2003) at the detour road surface and at the side slope of detour road. On Tuesday (May 27, 2003) the fill failed forming an arc shaped scarp approximately 100 m long at the top of the fill. The scarp dropped approximately 1 m and cracks approximately 10 to 150 mm wide formed. Soil cracks were also observed at the south flank of the slide area.

A slight soil bulging at the top of the culvert inlet was noted that could be the toe of the slide. There was no evidence of deep seated failure below the culvert crown elevation and no distress or deformation was noted in the culvert.

No water seepage or springs were noted at the slope surface during site inspection. The shape and orientation of the cracks indicated that the slope movement was in a westerly direction coinciding with the culvert axis.

At the time of the failure, it was considered that the embankment failure may have resulted from one of the following conditions:

- Fast fill placement at high moisture content, which may have generated excessive pore pressure in the fill material bringing the effective stresses to unstable condition, or
- Fill placement in a pre-sheared area in the foundation clay reactivating a previous slide.

5. GEOTECHNICAL INVESTIGATION

A geotechnical investigation and monitoring program were initiated to determine the cause of the embankment failure and assist with remediation.

The results of geotechnical instrumentation monitoring indicated that the slide was seated in the recently placed embankment backfill material above the crown of the culvert and was still moving at a relatively fast rate.

It was also noted that the backfill around the culvert was well above Optimum Moisture Content and partially frozen (zone of scattered presence of ice and snow in the fill). The failure appeared to be at the interface between the unfrozen and partially frozen fill as shown by the soil profile in Figure 2. There was concern that failure could progress lower into the partially frozen fill during subsequent thawing.

Two direct shear tests conducted on samples of the clay fill gave peak strength parameters of $c' = 18$ kPa; $\Phi' = 18^o$; and $c' = 9$ kPa, $\Phi' = 20^o$. Residual strength parameters were $c' = 0$; $\Phi' = 11^o$ and 13^o. Atterberg limit tests yielded liquid and plastic limits of 56 and 19 %.

A triaxial test conducted on a sample of recompacted fill at 95 % of Standard Proctor Density gave strength parameters $c' = 13$ kPa and $\Phi' = 19^o$ and pore pressure response A parameter ranging from 0.53 to 0.69.

6. REMEDIATION OPTIONS

The following options were considered to stabilize the west embankment fill and rebuild the embankment:

- removing the entire fill to the base of the culvert and rebuilding using geogrid reinforcement combined with a toe gravel trench;
- installation of a tangent pile wall at the toe of the slope; and

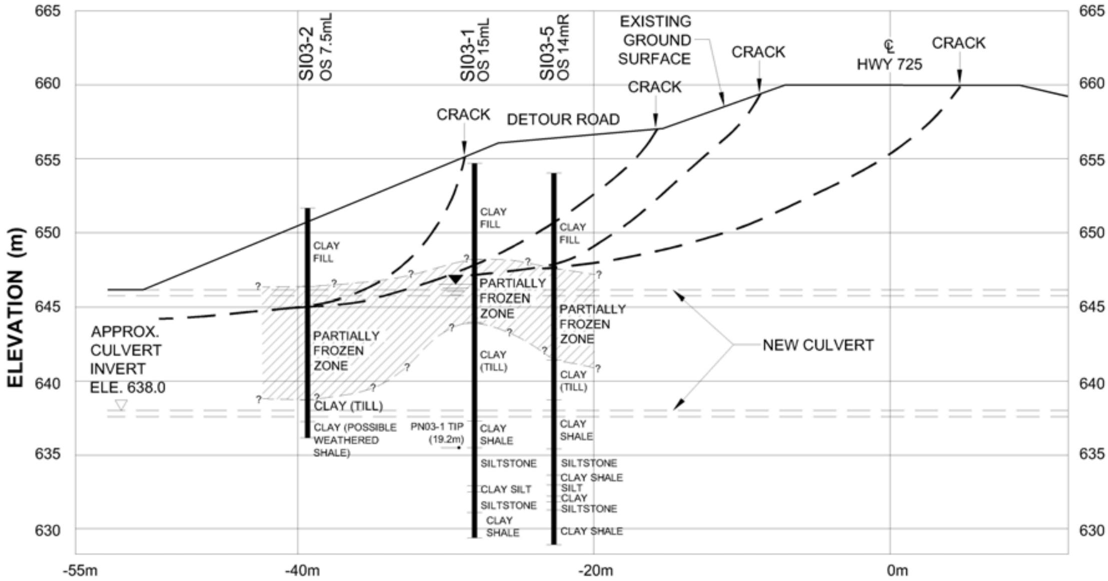

Figure 2. Soil profile of the west slide failure.

- partial removal of the embankment fill to the top of the culvert and installation of stone columns to reinforce the remaining (partially frozen) fill around the culvert.

Excavation and replacement of the entire backfill down to the base of the culvert was the preferred solution to remove the entire partially frozen fill; however, this would have required undercutting the toe of the valley slopes with significant risk of further slope movement within the northwest slope that had previously failed and was showing signs of creep.

Stability analyses indicated that installation of stone columns and reconstruction of the fill over the culvert would improve the factor of safety to about 1.2. The estimated cost was also comparable with the total excavation and gravel wedge solution.

Consequently, the repair option chosen consisted of removal of the slide material above the culvert crown, installation of approximately 300, 1.2 m diameter, stone columns on either side of the culvert to reinforce the frozen fill zone and reconstruction of the embankment fill above the culvert.

7. STONE COLUMN DESIGN AND CONSTRUCTION

The design of the stone columns reinforced fill is shown in plan and cross sections on Figures 3. Details of stone columns design are discussed elsewhere (FHWA 1983; Goughnour, et al. 1991; and Abramson et. al. 1995).

The embankment fill was removed down to the top of the culvert at a temporary slope of 2H:1V at the north and south ends and 3H:1V at the east end, due to concerns with slope instability of the existing highway. The detour road was moved to the east side of the highway embankment to allow the construction to proceed.

The construction pad for installation of the stone columns was graded from east to west to promote drainage (Figure 3C). A minimum of 4 m of fill was left directly over top of the culvert so that the construction equipment could move over the culvert.

The following installation specifications were used for installation of the stone columns:

- 1.2 m diameter stone columns were installed on a 2.2m by 2.5 m grid spacing drilled into the native soil below the base of the culvert to variable tip elevations as shown on Figure 3B;
- the stone column backfill consisted of 40 mm minus crushed gravel meeting Alberta Transportation Class 2 Des 40 (crushed granular base course material); and
- the gravel backfill was placed in approximately 1 m loose lift thickness. Each lift was backfilled by dropping a weight mounted to the Kelly bar of the piling rig, with approximately 8 drops per lift. At this point the gravel did not settle significantly after subsequent drops.

The existing fill was hauled off site, stockpiled, partially dried to obtain compaction to 95 % SPD within plus or minus 2 % of OMC. The moisture content limit was set at 22 %.

Excavation of the stone columns confirmed the presence of buried snow and ice within the lower part of the fill.

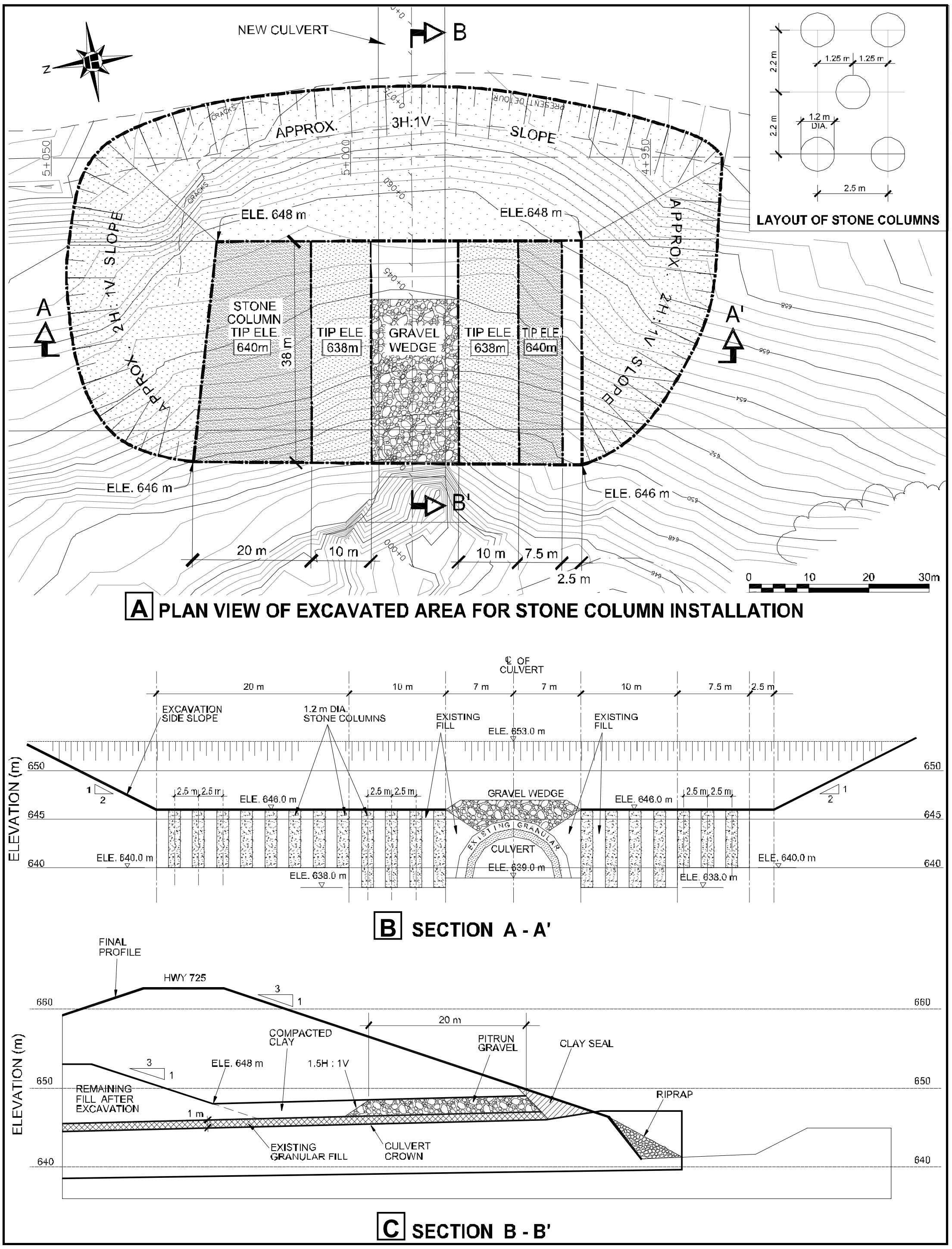

Figure 3 – Stone columns layout

Deep seated movements in the northwest slope accelerated during the excavation to the top of the culvert, confirming the choice of not removing the entire fill.

The stone column installation was carried out during September 2003 and backfilling commenced immediately.

The embankment was completed to design grade ready for gravel and asphalt completion at the end of 2003.

8. SLOPE MONITORING AND PERFORMANCE

Slope inclinometers and pneumatic piezometers were installed within the embankment fill and also in the northwest slope in June 2003, following the west embankment failure.

The locations are shown in Figure 4. The initial inclinometers 03-1 and 03-2 had sheared off at depths of 8.5 m and 6 m, respectively, before they could be initialized one day after installation, indicating the fast rate of slope movements. SI 03-3 and 03-4 installed in the northwest slope did not show any discernible movement over the initial several days.

SI03-5 was installed with sand backfill to allow immediate inclinometer initialization after installation avoiding the wait normally required for grout backfill to cure. This SI also sheared off, but the gross movements were confirmed above the culvert crown at approximate elevation 647.5m.

Slope movements in the northwest valley slope were detected at depths of 17.3 and 17.5 m (approx elevation 638.7 and 647 m). These movements increased during the embankment excavation in August 2003, and decreased once backfilling of the embankment was underway. A history of the northwest slope rate movements is shown in Figure 5.

These SIs in the west embankment were replaced at several stages during construction as they became inoperative due to excessive displacements.

A pneumatic piezometer was installed in between the stone columns to monitor construction pore pressures during embankment reconstruction. High pore pressures developed in the clay fill during backfilling and significant lateral spreading occurred within the stone column reinforced zone. The pore pressures have dissipated since construction as shown on Figure 6.

The west embankment fill initially underwent significant deformation during and following fill construction (Figure7). The movements have slowed down as shown on the rate of movement plot in Figure 8 and the current rate of movement is 82 mm/year.

Once the upper slope movements reduced, minor slope movements were detected in SI03-8 at elevation 634 m which was approximately 4 m below the culvert base. These deep movements were subsequently confirmed in SI 04-3.

When compared with the valley slope movements in SI03-3 and 03-4 in the northwest slope these are at the same approximate elevation indicating that deep seated movement of the north slope is also extending below the culvert.

A crack was noted in the wall of the culvert more or less at the junction between the Stage 1 and Stage 2 culvert construction once the embankment had been completed. Further investigation indicated that the crack extended through the 1 m thick base slab across the culvert, and to a lesser extent on the south wall of the culvert.

The deformations were consistent with rotation of the west end of the culvert resulting from the deep seated northwest valley slope movements.

A structure monitoring program was initiated in March 2004 to determine the direction and rate of movement of

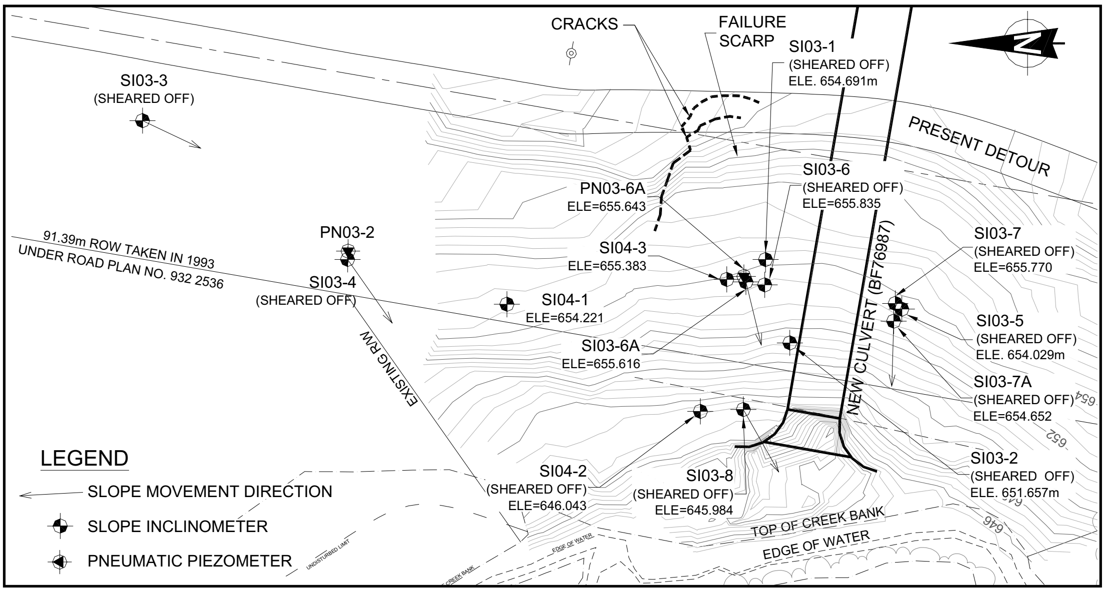

Figure 4. Site plan showing the location of the instrumentation (contour lines represent ground condition after west slide failure)

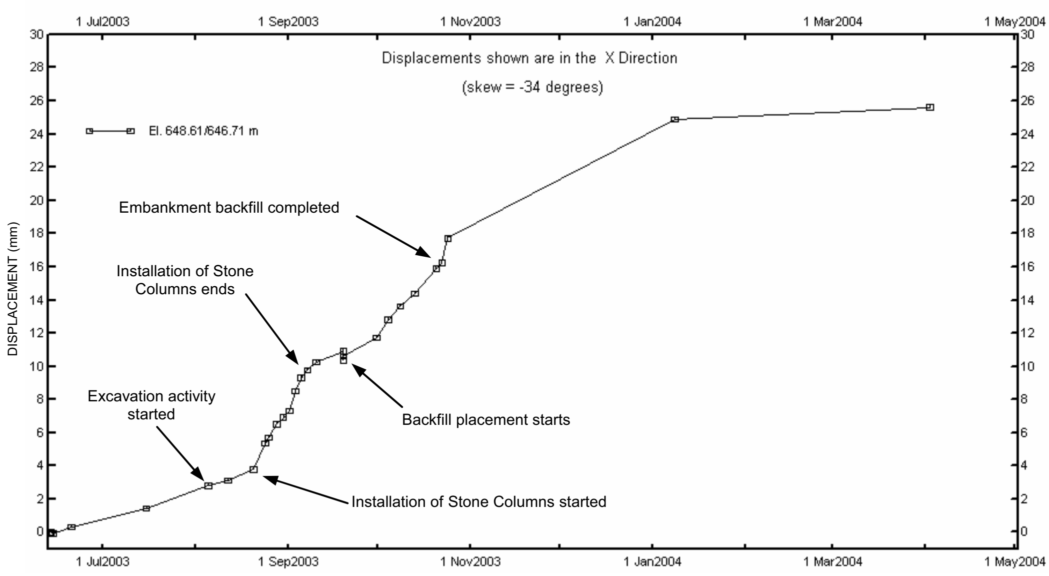

Figure 5. Northwest slope rate of movement with time (SI03-3).

the culvert structure. At the time of preparation of this paper, only the initial few sets of reading had been taken.

The results confirm that the west slope where the stone columns were installed is still moving but at a decreasing rates towards its stabilization. Yarechewski and Tallin (2003) reported a similar project using stone columns to stabilize a slope observed that slope movement was still occurring 2 year after construction but at steady decreasing rates.

The northwest slope (deep seated movement), which has not been remediated by stone columns appears to be more active and both SIs 03-3 and 03-4 sheared off in spring 2004. The structure integrity of the culvert is not presently in danger. Future remediation of the culvert may consist of grouting the cracks once the movements have reduced.

9. CONCLUSION

Excavation and reconstruction of the embankment fill combined with installation of 300 stone columns was used to stabilize the embankment fill and allow the completion and opening of the highway. Traffic for the public was maintained throughout the construction period.

Costs for the slope reconstruction were approximately $1.5 M.

Results of the recent monitoring indicate that the rate of the embankment slope movement is continuing to decrease.

Deep seated movements below the creek level resulting from the unstable northwest valley slopes causing movement of the culvert and will need to be addressed.

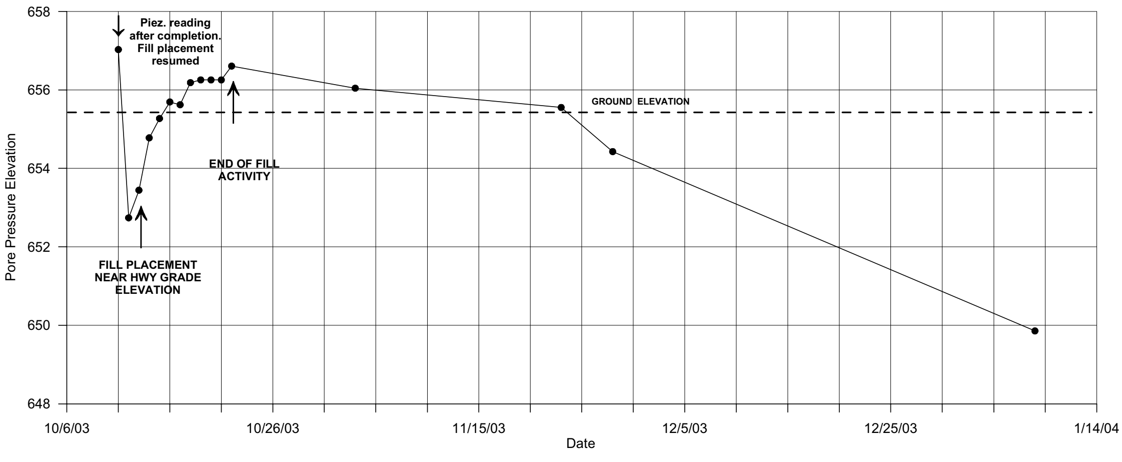

Figure 6. Pore pressure measurement during and after construction.

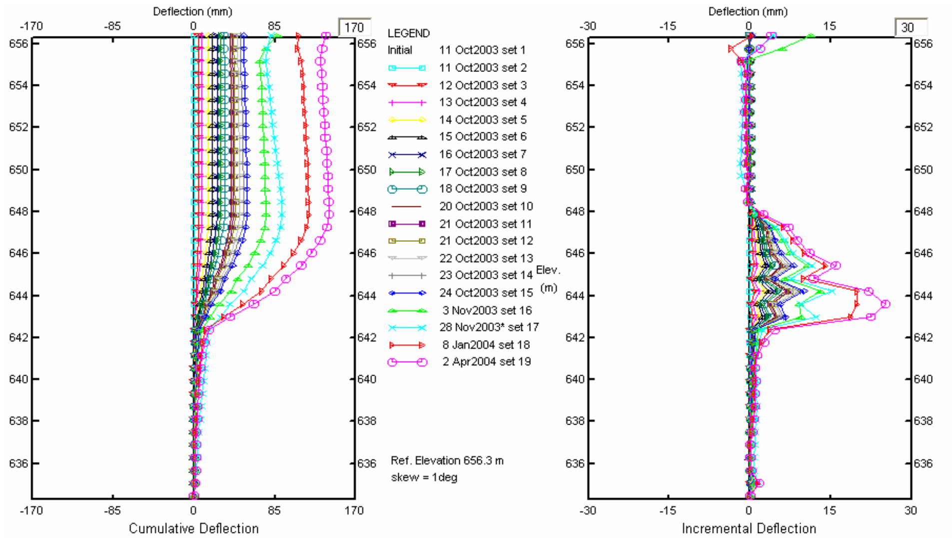

Figure 7 – Slope indicator (SI03-6A) readings at resultant direction.

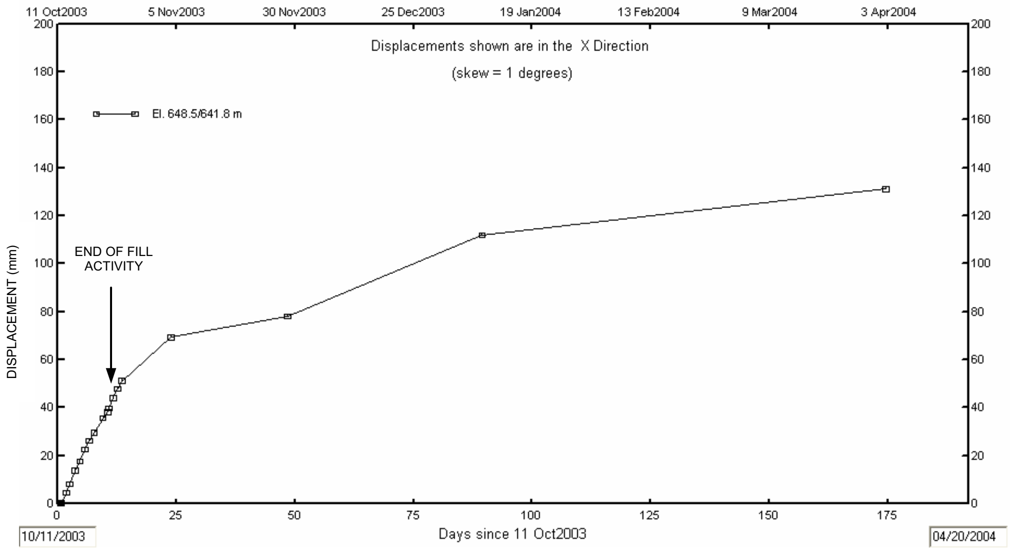

Figure 8 – West embankment slope rate of movement (SI03-6A).

ACKNOWLEDGEMENTS

The authors wish to acknowledge Alberta Transportation Peace Region personnel for their support throughout the project and for their permission to publish the results, and Mr. Dave Regehr, P.Eng of Thurber Eng. Ltd. who was full time on site during the remedial measures. The authors acknowledge the contribution of EXH Eng. during the embankment reconstruction.

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