APPROACH FILL DESIGN OF NORTH SASKATCHEWAN RIVER BRIDGE

A.F. Ruban, EBA Engineering Consultants Ltd., Edmonton, Alberta, Canada

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1. INTRODUCTION

A perimeter ring road is being constructed around the City of Edmonton that is referred to as Anthony Henday Drive. Funding for the project is being provided by Alberta Transportation. The ring road alignment includes a crossing of the North Saskatchewan River in the southwest corner of the city, which is presented on Figure 1.

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 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 was 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 sideslopes 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.

The primary focus of this paper is the approach fill design for the east side of the river. Figure 2 presents a site plan identifying the limits of the east and west approach fills at the proposed river crossing.

2. SUBSURFACE STRATIGRAPHY AND GEOMORPHOLOGY

Within the floodplain, 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 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 stratum. 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. Figure 3 presents a typical section for the east approach fill.

Landslides are common along the North Saskatchewan River valley through the City of Edmonton. Upstream of the proposed crossing, smaller scale ancient landslides are visible on the west valley slopes. Downstream of the proposed crossing the river valley exhibits signs of deep-seated, large-scale landslides on both sides of the river. The ground surface on the west valley slope indicates a stepped topography, which is commonly associated with historic landslide activity. As a result, it was speculated that landslide activity may have occurred in the geologic history of the site. This raised the possibility of the presence of presheared failure surfaces within the bedrock and associated impact on the bridge headslope stability. Although a detailed review of the bedrock core was performed, no evidence of slickensides was detected.

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 indicates 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°.

Previous studies in the North Saskatchewan River valley include the construction of the north portal for the Edmonton South Light Rail extension (Gerber et al., 1992). 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 result 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 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 (20 m) 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 would extend down to the bedrock. Although the bedrock is heavily over-consolidated, high pore pressure response ($\bar{B}$) is possible. Therefore, some increase in pore pressure was anticipated during construction, which would impact the factor of safety.

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. Other options considered included using granular fill for the abutment headslopes and high strength geogrid. A fourth option for increasing the factor of safety included the installation of large diameter, steel reinforced, concrete piles near the toe of the headslope to provide an increased resistance to sliding. 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 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 could 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 and 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 comprised medium to high plastic silty clay that was typically 10% 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.

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 16 original 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 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 attributed to the fact that the fill was placed below optimum moisture content and therefore did not generate excess pore pressures.

A review of the response of the piezometers installed in the floodplain deposit revealed 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.

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 $\bar{B}$ 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 actual pore pressure data that could be used in the detailed design. 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 various landslides including 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 adopting an observational approach for construction of the embankment fills. Stability analyses for deep-seated failure surfaces were evaluated using friction angles 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	$\bar{B} = 0.3$
Floodplain Deposit	0	25	$\bar{B} = 0.3$
Gravel	0	35	0
Bedrock (Cross Bedding)	50	25	$\bar{B}$ = varies
Bedrock (Horizontal)	0	8 or 14	$\bar{B}$ = 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 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 variable 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 re-run 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 uses 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 used 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 were 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 by installing 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, additional 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 1500 mm would be required to provide the shear resistance for the pile wall.

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 (4.5 m), 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°. Assuming one row of piles installed at a 4.5 m spacing indicated that the yield moment of the pile shaft was exceeded. Additional analyses were performed to determine that a second row of piles spaced at 1.7 m was required to supplement the first 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 m 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 and 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 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 with 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.

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% of the design bending moment and 35% of the design shear force was mobilized by the piles. As a result, no supplementary piles were installed.

A similar approach was taken for the west approach fill. Although the details of the west approach have not been presented, the general approach adopted was the same for both sides of the river. Similar to the east approach fill, the west side only required one row of piles.

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 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 regarding the appropriate friction angle for the underlying bedrock. 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.

ACKNOWLEGMENTS

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Figure 1. Location Plan

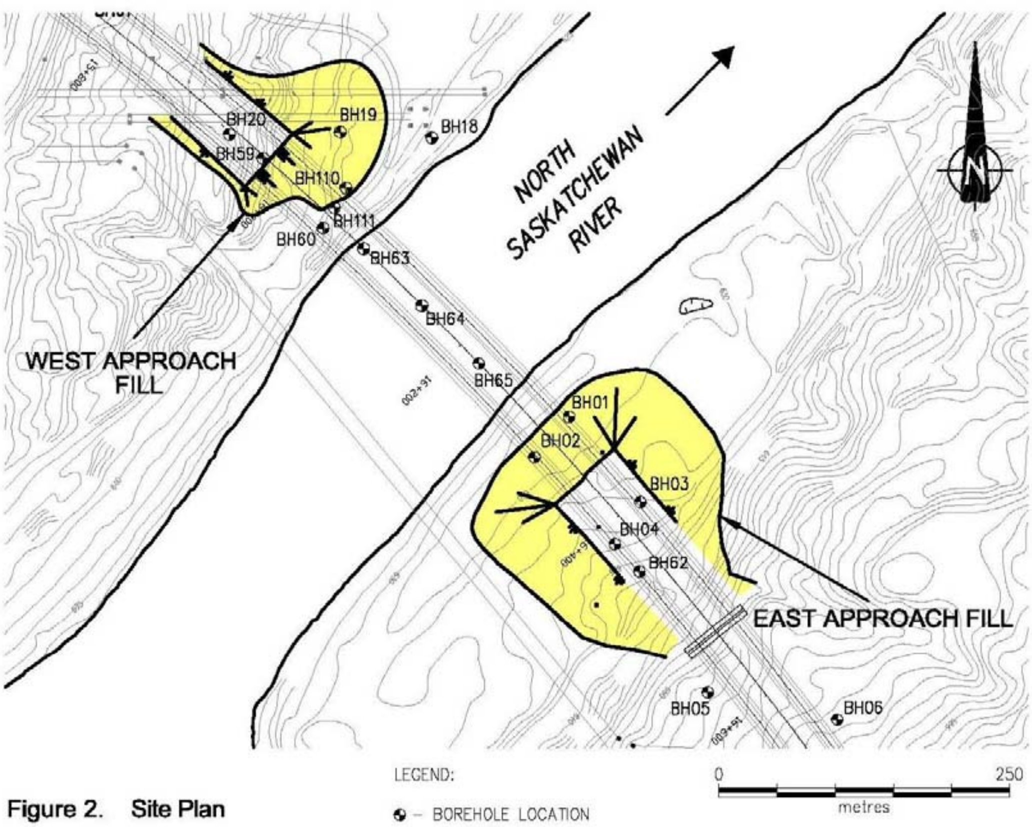

Figure 2. Site Plan

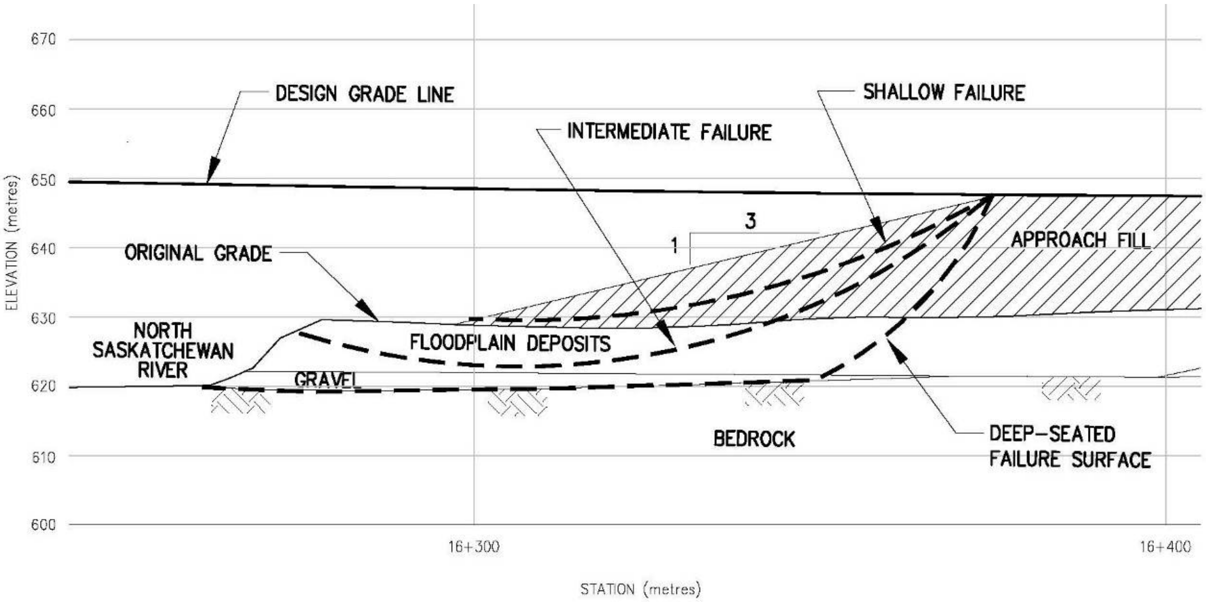

Figure 3. East Approach Fill Stratigraphy and Typical Failure Surfaces

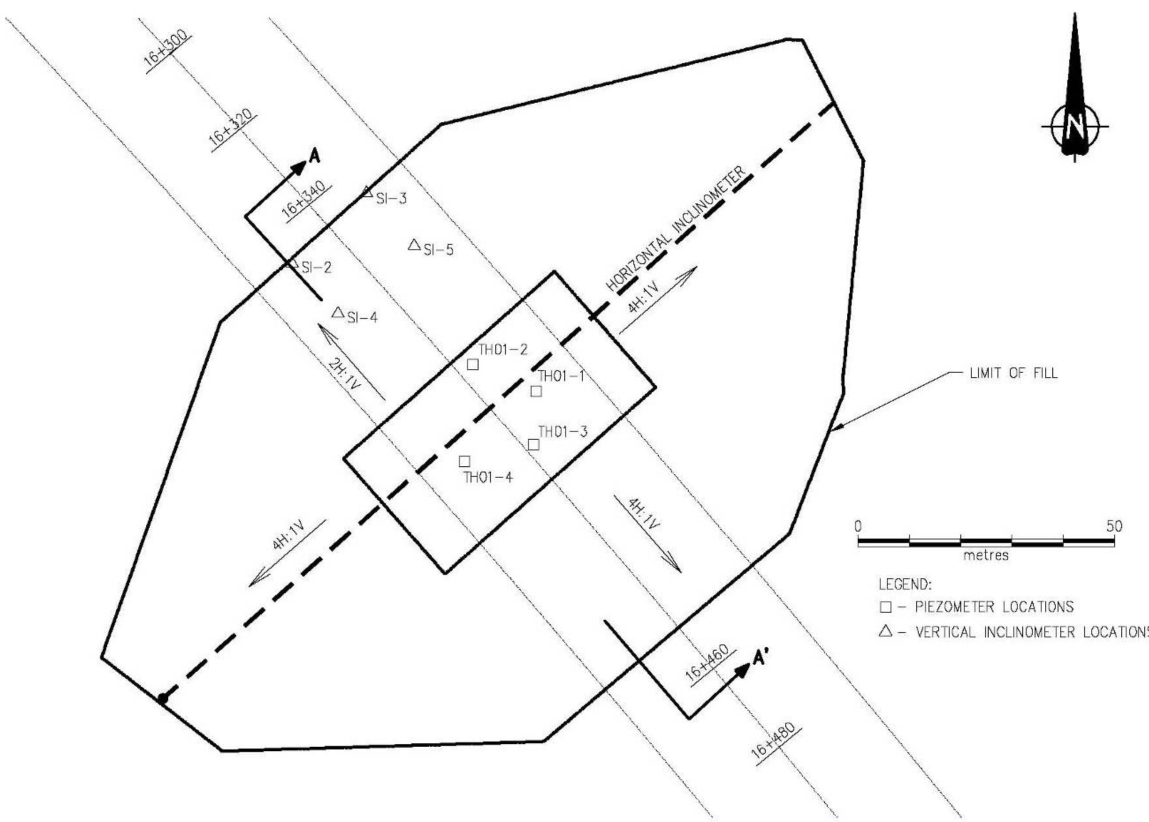

Figure 4. Test Fill Instrumentation Layout

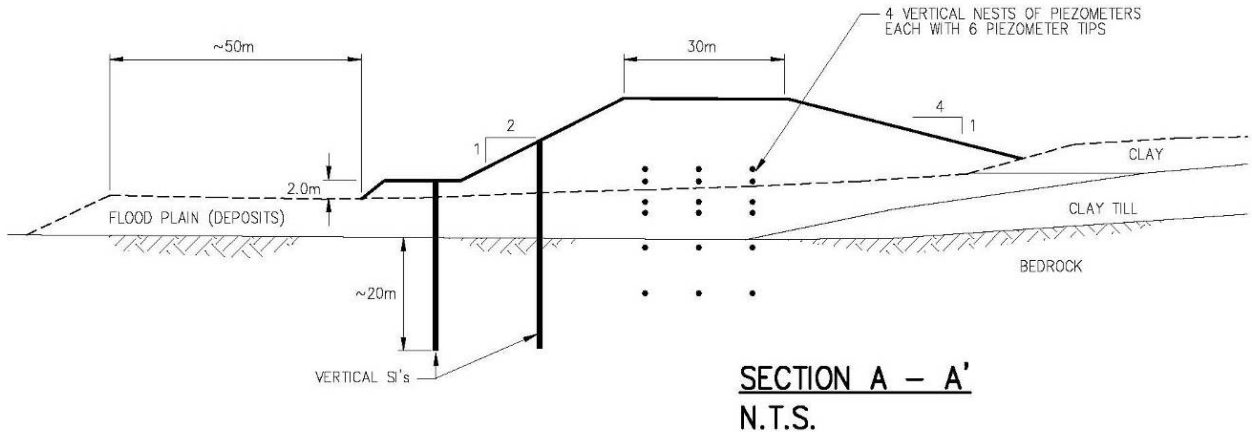

Figure 5. **Section for Test Fill Instrumentation**

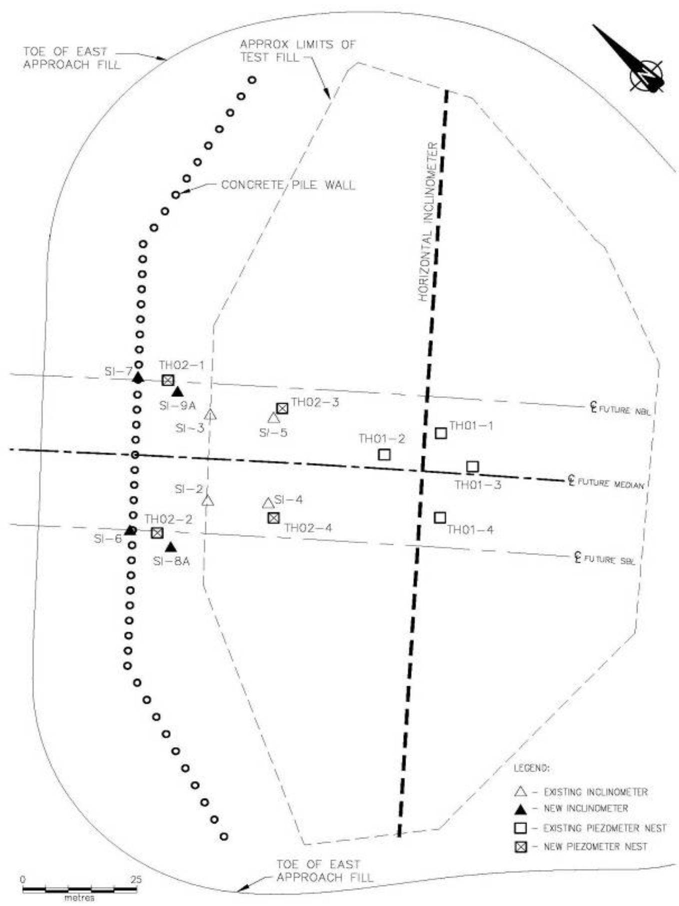

Figure 6. **Instrumentation and Pile Wall Layout**