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APPROACH FILL DESIGN OF NORTH SASKATCHEWAN RIVER BRIDGE

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ABSTRACT

A new ring road being constructed for the City of Edmonton includes a crossing of the North Saskatchewan River in the southwest corner of the city. Downcutting of the valley during post-glacial time exposed Cretaceous clay shale in the base of the river valley, which was subsequently covered by alluvial deposits. The clay shale contains bentonitic layers and is subject to pre-shearing, thus slope stability for the bridge headslopes was a major issue. To better assess the response of the underlying bedrock, a test fill was constructed where both pore pressures and slope movements were monitored. It was determined that large diameter concrete reinforcing piles provided the most cost-effective option. An observational approach was adopted which resulted in significant savings to the Owner.

RÉSUMÉ

Une nouvelle route périphérique construite à Edmonton comprend un pont traversant la rivière North-Saskatchewan dans la partie sud-ouest de la ville. Une couche de schiste argileux du crétacé située à la base de la vallée a été exposée durant la période post-glacière et a ensuite été recouverte par des dépôts alluvionnaires. Le schiste argileux, qui contient des couches de bentonite, est sujet à des contraintes de pré-cisaillement. La stabilité de la pente de la vallée est donc un aspect important à considérer pour maintenir l'intégrité du pont. Afin de mieux établir le comportement du roc support, un remblai a été construit dans on a suivi l'évolution des pressions interstitielles et du mouvement de la pente. Il a été établi que l'option offrant le meilleur rapport coût-efficacité consiste en un renforcement à l'aide de pieux de grand diamètre. Une approche observationnelle a été adoptée pour minimiser les coûts des pieux afin de permettre la réalisation d'importantes économies pour le propriétaire.

1. INTRODUCTION

The portion of the CANAMEX North-South Trade Corridor being constructed by the Province of Alberta includes a ring road around the City of Edmonton, which is referred to as Anthony Henday Drive. The ring road alignment includes a crossing of the North Saskatchewan River in the southwest corner of the city. Figure 1 presents a general location plan for the site.

A 360 m long bridge was proposed for crossing the North Saskatchewan River. The short term design for the crossing included two lanes in each direction with an ultimate design of a total of eight lanes. To accommodate the future lane widening, the structural design comprised two separate bridge structures at a total cost of about $33 million. Significant approach fills for the proposed bridge structures were required on both sides of the river. Based on the proposed vertical alignment, the maximum difference in height from the crest of fill to river level is approximately 35 m on the west side and 28 m on the east side. These fill heights are among the highest in the Edmonton area.

The width of the embankment at the crest is approximately 55 m, with side slopes of 4H:1V. This equates to a footprint width of approximately 200 m on the floodplain that is present along the east side. Headslope angles for the embankments of 3H:1V were proposed, with a minimum setback of 20 m, for environmental considerations, from the crest of the riverbank to the toe of the approach fill.

Figure 1. Location Plan

The primary focus of this paper is the approach fill design for the east side of the river. Figure 2 presents a site plan with borehole locations at the proposed river crossing. Figure 3 presents a typical section for the east approach fill.
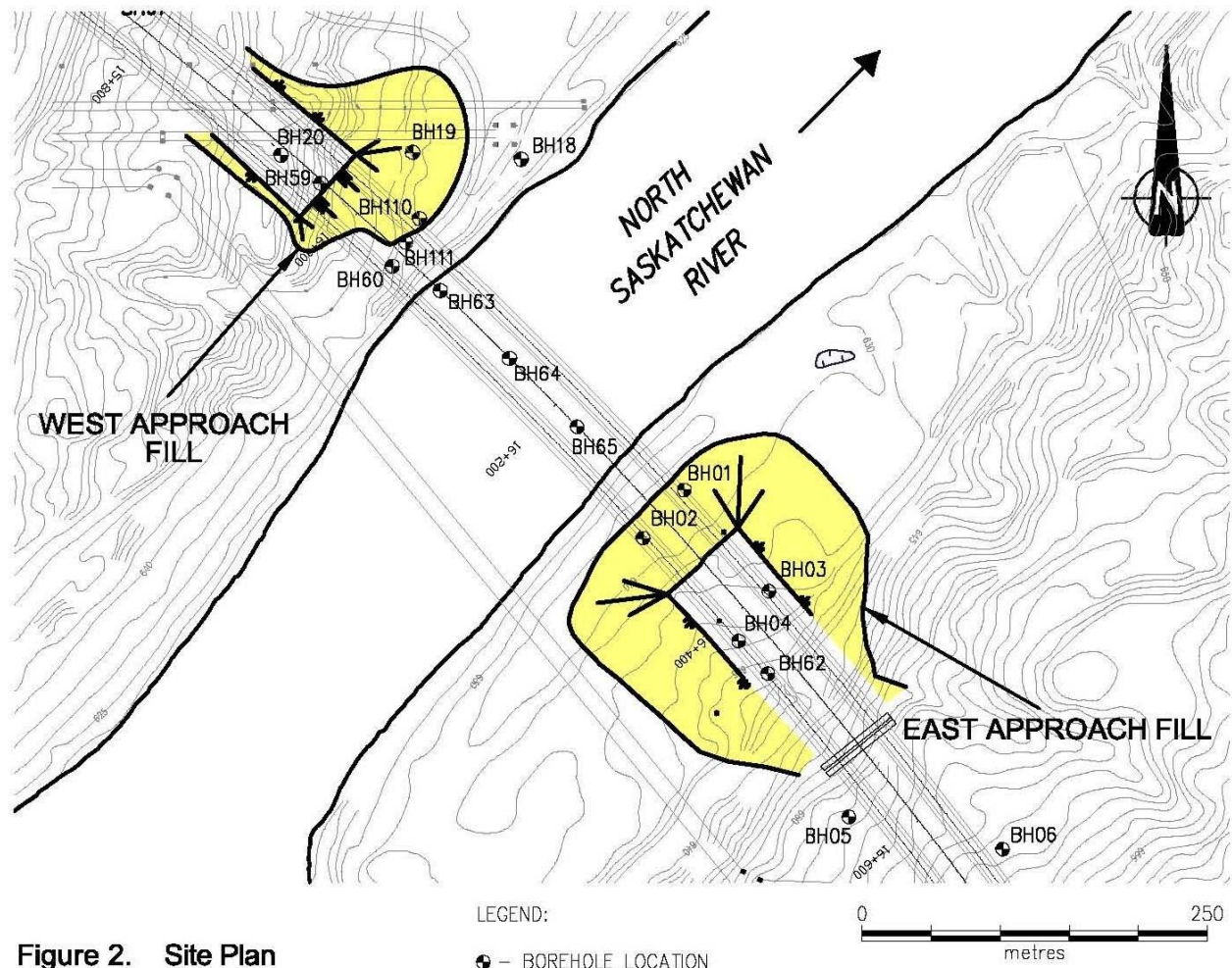
Figure 2. Site Plan

LEGEND:

⊕ – BOREHOLE LOCATION

2. SUBSURFACE STRATIGRAPHY AND GEOMORPHOLOGY

In the floodplain area, the stratigraphy comprises a fine-grained alluvial deposit overlying alluvial sand and gravel, which is underlain by bedrock. Closer to the valley slope, the gravel thins and pinches out, and a thin layer of clay till is present. The upper alluvial deposit varies in thickness up to 9 m on the east side and comprises generally clay, which is medium plastic near the surface and decreases in plasticity with depth. Atterberg limits indicate that the deposit is primarily a low plastic silty clay. Occasional sand seams were identified in the lower portion of this strata. The consistency of the clay varies from very stiff at the surface (desiccated crust) to firm at the base of the deposit.

The underlying gravel layer is sandy with a trace of silt. This layer is a maximum of 1 m thick on the east side. The underlying bedrock comprises interbedded clay shale, siltstone and sandstone of Upper Cretaceous age. The bedrock is generally uncemented and weathered near the surface, becoming more competent with depth. Liquid limits for the clay shale typically varied from 50 to 100%. One sample (from Borehole 4) had a liquid limit of over 200%. Several layers of highly bentonitic clay shale were also identified.

Landslides are common on the valley slopes along a significant portion of the North Saskatchewan River valley through the City of Edmonton. Upstream of the proposed crossing, ancient landslides are visible on the west valley slopes. Downstream of the proposed crossing the river valley exhibits indications of deep-seated large-scale mass movement. The ground surface on the west valley slope indicates a stepped topography, which is commonly associated with past landslide activity. As a result, it was speculated that landslide activity may have occurred at some time in the geologic history of the site. This raised the possibility of the presence of presheared failure surfaces within the bedrock and the associated effect on the bridge headslope stability. Although a detailed review of the bedrock core was performed, no evidence of slickensides was detected.
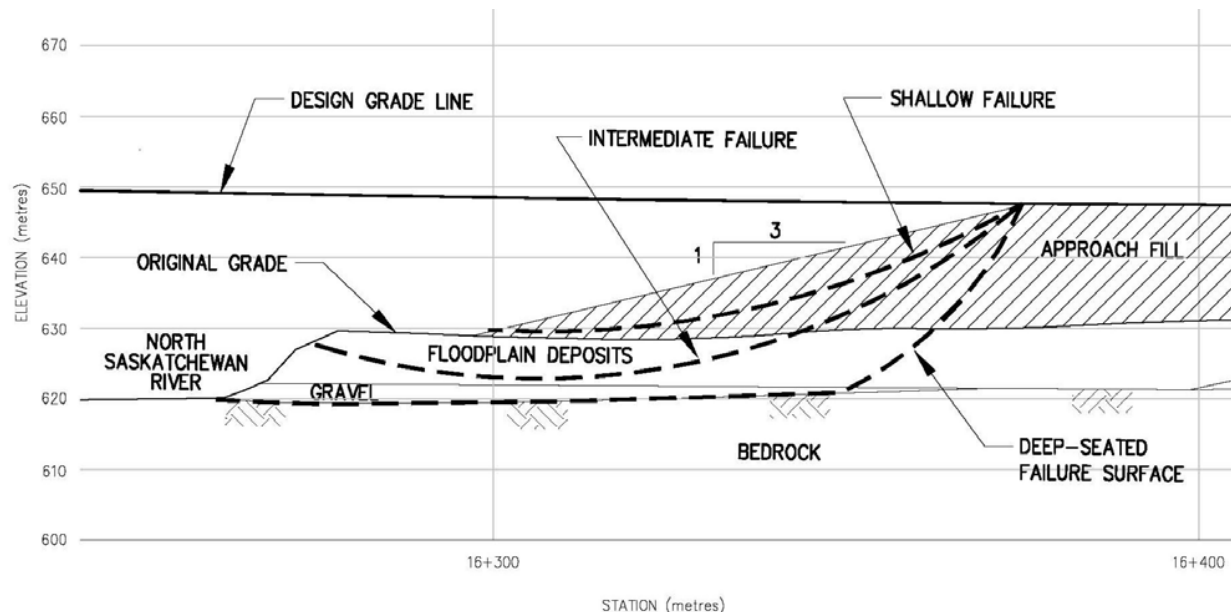
Figure 3. East Approach Fill Stratigraphy and Typical Failure Surfaces

3. PRELIMINARY STABILITY ANALYSIS

Fill for the abutments would be obtained from the roadway cut located upslope of each abutment. The majority of the excavation would comprise a medium to high plastic lacustrine clay for both abutment fills.

Shear strength parameters used in the stability analyses were selected based on the results of the drilling program, laboratory index testing, published reports and shear strength testing carried out on similar soil and bedrock from the Edmonton area. One of the key parameters that significantly influences deep-seated failure surfaces is the shear strength assumed for the bedrock. Published data from the Edmonton area indicate that clay shales with liquid limits in the 50 to 100% range, have a peak friction angle of 14 to 24° (Thompson and Yacyshyn, 1977). The residual friction angle for these clay shales is 12 to 17°. For bentonitic shales (with liquid limits in the 125 to 220% range) the peak friction angle is 14° with a residual friction angle of 8 to 10°.

Gerber et al (1992) addressed the construction of the north portal for the Edmonton South Light Rail extension. This paper presented experience with a deep-seated slide in the bedrock during construction. Prior to construction, it was determined that there was no evidence of deep-seated movement. The paper states that construction involved a cut of 1 to 2 m and a fill of 2 to 3 m combined with a period of high precipitation that was sufficient to initiate slope movements. A back analysis of the slide indicated that a friction angle of 10° was mobilized on a horizontal failure surface within the bentonitic bedrock.

Matheson et al (1973) have described a process referred to as "valley rebound". Flexure and slippage between bedding planes that occurs during valley rebound results in presheared surfaces and significantly reduced frictional values for the bedrock, even though no slope movement has occurred. Knowing also that in the floodplain areas the upper bedrock has been softened and weathered due to water flowing over the bedrock at an earlier time (in geologic history), it was considered prudent to initially adopt a residual friction angle of 8° for the bedrock for the stability analyses.

It was anticipated that during construction of the abutment fills there would be an increase in pore pressure in the native subsoils (alluvial deposit and bedrock) and possibly within the clay fill. It is difficult to predict with a high degree of certainty the pore pressure response within these three strata. Accordingly, a range of values was used to evaluate the sensitivity of the factor of safety to varying pore pressures.

Stability analyses were conducted using commercial software, which uses limit equilibrium to evaluate the factor of safety. Three modes of slope failure were analyzed as illustrated in Figure 3. Analyses were conducted to evaluate the factor of safety for shallow failures confined to the embankment fill. The second set of the analyses examined failure surfaces extending into the underlying floodplain deposit. The third set of analyses evaluated deep-seated failures into the bedrock.

The stability analyses focused on the headslope stability for several reasons. Every metre of the bridge structure costs about $85,000, therefore significant costs could be saved if the headslopes could be steepened. The consequence of failure of the headslope is greater than for the sideslope due to the presence of the bridge abutment at the crest of the headslope and possible

involvement of the pier. Supplementary support or mitigation of headslope movement is also typically more difficult due to limited access.

Results of the preliminary stability analyses indicated that all the various potential failure surfaces were sensitive to pore pressure development within the embankment fill, native floodplain deposits and bedrock. Deep-seated failures extending into the bedrock generally indicated the lowest factors of safety. Although the shallow and intermediate failures were a concern, the primary focus was the deeper-seated failures.

Due to the proximity of the embankment fills to the riverbank and the design grade line, the height of the slope being analyzed on the east side of the river was 28 m including both the embankment fill (20m) and the natural riverbank. The bedrock contact is at approximately the same elevation as the river level, which makes a potential failure surface breaking out near river level a realistic failure mechanism (see Figure 3).

Due to the significant footprint of the embankment fill, the increase in vertical stress associated with fill placement will extend down to the bedrock. Although the bedrock is heavily over-consolidated, high pore pressure response ($\bar{B}$ values) is possible. Therefore, some increase in pore pressure is anticipated during construction, which will influence the factor of safety. A parametric study was performed varying the value of $\bar{B}$ from 0.0 to 0.4. In addition, the effect of using different setback distances (20, 30 and 40 m) from the riverbank and the impact of varying headslope angles was also evaluated.

Stability results indicated that as the setback increased from the riverbank, the factor of safety increased. However, the option of lengthening the twin bridges was costly, consequently efforts to minimize the bridge length were a priority. With a 3H:1V headslope, a 20 m setback and no excess pore pressure the computed factor of safety was 1.0. By increasing the setback to 40 m the factor of safety increased to 1.1. With $\bar{B}$ values of 0.4, the factor of safety decreased to 0.76 and 0.86, for setbacks of 20 and 40 m, respectively. With these relatively low values, alternative methods for increasing the factor of safety were evaluated.

Due to road geometric constraints it was not possible to lower the height of the fill, which would increase the factor of safety. One feasible option of increasing the factor of safety included constructing the embankment headslope with gravel and high strength geogrid. Analyses indicated that with a $\bar{B}$ value of 0.4 and 15 layers of geogrid, the factor of safety was 1.2.

Other options for increasing the factor of safety include the installation of large diameter, steel reinforced, concrete piles near the toe of the headslope or the installation of anchors to provide an increased resistance to sliding. Stability analyses indicated that a maximum shear force of 2,100 kN/m was required to provide an adequate factor of safety. Calculations by the structural consultant indicated that the required shear force could be provided with a heavily reinforced, 1500 mm diameter, cast-in-place concrete pile.

Review of the analyses indicated that two parameters, namely the friction strength of the bedrock and the pore pressure response of the bedrock, had significant uncertainty associated with their selection. Based on available literature, an upper bound friction angle of 15° could be used in the design of the embankment fill. This value would be considered to be optimistically high if presheared or bentonitic layers existed within the affected portion of the bedrock, in which case a value of 8° was considered more reasonable.

Uncertainty was associated with estimation of bedrock pore pressure generation and dissipation. Depending on the magnitude of pore pressure increase, the factor of safety can vary dramatically. Vertical wick drains or sand drains were evaluated as potential means of controlling pore pressure development. Problematic installation conditions and high costs were associated with these two options.

To gain a better understanding of the pore pressures response to the embankment surcharge, a test fill was recommended, with instrumentation to monitor pore pressure response and ground deformation.

4. TEST FILL CONSTRUCTION AND MONITORING

4.1 General

The test fill was constructed to coincide with the footprint of the roadway embankment for the east approach fill. To ensure that the test fill did not generate a deep-seated failure in the bedrock, it was decided to maintain the toe of the test fill approximately 50 m from the riverbank. Sideslopes of the test fill of 4H:1V were proposed with a headslope of 2H:1V. Figure 4 presents a plan of the proposed test fill and the instrumentation, which included: eight pneumatic piezometers in the bedrock, eight pneumatic piezometers in the floodplain deposits, eight pneumatic piezometers in the embankment fill, one horizontal inclinometer, and four vertical inclinometers.

Test fill construction commenced in late June 2001 and was completed on September 7, 2001. The average ground surface elevation in the centre of the test fill was 631 m. The maximum elevation of the test fill was 647.5 m, which equates to an embankment fill height of approximately 16.5 m. The fill material used to construct the embankment originated from the roadway cut upslope of the floodplain. The fill comprised medium to high plastic silty clay that was typically 10 percent over the optimum moisture content for compaction. Based on concerns about developing pore pressures within the fill, the compaction specifications required that the fill be placed below the optimum moisture content.
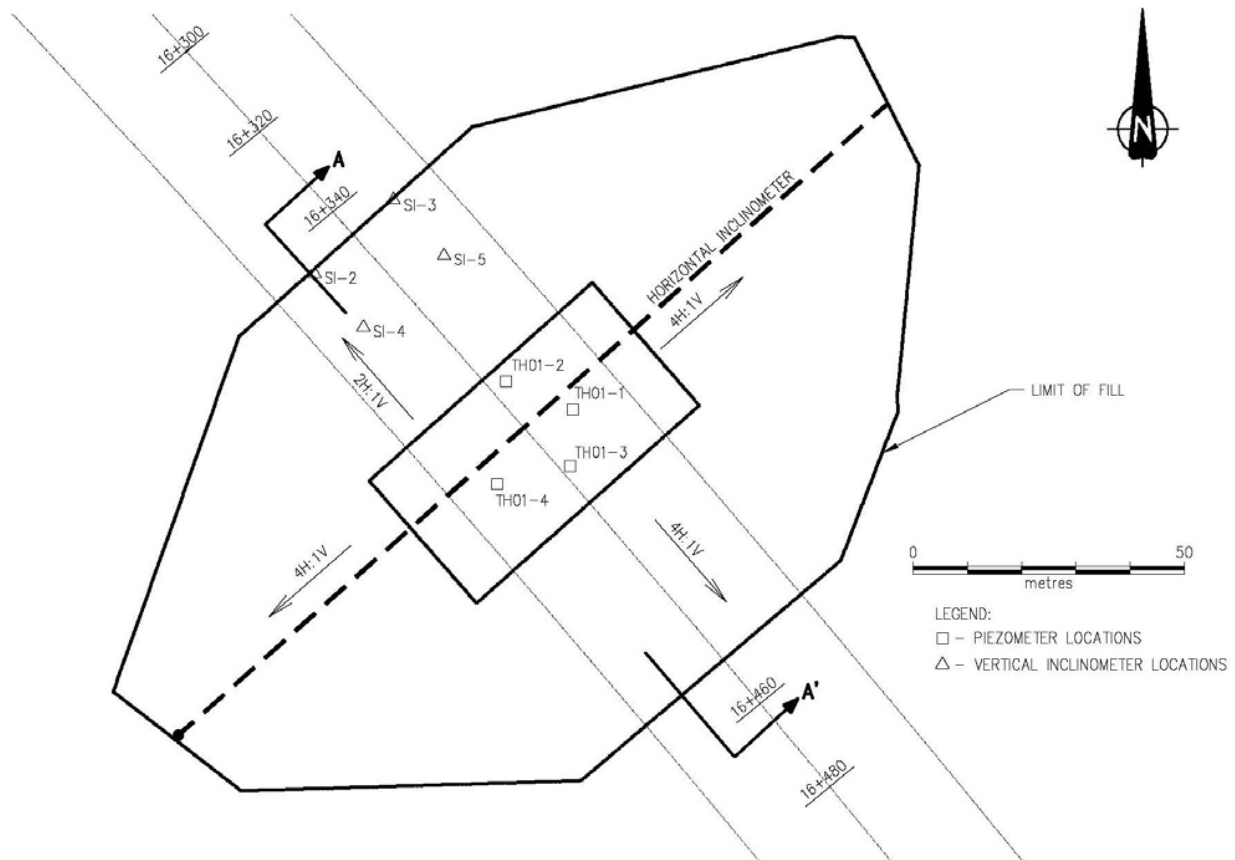
Figure 4. Test Fill Instrumentation Layout

Instrumentation was installed at strategic times during the fill construction. Prior to fill placement eight pneumatic piezometers were installed in the native floodplain deposit and eight pneumatic piezometers in the underlying bedrock. Piezometer leads were extended to the north side of the test fill to a readout box. The lower portion of the test fill was placed to a height of approximately 1.5 m, at which time a trench was constructed across the middle of the test fill for the installation of a horizontal inclinometer, which was installed to monitor the settlement of the test fill.

Two vertical inclinometers were installed near the toe of the test fill to a depth of approximately 20 m into bedrock. With the fill height at approximately 3 m, eight "drive point" pneumatic piezometers were installed within the embankment fill. The final two vertical inclinometers were installed when the fill height reached approximately 8 m. Figure 5 presents a section through the test fill illustrating the piezometer tip locations and vertical inclinometers.

4.2 Piezometer Response

The pneumatic piezometers were read daily during the test fill construction. After completion of the test fill, the reading frequency was decreased to weekly and then monthly. Shortly after completion of the test fill problems were encountered with the pneumatic piezometers installed in the floodplain deposit and bedrock. Within a few weeks to several months, all the original 16 piezometer installations had failed. It is believed that consolidation of the floodplain deposits, resulted in pinching of the piezometer leads. Replacement piezometers were installed to continue monitoring of pore pressure response in the bedrock and floodplain. It is interesting to note that of the eight piezometers installed in the embankment fill, none exhibited pore pressure response. This is attributable to the fact that the fill was placed below optimum moisture content and therefore did not generate excess pore pressures.

Reviewing the response of the piezometers installed in the floodplain deposit, there was negligible response during placement of the lower 9 to 10 m of fill. Once the fill height reached an elevation of approximately 640 m, several piezometers in the floodplain started to respond. It was known that these deposits were unsaturated and it appears that a surcharge load of approximately 200 kPa was required before pore pressures started to generate. If this response is applied to the full height of the fill (16.5 m), then the $\bar{B}$ response of the floodplain deposits was about 0.3.
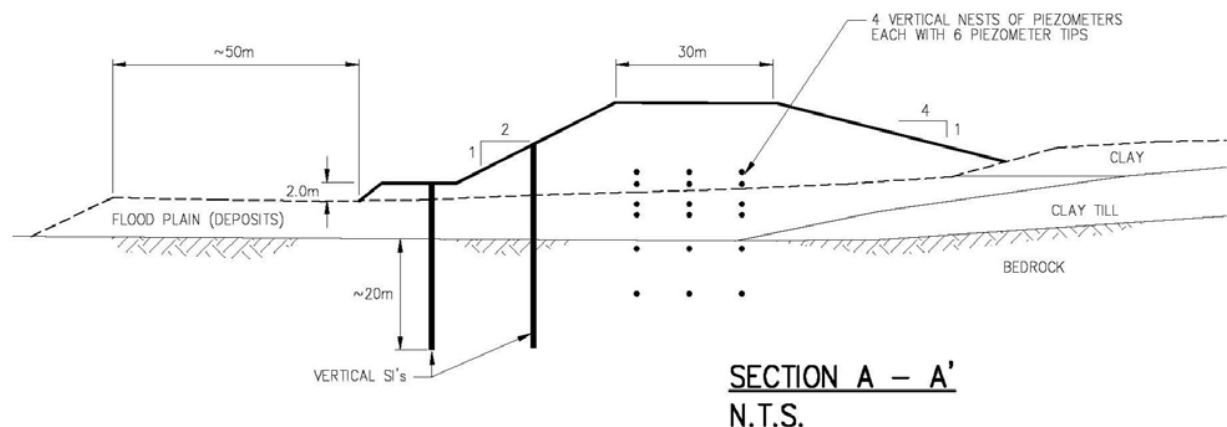
Figure 5. Section for Test Fill Instrumentation

The piezometers installed in the upper bedrock did not exhibit any appreciable response. By comparison, three of the four piezometers in the deeper bedrock showed a significant response. The maximum response was in the order of 230 kPa, which equates to an average $\overline{B}$ of 0.7. The inclinometer data indicated that no concentrated movements were triggered in the bedrock by the test fill placement.

5. DETAILED DESIGN

5.1 Strength Parameters and Pore Pressures

The test fill program provided real world pore pressure data that could be used in the stability analyses. Preliminary analyses had made a variety of assumptions regarding possible response in the embankment fill, floodplain deposits and bedrock. From the test fill data it was observed that if the moisture content of the embankment fill is kept below optimum, no excess pore pressures would develop within the embankment fill. It was also shown that pore pressures would likely develop within the floodplain deposits and the underlying bedrock.

The possibility of presheared failure planes and weak bentonitic layers within the bedrock substantiate the use of a residual friction angle of 8° for horizontal failure surfaces within the bedrock. This value is based on historical laboratory testing as well as back-analysis of the Leseur landslide by Thomson (1971) and the Devon slide by Eigenbrod and Morgenstern (1972).

Based on discussions with Alberta Transportation and the favourable performance of the test fill, it was agreed that an upper end value of 14° could be adopted for the shear strength of the bedrock underlying the east approach fill. Adopting this higher value permits a significant reduction in the cost of mitigative measures that will have to be implemented to increase the factor of safety for the east approach fill. The use of the higher shear strength values was contingent on adoption of an observational approach for construction of the embankment fills. Stability analyses for deep-seated failure surfaces were evaluated using both a friction angle of 8° and 14°.

The critical failure mechanism comprised deep-seated failure planes extending into the bedrock, which would mobilize shearing across the bedding planes of the bedrock. To select appropriate shear strength parameters for this portion of the failure plane, reference was made to previous work undertaken at the Edmonton Convention Centre site. Based on a report by EBA and Morgenstern (1979), the value for cross bedding shear strength of the bedrock adopted for the design was c' = 50 kPa and a friction angle of 25°.

A summary of the shear strength and pore pressure parameters utilized in the stability analyses is presented on Table 1.

Table 1. Soil Properties and Pore Pressures Parameters

Soil Type / Strata	c' (kPa)	Ø' (degrees)	Pore Pressure Parameter
Clay Fill	0	25	$r_u = 0.1$
Native Clay	0	25	$\overline{B} = 0.3$
Floodplain Deposit	0	25	$\overline{B} = 0.3$
Gravel	0	35	0
Bedrock (Cross Bedding)	50	25	$\overline{B}$ = varies
Bedrock (Horizontal)	0	8 or 14	$\overline{B}$ = varies

Based on the results of the test fill, a design pore pressure response within the bedrock was adopted that incorporates the maximum pore pressures obtained from the test fill. The upper bedrock exhibited no increase in pore pressures. Piezometers approximately 3 m below the top of bedrock indicate minimal pore pressure

increases. One piezometer installed approximately 5 m below top of bedrock indicated a $\bar{B}$ response of approximately 0.6. In the deeper piezometers, a maximum $\bar{B}$ response of approximately 0.7 was calculated. A design pore pressure distribution was been assumed for the bedrock that includes a $\bar{B}$ of 0.2 within the upper 2 m of the bedrock, increasing linearly to 0.7 between 2 and 10 m. Below a depth of 10 m, the value of $\bar{B}$ was assumed to be 0.7. This varying pore pressure distribution for the bedrock is shown in the above table as $\bar{B}$ = varies.

5.2   Stability Analysis

Based on the results of the test fill, the preliminary analyses were rerun utilizing the parameters given in Table 1 to assess the impact of the actual pore pressure response attained during the test fill program .

Based on the test fill results and discussions with Alberta Transportation, it was agreed that the design factor of safety for these stability analyses would vary depending on whether the residual or peak strength of the bedrock was adopted. The design factor of safety also varied depending on whether the analysis was for the long-term condition (with no excess pore pressures) or for the end of construction condition, which incorporates elevated pore pressures. For the short-term case, which utilizes estimated pore pressures generated during construction, a factor of safety for either peak or residual shear strengths of 1.3 was adopted as the minimum acceptable factor of safety. For the long-term condition, which utilized residual strength parameters for the bedrock, an acceptable design factor of safety of 1.3 was selected. If peak strength parameters were used to analyze the long-term case, the minimum factor of safety was 1.5. The final case analyzed was the factor of safety under seismic loading. For a seismic loading event, a minimum factor of safety of 1.1 was adopted for both sets of strength parameters.

The initial analyses determined that the critical failure surface was located within the upper bedrock. The analyses also indicated that regardless of whether peak or residual parameters were used, none of the criteria for minimum factor of safety was achieved. The analyses indicated that if the failure surface could be forced deeper into the bedrock, then a higher factor of safety could be developed. The most economical and practical means of achieving this objective was determined to be installation of large diameter reinforced concrete piles across the headslope width. Based on these analyses it was determined that reinforcing piles providing increased stability must extend to a depth of 15 m to achieve the required factor of safety for end of construction.

5.3   FLAC Analysis

To analyze the stresses within the reinforcing piles supplementary analyses were performed using the computer program FLAC (Fast Lagrangian Analysis of Continua). Preliminary design for the cast-in-place concrete piles indicated that a pile diameter of 1.5 m would be required to provide the shear resistance for the pile wall. The plastic yield moment of the piles provided by the structural engineer was determined to be 2800 kN-m. Preliminary results indicated that if an uncracked pile section was used, the bending moments and shear forces were substantially higher than compared with a cracked pile section. Accordingly, it was decided to adopt the moment of inertia for a cracked pile for the remaining FLAC analyses.

The initial analyses assumed a peak bedrock strength of Ø' = 14°. Failure surfaces were assumed to exist at any elevation within the upper 15 m of bedrock. With a centre to centre spacing of three pile diameters (eg 4.5m) it was determined that the maximum bending moment for a single pile was approximately 1500 kN-m and the maximum shear was approximately 2000 kN. Based on these analyses, it was determined that the piles are not overstressed, although some pile deformation was expected. These results reflect the best-case scenario for design.

However, it is possible that a residual shear strength of only 8° may exist within the bedrock. Accordingly, analyses were performed to evaluate a scenario where the bedrock only mobilizes a shear strength of 8°. An initial row of piles was installed at the same 4.5 m spacing with a yield bending moment of 2800 kN-m. These analyses indicate that 1.0 to 2.5 m of the pile shaft exceeds the yield moment of the pile. The maximum shear force is approximately 5,000 kN per pile.

Additional analyses were performed to determine the spacing of a second row of piles to supplement the first row. As the first row was assumed to have failed, some resistance could still be provided, however, a more conservative approach was considered necessary. Using a pile spacing of 1.7 m for the second row a maximum bending moment of approximately 1,000 kN-m and maximum shear force of 1,500 kN per pile was calculated for the second pile row.

Based on these analyses, if a shear strength of 14° is mobilized in the bedrock, the design bending moments and shear forces are reasonable and only one row of piles spaced at 4.5 centres is required. However, if a bedrock residual strength of only 8° is mobilized, the bending moments and shear forces exceed the capacity of the first row of piles. A second row of closely spaced piles would be required.

6.   CONSTRUCTION

An observational approach was proposed and adopted for the east approach fill. A single line of piles would be installed and monitored during fill placement. If conditions deteriorated to a predetermined state the second line of piles would be quickly installed. The incentive for this observational approach was the potential cost saving if the second row of piles could be eliminated. This additional cost was estimated to be $3,000,000.

A single row of 49 piles (1.5 m diameter) approximately 25 m long, were installed at a spacing of 4.5 m. The line of piles was located about 30 m from the toe of the headslope. Instrumentation was installed to monitor the subsurface movements and pore pressures. Two inclinometers were installed within the piles, which extended to a depth of 25 m. Two additional inclinometers were installed 8 m upslope of the pile wall to detect possible formation of a failure plane. The two upslope inclinometers extended to 5 m below the tip of the piles. Additional piezometers were installed to monitor pore pressure response to the balance of the approach fill construction. The line of piles and the instrumentation locations are show on Figure 6.
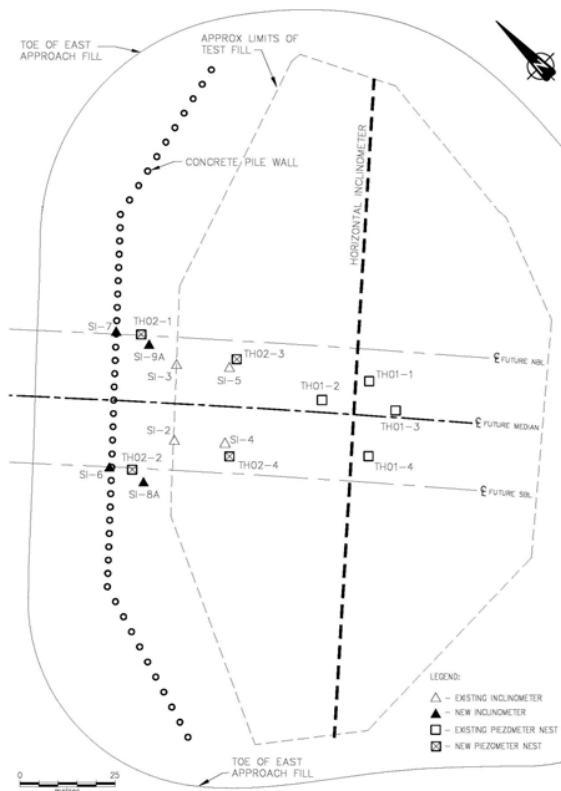

Figure 6.   Instrumentation and Pile Wall Layout

Results of the inclinometer monitoring indicate no movement in the bedrock and a gradual lateral deformation of the piles. The deformation profile was characteristic of lateral spreading or bulging of the alluvial deposits in response to the embankment loading. This mode of deformation was expected. A maximum pile movement of 75 mm was recorded after a period of three months. Back-analysis of these deformations by the structural engineer indicates that 80 percent of the design bending moment and 35 percent of the design shear force was mobilized by the piles. As a result, no supplementary piles were installed.

7.   CONCLUSIONS

Uncertainties were associated with the response of foundation materials to the construction of a high embankment structure in the valley of the North Saskatchewan River. A 16.5 m high instrumented test fill was constructed in order to provide realistic design strength and pore pressure response parameters for the stability analyses. Although the test fill provided confidence regarding pore pressure response, there was still doubt about the appropriate friction angle. The design team looked closely at the technical, as well as construction and risk issues to develop a cost effective solution to the approach fill. The Owner was involved in these meetings and was able to make an informed decision regarding this aspect of the project.

Based on the results of the test fill, an observational approach was adopted for the bridge headslope and approach fill construction and ensured that the minimum number of piles were installed. This approach permitted the opportunity for the Owner to save several millions of dollars in costs compared to a more conventional design for the bridge embankment.

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