DESIGN IMPLICATIONS OF SOFT SOILS FOR HIGHWAY IMPROVEMENTS NEAR VICTORIA

By: Ian Pilkington, Stephen Alexander, and Don Lister
Geotechnical & Materials Engineering Branch,
Ministry of Transportation and Highways
Victoria, B.C.

The Ministry of Transportation and Highways, as part of the Vancouver Island Highway Project is widening 7.8km of the Trans Canada Highway west of Victoria.

The geology of the area consists of an irregular bedrock surface overlain by soft glaciomarine clays with some peat deposits.

The design concerns were: 1. an area of peat 7m thick underlying a proposed highway underpass; 2. the design of a 16m high embankment overlying very soft, compressible glacio-marine clay; and, 3. the extension of existing culverts, to accommodate highway widening, over very soft clay. Options considered for the peat area included consolidation by preloading and excavation and replacement; for the 16m high fill, options included preloading, ultralightweight fill, excavation and replacement, and soil improvement using stone or vibro concrete columns or steel H piles; and for the culvert extensions, possible solutions were preloading (including sacrificial culverts), ultralightweight fill and flexible connections between the new and old sections of culvert.

This paper discusses the advantages and disadvantages of the above design alternatives.

INTRODUCTION

West of Victoria, the Trans Canada Highway is to be widened and upgraded to freeway status. The section, forming part of the Vancouver Island Highway Project, extends from Langford in the west to View Royal in the east, a distance of 7.8km (see Figure 1).

The eastern 2.5km will be a six-lane facility while the western section will be four lanes wide. Three full and one partial grade separated interchanges are to be constructed. Portions of the Esquimalt and Nanaimo railway require relocation. Construction and reconstruction of 6km of local roads is also required. Preliminary design work started in 1990, and detailed design commenced in 1993. Construction started in the spring of 1995. During field investigations, the extent of soft soils was found to be greater than originally anticipated. Due to the limited choices of highway alignment alternatives, a number of complicated design issues arose. A number of alternative methods were

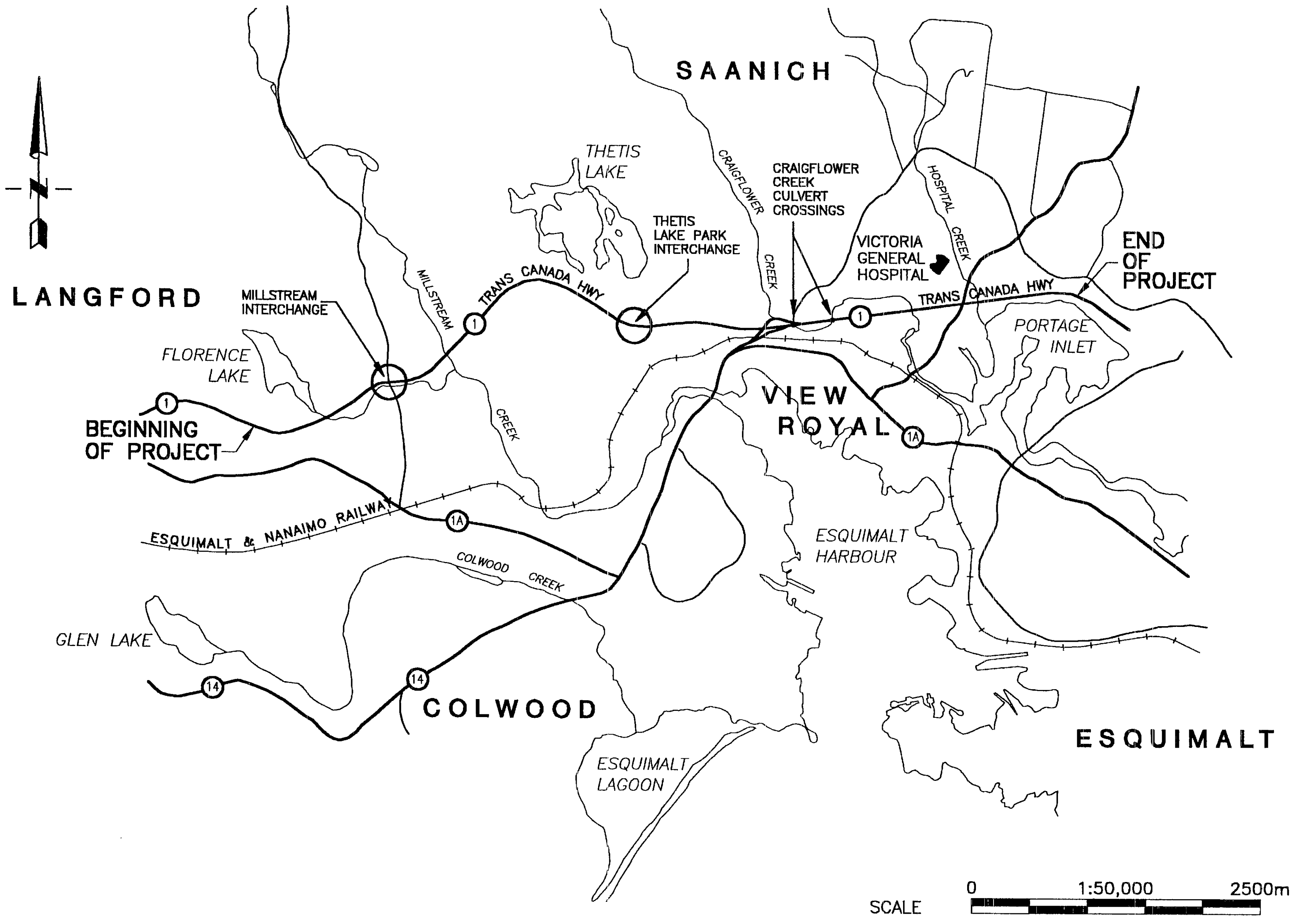

Figure 1: Trans Canada Highway – Victoria Approaches Location Plan

considered to minimize the effects of the soft soils. These methods are described with reference to three specific locations on the project.

GEOLOGY

The local bedrock throughout the project area is the Wark Gneiss, a dark coloured unit of an early Paleozoic metamorphic complex that underlies much of the Greater Victoria region. The Wark Gneiss is a well foliated to massive hornblende and plagioclase rock with minor quartz and magnetite (Muller 1983). The rock is moderately strong with closely spaced discontinuities. Typically, in the Victoria area the rock surface is very undulating with irregular depressions often with very steep sides. This undulating surface was produced as a result of glaciation. Glacial grooving and striation is common on the rounded and smooth rock surface. At the close of glaciation, silty marine clays were deposited, blanketing the Victoria area. In the deepest depressions, soft blue-grey clay of up to 30m thickness was deposited. Often, a thin (approximately 2m thick) layer of glacial till is present between the rock and the clay. At higher elevations, the clay was either never deposited or subsequently eroded such that the local topography consists of bedrock hills or knolls surrounded by low lying depressions filled with clay. As sea levels fell after the close of the Pleistocene epoch the upper few metres of the glaciomarine clay dried out changing from a soft blue grey clay to a very stiff brown clay.

During deglaciation, glaciofluvial outwash was deposited as an extensive sand and gravel deposit in the Colwood and Langford area close to the western end of the project. Throughout the outwash deposit are the old stream channels and kettle holes that have been infilled with peat and other organic soils up to 8m thick.

GEOTECHNICAL DESIGN CONCERNS

Problems associated with excessive settlement and slope stability are abundant throughout the project area. Three of the most challenging areas are discussed, including ground conditions, engineering concerns and the solutions to these problems. The areas discussed are:

1. Millstream Interchange, where a highway underpass is proposed over a peat deposit;

2. Highway widening at Thetis Lake Park requiring the construction of a 16m high embankment over very soft clay; and,

3. Craigflower Creek, where the extension of three existing culverts, the design of two new bridges and the realignment of 150m of Craigflower Creek is required.

MILLSTREAM INTERCHANGE

An underpass is required to carry Millstream Road over the Trans Canada Highway at the location of a current at grade intersection. The preliminary bridge design comprise two span structure of 25 and 33m lengths. The width of the underpass is 35m wide and will be located approximately 25 m to the west of the existing Millstream Road.

Ground Conditions

The local topography is flat to the south, and along the mainline of the Trans Canada Highway to the west and east. To the north, bedrock quickly rises to elevations of 20m above the highway on either side of Millstream Road. A drainage channel, called Kershaw Canal flows through the interchange area into Millstream Creek, a fish bearing stream located approximately 550m to the east.

The soil stratigraphy consists of loose to compact sands and gravels having a range of Standard Penetration Test Blowcount (SPT) values of 3-17, and a variation in thickness of 0-5m. The granular deposits overlie a peat layer that is 0-7m in thickness with a range of SPT values of 1-5. Below the peat layer is a second layer of sands and gravels having a range of SPT values of 11-58 and a thickness of 0-8m overlying bedrock (see Figure 2). A discontinuous layer of clay 0-5m thick was encountered with SPT values of 5-20.

The peat extends 45m north of the Trans Canada Highway to 125m to the south. The peat is bordered by Millstream Road to the east and extends approximately 100m to the west.

The peat, based on field and laboratory testing, has a range of moisture contents of 100-930%, laboratory vane shear strengths of 23-98kPa, field vane shear strengths of 21-43 kPa, and undrained shear strengths of 5-14 kPa. The settlement characteristics of the peat, based on a consolidation test, consist of a coefficient of consolidation (C_c) of 4.0 and an initial void ratio (e_o) of 9.5.

The thickest sequence of peat coincided with the approximate location of the south abutment, including the on and off ramps. Fill heights range from 3 to 8m. At the highest fill sections, primary settlements of up to 2.5m were calculated. The primary settlement would be expected to occur rapidly with 90% occurring within the first year following construction. Stability of the fills was also identified as a major concern.

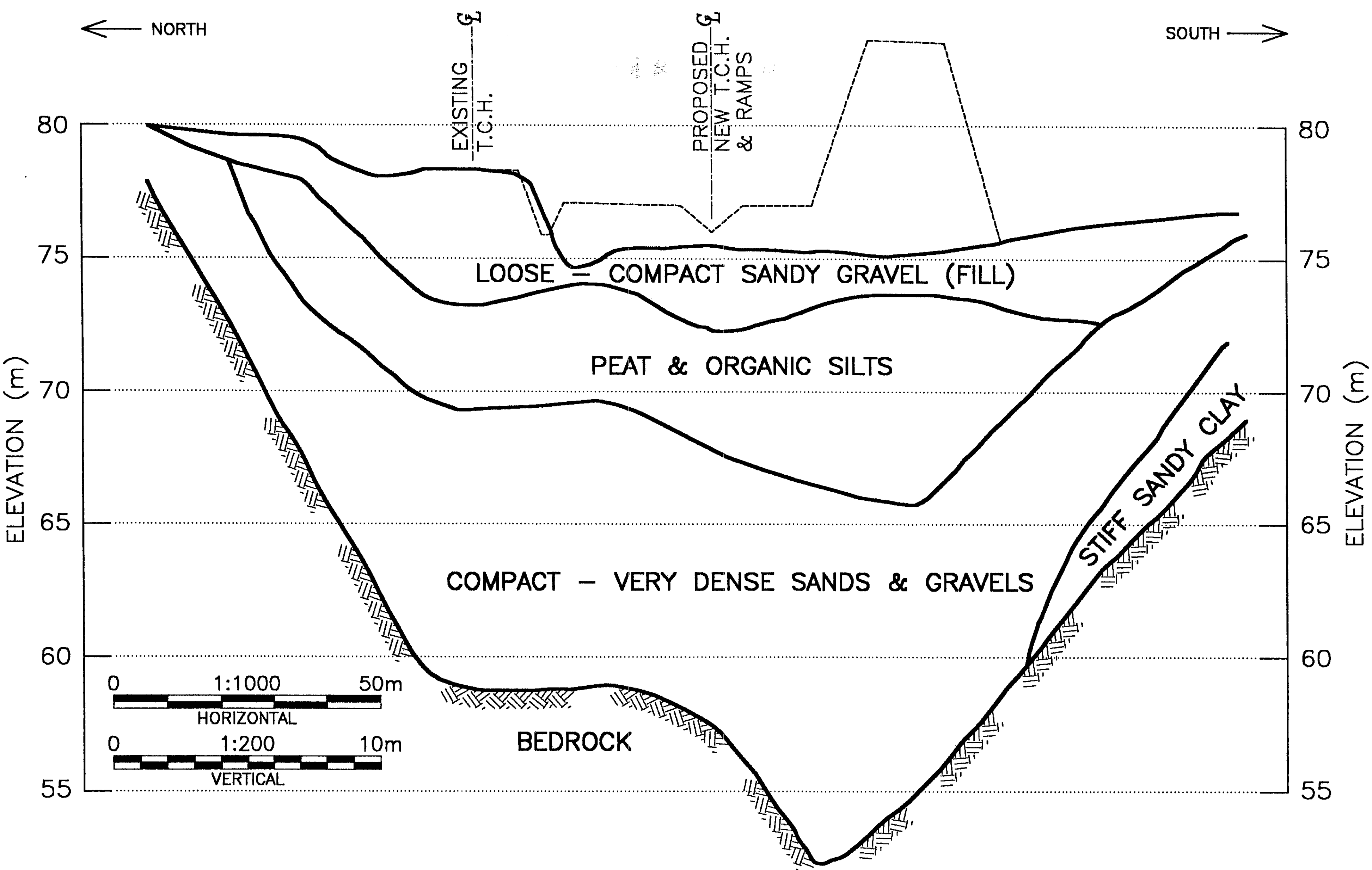

Figure 2: Trans Canada Highway – Victoria Approaches Millstream Interchange – Schematic Cross Section

Design Options

Several options were considered to reduce or eliminate problems of excessive settlements and improve the stability of the structure and associated ramps. They are as follows:

1. Surcharging of the Peat Zone
2. Lightweight/Ultralightweight Fill
3. Excavation of Peat

1. Surcharging of the Peat Zone

The benefits of dealing with the settlement problem by surcharging were relatively low cost and relatively easy construction techniques. However, disadvantages included the length of time required, which was long for the accelerated construction period; concern of the effects of long term secondary consolidation, and the potential for excessive settlements in the existing Trans Canada Highway which has to be kept open during the construction period.

2. Lightweight/Ultralightweight Fill

Lightweight (woodwaste) and ultralightweight (polystyrene) fill was considered in an effort to reduce the loadings and consequently the settlements. However, several problems were identified including the very high cost of the ultralightweight fill and environmental concerns of placing lightweight fill (woodwaste) in the vicinity of a creek.

3. Excavation of Peat

Merits of the removal of peat included elimination of both construction and long term settlements as well as improvements of slope stability. Additionally, it allows for the structure to be founded on spread footings as opposed to piles.

After consideration, Option 3, excavation of the peat, was selected. The peat is to be removed in sections in an "excavate and replace" sequence while pumping will maintain a dry hole. It was discovered during the investigation that water pumped from the excavation could not be discharged to the adjacent Kershaw Canal due to high levels of several metals. To deal with this concern, water pumped from the excavation will be transported to the north side of the Trans Canada Highway through a conduit to a natural storage area where it will be allowed to gradually filter back towards the peat area. After the peat has been removed and replaced with compacted granular material the structure will be placed on spread footings.

The excavated peat will be stockpiled for later use in landscaping.

HIGHWAY WIDENING AT THETIS LAKE PARK

The Thetis Lake Park Interchange provides access to Thetis Lake Park and is a subsidiary interchange to the main Colwood Interchange. Adjacent to Thetis Lake Park the Trans Canada Highway is to be widened and moved southwards from the existing alignment into a large depression. The local topography consists of a small valley running approximately north-south, with bedrock bordering along both the west and east sides. The valley fans out as it progresses south. The present two lane highway crosses the edge of the depression with a 13m high fill.

Ground Conditions

In general, the area between the valley walls consists of a sequence of glaciomarine clays and till overlying bedrock. While bedrock outcrops along both walls of the valley, it is up to 20m deep near the valley centre.

Subsurface investigations carried out to date has included 12 boreholes, 2 test pits, and 6 electric cone holes to delineate soil boundaries and collect samples in the area of the proposed fill. The depths of holes varied from 3 to 32 metres. For a majority of the test holes, Standard Penetration Testing (SPT) and undisturbed Shelby sampling was carried out at 1.5m intervals. Insitu vane shear testing was carried out in selected holes in the fine grained soils.

The laboratory testing program consisted of visual identification and moisture content determination of the granular soils, while testing of the fine grained soils included Atterberg Limits, laboratory vane shear strength, triaxial testing, unconfined compression strength, and consolidation tests.

Typically, the soil profile consists of 3-5m of stiff to very stiff desiccated brown clay over 0-16m of very soft to firm clay over 0-2m of very dense till on the bedrock surface (see Figure 3).

Field and laboratory testing was conducted to determine the shear strength profile of the upper and lower clay. Results from samples recovered in the upper clay had a range of laboratory vane shear strengths of 87-150kPa and unconfined compression shear strengths of 50-70kPa. Triaxial testing indicated that the upper clay had a friction angle (Ø) of 25°-26° and a cohesion of 14-22kPa. Classification tests indicated that the samples had moisture contents of 19-41% and liquid limits of 38-67%.

Test results from samples recovered in the lower clay had a range of field vane shear strengths of 26-30kPa, laboratory vane shear strengths of 46-87kPa and unconfined shear strengths of 26-47kPa for the area close to the toe of the existing fill. Further to the south, in the area of the proposed ramp, the strengths of the clay tends to be weaker with values in the range of 13-23kPa for field vane shear strengths, 18-32kPa for the laboratory vane shear strength, and 7-14kPa for the unconfined shear strengths.

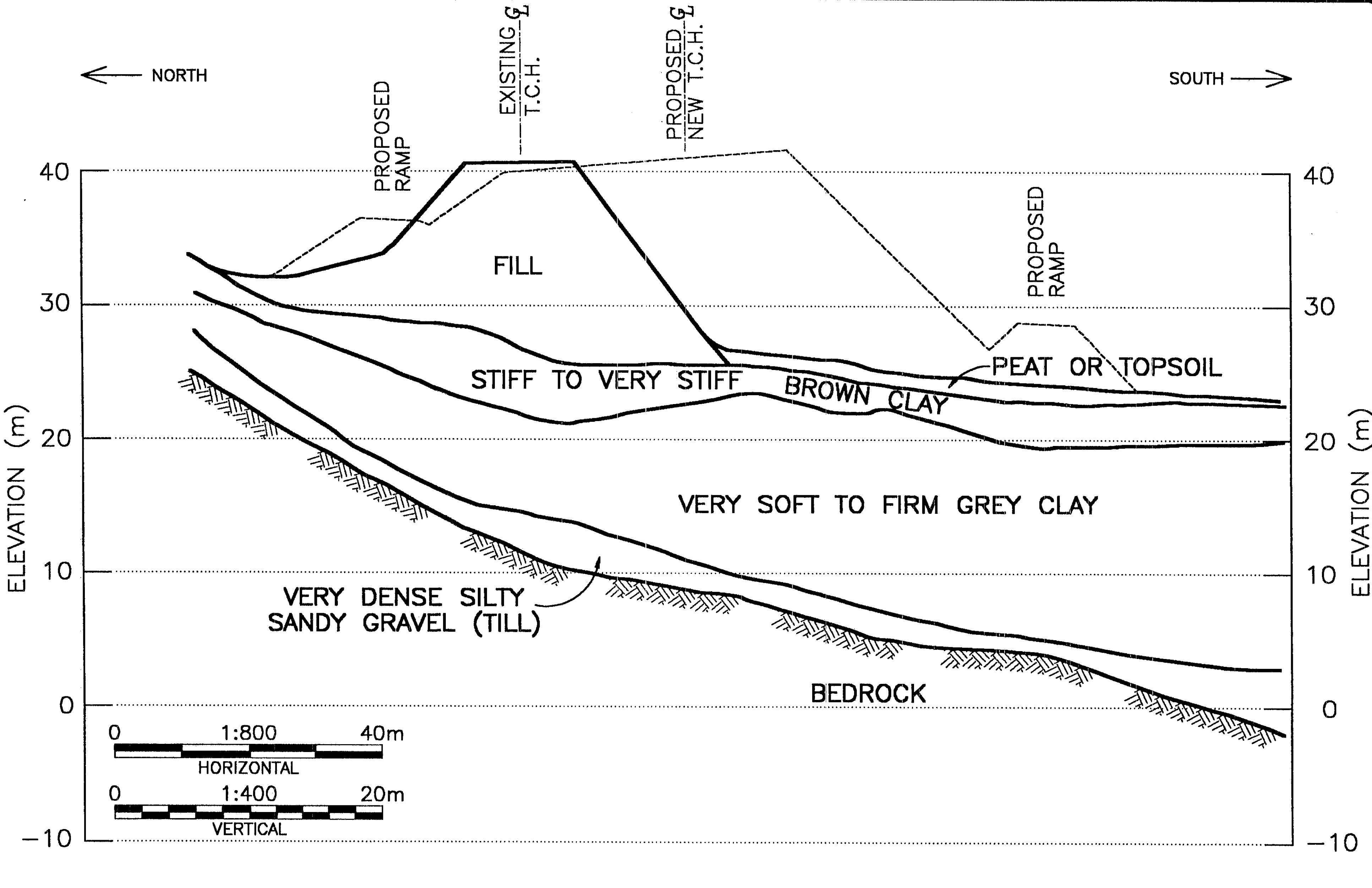

Figure 3: Trans Canada Highway — Victoria Approaches
Thetis Park Interchange — Schematic Cross Section

Additionally, triaxial tests determined that the lower clay has a friction angle (Ø) of 19°-22° and a cohesion of 7-11kPa. Moisture contents of 32-52% and liquid limits of 36-60% were obtained from samples recovered from the lower clay. The liquidity indices ranges from 1.0 to 1.7.

The compressibility of the lower clay is high, with values of C_c ranging from 0.26 to 0.50 based on consolidation tests. However, the electric cone data indicates some overconsolidation in the upper and lower zones of the soft clay.

The site presents difficult soil conditions for the proposed fill. An analysis of the stability of the existing fill indicated a current static factor of safety of about 1.1. Despite its low apparent level of stability the existing embankment shows no sign of distress or evidence of significant settlements. Applying a seismic coefficient of 0.15 to represent seismic loading led to a prediction of excessive deformation with a pseudo-static factor of safety of less than 0.9.

Analyses were also conducted to assess the stability of the proposed embankment assuming that it would be constructed of mineral fill and no staging of construction would be possible. The results of these analyses indicated that the fill would fail prior to its full height being reached. In addition, settlement analyses indicated settlements in the range of 1.5-2.0m, although this would extend over many years with about 30% expected during construction.

Ground Improvement Options

As a result of these analyses it was determined that improvements to the proposed embankment foundations will be necessary to achieve a satisfactory performance. The various options considered for improving the foundation are as follows:

1. Mineral Fill-Staged Construction
2. Piles with Mineral Fill
3. Piles with Mineral Fill & Woodwaste
4. Insitu Columns
5. Bridge
6. Woodwaste & Polystyrene Fill
7. Retaining Wall with Polystyrene Fill
8. Excavation of Clay

1. Mineral Fill - Staged Construction

The construction of a mineral fill embankment at normal construction rates would lead to the generation of high pore water pressures in the soft clay, due to the very low permeability of the clay. Failure would occur before the embankment was completed (probably after reaching 5-6m in height) unless the pore water pressures were allowed to dissipate. Staged construction is commonly adopted in these circumstances, but the construction schedule does not allow sufficient time for this, which may require one year or more. The installation of wick drains was considered, but local experience in similar soils with this technique was limited and not encouraging. Between 1.5 and 2.0m of total settlement is estimated to take place over a fifty year period.

2. Piles With Mineral Fill

305mm section (53lb/ft) H-piles would be driven to refusal on a 1.6m triangular grid pattern. There would be approximately 1800 piles with an average length of 14m. The piles would be installed to create a reinforced block of soil approximately 40m in width and 100m in length beneath the new embankment. The piles would be cut off at ground level and a 290mm x 290mm square, 1/4" (6.35mm) thick steel plate placed on the pile head. A relieving platform would then be constructed over the piles to spread the load from the embankment fill. The relieving platform would consist of 1.2m of minus 75mm granular fill with 3 layers of biaxial geogrid.

The factor of safety under seismic loading is estimated to be about 1.1 The advantages of this reinforcement method are the ready availability of suitable piles, together with the relatively short construction time frame of about 8 weeks.

3. Piles With Mineral Fill and Woodwaste

Partial replacement of the mineral fill with woodwaste encapsulated with polyethylene sheeting was considered as an alternative to the piles with granular fill. The woodwaste would require pre-compression with a temporary surcharge of granular fill of about 2m thickness. This alternative would reduce the number of piles needed while maintaining the stability of the existing embankment in a design earthquake.

4. Insitu Columns

Hayward Baker Inc., a specialist ground modification contractor, was contacted for information on possible ground improvement options for the site. Originally, either vibro concrete columns (VCC) or stone columns placed at approximately 2m spacing through the clay were considered. The columns would be covered with a granular layer reinforced with geogrid. After a review of further information, some technical difficulties with the options became apparent. The VCC option could cause disturbance of the sensitive clays resulting in temporary loss of total support with bulbs or necking of the concrete columns. A field trial would be required to address these concerns. The stone columns would be at the limit of their theoretical design and Hayward Baker suggested that they should be combined with the installation of lime columns.

5. Bridge

A bridge option with three 30m spans was developed by Reid Crowther, geometric designer for the project. The option consisted of two separated parallel sets of spans, with piers supported on piles driven to rock. One set of spans required removal of about 3m of the existing embankment and the driving of piles through the existing fill. The other spans would be supported on piles adjacent to the toe of the embankment. The proposed piles, which totalled 13 for each pair of piers, would be 600mm diameter steel pipes filled with reinforced concrete.

In the event of moderate to high earthquake activity the embankment is, as noted above, expected to fail. Any structural elements, such as piles, passing through the failed zone will experience very substantial bending and shearing forces as the embankment and its foundation respond to the seismic forces. Preliminary calculations indicate lateral forces of the order of 600kN per lineal metre of embankment for a horizontal seismic coefficient of 0.15. The piles will pick up more load than their individual width, due to load concentrations, and may carry embankment forces from many metres of embankment, possibly up to 10m. In addition, the lateral load capacity of the pile group is reduced as the spacing decreases. The proposal above has a pile spacing of about 5 times the pile diameter, which is equivalent to a lateral load capacity reduction factor of about 0.6. Therefore, substantial and possibly impractical reinforcement of the piles and ground around the piles would be necessary.

6. Woodwaste and Polystyrene Fill

Construction of the embankment using lightweight materials would reduce the overall mass of the fill and allow construction to take place with reduced risk of failure and reduced settlements. Woodwaste was originally considered but with a unit weight of $6kN/m^3$ the fill would still be too heavy and failure of the soft clay is likely. A combination of approximately 10m woodwaste and 6m polystyrene was considered and found to be light enough to be stable. The woodwaste would be encapsulated and covered with polystyrene which in turn would be covered with approximately 1.5m of sand.

7. Retaining Wall With Polystyrene Fill

In order to minimize the volume of ultralightweight fill (polystyrene) a wall option was considered. The fill behind the wall would be composed of polystyrene and the wall itself would simply be some kind of cladding to protect the polystyrene. Like option 6, this option does not improve the stability of the existing embankment.

8. Excavation of Clay

This option consists of driving a sheet pile wall at the toe of the existing highway embankment, excavating and hauling to waste $125,000m^3$ of clay, backfilling with imported sand or rock and constructing the $60,000m^3$ embankment using mineral fill.

Waste sites had been identified on the adjoining properties now owned by the Ministry. The wasted clay would be placed to depths of 3m over an area of 4ha. and that the site would take a minimum of 3 years to dry sufficiently prior to offering the land for re-sale. As an alternative to wasting the excavated clay on land, barging the material to sea and dumping under permit from the Department of Fisheries and Oceans has been considered. The main concerns with this option were the difficulty of clay removal coupled with the potential failure of the existing highway embankment during excavation.

Each option was considered in terms of cost, risk of failure, risk of schedule delay, risk of cost over run and impact on long term maintenance. Table 1 outlines the options and their relative ratings.

Table 1 - Option Ratings

No.	Option	Relative Cost	Static Failure Risk	Delay Risk	Cost Risk	Maintenance Cost
1	Mineral Fill-Staged Construction	VL	VH	VH	L	H
2	Piles with Mineral Fill	L	L	M	L	M
3	Piles with Mineral Fill & Woodwaste	M	L	M	L	L
4	Insitu Columns	M	M	H	H	M
5	Bridge	H	L (1)	L	L	M
6	Woodwaste & Polystyrene Fill	VH	M (2)	M	M	M
7	Retaining Wall with Polystyrene Fill	H	M (2)	M	L	M
8	Excavation of Clay	H	VH	H	L	L

Key

VL	Very Low
L	Low
M	Medium
H	High
VH	Very High

Notes: (1) High risk to pier foundation in earthquake
(2) Existing embankment fails with seismic coefficient of 0.15

Mineral fill with staging would be the typical construction method but with insufficient time available and the high possibility of a failure occurring this option was rejected. As an analysis of the existing highway embankment indicated failure would occur in a moderate-large earthquake the lightweight fill and retaining wall options were also rejected. Excavation of the clay by trenching and back filling with blasted rock was considered until a test excavation highlighted the difficulties involved.

Maintaining the stability of the existing highway was a major concern and the cost was high compared with other methods. The bridge option was considered to be a relatively straight forward solution until consideration of the earthquake stability of the existing highway embankment required the design of an expensive piled foundation to resist the increased lateral loads. The ground improvement options of piles or columns were considered to be practical solutions. Piles were chosen over the column option because of the lower estimated cost and the greater uncertainties involved in the use of vibro concrete columns or stone columns.

CRAIGFLOWER CREEK

A small fish bearing stream called Craigflower Creek meanders through the Colwood Overpass section of the Victoria West Approaches. The Creek crosses the existing Trans Canada Highway and the proposed access ramps at six locations (see Figure 1). Two criteria were established for the creek by the Department of Fisheries and Oceans. Any crossings that are presently accommodated by an existing culvert can utilize a culvert extension. (There are three existing culverts, all requiring culvert extensions). Any crossings that are at present untouched or accommodated by a bridge will require a bridge structure. (Two areas are new crossings and one area has an Acrow structure spanning the creek). Also, a 150m long section of the creek is to be realigned to accommodate the widening of the east approach fill of the Colwood Overpass.

Ground Conditions

The ground conditions at the creek crossings are typical of the Victoria West Approaches area. There is a layer of brown, desiccated, very stiff clay from the surface to 3-5 m depth. Below the brown clay is a very soft, sensitive, highly compressible, grey clay that has a moisture content close to or above the liquid limit. The depth of the grey clay varies, but at the creek crossings it is typically 15-25m deep. Below the grey clay is a thin, very dense glacial till layer overlying bedrock (see Figure 4). The creek channel bottom is usually at the brown/grey clay boundary.

Creek Structures

The geotechnical considerations of Craigflower Creek can be divided into three areas:

1. Culvert extensions
2. Bridge structures
3. Creek realignment

1. Culvert Extensions

Craigflower Creek crosses the existing Trans Canada Highway at three locations. These crossings are accommodated by large cast-in-place concrete box culverts. Two of the crossings were originally single box culverts, 3.7m wide x 2.5m high, approximately 30m long, which were constructed in the early 1950's. The culverts were then extended approximately 6m on either end, and a matching twin culvert was placed adjacent to the existing culverts in the late 1970's. The third crossing is a single concrete box culvert, 3.7m x 3.7m, originally about 25m long, which was later extended

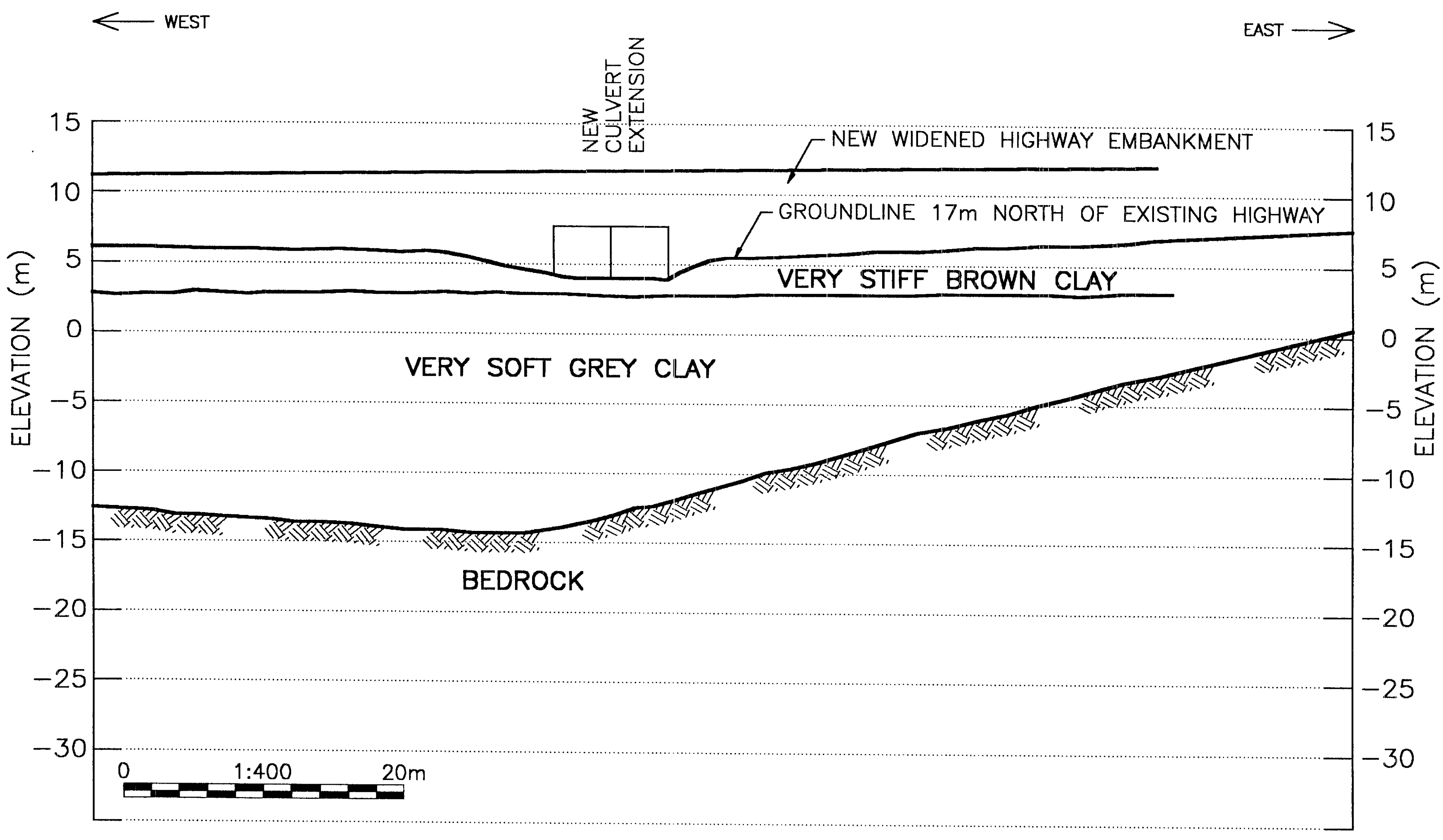

Figure 4: Trans Canada Highway – Victoria Approaches
Schematic Profile at Craigflower Creek Crossings

about 19m in the late 1970's. All three culverts have undergone differential settlements, with the centre of the culverts approximately 0.4-0.6m lower than the culvert ends. All three culverts will be further extended, with the longest extension being 47m. Two main geotechnical concerns were emphasized in the culvert extension design. The first is the undrained stability of the soft grey clay during construction of the large fills that are to be placed around the culverts. The fills will be up to 47m wide and 7-8m high. Undrained stability analyses were performed using the slope stability program G Slope.

The stability analyses indicated that the fills can be constructed up to a maximum height of 4-6m in one stage. At that point, the pore pressures within the grey clay will be at a critical level for stability, and instrumentation results will dictate if construction can proceed. The instrumentation that will be required to monitor the performance of the clay are pneumatic piezometers and settlement plates placed in the clay under the fill, slope inclinometers placed at the toe of the fill, and movement hubs placed mostly at the toe of the fill.

The second geotechnical concern was the long term settlements of the culverts and surrounding fill, which are estimated to be in the order of 1.0-1.5m, and the differential settlement between the existing and the new culverts. Several options were considered:

The first option was to utilize a sacrificial culvert and preload the soft area to remove as much of the settlement as possible. The deformed culvert would be later removed and replaced with a permanent culvert. The advantage of this option is the simplicity and relatively low cost of construction. Unfortunately, the settlement analysis of the soft grey clay indicated that the settlement rate of the clay is very low, and only 20-40% of the settlement would be obtained within the first year of preloading. This is assuming that the fill can be constructed to preload height fairly quickly which, as mentioned above, may not be possible due to the undrained stability problems of the fill above 4-6m height. A tight construction schedule precluded this option.

The second option was to construct a short bridge structure founded on piles to bedrock, which would eliminate any culvert connection or differential settlement problems. The disadvantages were the high cost, and the problems of long term settlement of the approach fills to the bridge.

The third option was to use ultralightweight fill (polystyrene) around the culvert. This would practically eliminate the settlement problem and the undrained shear strength problem during construction. The tight construction schedule would also be accommodated. The disadvantages are the extremely high cost of the ultralightweight fill (approximately $100/m^3), and the fact that the area of soft grey clay is quite large. Therefore, if ultralightweight fill was used around the culvert extension, there would still be the construction and settlement problems away from the culvert as the ultralightweight fill tapers out and mineral fill loading takes effect.

The final option, which is the one chosen, is to use a flexible culvert, comprising concrete box segments with flexible connectors. The culvert will be oversized, to accommodate the loss in hydraulic capacity after settlement. If the water flow after settlement is considered inadequate, gravel may be placed along the culvert bottom to create a level channel. This option unfortunately does not reduce the long term settlement or construction problems, but it does allow the culvert to deform with the fill settlement, regardless of how long the settlement takes. The advantage of this option, from a geotechnical point of view, is that it reduces the reliance put on the geotechnical input parameters, and puts the onus on the structural parameters. The primary geotechnical input is the calculation of a long term settlement profile along the culvert.

2. Bridge Structures

Two new bridge structures and a replacement structure for an existing Acrow bridge will be required. The soil stratigraphy at one of the new bridge structures is a stiff brown clay from the surface down to bedrock at shallow depth. Therefore, no foundation problems are anticipated. The other two crossings, which are adjacent to each other, have a thick soft grey clay layer below the stiff brown clay layer. The bridge structures themselves will be supported on piles driven to bedrock, which will eliminate any settlement problems for the structures.

The approach fills, however, are estimated to settle 0.3-0.5m. The fill heights are in the order of 3-5m, which should not cause a stability concern during construction. The tight construction window, however, will eliminate the possibility of preloading, which means the long term settlement will cause a bump to occur at the bridge abutments. Another problem is the high dragdown forces that will be applied to the foundation piles by the soft grey clay during consolidation. This will require large piles to overcome these loads. The solution at these locations is to use ultralightweight fill for the approach fills, which will eliminate the settlement and dragdown force problems, and also reduce the lateral loading on the bridge abutment walls during a seismic event.

3. Creek Realignment

Approximately 150m of Craigflower Creek is to be realigned just east of the existing Acrow structure to accommodate the widening of the east approach fill for the Colwood Overpass. To get approval from the Department of Fisheries and Oceans, a proposal was developed to try and make the new creek channel a more natural environment for the fish. This is to be accomplished by four "revetment structures". These structures will be reinforced concrete slabs approximately 3m wide, supported on the creek side by foundation piles and anchored into the creek bank. Soil and vegetation will be placed on top of the slab, with the idea that the vegetation overhanging the edge of the slab will provide shade for the fish.

Since the feasibility of the proposal hinged on the cost of these structures, the foundation piles could not be elaborate or expensive. A subsurface investigation and foundation analysis determined that the loads on top of the structures would be low, and therefore, the pile capacities would not be large. The foundation design concluded that 305mm diameter timber piles, 10-15m long could be used. In the locations where the piles could reach a dense sand and gravel layer, the piles would be spaced 6.5m apart. In the location closest to the existing Acrow structure, where the soft grey clay is thick, the piles would only take capacity in friction, and therefore, the pile spacing would be 1.5m.

CONCLUDING REMARKS

A tight design and construction schedule for the project has required the use of alternative methods to accommodate the problems associated with the stability and settlement of soft soils. At each of the three areas with the most severe design constraints, various alternatives were considered. Final selection of the option to be employed was based on a balance of cost and time available for construction.

ACKNOWLEDGMENTS

This paper is presented with the permission of Mr. V. Collins, Deputy Minister of Transportation and Highways. The authors would like to thank David Broomhead, Turgut Ersoy, and Ivan Grof for their assistance and comments.

REFERENCES

Geotechnical and Materials Engineering Branch, 1995, Trans Canada Highway No. 1, Victoria Approaches, Vancouver Island Highway Project - Final Geotechnical Design Report, Ministry of Transportation and Highways, Victoria, B.C.

Muller, J. E., 1983, Geology Victoria, B.C. Map 1553A, Geological Survey of Canada.