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GEOTECHNICAL ASPECTS OF HIGHWAY AND BRIDGE CONSTRUCTION ACROSS IOSEGUN RIVER VALLEY CROSSING, NORTHERN ALBERTA

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ABSTRACT
The Iosegun valley is a deep meltwater channel partially infilled with very soft deposits. Twinning of Highway 43 required a new crossing of the Iosegun River and associated valley. The site conditions presented geotechnical challenges for the highway and bridge design that were mitigated with innovative design and construction techniques. Stability and settlement of the 6 m high bridge headslope, and 13 m high approach fill embankment, were the primary concerns. Secondary issues included stability of approach cuts into the valley wall, seepage and erosion control. Mitigative design features included: saddle berms; wick drains; drainage layers; surcharge fills; staged construction; pore pressure, settlement and lateral deflection monitoring; geogrid reinforced bridge headslopes; and bitumen coated bridge piles. The paper provides an overview of the salient geotechnical issues for the investigation; design and construction of the twinning across the Iosegun River valley. A companion paper deals specifically with the instrumentation details.

RÉSUMÉ
La vallée Iosegun est un canal meltwater profond partiellement infilled avec des dépôts très doux. Le jumelage de Chaussée(Route) 43 a exigé un nouveau croisement de la vallée Fluviale et associée Iosegun. Les conditions de site ont présenté des défis géotechniques pour la conception de la route et de pont qui a été atténué avec la conception innovatrice et des techniques de construction. La stabilité et le consolidation de 6 m de haut headslope et de 13 m de haut approche, étaient le souci primaire. Des questions secondaires ont inclus la stabilité de d'approche excavations dans le mur de vallée, suintement et le contrôle d'érosion.Détails du projet incluses: selle bermes; drains de mèche; couches de drainage; surtaxe se remplit; construction organisée; surveillance la pore pression, consolidation et le déformation latéral; geogrid pour pont renforcé headslopes; et bitume piles. Ceci papier fournit une vue d'ensemble des soucis géotechniques pour la conception et construction du jumelage à travers le Iosegun River vallée. Un papier de compagnon a affaire spécifiquement avec les détails d'instrumentation.

1. INTRODUCTION

Highway 43, in northern Alberta, forms a part of the Alberta portion of the CANAMEX North-South Trade Corridor (NSTC) being constructed by Alberta Transportation. The design requirements for the NSTC include twinning of the highway at the Iosegun River valley crossing, located about 40 km northwest of Whitecourt, Alberta (Figure 1).

The old two-lane highway now forms the northwest bound traffic lanes of the future four-lane divided highway. The alignment of the old highway consists of a double S-curve with low fill heights across the valley bottom. The new twinned southeast bound traffic lanes take a more direct route across the valley, and required higher fills and associated deeper cuts where the vertical alignment transitioned to the upland terrain, Figure 2. A new river crossing was required, which entailed a new bridge and major river diversion. Other site restrictions included a pipeline running along the south side of the alignment that limited cut slope options.

Figure 1. Site Location Plan

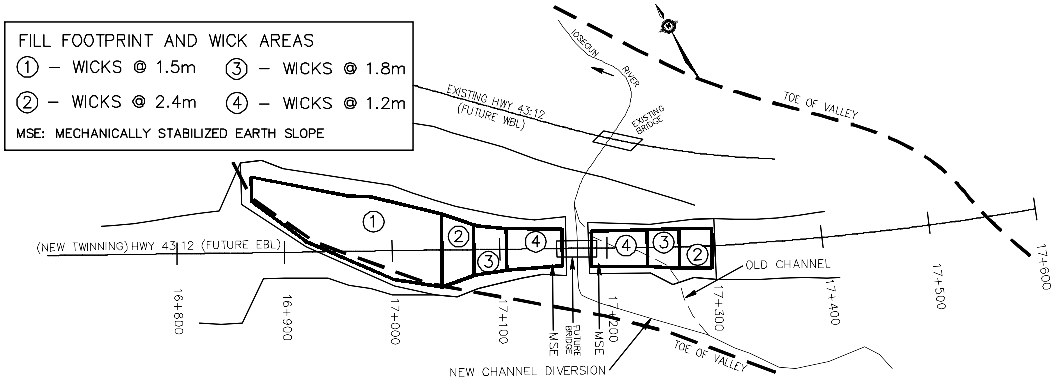

Figure 2. Plan of Highway Twinning Alignment and Iosegun River Diversion

Historical records indicates that the valley crossing was difficult to construct and that the bridge fills continue to settle, creating a maintenance nuisance. The investigation and assessment conducted for the twinning project indicated a wide variety of geotechnical challenges at the valley crossing. This paper presents a summary of the geological setting, past construction problems, performance of the existing fill and bridge, details of the recent site investigation, design and construction issues, and mitigative work undertaken. Issues related to realignment of the river channel, erosion control and other environmental considerations are not included in this paper, although these issues were also major factors at the site. A companion paper discusses the instrumentation program and summarizes the results of CPT testing performed prior to and after fill placement, which was undertaken to confirm design assumptions.

2. BACKGROUND

2.1 Geological Setting

The present day Iosegun River valley is a remnant of an outflow channel of glacial Lake Iosegun. The base of the outflow channel was deeply incised into cretaceous Paskapoo Formation sandstone bedrock. The lateral margins of the ancient channel roughly define the approximate present day valley slope boundaries, as defined by sandstone outcrops visible along the steep west valley slope.

The valley width, measured at crest elevation is about 3.0 km, while the valley floor is about 500 m wide along the new alignment. The upland level is at approximately Elevation 870 m, the valley bottom is approximately 800 m, while the base of the buried outflow channel is approximately 761 m.

Subsequent to the development of the ancient outflow channel, the site was subjected to glacial advances and retreats, damming and erosion, and as a result has been infilled to the present elevation with till, alluvial, fluvial and glaciolacustrine deposits. The deposits were found to be spatially highly variable, but were primarily fine sand, silt and clays overlying a clay till strata that conforms to the eroded bedrock surface. The infill material incorporated organic materials throughout the strata, and contains layers of sand and gravel that are considered to be past riverbed locations. The valley bottom at the crossing site is mantled with poorly drained muskeg.

2.2 Site History

2.2.1 Construction Records

Review of the highway construction records showed that major construction problems were encountered during the original highway embankment construction in 1954. Conventional earthmoving equipment of the day was not able to successfully travel on the soft ground conditions. At one point a dredge line was brought to site in an attempt to excavate the soft material and find a firm base. After several days of digging the effort was abandoned and displacement techniques were used to incrementally advance the fill across the valley. Approximately three times the anticipated fill volume was required to achieve the design grade.

2.2.2 Existing Bridge Headslope Performance

The existing bridge is a 3 span (12m-12m-12m) bridge across the Iosegun River supported by 25 m long pipe piles bearing on bedrock. The bridge was built in 1980 and replaces an older bridge built in 1955. The 4 to 5 m high approach fills at the existing bridge have continued to settle for over 22 years. Annual asphalt patching is required to maintain a smooth approach to the bridge. According to anecdotal maintenance records, a yearly settlement of around 7.5 cm is estimated and a total settlement of 1.5 m has accumulated over the past 22 years.

Construction records from 1979 indicate that wick drains were installed prior to bridge fill placement. The wicks were placed at 2.5 m centre spacing, and were advanced to about 20 m depths. The drainage design included a sand blanket over the footprint of the bridge headslope fills. The sand blanket was constructed from locally available fine sand with fines content of 10 to 15%.

2.2.3 Abutment Fill Settlement

Possible factors that contributed to the persistent settlement of the existing bridge approach fill were considered to be: slow rate of seepage transmittivity of the sand blanket due to the high fines content (15%) of the local sand materials; Excessively wide wick drain spacing, that resulted in longer drainage paths and reduced drainage efficiency; Insufficient surcharge loads placed during construction; Reduced or inhibited vertical drainage due to possible micro-folding of the wicks, and; Lateral spreading and creep movement of the headslope fill.

2.3 Drilling & Stratigraphy

The field investigation was carried out by conventional solid stem dry auger drilling as well as Cone Penetration Testing (CPT). Eight CPT holes were advanced. Five deep boreholes (40 m depths) and 9 CPT holes (32 m depths) were advanced. Disturbed samples were retrieved using Standard Penetration Testing (SPT) and grab-sampling methods, while undisturbed samples were retrieved by pushing thin walled Shelby tubes. Dynamic cone penetration testing and vane shear testing was also undertaken. A plan of the testhole locations is presented in the accompanying paper. The soil stratigraphy profile and soil strength are presented in Figure 3.

The soil stratigraphy of the Iosegun River Valley entails a thick upper layer (up to 30 m) of very soft channel deposits. These deposits are variable but can be generalized as silty clay overlying clayey sand overlying sandy clay and/or clayey silt. Organics inclusions were common, comprising up to 33% by volume in some layers, and consisted of decomposed to non-decomposed wood fragments, shells, marl and peat. The soft material is underlain by approximately 8m of firm to stiff clay till, followed by competent sandstone bedrock at approximately 38 m to 40 m.

2.4 Field Strength Testing and Laboratory Testing

Field CPT, shear vane and SPT testing was undertaken to estimate the undrained soil strength of the soft deposits. The undrained soil strength of the channel deposits were divided into zones as shown on Figure 3. The western half of the site was broadly characterized as having undrained shear strengths (S_u) of 20 to 40 kPa, while the eastern half had S_u values of 50 to 60 kPa.

The laboratory results include moisture contents, Atterberg limits, and gradation testing. Consolidation testing was performed in both vertical and horizontal directions. Using consolidation test results a model consisting of five idealized soil types was developed for use in the settlement analysis, as shown on Table 1.

It was determined that the channel deposits were normally consolidated with higher silt and organics content within the upper 10 m, higher clay content for the middle portion between 10 m to 20 m, and primarily sandy silt in the lower 20 m to 30 m. The top 10 m contained a high volume of organic matter, which was determined to be highly compressible. The zone between 10 m and 20 m was clayey and had relatively low permeability values. The lowest 10 m had greater permeability, and was less compressible than the upper 20 m.

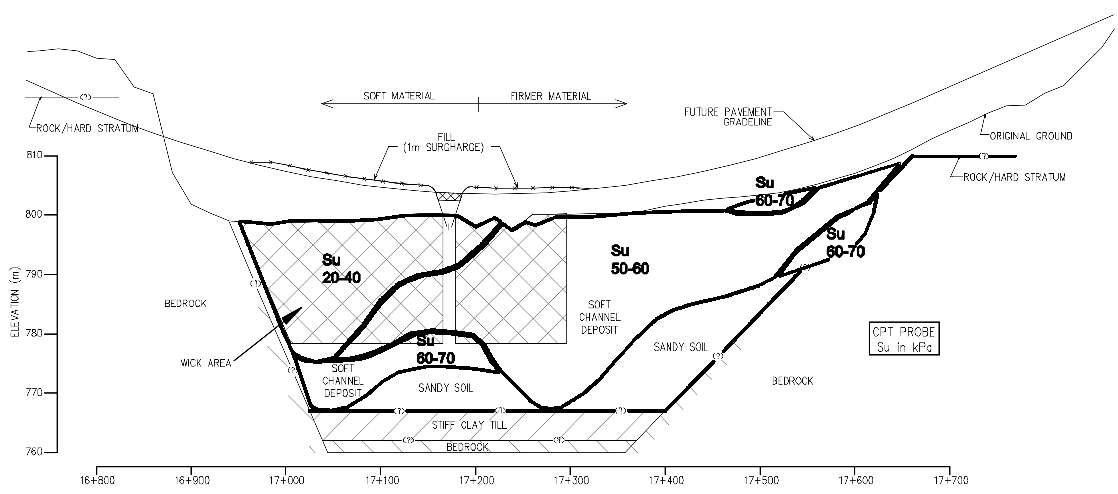

Figure 3. Stratigraphic section with zones of undrained shear strength (Su) from the CPT results

Table 1. Consolidation Model

Zone	Soil Depth (m)	Soil Type	e_o	C_c	C_v (m²/yr)	Estimated Primary Consolidation Settlement (m) 4 m Fill	11 m Fill
1.	0 – 4	silt, clay	1.6	0.5	12.59	0.51	1.03
2.	4 – 8	silt+organics	3.2	1.1	63.70	0.59	0.97
3.	8 – 10	sand+organics	1.5	0.2	105.17	0.07	0.12
4.	10-20	clay, silty	1.1	0.35	3.10 c_h=2 m²/yr	0.50	0.94
5.	20-30	silt, sandy	0.6	0.12	65.70	0.12	0.27
					Total:	1.80	3.33

Note:

P_c = Pre-consolidation Pressure
P_{vo} = Existing Effective Stress
C_c = Compression Index
C_r = Recompression Index
C_v = Co-efficient of Consolidation Along Vertical Axis
C_h = Co-efficient of Consolidation Along Horizontal Axis
e_o = Initial void ratio

3. GEOTECHNCIAL ASSESSMENT

The past site problems and present geotechnical assessment both predicted difficult construction and long-term problems related to several concerns. At the planning stage of the project a very tight construction schedule was demanded, and as a consequence the short-term stability of the high embankment fills and ridge headslopes were a primary concern. Long-term stability and settlement of the embankment and bridge approach fills were also a concern. The assessments for these issues are discussed below.

3.1 Slope Stability

From observation of the existing roadway embankment, it was evident that low fills, 4 m to 6 m in height, with gentle sideslopes, 5H:1V are stable. However, the stability assessment found that both the proposed 4H:1V high embankment fill, and 2.5H:1V low bridge headslope fill were only marginally stable.

Both total and effective stress analyses were undertaken to assess the stability of the proposed highway embankment and bridge headslope. All stability analyses were run using SLOPE/W software. The short-term factor of safety (during construction) was calculated using total stress analysis. The long-term factor of safety (post construction) was calculated using effective stress analysis, in accordance with the pore pressure dissipation conditions predicted. During fill construction the factor of safety will improve from short-term (construction) to long-term (post-construction) as dissipation of pore pressure occurs.

Strength parameters used in the analysis are summarized in Table 2. The results of the stability assessment are provided in Table 3. Figures 4 and 5 depict the results of the total and effective stress analysis respectively for the low and high embankment fill areas.

Skempton's (1957) relationship between plasticity index, surcharge and undrained shear strength predicts an increase in shear strength due to an increase in fill height. This increase in shear strength was not incorporated into the stability analysis due to the complexity and variability of the soil conditions. An increase in strength was observed from post-construction CPT testing. The results of the CPT testing are included in the accompanying paper in these proceedings.

3.2 Settlement

The fill settlement was assessed using Terzaghi’s one dimensional settlement equation. The channel deposits were idealized into 5 zones. The settlement and permeability characteristics used for each zone are summarized in Table 1.

Table 2. Strength Parameters

Soil Type	Fill	Channel deposits
Effective cohesion, c' (kPa)	0	0
Effective friction, ϕ'	30	20
Undrained shear strength, Cu (kPa)		30

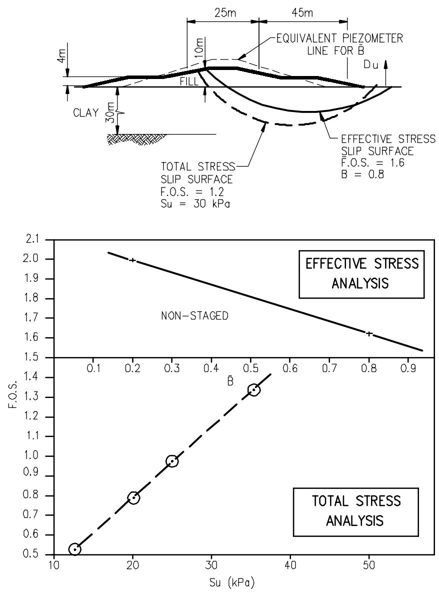

Figure 4. Slope Stability analysis

Wick spacing was determined using Barron's Formula. The clayey soil between 10 and 20 m depth, was found to control the wick drain spacing requirements. The results of the wick drain spacing analysis are provided in Figure 6.

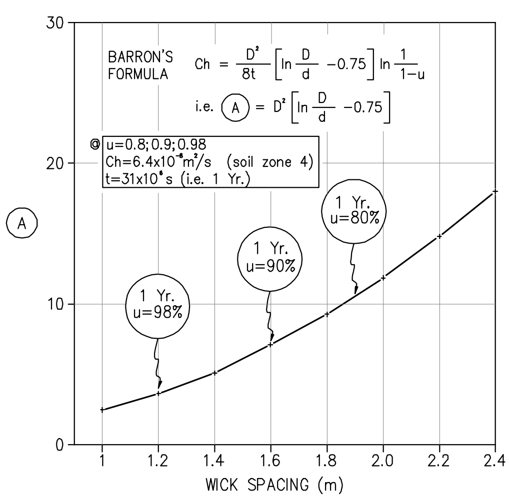

Figure 6. Wick drain spacing for Soil 4

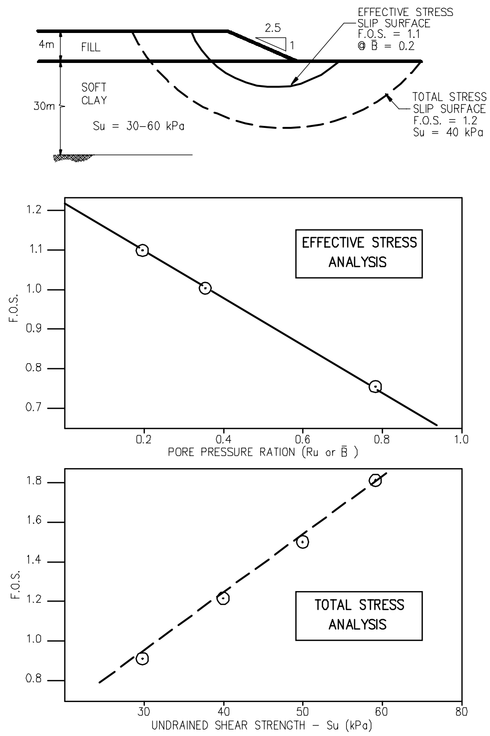

Figure 5. Slope Stability Assessment

Table 3. Range of Factor of Safety

Fill height (m)	4 to 6	4 to 6	10
Fill slope	5H:1V	2.5:1V (head-slopes)	4H:1V with toe berm
Short term F.O.S. (Total Stress)	1.5	0.9	0.9
Long term F.O.S. (Effective Stress)	+1.5	1.2	1.5

The wick drain spacing of 1.2 m was designed to achieve 98% of the ultimate consolidation within one year after fill placement at the steep headslope location. At the high fill saddle berm area 1.5 m spacing was used. The increase in spacing between the headslope and high fill area is a reflection of project economics, the desire to reduce downdrag on the bridge piles, and the reduced consequences of long-term settlement in the high fill area. A 1 m thick surcharge fill was included in the design to complement the wick drainage over a planned one year duration.

4. MITIGATIVE DESIGN FEATURES

The analysis, and past site history, indicated that several geotechnical mitigative design features were required to successfully construct the project, and achieve satisfactory long-term performance of the embankment and bridge fills. These features included a saddle berm constructed along the high fill area, mechanical stabilized earth (MSE) headslopes, bitumen coating of bridge piles, surcharge fill, staged construction, and deep drainage measures. In addition, geotechnical instrumentation was used to monitor pore pressure dissipation, settlement and lateral movement of the fill. The instrumentation details are presented in an accompanying paper.

4.1 Saddle Berms

A saddle berm was required along the high fill area in order to maintain the fill stability during construction. The berm was initially sized using reference charts provided in NAVFAC DM 7 and subsequently analysed using SLOPE/W to fine-tune the design. The final berm design was 30 m wide and 6 m high, with 4H:1V sideslopes.

The stability analysis of the low fill area indicated that saddle berms were not required. The low fills (less than 6 m in height) were designed with 5H:1V sideslopes.

4.2 Mechanically Stabilized Earth Headslope Design

The stability analyses determined that the proposed 2.5H:1V headslopes would not be stable. It was determined that a MSE reinforced slope was cost effective compared to flattening the headslopes and lengthening the bridge structure. In addition to stabilizing the headslope, the MSE was expected to reduce the potential for lateral spreading of the fill, as may be the case in the existing bridge fill.

The MSE design incorporated 4 horizontal layers of Tensar BX1100 biaxial geogrid at 0.5 m vertical spacings within the upper half of the fill, above the level of the basal sand drainage blanket. The geogrid extended 30 m away from headslope. The exposed geogrid was wrapped upward and lapped into the next higher grid level. As part of the realigned channel design riprap was extended up the headslope on top of the geogrid facing.

4.3 Negative Skin Friction Design

The bridge will be supported by 35 m to 40 m long pipe pile foundation bearing on bedrock beneath the soft channel deposits. Without mitigative measures, significant negative skin friction forces would act on the pile, reducing the pile capacity and potentially deflecting the pile.

The use of bituminous coating on the piles was considered as an effective mitigative design to reduce the potential effects of negative skin friction. After an extensive search it was determined that Shell SL Compound met the kinematic viscosity and shear strain rate criteria for bitumen pile coating and this material was specified. The bitumen coating design involves coating the upper 30 m of the pile with bitumen. The bottom 10 m of the piles are not coated in order to mobilize frictional resistance within the clay till strata and bearing resistance of the sandstone bedrock. It is expected that the potential negative skin friction force of 15 kPa to 25 kPa within the upper 30 m of the fill could be reduced to about 3 kPa through the use of bituminous coating.

4.4 Surcharge Fill Design

In order to reduce future pavement maintenance related to differential settlement of the bridge fill and bridge abutments, it was decided to incorporate a surcharge fill into the grading design. A one-metre thick surcharge fill was placed during the first construction season (ending Nov. 2001). It was expected that approximately 0.7 m of surcharge would remain above the final pavement grade after a 9 month consolidation period. During the second construction season (Summer 2002) the 0.7 m of excess grade, and 0.6 m of fill corresponding to the design pavement structure thickness, would be removed and the pavement structure placed.

4.5 Staged Construction Design

The construction contract contained special provisions that notified the contractor that the rate of fill placement would be varied according to results of a monitoring program. As a guide for bidding purposes the rate of fill placement was estimated to be 1.5 m per week. The possibility of "holiday" (breaks for pore pressure dissipation) in the fill placement were noted in the contract.

4.6 Deep Drainage Design

The ability to control the foundation soil pore pressure is directly related to the stability of the embankment fill during construction. In addition the ability to expedite the consolidation of the soft channel deposits also affects the long-term performance of the embankment. In order to meet these objectives the following deep drainage design was developed.

- Non-woven geotextile placed directly over the cleared ground surface.
- 1 m thick layer of manufactured sand with less than 5% fines content placed onto the non-woven geotextile.
- Install wick drains to 20 m depths.
  - wick spacing at:
    1.2 m for 4 to 6 m low fills at headslopes
    1.5 m for 10 m high fills
    1.8 m at transitional area
    2.4 m at outer transitional area

The layout of wick installations is presented in Figure 2.

5. CONSTRUCTION ASPECTS

Minor changes to the designs were implemented during construction due to site specific conditions and contractor related issues.

5.1 Shortage of Manufactured Sand

After award of the contract, the contractor found they could not meet the construction schedule for the production of coarse manufactured (MF) sand intended for use in the sand drainage blanket. Locally available silty sand was available and an alternative design was developed to make use of this material and thereby reduce the volume requirement for MF sand. The design was somewhat conservative since previous lab testing had determined the local sand had relatively high fines content, and the same sand source had been used at the existing bridge headslope with poor results.

The revised design consisted of:

- Non-woven geotextile placed directly on the native ground.
- 1 m of locally available fine sand, with fines content of 10 to 15%.
- Install wick drains to 20 m depths using the same spacing as originally designed.
- Horizontal placement of prefabricated strip drains to directly connect the wick drains to supplement the decreased transmittivity of the local sand
- Placement of 0.5 m layer of MF sand over the strip drains
- Non-woven geotextile placed over the MF sand to separate the subsequent fill above.

An additional design modification included the placement of non-woven geotextile and riprap along the sand blanket drain outlet. This was done in response to observed piping of the sand drain materials.

The revised design was successful in meeting the drainage requirements and construction schedule of the project. The original design was maintained below the bridge headslopes since the MF sand had superior strength characteristics than the local silty sand.

5.2 Soft Ground Construction Methods

Some common construction methods were implemented by the contractor to deal with the soft ground conditions. During the site clearing and grubbing operations the trees were cut off at ground level and the stumps left in place. Heavy construction equipment was not able to traverse portions of the cleared site and placement of the non-woven blanket was done manually. Workers wearing hip waders rolled and unfolded the fabric in advance of sand fill placement. During placement of the design 1 m thick local silty sand layer, the sand was noted to rapidly settle during fill placement operation. To compensate for the fill settlement, irregular dips in the existing ground, and to produce a useable working pad across the site, about 1.5 m of local sand, on average, was required. The sand was also noted to saturate rapidly, and the wick drain installations had to be done immediately after pad preparation. Any delays would have resulted in a working pad that could not support the track equipment used to install the wick drains. Similarly, after insertion of the wick drains, water was observed to rapidly seep onto the working pad surface through the wick drains. Placement of the synthetic strip drains and upper drainage layer of MF sand was undertaken directly after wick installation.

5.3 Costs

The overall project costs were about $7,000,000, which included the Iosegun valley crossing and 7 km of adjoining highway grading. The total construction cost of geotechnical-related work was about $2,200,000 as summarized in Table 5. Costs for the bitumen pile coating were not available at the time this paper was prepared.

Table 5. Geotechnical-related costs

Item	Quantity	Amount
Local Sand	94,000 tonnes	$752,000
MF Sand	41,000 tonnes	$522,000
Wick Drains (9,000)	174,000 m	$387,000
Strip Drains	7800 m	$119,000
Non-woven fabric	60,000 m²	$120,000
Investigation, instrumentation, monitoring	N/A	$300,000
TOTAL		$2,200,000

5.4 Changes to the Construction Schedule

Changes to the NSTC construction schedule and budget permitted a slowdown of the pace of construction and as a result the time for consolidation was 1.5 years instead of the design 9 months. The majority of the fill was placed during the first year of construction allowing a 9 month consolidation period thereafter, but the upper metre and surcharge fill was not placed. The rate of fill placement was done according to the design, 1.0 to 1.5 m of fill per week. In the second year, the remainder of the fill and the entire 1 m surcharge was placed in the summer of 2001 and allowed to consolidate under the surcharge for 8 to 9 month. It is anticipated that paving will be done in the summer of 2002.

The delay in construction has some benefits for the long-term embankment performance. By permitting the consolidation under a majority of the design fill for nine months prior to completion of the fill it can be shown that the actual surcharge fill thickness is about 0.2 to 0.3 m thicker than it would have been if the surcharge was placed in the first construction season. By permitting an additional 9 months of consolidation beyond the original schedule the long-term performance of the pavement surface at the bridge abutment and cut-fill transitions should be further enhanced.

For the bridge headslope, it is anticipated that piling construction will start in summer 2002. A consolidation period of slightly over 1.5 years will help to reduce the downdrag effects on the bridge pipe piles.

6. DISCUSSION

The following presents a summary of the critical facets associated with the construction of this difficult river crossing.

- The construction of a high fill over deep, very soft channel deposit was successfully carried out using staged fill placement.
- The wick drain, lateral drain and sand blanket system performed very well to dissipate pore pressure for consolidation of soft subgrade soil during and after placement of the fill. The sand blanket exit in the toe berm area is still wet two years after placement, indicating continued seepage from the wick drains.
- The gradation and modified design of the sand blanket (using strip drains and a top zone of clean MF sand) were important design aspects to provide effective lateral drainage capacity for the vertical wicks.
- Substantial water flow was observed at the edge of the sand blanket and non-woven geotextile with a riprap cover was placed over the daylighted sand blanket to prevent piping of the sand.
- The estimated settlement derived from the five-soil model is close to the monitored settlement. The actual fill height constructed included the fill settlement, design height and surcharge thickness.
- The use of surcharge (1m above pavement elevation) will provide additional consolidation of the foundation soil and will reduce future settlements. Long-term performance of the pavement structure, particularly at the bridge abutment will be improved, resulting in reduced long-term maintenance costs.
- The MSE bridge headslopes were successfully constructed and appear to be behaving as a rigid soil mass. No lateral spreading or instability of the headslope has been detected to date.
- Bridge piles will be installed in summer, 2002, after the preparation of this paper. Bitumen coating of the piles will reduce the anticipated downdrag forces on the piles.
- Early placement of the majority of the fill followed by placement of a surcharge fill in the second year appears to be a preferred alternative to complete fill placement in one year.
- From observation of pore pressure results, it is anticipated that improvement to soil strength can be more effective at high fill area due to the amount of overburden pressure imposed.
- Discussion of the monitoring program is provided in the accompanying paper in these proceeding. In general the monitored results are in agreement with the design parameters and design predictions.

7. CONCLUSION

The twinning of Highway 43 over the Iosegun River valley presented significant challenges in geotechnical engineering. The fundamental lessons of soil mechanic can, and do, play a part in everyday designs, even in complex and high profile projects such as this one. This is not to say that advanced modeling techniques should not be used to tackle complex problems. Rather, there is a place for sound engineering judgment, especially when accompanied by anecdotal and factual past site history, field and lab testing to adequately define the problem, analysis to determine the primary site issues, designs to mitigate these issues and the use of the observation approach to confirm the designs.

8. ACKNOWLEDGEMENTS

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9. REFERENCES

Skempton, A.W. and Bjerrum, L. 1957, A contribution to the settlement analysis of foundations on clay. Geotechnique, 7: 168-178

NAVFAC, Design Manual 7, 1971.