# Slope Redesign and Slope Remediation during Design/Build Construction at Kicking Horse Canyon– Phase 2

By Richard Humphries, P.Eng.
Don Lister; P.Eng.
and
Paul Schlotfeldt, P.Eng.
Golder Associates Ltd., BC

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## ABSTRACT

This paper describes the adjustments to the design and stabilization of the excavated highway slopes during construction of the Phase 2 Upgrades to the Trans Canada Highway in Kicking Horse Canyon between Golden and Yoho National Park. After the start of the construction of the design/build project, the detailed snow avalanche analysis indicated that the bid design for slopes on the western end of the project that were proposed to be cut at an angle of 1:1 for geological reasons would generate an unacceptable avalanche risk. Consequently, it was necessary to steepen the rock and overburden cuts from 1:1 to 0.5:1 and install slope support. In some areas the rock structure varied significantly over short distances and caused unexpected, local failures which required "on-the-fly" designs of additional rock and overburden support in the form of steel mesh, shotcrete and rock bolts.

The Kicking Horse Phase 2 Project consisted of 6 kilometres of new 4 lane highway which included the 400m long, 100m high New Park Bridge over the Kicking Horse River. The project is in the heart of the Rocky Mountains in an area that is subject to rockfalls, snow avalanches and debris flows. The natural slopes in the area of the project are very high and steep and many of the soil and rock units are relatively weak – a combination that provided significant geotechnical challenges and resulted in slope instability in places that required remediation to constructed works.

This paper describes the initial and revised designs and construction of the up to 75m high rock slopes and the support for the till and colluvium slopes above the rock cuts. The paper also describes the revisions to the design of rock support for one of the large bridge piers as a result of discovering adversely dipping rock jointing after overburden removal.

Construction of the Phase 2 Project was carried out under a design-build contract which was a part of the Design-Build-Finance-Operate Concession. This contract delivery method enables projects to be brought into service in compressed schedules and has been praised by the politicians, but it involves significant challenges and additional risks for design engineers. These aspects of the Kicking Horse Phase 2 Project are also addressed in the paper.

## INTRODUCTION

The British Columbia Government is in the process of upgrading the Trans Canada Highway between Golden and Yoho National Park in the Rocky Mountains. This section of the Highway 1 was built in the mid-1950s and has had no significant upgrades since that time. Before the start of the upgrades, the section was one of the highest rock fall hazard areas in the Province with an accident rate double the Provincial average. The highway carries 10,000 vehicles per day in the summer of which up to 25% are heavy trucks. Traffic volumes are expected to increase substantially in the next 25 years.
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Upgrading the highway between Golden and Yoho Park is proceeding in the following phases:

| Phase | Main Features | Cost | Complete |
|---|---|---|---|
| 1 – Yoho Bridge | 3.2 km of highway upgrades, 2 new (twin) bridges | $64m | 2006 |
| 2 – Park Bridge | 6.2 km of new 4 lane highway, replacement of Park Bridge | $143m | 2007 |
| 3 - Brake Check to Yoho Park and Golden Hill to West Portal | 4-lane upgrade, replace Mt. Hunter Creek bridge, grade reduction, environmental initiatives | $134m | 2013 |
| 4 – West Portal to Yoho Bridge | Tunnel and surface alternatives under consideration | $630m | ? |

The geotechnical issues and design changes in Phase 2 are described in this paper. This phase of the project was completed as a Public-Private Partnership. Trans Park Highway General Partnership is the concessionaire who arranged financing and construction and will maintain the highway for 25 years. Trans Park Highway Constructors was the design-build contractor. The Ministry of Transportation (MoT) web site states that construction was completed 21 months ahead of schedule, with approximately

$18 million in savings. The project was the recipient of the Premier's award and won the 2008 Canadian Consulting Engineers Award of Excellence in the Transportation Category.

The Design-Build-Finance-Operate contract delivery method enables projects to be brought into service in compressed schedules but it involves significant risks for the designers and constructors, especially the geotechnical engineers. Because of the steepness of the terrain, difficulty of access and time constraints, pre-construction investigations were minimal. Rock outcrops were also sparse along most of the new alignment, especially in the western half of the project, so preparation of the firm-price bid for the design-build project involved significant geotechnical extrapolation and considerable uncertainty. As a result some of the geotechnical design prepared for the design-build bid had to be adjusted when the actual site conditions were exposed during the excavation.

## SITE CONDITIONS

The natural slopes in Kicking Horse Canyon area are very steep and rise to approximately 900m above the river. The most prevalent natural hazards in the canyon are snow avalanches, rockfalls and debris flows which have caused highway closures several times in most winters. The natural hazards extend far above the cuts for the highway and cannot all be mitigated by the upgrades.

There is a 200m elevation difference between the East and West ends of the Phase 2 project. This requires high side-hill cuts at both ends of the project with a high through-cut and the bridge in the central part of the project.

The bedrock geology of the project consists of a sequence of tectonically deformed and layered Lower Paleozoic sedimentary rocks striking NW-SE and steeply dipping to the NE. All of the rock units have been subjected to strong, eastward directed, thrust faulting and as such contain numerous minor shears. The main rock types along the alignment comprise:

- The Mount Wilson Formation and the Upper Glenogle Formation consisting of white, grey and brown quartz sandstone or quartzite with siltstone and minor shale. These two units are exposed on the slopes along the south side of the new highway, west of the proposed bridge site.
- The Lower Glenogle Formation consisting of dark grey to black shale, argillite and argillaceous limestone. This formation is exposed mainly along the highway west of the new bridge.
- McKay Group Unit 6 which consists of medium grey, thin to thick bedded limestone with interbeds of slate and calcareous shale. This resistant unit forms the bulk of the steep ridge located to the east of the bridge.
- McKay Group Unit 5 consists of grey calcareous slates, with thin limestone and siltstone interbeds.

The surficial materials along the highway consist of mixtures of colluvium, ice contact deposits and glacial till deposits. These materials vary considerably in thickness from less than 1 m up to 50 m. The colluvium consists primarily of angular rock fragments generally 300 to 1000mm in size. The ice contact materials are variable in thickness and composition, and typically consist of sub-rounded gravel, sand, silt and clay mixtures of varying amounts. These deposits tend to form in pockets ranging in size from a few metres to tens of metres. The glacial till is a very hard dense mixture of silty clay, sand and gravel.

## SLOPE DESIGN, REDESIGN AND CONSTRUCTION

***Slopes East of New Bridge:*** The most eastern third (approximately) of the new highway alignment involves side-hill cuts in calcareous siltstone of the McKay Group Unit 5. The cuts are up to 65m high and are predominantly in rock which is overlain by a thin veneer of till overburden. The rock is moderately strong with variable jointing; however, there is a joint set that dips at around $70^{o}$ to the west. The selected slope design for the design-build bid was for a ½:1 cut in rock with an 8m wide rockfall catch ditch, as shown in Figure 1 and Photograph 1. The limited thickness overburden was laid back at 1.5:1. The wide-catch-ditch design was selected so that slope rock support could be minimized and so that rockfalls from the cut slopes will be caught in the ditch. Construction proceeded according to this design and few problems were encountered.

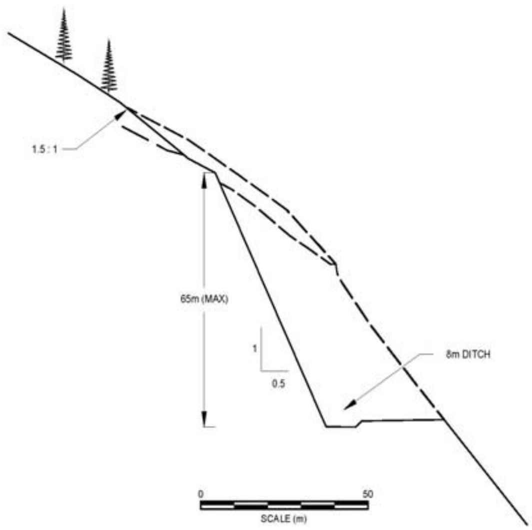



Figure 1: East End Road Cuts

Photograph 1 – East End Road Cuts

At the east end of the bridge, in the central third of the project, the new highway alignment cuts through a high, steep rock dome of limestone of the McKay Group Unit 6. The bedding exposed in the double sided cuts varies from near vertical to about 60 $^{o}$ and strikes across the highway alignment. The rock mass is relatively strong, so selected slope design for the 70m high rock cuts for the bid was for near vertical (1/6:1), cuts with pattern rock bolt support and 8m to 10m wide rockfall catch ditches, as shown in Figure 2 and Photograph 2. As with the far east end of the project, very few problems were encountered during construction and the rock turned out to be of better quality than expected from surface exposures, so the required rock bolting was minimal.

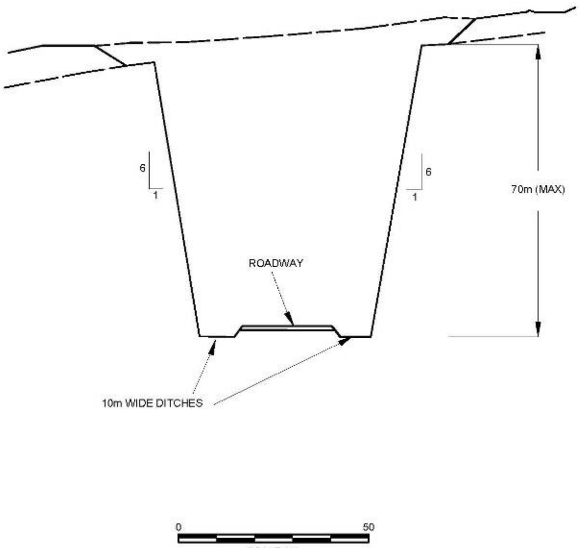



Figure 2: Through Cuts – East of Bridge

Photograph 2: 70m high through cut East of Bridge

***Slopes West of New Bridge:*** While the design and construction of the rock slopes in the central and eastern sections of the project went smoothly and followed the bid design, the opposite happened for the 1.6 km of slopes on the western third of the project. In this section of the project, there were only two boreholes and sparse rock exposure on which to base the bid design of the slopes. This information indicated that there would be limited thickness of overburden overlying Glenogle shale, which has bedding planes that dip at $45^o$ towards the new highway alignment. The upper natural slopes above the new highway cuts extend upslope at angles of between $35\ ^o$ and $50^o$ for over vertical 500 m. The bid design selected had rock cuts at the angle of the bedding (1:1) with slightly flatter slopes (1.1:1) in the overburden, as shown in Figure 3.

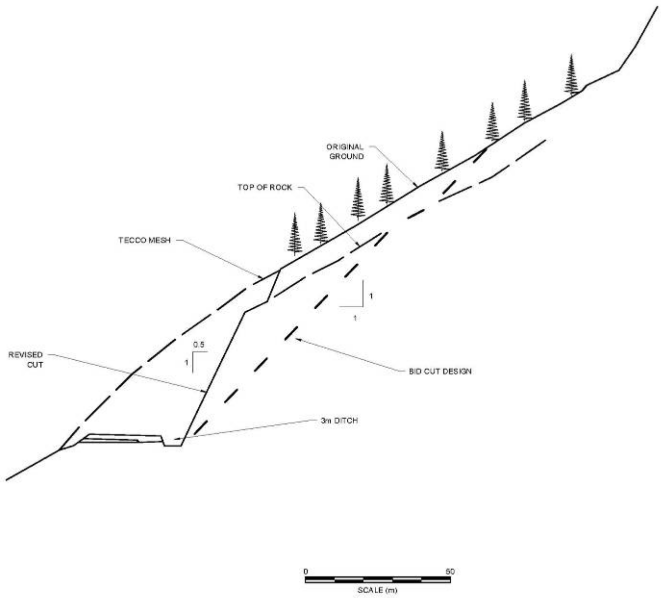
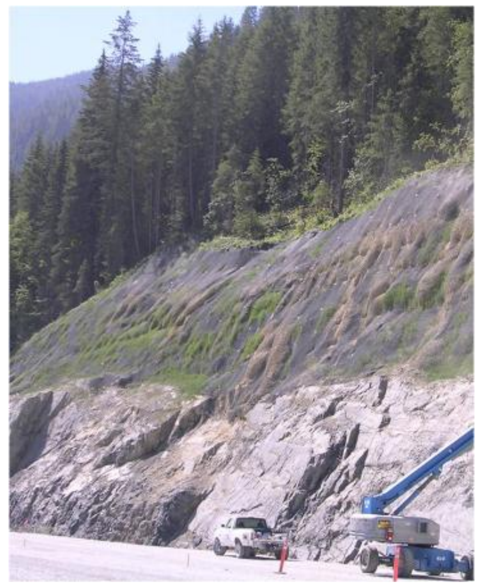



Figure 3: West End Road Cuts



Photograph 3: West End Cuts in Quartzite with Tecco Mesh

The excavation for these slopes had started when further hazard evaluation indicated that such a large exposure of 1:1 slope would increase the risk of generating snow avalanches. The initial excavations showed that the ground conditions were far more complex than the initial data indicated: the overburden was considerably thicker and the underlying material varied from Mt. Wilson quartzite, to Glenogle shale, to cemented black till, to a mix of calcareous tufa and cemented colluvium. Instead of a simple 1:1 cut, as initially designed, the cut slopes had to be redesigned to minimize the snow avalanche hazard and to suit the actual geotechnical conditions.

Since this was a design-build contract, time was always of the essence, particularly with the west end slopes which were one of the last items on the constructed schedule and, consequently, were close to the substantial completion and early-completion-bonus dates. Whenever geotechnical conditions varied, there was insufficient time to evaluate the problems, collect data, carry out analyses and have them checked, develop designs, prepare design drawings, and develop work plans. With such rapidly changing geology along this part of the alignment, many closely spaced station-specific support designs were required and the designers were always pressured to keep construction costs to a minimum and to develop designs that shortened the construction schedule. Under these conditions, inevitably some corners were cut and slope stabilization measures simplified to speed construction and, as a consequence, there were some local slope failures which had to be remediated.

The redesign of the slopes and the requirements for remediation of the local failures were as follows (from the west end of the new bridge to the west end of the project):

- 670m of ½:1 rock cuts in quartzite with a 3m to 6m wide avalanche/rockfall ditch at the toe of the slope and Tecco mesh and anchors to stabilize the overburden at ½:1, as shown in Figure 3 and Photograph 3. The Tecco mesh is a proprietary high tensile galvanized steel wire mesh that was supported and anchored by 25mm galvanized thread bar anchors installed into the overburden on a 3m by 2m triangular grid spacing. The rock cuts are up to 30m high and the overburden thickness varies from about 2m to 8m. The initial excavations of the rock indicated that the dip of the major rock structure (bedding and cleavage) in the quartzite was about $65^o$ so the cut slopes were designed to be just flatter than $65^o$ i.e. at ½:1. As it turned out the dip varied significantly and, in some sections, the dip was flatter than the cut slope angle which resulted in local rock slope failures, which undermined Tecco mesh anchors. Slope remediation, in the form of installing rock bolts in the lower rock slopes and replacing the Tecco mesh and associated anchors, together with shotcreting, was required in these areas;

- 360 m of 1:1 rock cuts in Glenogle Shale with Tecco mesh and anchors or flattened slopes in the overburden. Road-side avalanche walls were required in the high hazard areas. The shale cuts are a maximum of about 20m high and the overburden thickness varies from about 1m to 4m. These cuts behaved well except where there were local variations in the rock structure which caused slabs of shale to slide out of the slope. A few small remedial shotcrete buttresses with dowels were required to support overhangs and prevent progressive raveling;

- 350m of 1:1 cuts 20m to 30m high in cemented black till with the face protected by a rockfill blanket. The till was very hard and had to be excavated by ripping and local blasting. There were a number of face sloughs in the first spring after construction as the rockfill protective blanket was not initially installed in the haste to open the highway; and

- 210m of ½:1 to 1:1 cuts up to 60m high in calcareous tufa, cemented colluvium, quartzite and siltstone. This section is heterogeneous but the overburden soils and the rock units are well cemented from the calcium-rich seepage water so the slopes have remained stable at relatively steep slopes.

The Tecco mesh and associated anchors were designed to support the overburden at slopes of ½:1, as shown in Photograph 3. The overburden was initially assumed to be dense glacial till but some areas turned out to have a 2m to 3m thick layer of colluvium overlying the till. The anchors were designed to provide overall stability while the Tecco mesh was designed to support the face of the overburden. During construction, the till had sufficient cohesion to be stable at ½:1 in the cuts between excavation lifts while the colluvium sloughed to a flatter angle. Since construction, the Tecco mesh system has generally behaved adequately, though experience has shown that it is not practical to construct a smooth overburden face with neat “dells” at the heads of the anchors in difficult, steep terrain, so the mesh cannot be adequately tensioned against the face of the overburden. Consequently, the mesh does not provide a uniform, positive support pressure so local sloughing has developed behind the mesh in some areas.

Tables 1 and 2 summarize the initial and modified slope design criteria for the various sections on the project.

**Table 1**
**Initial Cut Slope Design**

| Station (West to East) | Material Type | Cut Height (m) | Rock Cut Slope | Overburden Cut Slope | Catchment Width (m) |
|---|---|---|---|---|---|
| 98+00 – 114+80 | Shale | 0-50 | 1:1 | 1.1:1 | 3 |
| 114+80-119+00 | Bridge | | | | |
| 119+00 – 120+40 | Limestone | 5-30 | 0.5:1 | 1.5:1 | 8 |
| 120+40-123+30 | Limestone | 10-80 | 0.17:1 | 1.5:1 | 10 |
| 123+30-124+40 | Soil | 5-10 | 1.5:1 | 1.5:1 | 1 |
| 124+40-136+00 | Calcareous Shale | 10-60 | 0.5:1 | 1.5:1 | 8 |
| 136+00-138+40 | Calcareous Shale | 0-5 | 0.5:1 | 1.5:1 | 3 |
| 138+40-146+40 | Calcareous Shale | 10-40 | 0.5:1 | 1.5:1 | 8 |
| 146+40 – 156+00 | Soil | 0-10 | 1.5:1 | 1.5:1 | 1 |

**Table 2**
**Final Cut Slope Design**

| Station (West to East) | Material Type | Cut Height (m) | Rock Cut Slope | Overburden Cut Slope | Catchment Width (m) |
|---|---|---|---|---|---|
| 97+50 – 99+60 | Quartzite, Siltstone | 0-50 | 1:1 | 1:1 | 5 |
| 99+60 – 103+10 | Glacial Till | 20-50 | 1:1 | 1:1 | 5 |
| 103+10 – 106+70 | Shale, Quartzite | 20-50 | 1:1, 0.5:1 | 0.5:1 | 5 |
| 106+70 – 114+80 | Quartzite | 0-50 | 0.5:1 | 0.5:1 | 3 [1] |
| 114+80 – 119+00 | | Bridge | | | |
| 119+00 – 123+30 | Limestone, Calcareous slate | 5-80 | 0.17:1 | 1.5:1 | 8-10 |
| 123+30 – 124+40 | Colluvial Soil | 5-10 | 1.5:1 | 1.5:1 | 3 |
| 124+40 – 146+40 | Calcareous Shale | 10-40 | 0.5:1 | 1.5:1 | 8[1] |
| 146+40 – 156 + 00 | Colluvial Shale | 0-10 | 1.5:I | 1.5:1 | 3 |

1. Locally wider for snow avalanche.
2. Landscape matting hydroseeding, rock blankets as required.
3. Anchored Tecco mesh.

## BRIDGE FOUNDATIONS AND PIER FOUNDATION STABILIZATION

The 400m long curved, steel girder New Park Bridge spans the previous highway alignment, the Kicking Horse River and CP Rail tracks. The five piers vary in height from 50m to 100m high and where labeled sequentially from the west end of the bridge to the east end of the bridge . All piers are supported on 900mm diameter piles which are socketted into rock – typically between 9m and 12m long into rock. A profile of the bridge is shown in Figure 4 and Photograph 4 and the bridge girders being launched is shown in Photograph 5.

The piles for Piers 1 through 4 are socketted in Glenogle Shale, which is a relatively weak, calcareous shale unit. For Piers 1 through 3 the piles were steel cased through the deep overburden. The overburden was ignored in the socket analysis of down-thrust loading but the overburden pressure was assumed to act on the cones of rock for uplift loading. The steep valley sides added to the design and construction challenges, however, foundation construction and pile drilling went relatively smoothly and in accordance with the bid design.

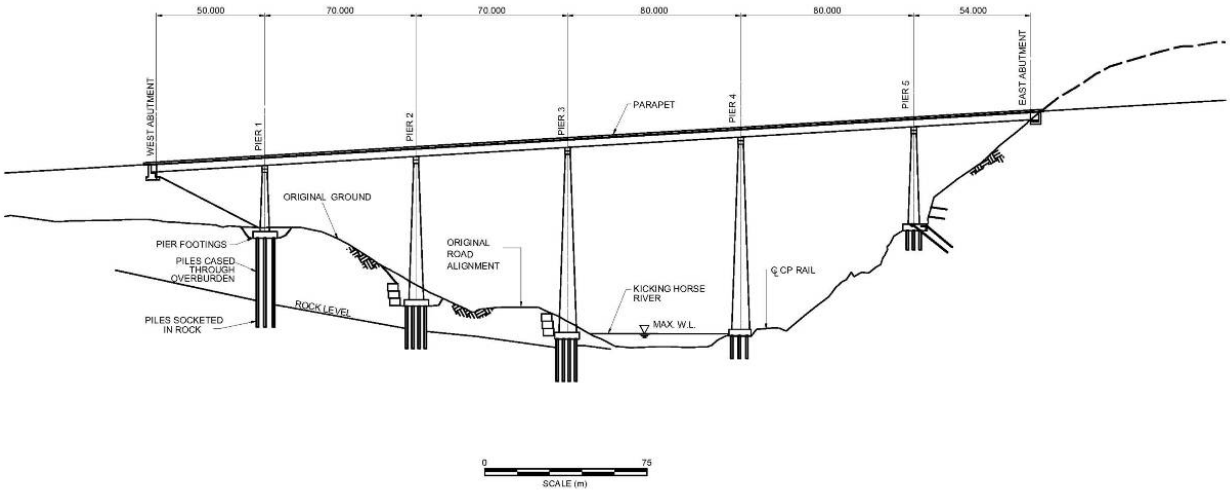



Figure 4: Profile of New Park Bridge

Photograph 4: New Park Bridge

Photograph 5: Launching Bridge Beams

Pier 5 (east end of the bridge) is on the McKay Group 6 limestone which is stronger than the shale but has adversely oriented rock structure. This pier is on a steep slope, about 50m above river level on the east side of the valley. The natural slope was covered by a veneer of colluvium and rock exposures were limited before construction started. Excavations for the pier foundations exposed a set of joints that dip out of the slope at about 1:1 which provides a potential sliding plane below the pier footing, as shown in Figure 5. Detailed on-rope geotechnical mapping was required to re-analyze the stability of the footing. The resulting stabilization measures are shown in Figure 5 and include extending the downhill side piles to increase the shear resistance and transfer load to below the potential sliding plane; installing nine 15m long, 65mm diameter rock anchors back into the slope from the pile cap; drilling drain holes across the sliding planes from the exposed rock face; and installing rock bolts in the face below the pile cap to prevent surface raveling.

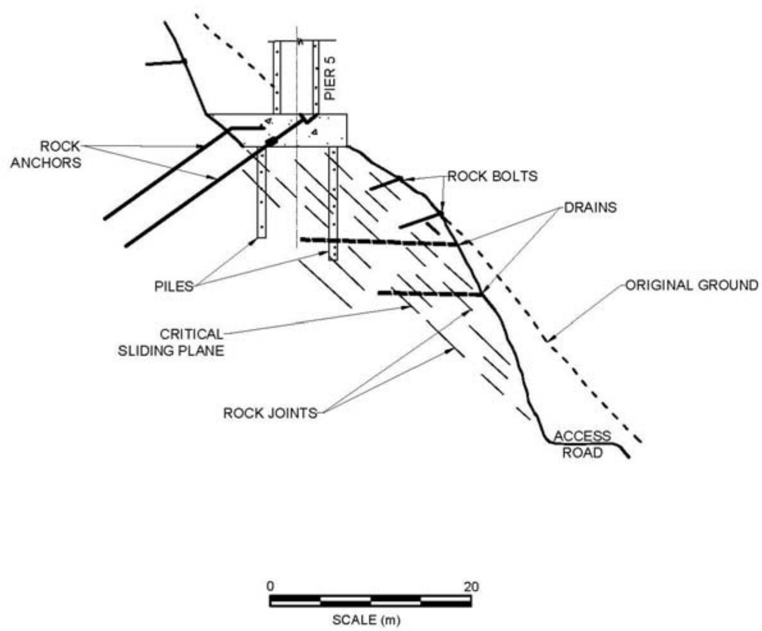



Figure 5:  Pier 5 Stabilization of Foundation

## LESSONS LEARNED

A number of design/build process lessons were learned on the geotechnical aspects of the project. These include:

- Design-build project procurement can enable project to be completed in a shorter time than conventional design-bid-build procurement;
- Design-build highway construction shifts the subsurface risks from the MoT to the design-build team – especially to the geotechnical designers;
- Schedule becomes critical on these contracts and there is often a rush to complete the project and pressure on designers to cut corners and develop designs that can be built quickly.  The system risks becoming build-design, rather than design-build;
- Unless the pre-bid investigations are thorough, unexpected ground conditions and design changes should be expected during construction;
- Design changes during construction usually have to be done in a rush which requires the senior geotechnical design engineers to be on site during much of the construction.