GEOTECHNICAL ASPECTS OF THE KING STREET INTERCHANGE CONSTRUCTION, FORT McMURRAY, ALBERTA

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ABSTRACT: A $22 million (CAN) grade-separated interchange was constructed at the intersection of Highway 63 and King Street in Fort McMurray, Alberta. The interchange conforms to a tight diamond overpass configuration with an adjacent raised “European” style roundabout that ties into the urban local road system. The primary geotechnical concern was the stability of the west overpass headslope and associated North-South access ramp which were constructed over colluvium material at the base of an historically unstable valley slope. Secondary issues included stability of the valley wall, design of a mechanically stabilized earth (MSE) wall to support the east edge of the 11 m high traffic circle embankment and control of surface and subsurface water. Mitigative design features included: judicious siting of the interchange; restrictions on cuts into the valley slopes; wick drains; a blanket drainage layer; tight control over embankment fill placement rate based on pore pressure and lateral deflection monitoring; geogrid reinforced embankments; and a sub-surface cast-in-place concrete pile retaining wall along the west headslope. The paper provides an overview of the salient geotechnical issues for the investigation; design and construction of the interchange, with an emphasis on the west headslope stability considerations.

Key words: interchange, embankments, colluvium, slope stability, wick drains, instrumentation, pile wall

RÉSUMÉ: Un inter change routier qui coûte $ 22 Million (CAN) a été construit au croisement de Highway 63 et King Street dans la ville de Fort McMurray en Alberta. Le genre de l’inter change ressemble à une configuration de diamant étroit adjoint à une intersection qui consiste à un cercle de trafic élevé (genre Européen) qui s’unit avec le système routier urbain local. Le problème primordial était la stabilité de la rampe d’accès du pont de l’ouest et la rampe d’accès nord-sud qui ont été construit sur des matériaux d’éboulis qui se trouve à la base d’un versant historiquement instable. Les problèmes secondaires étaient la stabilité de la pente de la vallée, la construction d’un mur de sol mécaniquement stabilisé (MSE wall) qui supportera la partie est de la rampe du cercle élevé; et le contrôle des eaux de surfaces et souterraines. Les mesures de contrôle consistent à la sélection judicieuse de l’emplacement de l’inter change; aux restrictions d’excavations dans le versant de la vallée; à l’installation de mèche de drainage vertical (wick drains) et d’une blanquette de drainage horizontal; au contrôle de la vitesse de placement du remblais basé sur pression des pores du sol et des mesures de déflection latérale; la construction des rampes avec du geogrid renforcé; et un mur souterrain de rétention formé de pieux de béton armé pour supporter le pente frontal du remblais du pont ouest. Cet article présente une revue des issues importantes du programme d’investigation; de l’analyse et de la construction de l’inter change avec considération spéciale sur la stabilité de la pente frontal du pont ouest.

Mots clés : inter change, remblais, éboulis, stabilité de talus, mèche de drainage, instruments, mur de rétention.

1. INTRODUCTION

Highway 63 provides the primary link between the northeastern Alberta oilsands mines, located north of Fort McMurray and the rest of Alberta. Within the city of Fort McMurray the highway is a four lane twinned structure that provides the primary north-south link through the city. An at-grade intersection was located at King Street. Approximately 20,000 vehicles per day pass through this intersection with a peak turning movement of about 400 vehicles per hour (vph), projected to increase to 2000 vph within 10 years. Capacity limitations and safety concerns precipitated the need for a grade-separated interchange.

The project site is constrained by geometric and geotechnical issues, which required an innovative interchange design. This paper presents a summary of the physical and geological setting, past construction problems, details of the most recent site investigation, design and construction issues, instrumentation details and monitoring results, and mitigative work undertaken for the west headslope and north-south ramp embankment fills of the overpass.

2. PHYSICAL SETTING AND CONSTRAINTS

The project site is located at the bottom of the Clearwater River valley slope as shown on Figure 1. The hillside is locally known as Beacon Hill in the vicinity of the project site. A tributary of the Clearwater River, the Hangingstone River, forms the north and east boundary of the project site, while the Clearwater River valley slope forms the south and west boundary. To the south the highway rises at a 6% grade out of the 70 m deep valley to the upland terrain.

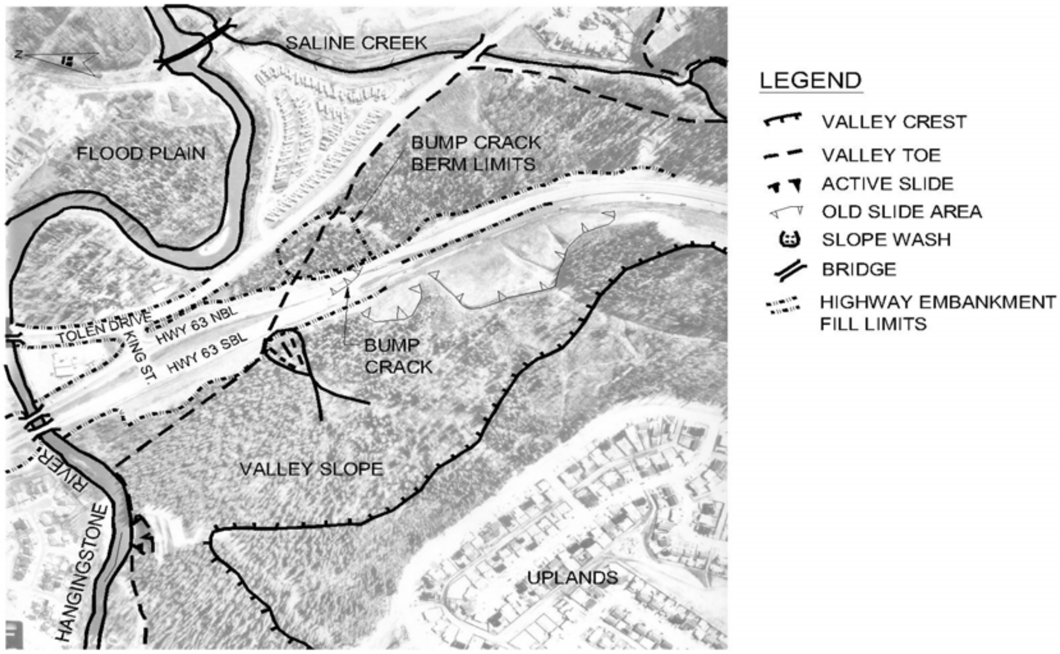

Figure 1. Site location plan showing topographical constrains and existing features prior to interchange

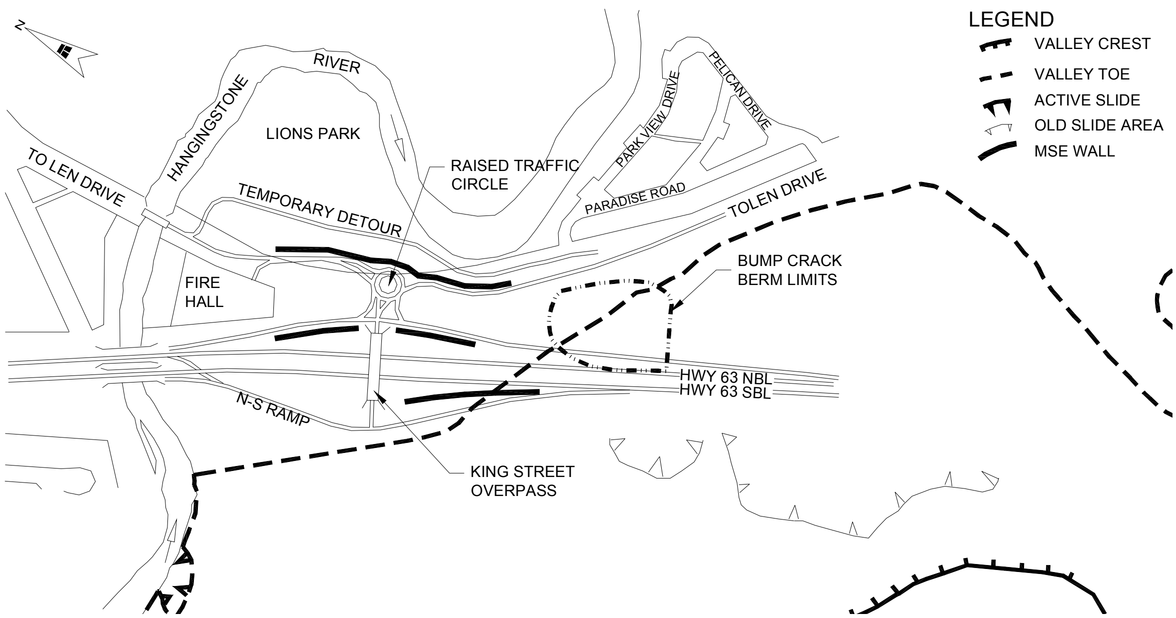

Figure 2. Design layout selected for the interchange

Geometric constraints were related to the presence of three nearby bridge structures, an adjacent arterial local roadway named Tolen Drive, adjacent steam banks of the Hangingstone River, access requirements to the regional fire hall, and limited traffic weaving and turning movement storage capacity.

3. GEOLOGICAL SETTING AND GEOTECHNICAL CONSTRAINTS

The project site is located along a flank of a partially infilled glacial meltwater channel within which the present day Clearwater River flows. The meltwater channel incised deeply through Pleistocene deposits, primarily glacial tills, Cretaceous period Clearwater Formation marine clay shale and sandstone, and into McMurray Formation oilsand. Devonian Limestone underlies the

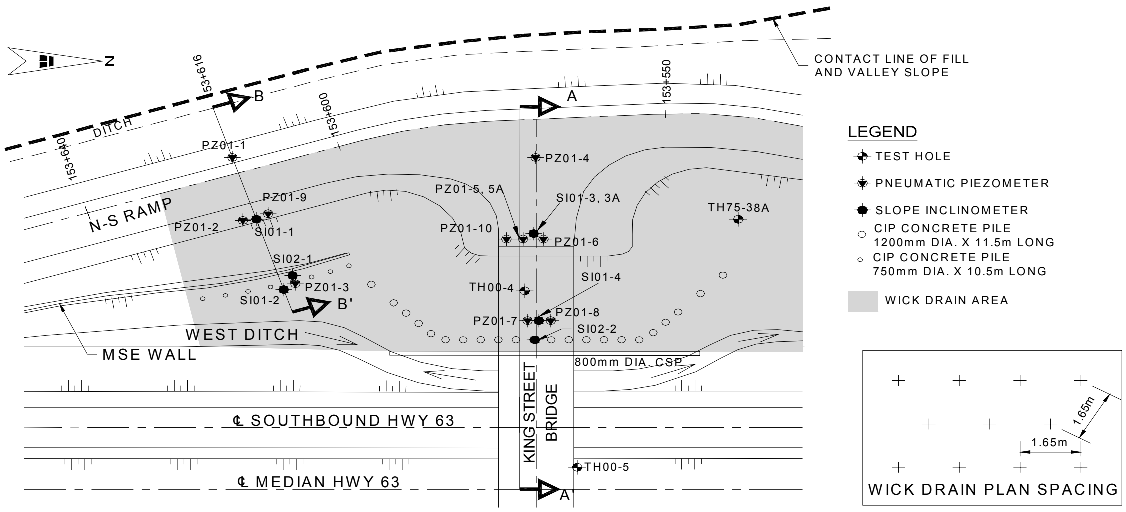

Figure 3. Plan of King Street west headslope area

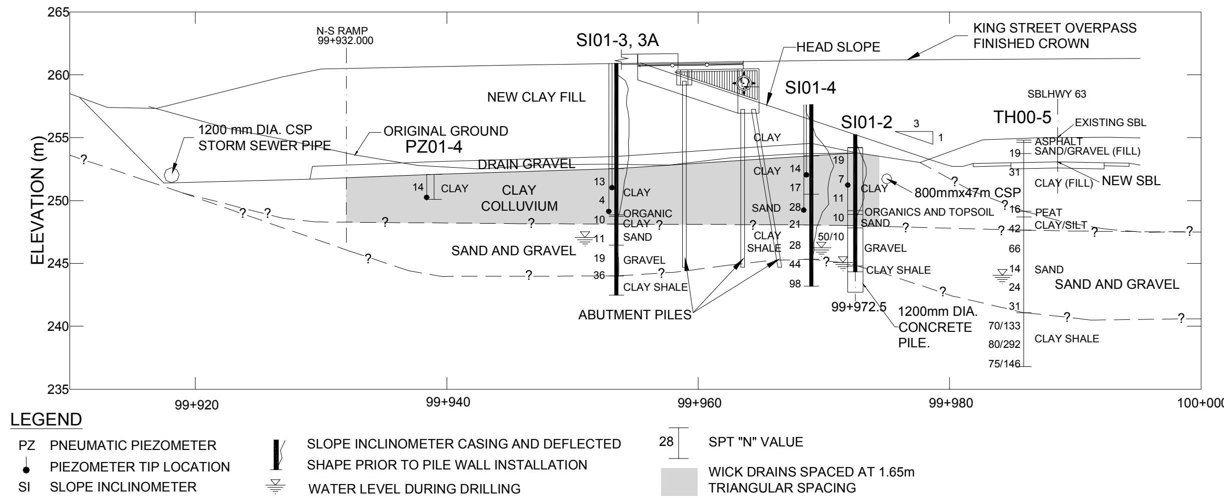

Figure 4. Stratigraphic cross-section A-A' through king street west headslope

Cretaceous deposits. The upland area defined by the Clearwater River valley and its tributary Hangingstone River valley forms Beacon Hill. The local geology is dramatically exposed at an eroded cliff along the Hangingstone River, approximately 200 m from the project site.

The meltwater channel has infilled with alluvial sand, silt and gravel deposits. Along the flanks the alluvial deposits are interbedded and mixed with colluvium materials derived from the clay till and bedrock.

The Clearwater Shale and clayey colluvium derived from the shale are known to be prone to slumping. Sharma, 1978, and Miller, 1985 provide reviews of geotechnical problems related to highway construction along the Beacon Hill area. At least five local areas of instability were encountered during the initial highway construction in 1966 or subsequent twinning construction in 1977. A specific instability, known as the "Bump Crack", which had been stabilized by constructing a significant toe berm (Thurber Engineering Ltd. 1991), was located within the project footprint.

Geotechnical constraints in the proposed interchange site were mainly related to the presence of weak clayey colluvium soils covering the lower portion of the valley slope.

The interchange design options available were severely limited by these constraints, most notably by the restriction on excavation into the valley slope and Bump Crack toe berm area. Conventional trumpet and cloverleaf interchange templates were unable to accommodate this constraint.

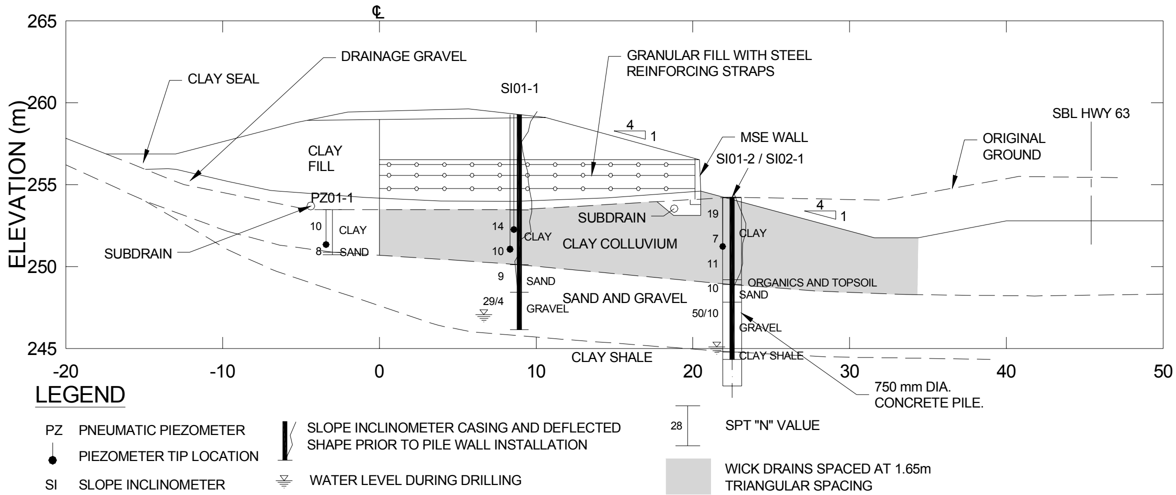

Figure 5. Stratigraphic cross-section B-B' through N-S ramp, Sta. 153+616

4. INTERCHANGE DESIGN LAYOUT

The selected design layout, shown on Figure 2, consisted of a tight diamond interchange with an elevated offset European style roundabout. This design was chosen because it accommodated the directional traffic movements with minimal excavation into the valley slope, and minimal impact on the Bump Crack berm stabilization works and adjacent river.

The design incorporated shallow MSE walls along the overpass ramps in order to reduce the overpass approach fill requirements and loading imposed on the colluvium. Highway 63 was lowered by about 2 m in order to meet geometric constraints and limit the fill placed onto the Bump Crack stabilization berm. An 11 m high MSE wall separates the traffic circle intersection from the lower level Tolen Drive and reduces encroachment of the interchange on the Lion's Park area. High load traffic will be accommodated by using the overpass approach ramps with gated crossing movements.

5. GEOTECHNCIAL INVESTIGATION

The geotechnical investigation included a detailed airphoto review and engineering site reconnaissance. Test Hole drilling was also carried out by conventional solid stem auger drilling at the locations shown on Figure 3. Disturbed samples were retrieved using Standard Penetration Testing (SPT) and grab-sampling methods. The interpreted soil stratigraphy along cross-sections through the proposed west headslope of the King Street overpass and through the north-south ramp are shown on Figures 3 and 4, respectively. The drilling indicated that the colluvium layer beneath the west headslope and north-south ramp varied from about 3 to 6 m in thickness.

The colluvium consisted primarily of high plastic clay with a Liquid Limit of 59%, Plastic Limit of 23% and natural moisture content of 26% to 42%. SPT "N" values ranging from 4 to 28 blows/300 mm of penetration corresponded to a wide range in stiffness of soft to very stiff. Later excavations into the colluvium during construction for installation of storm sewer pipes revealed that the clay colluvium contained pre-sheared (slickensided) surfaces indicative of past movement.

6. GEOTECHNCIAL DESIGN CONSIDERATIONS

Given the history of instability in the general area of the project site and the presence of the weak colluvium clay layer below the west part of the site, special attention was paid to assessing the stability of the west headslope and north south ramp embankments. At the planning stage of the project a very tight construction schedule was demanded, and as a consequence the short-term stability and post construction settlements of the high embankment fills and bridge headslopes were also of primary concern. The assessments for these issues are discussed below.

Slope stability analyses were carried out to assess the short and long term stability of the embankments. Drained analyses were employed using soil parameters selected based on experience from the previous slides with minor adjustments for local site variations as indicated by comparison of laboratory index test results. A partially remolded friction angle of 14 degrees and effective cohesion of zero were selected for the strength parameters of the clay colluvium. For the short term (end of construction) stability analysis, A Bbar value of 0.7 was assumed for the saturated zone below the water table and an Ru value of 0.2 was assumed for the zone above the water table.

Based on the results of these analyses, the following geotechnical design measures were incorporated into the design of the embankments to provide short and long term minimum factors of safety against slope instability failure of 1.5 and 1.3, respectively:

- Wick drains (Amerdrain 407) were installed through the existing clay fill and clay colluvium layer on a 1.65 m equilateral triangular grid spacing to accelerate dissipation of the construction induced pore water pressures. The wick drain spacing was designed based on a Coefficient of Consolidation, C_v of $2x10^{-7}$ m^2/sec and a required 80% dissipation of construction induced pore pressures over a 90 day period. It was assumed that with the wick drains in place the Bbar value used in the short-term stability assessment could be reduced to an equivalent value of 0.3.

- The tops of the wick drains were covered with 1 m thickness of free draining gravel layer. Perforated subdrain pipes, surrounded by washed gravel and non-woven geotextile, were installed in any low areas of the drainage gravel to collect potential drainage and seepage water;

- The west headslope was constructed with an inclination of 3H:1V and all embankment side slopes were constructed at 4H:1V where space permitted;

- Due to limited available space, a two metre high MSE retaining wall was incorporated mid-height along the south portion of the east sideslope of the north-south ramp embankment. The reinforced structural fill zone of the wall was extended back to the centreline of the ramp to provide additional bridging strength over the underlying colluvium layer;

- A maximum allowable fill placement rate of 1 m per week was specified in the contract for construction of the west headslope and north-south embankment fills;

- The overall designs of the interchange ramps and highway embankments were reviewed to provide a graduated stepped fill extending from the valley slope eastward to the flat lying floodplain. Restrictions were placed on the highway design regarding permissible maximum lowering of the existing highway lanes. In addition, no cutting was allowed into the existing Bump Crack toe berm. The berm was extended by placing additional fill in a low area at the north end of the berm;

- Instrumentation, consisting of slope inclinometers and pneumatic piezometers, was installed to assess the effectiveness of the design measures and specified fill placement rate using the Observational Approach.

7. INSTRUMENTATION MONITORING RESULTS

The deflected shapes of the slope inclinometers at the end of fill placement at the two instrumentation cross-sections are shown on Figures 4 and 5.

A plot of piezometric elevation versus time for P01-6, located under the crest of the west headslope with the tip installed in the clay colluvium, is shown on Figure 6. The corresponding elevation of the fill placed at this location is also shown on Figure 6. The pore pressure response to fill loading in this piezometer corresponded to an equivalent Bbar value of 0.25. Based on Figure 6, it took approximately 150 days for the wick drains to achieve 80% dissipation of the construction induced pore pressure from the clay colluvium.

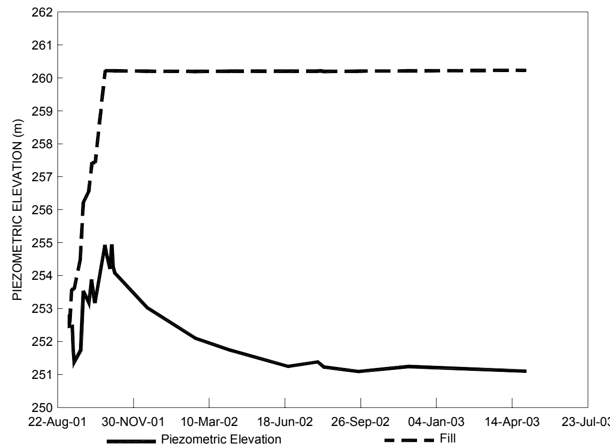

Figure 6. Porewater presure dissipation in piezometer P01-6 with tip in clay @ EL 251.03m

The slope inclinometers (SI's) indicated that the clay colluvium was squeezing laterally in an eastward direction in response to the fill loading. A plot of ground displacement versus time for SI01-3A located behind the crest of the west headslope fill is shown on Figure 7. During the embankment construction, fill placement was paused after each meter of fill height to allow the creep rate to diminish before the next meter of fill was placed. However, after the fill reached the design maximum height, a steady rate of about 25 mm/year was noted in the clay colluvium layer which did not diminish with time. Ongoing creeping of the headslope was considered unacceptable do to the potential impact that long term movements could have on the bridge structure.

The noted creep rate corresponded to a strain rate of $6.5x10^{-3}$ when taken over the thickness of the clay layer. Based on previous work by Crooks et al (1986) it was assessed that this strain rate corresponding to an overall slope stability factor of safety (FOS) of about 1.15 and effective friction angle of 10 degrees for the slickensided clay colluvium. Since this FOS was less than the desired minimum value of 1.3, it was decided that further stabilizing measures would be required to increase the FOS and stop the lateral creep movements before bridge construction could be carried out.

A pile wall, as described in the following section, was installed in February 2002. Additional SI's (SI02-1 and SI02-2) were installed in two of the piles to monitor the response of the piles to loading. As shown Figure 7, the piles were successful in significantly reducing the creep rate of the colluvium to the point where the abutment piles (driven steel H piles) could be installed. The slope experienced some additional lateral movements during installation of the abutment H piles in July 2002, however the movements halted immediately after pile installation which allowed bridge abutment and deck construction to proceed.

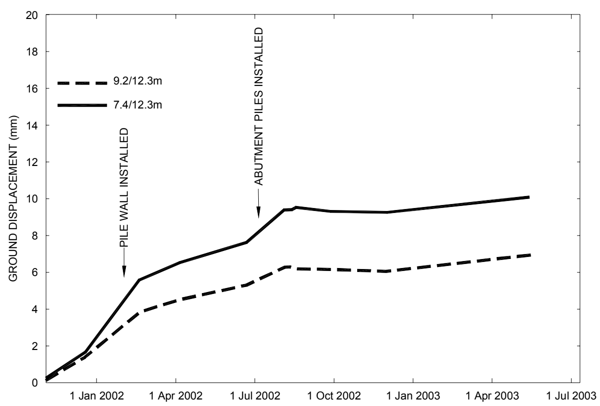

Figure 7. Ground displacement versus time in SI01-3A

8. PILE WALL

A subsurface pile wall was identified as the most practical and timely solution for providing additional support to the weak clay colluvium foundation soil underlying the embankment fills. The plan and cross-section arrangements of the piles are shown on Figures 3, 4 and 5. The piles consisted of cast-in-place bored reinforced concrete piles. The piles installed around the nose of the headslope fill consisted of 1200 mm diameter piles, 11.5 m in length, designed to provide an additional 175 kN/m (unfactored) of lateral support to the slope.

Along the front and front edges of the headslope the piles were installed at 2 pile diameters centre-to-centre spacing, while further back along the sides of the headslope and along the toe of the MSE wall, the piles were installed at 4 pile diameters centre-to-centre spacing. The piles installed along the toe of the MSE wall consisted of 750 mm diameter piles, 10.5 m in length, designed to provide an additional 50 kN/m (unfactored) of lateral support to the slope.

In the design of the piles, it was assumed that the portion of the pile located in the clay colluvium was unsupported and was required to resist a load imparted by the creep deformation of the colluvium clay. This load was assessed using a limit equilibrium slope stability analysis as the external force that would be required to bring the FOS up to 1.3. The length, diameter and reinforcing of the piles were designed to provide sufficient toe support in the gravel and shale and limit deflections to about 30 mm at the head of the piles.

9. COSTS

The overall project costs were about $22,000,000, which included the interchange, MSE walls, improvements to bridges, drainage improvements, utility relocation and other associated costs. The total cost of the geotechnical work was about $935,000 as summarized in Table 1.

Table 1 Geotechnical costs

Item	Quantity	Amount
a) Gravel Drainage Blanket		$450,000
b) Wick Drains	9056 m	$45,000
c) Extra Reinforcement in the N-S MSE wall ramp		$50,000
d) Investigation, Instrumentation and monitoring	NA	$140,000
e) Pile Wall	30 Piles	$250,000
TOTAL		**$935,000**

These costs do not include the MSE walls.

10. DISCUSSION AND CONCLUSIONS

This paper summarized the geotechnical aspects of the design and construction of a grade separated interchange in a location governed by geometric and geotechnical constraints. The main geotechnical constraint was the requirement to construct an overpass bridge abutment and associated access ramp embankments over a weak sloping clay colluvium deposit.

A key consideration in the success of the project was the knowledge of past area instabilities, early identification of the main geotechnical constraints in the project area and close corroboration between the Prime Consultant, Owner and the Geotechnical Consultant in optimizing the layout and design of the interchange to accommodate the geotechnical constraints

The use of wick drains and a gravel drainage blanket combined with a prescribed rate of fill placement was successful in controlling construction induced pore water pressures within tolerable levels and allowed the north-south access ramp to be constructed under a tight construction schedule.

This paper highlights the importance of geotechnical instrumentation for tracking the response of foundation soil to fill loading. In this case the results of the instrumentation monitoring formed the basis for fine tuning the allowable fill placement rate, and ultimately identified the need for additional stabilizing measures, consisting of a pile wall to stop creep ground movements prior to bridge construction.

The cost of the geotechnical measures was about 4.25% of the total construction costs which may serve as an approximate guideline for similar projects in the future.

Figure 8 shows the interchange as it looked in Fall 2002.

Figure 8. Interchange under construction (Fall, 2002)

11. ACKNOWLEDGEMENTS

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