# 1 INTRODUCTION

## 1.1 Purpose

This report presents initial recommendations for the preliminary design of the rock fill embankments and cuts in overburden soils required for a proposed railway line, about 150 km long, that will carry iron ore between the Mary River Mine and Port Steensby on Baffin Island.

Initial design recommendations for rock cuts and two tunnels, which are also required for the railway, are presented in a separate report (EBA Engineering (2011). Initial geotechnical recommendations for the preliminary design of bridges and culverts for the railway line are also presented in a separate report (Thurber Engineering Ltd., 2011b).

*This report is subject to the Statement of General Conditions, which is included at the end of the text. The reader's attention is specifically drawn to these conditions as it is considered essential that they be followed for the proper use and interpretation of this report.*

## 1.2 Project Background

Baffinland Iron Mines Corporation is planning to extract iron ore from a deposit located near the Mary River on Baffin Island, about 800 km northwest of Iqaluit, Nunavut. The development will require construction of an open pit mine near Mary River and a port facility on Steensby Inlet so that the ore can be loaded onto ocean going vessels for transportation to world markets. The ore will be transported from the mine site to the port facility by rail, a distance of about 150 km.

The locations of the open pit mine, the port facility on Steensby Inlet and the proposed railway alignment are presented in Appendix D, Figure D.1.

The railway grade will be constructed by excavating cuts in topographic high areas and placing fill in low areas along the alignment. Cuts will be excavated in both overburden soils and rock, while the embankment fills will be constructed of blasted granite excavated from rock cuts or quarries. The railway grade will be constructed during both the winter months and the short thaw season, and will take several years to complete.

### 1.3 Scope of Report

The geotechnical recommendations presented in this report are intended to enable a more accurate estimate of the total volume of earthworks that will be required to construct the railway. The recommendations are intended to minimize the volume of earthworks required while limiting track settlements, culvert deformations and cut slope instabilities to a level that is expected to be manageable during railway operations.

The initial geotechnical recommendations presented in this report are based on a review of previous reports and technical memorandums, but primarily on the following:

1) The proposed railway alignment and profile (Canarail, 2010);

2) Geotechnical sections along the railway alignment (Knight Piesold, 2010a), which summarize subsurface conditions as observed in boreholes drilled in 2008;

6) Maps delineating the surficial geology within the railway corridor as presented on a series of map sheets (Knight Piesold, 2008);

3) The results of geophysical surveys undertaken at various locations along the alignment (EBA Engineering Consultants Ltd., 2008b);

4) Logs for boreholes drilled in 2006 and 2007, but which were not plotted on the geotechnical sections (Knight Piesold, 2010b); and

5) Logs for boreholes drilled along the railway alignment in 2011 (Thurber Engineering Ltd. 2011a).

In addition, a helicopter site inspection of the railway corridor was completed during the period from July 20 to July 23, 2011, by Messrs. Ramli Halim of Hatch Ltd. and Bruce Smith of Thurber Engineering.

It should be noted that at the time the data analyses described this report were undertaken (June, 2011), the information from the boreholes drilled in 2011 was not available and was therefore not included in the analyses. In view of the time constraints for preparation of this report, the data were not re-analysed to include the 2011 borehole information.

The analyses described in this report were undertaken in July to allow alignment design work to proceed on schedule. At that time, very little information from the drilling investigations undertaken during 2011 was available to incorporate into the analysis. Once additional 2011 site investigation information became available, the data was reviewed to confirm that the conditions used in the analysis were still applicable.

It is expected that during the next stage of the design process, the cut and fill configurations along the railway alignment will need to be refined based on site specific conditions at each location. This more detailed review should, of course, include the site specific subsurface information available from all of the previous geotechnical investigations, included those completed in 2011. It can be expected that this review will lead to modest changes in the estimated volumes of earthworks.

### 1.4 Field Investigations

The geotechnical investigations for the Mary River Project commenced in 2006 and continued in 2007 and 2008. A summary of these investigations, including borehole logs, is presented in a report prepared by Knight Piesold (2010b).

During the period from April to August 2011, Thurber Engineering supervised geotechnical investigations for the Mary River Project, including drilling programs at Port Steensby, at the Mary River Mine Site and along the route for the Mary River Railway. A summary of these investigations is presented in a separate report (Thurber Engineering Ltd., 2011a).

All of the boreholes drilled during the period from 2006 to 2008, and most of the boreholes drilled in 2011, were drilled using wet rotary drills (diamond drills) with un-chilled water as the drilling fluid. These drills generally recovered excellent core samples in the bedrock; however core recovery in the overburden soils was typically poor because the drilling fluid often melted the unconsolidated overburden soils and washed much of the ice, fine sands, silts and clays from the samples. Despite these difficulties, the diamond drills were often successful in recovering partial core from thicker ice layers within the overburden.

As a consequence of the drilling procedure, the stratigraphy and properties of the overburden soils is uncertain. Unfortunately, the use of chilled brine, which is required to obtain satisfactory samples of the frozen overburden soils with this type of drill, was not possible under the terms of the drilling permit.

During the field work in 2011, up to 3 sonic drill rigs were used in an attempt to recover representative samples of the overburden soils so that the critical geotechnical properties of the overburden could be determined. The sonic drill advances a core barrel using high frequency percussion and drilling fluid was only required where cobbles, boulders or bedrock were present. This drilling method proved to be fairly successful and frozen cores of the overburden soils were recovered in most drill holes, allowing the geotechnical properties of the overburden materials to be established more reliably.

The success of the sonic drilling program presented an opportunity to compare the overburden stratigraphy found with the sonic drill rig with the stratigraphy found using the diamond drills. In particular, since the diamond drill rigs were only successful in recovering core in the thicker ice layers, it was important to determine if this method of drilling could be expected to consistently identify any thick ice layers within the overburden.

Six pairs of boreholes were drilled on the Mary River Mine Site with one borehole drilled using a sonic drill rig and a second borehole drilled within a few metres of the first, but at a later date, using a diamond drill. The pairs of holes were drilled by different drilling crews so that neither drill crew would be aware of the stratigraphy as encountered in the adjacent borehole.

Thick ice layers were encountered in 3 pairs of holes, and ice cores were recovered by both the sonic and diamond drills. The thickness of the ice in all 3 diamond drill holes was a metre or two less than in the adjacent sonic boreholes. This may be a result of melting of some ice by the diamond drill or it is possible that the ice thickness varied over the short distance between the diamond and sonic drill holes. It is reasonable to conclude, however, that if there are ice layers that are more than about 2 metres thick within the overburden, the diamond drill rig can be expected to reveal the presence of the ice layer but not necessarily its total thickness.

Core recovery with the sonic drill was generally very good. However, it was highly variable with the diamond drill, ranging from 25 to 90 percent, with better recovery at locations where the overburden soils consisted primarily of frozen sands containing no coarse gravel. Core recovery with the diamond drill fell below about 25 percent whenever the overburden contained a significant proportion of gravel or cobbles.

The following preliminary conclusions may be drawn from this limited comparison between boreholes drilled using sonic and diamond drill rigs:

1) If ice layers greater than about 2 metres in thickness are present within the overburden, then it is very likely that the diamond drill will reveal the presence of the ice but not necessarily the total thickness.

2) If core recovery of the overburden soils is very poor with the diamond drill (less than about 25 percent) then it is probably due to the presence of a significant proportion of gravel, cobbles or boulders within the core run.

3) If there is excess ice or ice layers less than about 2 metres thick within gravelly or cobbley sections of the overburden, then they would most likely not be detected by the diamond drill.

4) The samples recovered with the diamond drill are generally unsuitable for determining excess ice contents, water contents, pore fluid salinity and grain size distribution of the overburden soils, information that is critical to the evaluation of the mechanical behaviour of these materials.

THURBER

# 2 SITE DESCRIPTION

## 2.1 General

As indicated in Figure D.1 in Appendix D, the proposed railway runs southeast from the Mary River Mine to about kilometer post 35 (KP 35), and then turns south towards the proposed port facility on Steensby Inlet which is at about KP 145.

The ground surface along the route is dominated by exposed sands and gravels, boulders and frost shattered or intact bedrock.

Vegetation is very sparse and consists primarily of a variety of grasses in some locations where surface moisture is present during the summer months. No shrubs or trees exist in the area. No peat deposits were observed within the study area and organic soils, which are present in only a few isolated locations, are usually less than 100 mm in thickness.

It is recommended that the reader refer to the airphoto terrain maps (Knight Piesold, 2008) while reading the descriptions of the topography and subsurface conditions which follow.

## 2.2 Permafrost

The proposed railway is underlain for virtually its entire length by permafrost, which consists of frozen overburden soils of variable thickness underlain by bedrock.

The maximum annual depth of thaw (the active layer) ranges from about 1 metre in low lying areas where the water depth is near ground surface to 2 or 3 metres in locations where well drained sands and gravels are present.

In most locations the near surface soils along the railway corridor consist of coarse grained sand and gravels which are thaw stable during the summer months and if the near surface cobbles were removed and the surface leveled off, the ground would be trafficable to both wheeled and tracked equipment, except in a few, relatively small isolated locations where the water table is high and fine grained silts are present.

Ground temperatures, which have been measured in several locations along the railway alignment, indicate that the permafrost is relatively cold, ranging between minus 8 to minus 10$^{o}$C at depths below about 8 metres.

A deep thaw bulb is expected to be present below large bodies of water where water depths exceed 3 or 4 metres. As a result, average annual ground temperatures near the shoreline of deep lakes can be expected to be much warmer than average annual ground temperatures at locations some distance from the lakeshore.

The water depths in most of the rivers and streams along the railway alignment are less than about 1 metre in late summer, so that they freeze to the bottom each year. It is therefore expected that permafrost is present below most of the rivers and streams although ground temperatures may be somewhat warmer than in adjacent areas.

### 2.3 Ground Ice

Ground ice ranging up to thicknesses of about 20 metres was encountered within the overburden soils, primarily between the Mary River Mine and KP 86. Ground ice was not encountered in the granite bedrock in any of the holes and it is very unlikely that ground ice will be encountered in sound granite. Relatively thin ice lenses were observed in the sandstone bedrock in some boreholes.

The information from the 292 boreholes drilled prior to 2011 was reviewed to determine the frequency and distribution of thick ice lenses along the railway alignment. Significant thicknesses of either massive ice or ice rich soils were encountered in 54 of these boreholes, which represents an average of about 18 percent of the boreholes. However, the probability of encountering ice or ice rich soils is not the same at all locations along the railway corridor.

In an effort to identify those areas where the probability of encountering ice or ice rich soils is greatest, the railway was arbitrarily divided into 10 km long sections and the frequency of encountering ice or ice rich soil within each section was estimated from the logs for the boreholes drilled prior to 2011. The number of boreholes drilled prior to 2011, within each 10 km section, ranged from a maximum of 34 to a minimum of 17, with an average of about 20 boreholes. The boreholes are therefore fairly uniformly distributed along the corridor and should provide a realistic indication of the probability of encountering ice or ice rich material with each particular section.

The distribution of ground ice along the alignment, as encountered in the boreholes, is presented in Appendix B, Figure B.1. As indicated, the probability of encountering ice or ice rich soil within a particular section of the railway varies from about 18 to 40 percent in the sections of the line between the Mary River Mine and KP 75 and 0 to 10 percent south of KP 75 to Port Steensby.

The dominant terrain type along the railway alignment within each 10 km section was also determined and is also noted on Figure B.1. It appears that ice or ice rich soils are more common in those locations where the overburden thickness is significant (that is in locations where kame terraces, glacial fluvial deposits or a till blanket are present), and much less common in locations where the thickness of the overburden is relatively thin, such as in areas mapped as till veneer.

This finding is entirely consistent with the geological origin of the various deposits encountered along the railway alignment and it is speculated that the massive ice layers in the thicker overburden deposits may be relict ice remaining from the last continental glaciation.

It is recommended that all of the borehole logs from the various geotechnical investigations and all of the geophysical survey tracks, be plotted on the existing terrain maps. Once the geotechnical information has been compiled, it should be carefully reviewed to generate a more accurate subsurface stratigraphy with a particular focus on determining the extent and distribution of ground ice along the railway alignment. If the lateral extent of these deposits can be shown to be fairly limited, then creep settlements of the railway embankments may not be a major maintenance concern.

It is important to note that the foregoing evaluation of the frequency of encountering ice or ice rich soils within the various sections of the railway alignment is based on the assumption that the ice thicknesses, as described on the borehole logs, are representative of actual subsurface conditions. All of the boreholes used in this analysis were drilled with a diamond drill, which as discussed previously may under estimate the thickness of ice and ice rich soils. If this is the case, then the borehole logs may be underestimating the amount of thick ice deposits that are actually present in the overburden soils.

## 2.4 Mary River Mine Site to KP 35

From the Mary River Mine to about KP 35 (near the Ravn River crossing); the railway is located on the northeast side of a broad valley. The valley wall immediately northeast of the alignment consists of granite rock which rises about 100 metres above the valley floor.

Throughout this section, the railway alignment is underlain by overburden soils that consist primarily of sandy gravel of glaciofluvial origin, which ranged from about 5 to over 20 metres in thickness in the boreholes. Cobbles and boulders are present in the area, but they are generally confined to active river and stream channels.

The overburden is underlain by sandstone or by granitic bedrock. It is understood that the area southwest of this section of the railway line is underlain by sandstone bedrock and that the northeast side of the valley follows a discontinuity between the sandstone and the older granite rocks to the northeast.

The glaciofluvial deposits below the railway alignment are dissected by a series of small gullies and valleys, which are often oriented perpendicular to the railway alignment and which were formed by rivers and streams flowing from the uplands northeast of the valley.

Granite bedrock outcrops were observed in many of the river and stream valleys, particularly upstream (northeast) from the railway alignment.

Ground ice, which ranged up to about 20 metres thick, was encountered in many of the boreholes drilled along this section of the railway. The areal extent and origin of these ground ice deposits is not known, although it has been speculated that the deposits are relict ice remaining from glaciers which covered Baffin Island in the past.

## 2.5 KP 35 to KP 86

The railway alignment south of KP 35 to about KP 86 traverses an area which is flat to gently rolling.

The overburden soils along this section of the alignment consist primarily of a sandy gravel till which contains cobbles and boulders and has a variable silt content. Ground ice was encountered within the glacial till in some of the boreholes drilled in this section.

IN this section of the alignment the overburden is underlain by granite bedrock and areas of frost shattered or intact outcrops of the rock are visible at a number of locations, particularly south of KP 40. The depth to bedrock was found to range from surface to depths of over 20 metres in the boreholes drilled in this area.

Thick deposits of ground ice were encountered within the overburden soils in many of the boreholes drilled along this section of the railway, although the frequency and thickness of the ice deposits appears to be somewhat less as compared to the section from the mine site to KP 40.

### 2.6 KP 86 to KP 124

From KP 86 to KP 124, the railway alignment runs along the shores of the Upper and Lower Cockburn Lakes. From KP 86 to KP 95 the railway line runs along the west side of Upper Cockburn Lake and the alignment is restricted by the presence of the lake and the steep valley slopes beside the lake.

At KP 95, the alignment crosses the Cockburn River, which flows between the Upper Cockburn and Lower Cockburn Lakes, so that from KP 95 to KP 124 the railway is located on the east side of Lower Cockburn Lake. The alignment along this section of track is also confined to a narrow strip of flatter ground between the lake and the steep valley slopes.

Sidehill cuts will have to be excavated in overburden soils and granite rock in many locations along this section of the railway line. In addition, two tunnels will be required, one tunnel, about 1 km long, is located between KP 103 and 104, while the second tunnel, about 300 m long, is located near KP 108.

Bedrock is exposed on the valley slopes in many locations and the overburden, which is expected to be relatively thin in most locations, consists of frost shattered granite or bouldery till.

THURBER

In locations closer to the lakeshore, where the valley slopes are flatter, the overburden soils can range up 15 or more metres in thickness and consist of glacial tills and gravelly colluvium which has collected from the slopes above. Delta deposits consisting of sandy gravels with cobbles and boulders have been deposited on the lakeshore by large streams flowing into the lake from the uplands, for example near KP 96, 105, and 124.

Ground ice was encountered very infrequently in the boreholes drilled between KP 86 and KP 124.

## 2.7 KP 124 to Port Steensby

From KP 124 to Port Steensby the topography is irregular, consisting of randomly located bedrock hills and knobs interspersed by lakes and streams.

A significant number of sidehill and through cuts will need to be excavated in the bedrock hills and the blasted rock fill will be placed in topographic low areas to construct the railway grade through this area.

The depth to bedrock in low lying areas is highly variable and can range from a thin veneer over the bedrock to depths of over 20 metres. The overburden soils generally consist of glacial till, glaciofluvial and alluvial deposits, all of which consist primarily of sandy gravels, with some cobbles and boulders.

## 3 EMBANKMENT FILLS

### 3.1 Embankment Sideslopes

Experience on a number of earthworks construction projects, similar to that proposed for the Mary River Railway, has demonstrated that if the embankment fills are constructed of blasted granite rock, sideslopes of 1.5 horizontal to 1 vertical will remain stable, provided the embankment is constructed on a fairly level surface and the underlying soils consist of well drained sands and gravels.

In a few locations, where ice rich silts are present within the upper few meters of the ground surface, some small scale failures may occur near the toe of the embankments, particularly during the first few thaw seasons after construction. It is recommended that all embankment sideslopes be inspected during the first few thaw seasons so that any slope failures can be identified and remedial measures (such as construction of stabilizing berms) can be implemented as necessary.

A preliminary series of conventional stability analyses were undertaken to provide an indication of the range in factors of safety of the embankments against large scale landslides due to high shear stresses in the embankment and foundation soils.

The factor of safety against instability of the embankment fills will vary significantly depending on a number of factors, the most important being:

1) The height of the embankment;
2) The angle of the embankment sideslopes;
3) The strength properties of the embankment fill during the thaw season;
4) The strength of the underlying foundation soils during the thaw season; and
5) The water table below the embankment during the thaw season

The stability analysis indicated that the factor of safety of blasted rock fills with a sideslope of 1.5 horizontal to 1 vertical, placed on relatively level ground and underlain by well drained sands and gravels, such as are present below most sections of the railway, will generally be in the order of 1.5, which is considered adequate.

Under unfavourable conditions, (high fills on poorly drained, sloping ground) the factor of safety will be less than 1.5 and, in locations where the native soils are very weak, the factor of safety will be less than 1.0, indicating the embankment would fail.

Landslide failures of the embankments are most likely to occur under the following conditions:

1) Embankments constructed on relatively flat ground but where the foundation soils contain more than about 30 percent silt and clay size material and the near surface soils contain excess ice. These soil conditions are expected to be present at only a few locations along the proposed railway corridor.

2) Embankments constructed on steeply sloping terrain, such as occur along the Cockburn Lakes;

3) Early in the thaw season when the thaw front is advancing and the ground water table is high;

4) As a result of triggering events, such as stream erosion near the toe of an embankment or ground accelerations due to a seismic event.

The shear strength of frozen soils is relatively high and therefore large scale landslides or other mass movements are unlikely to occur during the winter months when the embankments and foundation soils are completely frozen.

## 3.2 Seismic Loading

The stability analysis was extended to provide an indication of the effect which seismic events could have on the stability of the rock fill embankments.

A seismic risk assessment was undertaken previously (Knight Piesold, 2007), which found that for a return period of 2500 years, the peak ground acceleration was 0.25 g (gravity) at the Mary River Mine Site and 0.12 g at Port Steensby. The calculated factors of safety for a blasted rock fill embankment constructed on level ground underlain by well drain sands and gravels were as follows:

| Location | Peak Ground Acceleration (g) | Factor of Safety of Embankment |
|---|---|---|
| All Locations | 0 | 1.5 |
| Port Steensby | 0.12 | 1.2 |
| Mary River Mine | 0.25 | 1.0 |

Under the American Railway and Maintenance of Way Association (AREMA) guidelines for the design of cuts and fills, an acceptable factor of safety against instability may be close to unity under low probability events. On this basis, the foregoing factors of safety can be considered acceptable.

However, in this particular case, the peak ground acceleration would have to occur sometime during the 3 months of the thaw season when the near surface soils below the embankment are thawed, a situation which, in effect, increases the return period of 2500 years to about 10,000 years. The peak ground acceleration for a return period of 625 years, which is equivalent to a return period in a non-permafrost region of 2500 years, would be about 0.15 g at the Mary River Mine Site and 0.08 g at Port Steensby.

In view of these considerations, the factor of safety of the railway embankments appears to exceed AREMA guidelines, even under seismic events which have a low probability of occurring during the design life of the railway.

Nevertheless, during detailed design, the stability of the railway embankments should be reviewed at each location, based on site specific conditions, so that critical cross-sections can be identified and the design sideslopes can be adjusted as appropriate.

THURBER

# 4 CUTS ON OVERBURDEN

## 4.1 General

The stability of cut slopes in permafrost depends on the proportion of fines (silts and clays) present in the overburden soils. In general, if the fines content is less than about 20 percent, then the overburden soil will drain faster than it thaws, such that while surface settlements will occur, mass movement and sloughing of the cut slope will be minimal and the slope should become relatively stable after 1 to 3 thaw seasons, at a slope of 2 horizontal to 1 vertical. The magnitude of surface settlements within the cut will depend on the total amount of excess ice in that portion of the overburden which thaws.

In locations where the fines content exceeds about 20 percent, then the slope will drain slower than it thaws and some sloughing and mass movements may occur in addition to surface settlements (Smith et al, 1989). In locations where the fines content exceeds 20 percent, it may be necessary to flatten the slope and take additional remedial measures to control slope instability, depending on the specific site conditions. For initial estimating purposes, it is expected that in most locations where the fines content exceeds about 20 percent, it will be possible to stabilize the slopes at an angle of 3 horizontal to 1 vertical. In locations where the fines content exceeds about 20 percent, and other site conditions are unfavourable, it may be necessary to construct the cut slopes flatter than 3 horizontal to 1 vertical, and take additional measures to stabilize them.

It can be expected that it will be necessary to blast the frozen overburden soil to facilitate excavation. A conceptual sketch of an ice rich slope excavated in sand and gravel, having a fines content less than 20 percent is presented in Appendix B, Figure B.12. It is expected that cuts will be blasted and excavated in benches during the winter months, such that at the end of excavation, the slope will have the configuration illustrated in Figure B.12a.

The benches will thaw, drain and stabilize during the first few thaw seasons after construction. During this period it will be necessary to re-grade the slopes, clean and grade the working area at the base of the cut and carry out other maintenance until a

THURBER

new thermal regime becomes established and the excavation stabilizes and the cross-section will have the configuration shown in Figure B.12b.

For initial volume estimates, the bench heights and widths presented in the following table are suggested. The actual bench heights and widths will depend on the subsurface conditions within each cut as well as contractor preferences.

| Final Design Slope | Bench Height | Bench Width |
|---|---|---|
| 2H to 1V | 3 to 4 metres | 2 times bench height |
| 3H to 1V | 3 to 4 metres | 3 times bench height |

## 4.2 Fines Contents

The fines content of the majority of overburden materials could not be reliably determined from grain size distribution tests carried out on core samples of the frozen overburden soils recovered from the boreholes because the drilling fluid thawed and washed the fine material from most of the core samples. However, test pits were excavated adjacent to many of the boreholes and grain size distribution tests were carried out on samples recovered from some of the test pits (Knight Piesold, 2010b).

A statistical analysis was undertaken on the results of grain size tests on 164 samples recovered from test pits excavated in various terrain types along the railway alignment. The results of the statistical analysis are presented in Table A.1 in Appendix A.

As indicated on Table A.1, the average fines contents in all of the major terrain units that exist along the alignment were found to be less than about 15 or 16 percent, except for samples of hummocky till, where the average fines content was found to be 26 percent. These findings are generally consistent with the geological origins of the various overburden materials.

The foregoing assessment indicates that in most of the terrain types along the railway alignment, it can be expected that the material will drain faster than it thaws and therefore while the cut may settle several metres or more, sloughing and mass movements should be minimal, except in some localized areas.

However, the fines contents of the samples were also found to be highly variable, as indicated by the results presented on Table A.1. For example, in the glaciofluvial deposits, the average fines content was found to be 8; however 67 percent of the 41 samples tested had fines contents between 0 and 21 percent while 95 percent of the samples had fines contents that ranged from 0 to 34 percent. These results indicate that while cuts in glaciofluvial deposits can generally be expected to drain faster than they thaw, some portions of the cuts will drain slower than they thaw, in which case sloughing and mass movements can be expected at some locations within this terrain unit.

### 4.3 Cut Slope Angles

Despite the variations in the fines contents of the materials along the railway alignment, an attempt was made to estimate the probable cut slope angles that can be expected to remain relatively stable. The results of this evaluation are presented in Table A.2, in Appendix A.

For this assessment, the railway line was divided in 5 km sections as shown on the table. The maximum depth of cut within each section was determined from the alignment and profile drawings (Canarail, 2010). The most extensive terrain unit within each 5 km long section of track was determined from an examination of the air photo interpretation maps prepared for the railway corridor (Knight Piesold, 2008). In most cases several different terrain types are present within each track section; however for this initial analysis the dominant terrain type was assumed to exist over each 5 km track section.

The average fines content and the range in fines contents of the most extensive terrain unit within each 5 km section of track could then be determined from the results presented on Table A.1. It was assumed that if the average fines content within each 5 km section of track is less than 20 percent, then the cut slopes within that entire section will drain faster than they thaw and therefore if cut slopes are excavated at 2 horizontal to 1 vertical, then the slope will become relatively stable within 1 to 3 thaw seasons after excavation. It is expected that within each 5 km section there will be some sections of some cuts for which the overburden soils will drain slower than they thaw and it may be necessary to flatten the cut slope and take other measures to stabilize these areas.

In locations where the average fines contents of the overburden exceeds 20 percent, it has been assumed that cut slopes will or can be made stable after 1 to 3 thaw seasons, at slope angles of 3 horizontal to 1 vertical. The sections of track where slope angles should be assumed to be 3 horizontal to 1 vertical for the volume estimate are shown on Table A.2.

The last column on Table A.2 shows the maximum thickness of ice or ice rich soil that was encountered in the boreholes drilled within the particular section of track. This value provides a very approximate indication of the probable maximum surface settlement that could occur within that section of track. It should be noted however, that the maximum amount of settlement would only occur if the entire ice rich deposit were exposed within the depth of cut. This will not be the case in those situations where the ice rich materials are present a significant distance below the base of the cut.

In some locations, the proposed cuts will expose significant thicknesses of ice or ice rich soil. Progressive long term thaw settlements in these areas will occur and it will likely be necessary to install rigid panels of extruded polystyrene insulation (such as Styrofoam panels) and place a gravel cover over such areas to control progressive thaw settlements. The thaw settlement that occurs each thaw season will depend on a number of factors, but will generally not exceed 1 or 2 metres of thawing and surface settlement during a single thaw season. It should be possible to identify most of the affected sites during the first thaw season after cut excavation and implement remedial measures to minimize further thaw settlements.

THURBER

# 5 COCKBURN LAKES SECTION

## 5.1 General

As mentioned earlier the railway alignment between KP 86 and KP 124 is located along the shoreline of the Upper and Lower Cockburn Lakes and will be constructed as a sidehill cut and fill in most locations. In locations where the railway line crosses existing gullies and streams, fills and culverts or bridges will be used to achieve the design grade.

## 5.2 Site Conditions

A series of topographic maps which show the railway alignment between KP 86 and KP 124 are presented in Appendix D, Figures D.2 to D.5.

Representative photos (courtesy of EBA Engineering Consultants Ltd.) at various locations along this section of the railway line are presented in Appendix C, Figures C.1 to C.8. As shown, boulder fields, which are believed to be 1 to 2 metres in thickness, are widespread on the ground surface on many of the slopes. The geological origin of these boulder fields is uncertain.

A preliminary estimate of the lengths of cut and fill required along the section of track between KP 86 to KP 124 found the following:

| Description | Approximate Length (km) | Approximate Proportion of Total Length (%) |
|---|---|---|
| Rock Tunnels | 2.5 | 6 |
| Cuts in Rock | 7.0 | 19 |
| Cuts in Overburden | 24.0 | 65 |
| Fills | 4.0 | 10 |
| **Total:** | **37.5** | **100** |

As indicated above, about 24 km of the railway embankment along this section of the track will involve cutting into the overburden soils, which consist primarily of gravel till and which contains some boulders in a matrix of gravel, sand and silt. It is believed that these materials originated as a lateral moraine deposited by the glacier that filled the Cockburn Valley.

Continuous permafrost is present below the entire area except below deep bodies of water, including the Upper and Lower Cockburn Lakes. The maximum depth of annual thaw ranges from 1 to 3 metres and the mean annual ground temperature ranges from minus 5 to minus 10$^{o}$C, depending on the proximity to the Cockburn Lakes.

Unfortunately, all of the boreholes that were drilled along this section of track were drilled with a rotary diamond drill with water circulation and as a result, the overburden soil melted and the fines were washed from most samples. Therefore, the fines content of the overburden soils is uncertain. However, based on data from other locations, it is expected that the overburden will consist of a mixture of sand, gravel and boulders with a silt content ranging from 10 to 40 percent and an average silt content of about 25 percent. The clay content is most likely less than 5 percent.

No thick ice layers were encountered in any of the boreholes drilled in this section of the railway alignment. Thin ice layers may be present within the overburden soils, but the excess ice content (and hence the in situ density) of the overburden is not known. It is very unlikely that ice is present within the unweathered granite bedrock that underlies the overburden in the valley.

### 5.3 Possible Cross-Sections

Sketches which illustrate three possible cross-sections for the railway embankment along this section of track are presented in Appendix D, Figure D.6. The 3 options shown on the figure have been drawn to the same vertical and horizontal scale and, for simplicity; the sketches assume that the valley slopes are 2 horizontal to 1 vertical (26.5 degrees to the horizontal). In most areas, the average valley slope above track level is about 2.5 to 1 (22 degrees) however there are short lengths of the alignment were the valley slope above track level ranges up to 1.7 to 1 (30 degrees). Slope angles at lower elevations are generally flatter than slope angles at higher elevations on the valley walls.

Option 1 on Figure D.6 illustrates a configuration in which the track structure is placed entirely within the natural ground by excavating into the slope. Since embankment construction will take place primarily during the winter months, it will be necessary to blast the frozen overburden in benches before it can be excavated. It is therefore assumed that the excavated material will not be used as fill but will be wasted at suitable locations. For this option to be viable, it is necessary that the cut slope remain relatively stable during the thaw season at a maximum slope as steep as 1.5 horizontal to 1 vertical.

Option 2 shows an embankment configuration in which the track structure is placed partly on the native soil and partly on fill. If construction were carried out during the thaw season and the excavated material were suitable, consideration could be given to using the excavated material to construct the fill. However, at this early stage of design, it should be assumed that all of the excavated material will be wasted and imported blasted granite rock will be used to construct the fill. It is assumed that both the cut and fill slopes will remain stable at a maximum slopes as steep as 1.5 horizontal to 1 vertical, as shown

Option 3 illustrates an embankment constructed of blasted granite rock placed over the existing ground surface, without cutting below the existing slope. The blasted rock fill would not be keyed into the natural ground and it has been assumed that the fill slope will remain stable at a maximum slope of 1.5 horizontal to 1 vertical.

### 5.4 Recommended Cross-Section

It is expected that all three of the cross-sections shown on Figure D.6 will be required to construct the grade within this section of the railway line, depending on the horizontal alignment and vertical profile of the railway and the existing topography at each location.

However, since most of the railway embankments will be constructed during the winter months, and considering the uncertainties associated with the thawed properties of the subsurface soils along this section of the railway, it is recommended that the cross-section shown as Option 3 be used wherever possible for initial design, in preference to the other options, since it offers the following advantages:

1) It will minimize the number and length of the cuts along the alignment and reduce the risk of instability and maintenance of the cut slopes;

2) It will minimize the volume of excavated overburden material that needs to be wasted;

3) It will maximize the use of blasted rock fill, for which the placement and properties can be controlled during both winter and summer construction; and

4) It leaves the existing surface layer of large boulders intact, which will minimize surface erosion during the thaw season.

The length of track along this section of the alignment that conforms to the Option 3 cross-section can be maximized by shifting the track centreline horizontally towards the Cockburn Lakes, thereby reducing cut volumes and increasing fill volumes.

The major disadvantage of using the cross-section illustrated by Option 3 is that it does not balance the excavation and fill volumes within each section of the railway, a technique that is commonly used in non-permafrost regions to minimize construction costs.

A second disadvantage is that it is more difficult to repair slope movements which occur in the fill downslope from track level, as compared to maintaining cut slopes above track level. It is therefore recommended that wherever the Option 3 cross-section is used, an access road be incorporated into the design cross-section near the toe of the fill, to provide access for large maintenance equipment.

### 5.5 Stability of Cut and Fill Slopes

As mentioned earlier, cut slope angles of either 2 horizontal to 1 vertical or 3 horizontal to 1 vertical are recommended, based on the fines content of the overburden, as listed in Table A.2 in Appendix A.

Therefore, cut slopes that are inclined at 1.5 horizontal to 1 vertical may need to be flattened if subsurface conditions are unfavourable, for example if thick sand or silt layers are present within the cut. If the topography is such that a flattened cut would daylight an unreasonable distance above track level, then consideration would have to be given to installing a mechanically stabilized earth (MSE) retaining structure above track level.

As recommended earlier, fill sideslopes of 1.5 horizontal to 1 vertical are expected to be stable, provided the blasted rock fill is placed on a competent, reasonably level surface. The stability of blasted rock fills on steep slopes will be governed by the strength of the native soils below track level and, if subsurface conditions are unfavourable, it will be necessary to flatten the fill slope accordingly. If this is not possible, (for example if the fill would extend an unreasonable distance downslope) then consideration would have to be given to constructing an MSE embankment, which would be keyed into competent material below the fill.

In general, the use of MSE retaining walls or MSE embankment fills should be minimized; since they must be constructed during the thaw season and they will be expensive and time consuming to build.

### 5.6 Estimating Earthworks Volumes (KP 86 to KP 124)

The probable maximum volume of cut excavation that may be required along this section of the alignment could be estimated by assuming that all of the cut slopes are trimmed to 2 horizontal to 1 vertical, irrespective of the vertical height of the cut. During detailed design, the cost of flattening some of the slopes to 2 horizontal to 1 vertical (or where necessary 3 horizontal to 1 vertical) can be compared to the cost of alternate stabilization measures such as the construction of MSE retaining structures.

The probable minimum volume of cut excavation that will be required can be estimated by assuming that in locations where the vertical height of the cut exceeds 11 metres (about 3 cut benches), that a slope excavated at 1.5 horizontal to 1 vertical will remain stable with occasional maintenance. Further geotechnical investigations will be required at each of these locations during detailed design to determine if these steep cut slopes can be justified.

The probable maximum volume of rock fill along this section of track can be estimated by assuming that, in those locations where the existing cross-slope is steeper than 6 horizontal to 1 vertical (about 10 degrees), that the fill sideslopes must be 1.75 horizontal to 1 vertical, while in locations where the existing cross-slope is flatter than 6 horizontal to 1 vertical, fill sideslopes of 1.5 horizontal to 1 vertical will be stable.

The probable minimum volume of fill that may be required can be estimated by assuming that all fill slopes will remain stable at an angle of 1.5 horizontal to 1 vertical, irrespective of the existing topography. Additional investigations will be required during detailed design at each location where the existing slope is steeper than 6 horizontal to 1 vertical, to determine if this steep fill slope will be suitable.

The estimated cut and fill volumes will need to be refined during detailed design, once the horizontal and vertical alignment for the railway grade has been finalized and the most suitable cross-section at each location has been selected based on the local topography and subsurface conditions.

It should be expected that additional modifications to the cut and fill sideslopes will have to be made during construction, based on field observations, the properties of the blasted rock fill, surface drainage and other factors. It should also be understood that maintenance of both cut and fill slopes will be required, particularly during the first 2 to 3 thaw seasons after construction, as the new thaw regime develops below the cuts and fills.

### 5.7 Recommendations

A number of methods can be used to improve the reliability of the geotechnical evaluation of this section of the railway line. Firstly, additional boreholes can be drilled in selected locations, using a sonic drill or another drilling method that will provide more representative samples of the overburden soils below the railway alignment.

Consideration should be given to undertaking seismic refraction surveys at selected locations down the valley slopes in order to determine the depth to bedrock below the valley walls. It is recommended that an experienced geophysicist be asked to review all of the existing information along this section of the track, recommend the most suitable geophysical technique(s) and provide an opinion with respect to the probability that meaningful data can be acquired.

Finally, the most effective method of assessing the properties and behaviour of the overburden materials will be to excavate a pioneer road and large test pits at suitable locations along the alignment. Large test pits are recommended in locations where major cuts are expected to be excavated and at each location where higher fills will be constructed. The test pits should be left open during the following thaw seasons so that the behaviour and properties of the overburden soils can be assessed. These observations will provide a much more reliable indication of the probable behaviour of the native soils in this area than can be established by drilling boreholes.

# 6 CREEP OF ICE RICH SOIL

## 6.1 General

In some locations along the railway alignment the embankments will be constructed on frozen overburden soils which contain layers of ice or ice rich soils, which can range up to 20 metres or more in thickness.

Research has found that ice and ice rich soils are viscous materials that deform with time under an applied deviator stress[1] and the rate of deformation depends primarily on the soil type, ice content, ground temperature and pore water salinity. In geotechnical practice, these time dependent deformations under constant applied deviator stresses are referred to as creep.

A limited number of creep analyses were undertaken during an earlier stage of design (EBA Engineering Consultants Ltd, 2008a), which found that once embankment heights exceed about 6 metres and sideslope angles become steeper than about 2 horizontal to 1 vertical, settlement at track level due to creep of the ice rich foundation soils could be significant and would affect railway operations.

Thurber Engineering completed a more extensive series of creep analyses, as described in the following sections, to confirm the results of the initial analyses undertaken by EBA Engineering, to assess possible methods of reducing creep deformations and to optimize the design of the embankment configurations.

## 6.2 Method of Analysis

The time dependent deformations of the railway embankment foundations were calculated using the computer program FLAC (Fast-Lagrangian Analysis of Continua), which is a two-dimensional, finite-difference formulation of the constitutive equations that govern the stress-strain behaviour of materials such as ice, soil or rock. The

[1] The deviator stress is the difference between the average confining stress on an element of a solid material and the maximum stress applied on one surface of the element. For example, in a standard unconfined compression test on a concrete cylinder, the average confining stress is zero and the maximum stress is the stress applied to each end of the cylinder. Under this loading condition, the deviator stress is equal to the stress applied to the ends of the test cylinder. The only stress that will cause a cylinder of ice to creep is the deviator stress, not the confining stress.

program can simulate the creep behaviour of these materials. FLAC provides a large strain formulation option that can capture geometric non-linearity in cases where significant deformations are expected, such as in ice or ice-rich soils.

The time dependent deformation behaviour of ice and ice rich soils may be estimated through the use of the following equation (Morgenstern et al, 1980):

$$\dot{\varepsilon} = B\sigma^3 \qquad (1)$$

Where:

$\dot{\varepsilon}$ denotes the strain rate (year$^{-1}$);

B denotes an empirical parameter which is primarily a function of the temperature of the ice rich soil and the salinity of the pore water (kPa$^{-3}$ x year$^{-1}$); and

$\sigma$ denotes the deviator stress (kPa).

Values for the empirical parameter B have been established in laboratory tests on ice rich soils as well as from a small number of field observations. Based on a review of the available data, Morgenstern et al (1980) have suggested the following values for B:

| **Soil Temperature ($^0$C)** | **B (kPa$^{-3}$ x year$^{-1}$)** |
|---|---|
| -1 | 4.5 x10$^{-8}$ |
| -2 | 2.0 x10$^{-8}$ |
| -5 | 1.0 x10$^{-8}$ |
| -10 | 0.6 x10$^{-8}$ |

It should be noted that in the temperature range between 0 and minus 1$^0$C, the value of B cannot be accurately determined, because soils which contain fine grained material freeze over this temperature range and contain a mixture of frozen and unfrozen pore water. Therefore it can be very difficult to differentiate between creep of that portion of the test sample that is frozen and consolidation (due to the dissipation of pore water pressure) of the unfrozen portions of the sample, because both processes are time dependent.

The thermistor data from locations along the proposed railway alignment were reviewed and supplemented by a thermal analysis to determine the probable distribution of ground temperatures below the embankments as a function of the time of year and the depth below ground surface. Based on this information, the following values for the parameter B were used in the creep analyses for both ice and ice rich soils:

| **Depth Below Original Ground Surface (m)** | **B in Summer ($kPa^{-3}$ x $year^{-1}$)** | **B in Winter ($kPa^{-3}$ x $year^{-1}$)** |
|---|---|---|
| 0 to 5 | 5.0 $\times 10^{-8}$ | 0.5 $\times 10^{-8}$ |
| 5 to 10 | 1.0 $\times 10^{-8}$ | 0.5 $\times 10^{-8}$ |
| Below 10 | 0.5 $\times 10^{-8}$ | 0.5 $\times 10^{-8}$ |

The B values given above for the summer months (when creep rates will be faster due to warmer temperatures in the upper 10 metres) were assumed to be in effect for 6 months, while the B values for winter (when creep rates are slower because ground temperatures in the upper 10 metres are colder) were assumed to be in effect for the remaining 6 months or the year.

It is expected that in locations where surface water does not flow through the base of the embankment during the thaw season, permafrost will aggrade upwards into the railway embankments over a period of several years and therefore ground temperatures in the foundation soils below the embankments will be colder than below the adjacent ground. However, since most of the deeper embankments will have culverts installed near the base of the embankment, ground temperatures in the vicinity of the culvert will be approximately the same those below the adjacent ground surface throughout the year. The creep analysis was therefore simplified by assuming that that the temperature distribution with depth in the foundation soils below the embankment was the same as the temperature distribution below the adjacent ground. As a consequence, creep settlements of embankments that are located some distance from culverts will be somewhat less than those predicted by the analyses.

As mentioned earlier, ground temperatures near large, deep lakes (which do not freeze during the winter) can be significantly warmer that in those areas further from the shoreline. If an embankment were constructed on ice rich soils near a deep lake, then the parameter B will be higher than given in the foregoing table and creep settlements will be greater than predicted.

## 6.3 Configurations Analysed

Total creep settlements that would occur at track level after a period of 25 years, were calculated for a variety of embankment and subsurface configurations.

The rate of creep will increase as the height of the embankment increases. Total embankment settlements at track level after 25 years were calculated for embankment heights of 5, 10, 15, 20 and 25 metres in height, which covers the range of embankment heights that will be required along the railway alignment.

The rate of creep will also increase as the embankment sideslopes become steeper. The total creep after 25 years at track level was calculated for embankment sideslope angles of 6 horizontal to 1 vertical, 3 horizontal to 1 vertical, and 1.5 horizontal to 1 vertical.

Since the magnitude of creep will also increase as the thickness of the ice rich soil below the embankment increases, total creep at track level was calculated for ice rich soil thicknesses of 5, 10 and 20 metres.

Based on a review of the borehole logs, it appears that in most locations, a cover of sand and gravel, which is not ice rich and will therefore not creep, is present over the ice rich soils. The rate of creep of an embankment will be reduced as the thickness of this soil cover is increased and therefore the total creep at track level after 25 years was calculated for soil cover thicknesses of 0, 2 and 5 metres.

Finally, creep rates of an embankment can be reduced by placing stabilizing berms on each side of the embankment. This approach effectively flattens the slope while reducing the volume of fill that would be required to flatten the slope. The total creep at track level after 25 years was calculated for no berm and berm heights equal to 1/6 and 1/3 of the embankment height. For example, if the embankment height is 20 metres, the analyses assumed that the height of the stabilization berms would be 0, 3.3 and 6.7 metres.

Clearly, there are a large number of variables that can affect track settlements and over 200 combinations of these variables were analysed. For convenience, only those results that are considered particularly relevant to the design of the railway embankments are presented and discussed in the following sections.

### 6.4 Calibration of Creep Parameters

The value assigned to the parameter B has a very significant effect on calculated embankment deformations. Unfortunately, there is considerable uncertainty in the values that should be used for this parameter since it has been derived primarily from small scale laboratory tests.

An effort was made to calibrate the parameter B against the observed behaviour of an embankment constructed for an aircraft taxiway and parking apron in Inuvik, NT.

The embankment was constructed in 1988 of blasted and crushed limestone that was placed on a gentle slope of about 4 percent, adjacent to an existing runway. The embankment required a maximum fill thickness of about 6 metres along the south side to create a level surface for a paved aircraft parking apron. The slope from the crest of the embankment down to the original ground surface was 3 horizontal to 1 vertical.

The native soil below the embankment consists of silty clay till which is ice rich to a depth of about 4 metres. The average annual ground temperature at this location is about minus $4^0$C, and the maximum depth of annual thaw below the base of the embankment fill is about 0.5 metres.

The parking apron was paved with asphalt about one year after the crushed rock fill had been placed, so that if significant ground movements have occurred in the 20 years since construction, some visible evidence of surface deformations should be evident in the pavement surface. Inquiries were made to the engineers who are familiar with the maintenance history of the facility and they reported that there has been no evidence of ground movements on the surface of the embankment or on the slope of the embankment. It appears that if creep movements of the embankment have occurred, they have been very small.

A creep analysis of the embankment was carried out, using the known geometry and subsurface conditions. The creep parameter B was assigned values with depth based on the estimated ground temperature distribution that would have existed in the subsurface soils below the embankment since construction. The analysis calculated a maximum total creep settlement of about 100 mm after 25 years where the depth of fill was greatest. Calculated differential settlements over the surface of the fill were less than 75 mm. These calculated values are considered to be in reasonable agreement with the observed behaviour of the embankment, particularly in view of the uncertainties associated with many of the variables that were assumed for this analysis.

While the foregoing results are encouraging, it would be worthwhile to calibrate the assumed creep parameters against field observations from other sites, particularly for embankments ranging up to 25 metres in height that have been constructed on significant thicknesses of ice or ice rich soils.

### 6.5 Creep Deformation Patterns

Figure B.2 in Appendix B presents the finite difference mesh used in the FLAC analyses and illustrates the pattern of deformations that might occur below an embankment after 25 years due to creep. In this example, it was assumed that a soil blanket (which is not ice rich) 2 metres thick overlies a layer of ice and ice rich soil that had a total thickness of 20 metres.

The analysis assumed that the original ground surface was a horizontal plane and the compacted rock fill was 25 metres high. At the start of the analysis, the finite difference mesh consisted of a series of equal size squares, the slope of the embankment was 1.5 horizontal to 1 vertical and the interfaces between the various colors shown on the figure were horizontal.

The figure illustrates the general pattern of deformations that can be expected to occur in the embankment and foundation after 25 years due to creep of the ice rich material in the embankment foundation.

In this example, the total vertical settlement of the embankment after 25 years was calculated to be about 7.6 metres. However, the total vertical settlement at the base of the embankment below track centreline was calculated to be about 3 metres, so that rock fill embankment itself has settled about 4.6 metres. This result may seem counterintuitive, since the rock fill does not exhibit creep behaviour, however an examination of Figure B.2 provides an explanation.

As indicated by the deformations in the mesh on Figure B.2, while the ice rich soil creeps vertically downward, it also creeps a significant distance in a horizontal direction. These horizontal deformations in the subgrade soils cause the slopes of the embankment to flatten with time, which causes the rock fill embankment to settle at track level. In this example the embankment slopes, which were originally at an angle of 1.5 horizontal to 1 vertical, have flattened to a slope of over 2 horizontal to 1 vertical after 25 years, due to horizontal creep movements in the foundation.

Figure B.3 illustrates the pattern of ground deformations that will occur after 25 years under the same subsurface conditions, if stabilization berms were constructed on each side of the embankment. Figure B.3a assumed stabilization berms 4.2 metres high, while Figure B.3b assumed berms 8.3 metres high.

Figures B.2 and B.3 also illustrate the mechanism by which the presence of a surface cover of ice poor soil and the construction of stabilization berms will reduce track settlements. The weight of the berms and the ice poor soil cover counteract the weight of the embankment and thereby reduce horizontal creep rates in the foundation below the embankment. That is, it is primarily the weight of the berms and soil cover that reduce total creep at track level, while the insulating value of the berms and soil cover is of secondary importance.

## 6.6 Strength of Embankment Fill

The results of the analyses indicate that the strength properties of the compacted embankment fill (which control deformations within the embankment) may have a significant effect on total track settlements. The friction angle (41 degrees) assumed for the embankment fill in all of these analyses was selected as being a conservatively low value for blasted rock fill derived from granite.

It is possible that if the use of a higher friction angle can be justified, it could result in a significant reduction in predicted track settlements. Consideration should be given to undertaking further creep analyses to determine the sensitivity of track settlements to the assumed strength parameters of the embankment fill.

Consideration could also be given to the use of soil reinforcement geogrid (such as Tensar) in the lower portions of the embankment to reduce deformations within the embankment fill. The use of soil reinforcement for this application is unconventional and would require full scale field verification test sections prior to widespread use. In the meantime, consideration can be given to undertaking additional creep analyses to assess the potential cost benefit of using soil reinforcement.

### 6.7 Effect of Embankment Height

Figure B.4a in Appendix B presents the calculated settlement after 25 years at track level, as a function of the embankment height. In all cases, it was assumed that the embankment was constructed with sideslopes of 1.5 horizontal to 1 vertical and that the ice rich soils were covered by a 2 metre thick blanket of ice poor sand and gravel that does not exhibit creep behaviour.

As noted on the figure, the relationship between embankment height and settlement at track level was calculated for 5, 10 and 20 metre thicknesses of ice rich soil in the embankment foundations. It is clear from these results that the total amount of creep at track level will vary significantly depending on the both the embankment height and the thickness of the underlying ice rich soil below the embankment.

### 6.8 Effect of Stabilization Berms

A series of creep calculations was made to determine the effectiveness of providing stabilization berms at the toe of the embankment. For this series of analyses, it was assumed that the embankments were constructed with sideslopes of 1.5 horizontal to 1 vertical, on a foundation soil that consisted of a 2 metre thick cover of ice poor sands and gravels (which will not exhibit creep behaviour) and which is, in turn underlain by 20 metres of ice rich soil.

Vertical settlements at track level were calculated for 3 stabilizing berm heights:

- no berm,
- a berm height equal to 1/6 of the embankment height, and
- a berm height equal to 1/3 of the embankment height.

In the latter two cases, the width of the stabilizing berm was determined by drawing a line, having a slope of 6 horizontal to 1 vertical, from the upper crest of the embankment down to the outer crest of the stabilizing berm. The results of this series of analyses are presented in Appendix B, Figure B.4b.

As indicated, the total settlement at track level can be significantly reduced by providing stabilization berms. For example, a berm 1/6 of the embankment height (4.2m) reduced the settlement of a 25 metre high berm from 7.6 metres to about 3 metres. Increasing the berm height to 1/3 of the embankment height (8.3m) reduces the total settlements at track level after 25 years to about 2 metres.

The configuration of the stabilization berms has not been optimized. For instance, reducing the width of the 4.2 metre high berm and placing most of the material closer to the toe of the embankment may reduce the total settlement after 25 years to less than 3 metres, without increasing the volume of fill required for the stabilization berm. Consideration can be given to undertaking additional calculations during later phases of design to determine total settlements at track level as a function of various configurations of the stabilization berms.

### 6.9 Rate of Creep Settlement

The total settlements presented on Figure B.4 are the settlements expected 25 years after construction. For example, if a 25 metre high embankment were constructed with sideslopes of 1.5 horizontal to 1 vertical on a foundation that contained 20 metres of ice rich material, then the total settlement after 25 years would be 7.6 metres, or an average of about 300 mm per year.

However, the rate of annual settlement for the 25 metre high embankment was found to decrease with time, as illustrated on Figure B.5. The annual rates of settlement for this case are summarized in the following table.

THURBER

| Time Since Construction | Settlement Rate (mm/year) |
|---|---|
| First 2 years | 900 |
| 2 to 5 years | 600 |
| 5 to 10 years | 400 |
| 10 to 25 years | 200 |

In contrast, as shown on Figure B.5, the rates of settlement for a 10 metre high embankment are seen to be more or less constant over the 25 year period after construction.

In view of these findings, there may be an advantage to constructing the higher embankments as early has possible during construction. This approach would allow the creep parameter B to be calibrated for higher fills, which would enable more accurate predictions of creep deformations, and in addition, would reduce the rate of creep settlement of the test embankment after the track has been laid.

Conceptually it would be possible to surface the track periodically throughout the year and maintain rail traffic; however, the practicality of surfacing would depend on the number of locations where significant track settlements occur. In addition, track settlements will be highly non-uniform along the length of the line, since they will vary significantly depending on the embankment height and thickness of ice rich material in the foundation soils, both of which will be highly non-uniform.

# 7 TRACK SETTLEMENTS

## 7.1 Interpolation of Calculated Track Settlements

Since the creep analysis considered only a limited number of embankment heights, and thicknesses of ice rich soils it is necessary to interpolate between the curves presented on Figure B.4.

In order to facilitate extrapolation of the FLAC calculations, the points shown on Figure 4.a were re-plotted as Figure B.6 in Appendix B, which shows track settlements as a function of embankment height for various thicknesses of ice rich soil in the foundation. The FLAC calculations are the points on Figure B.6, which have been connected with solid lines. The dashed lines on Figure B.6 have been interpolated from the FLAC data points. The interpolated curves presented on Figure B.6 allow track settlements to be estimated to an accuracy that is believed to be adequate for this initial evaluation.

The possible track settlements that can be expected to occur after 25 years, due to creep of the ice rich material in the embankment foundations, can be estimated from Figure B.6, for any embankment fill height and thickness of ice rich material expected to be present in the underlying foundations, if it is assumed that all embankment sideslopes are constructed at 1.5 horizontal to 1 vertical, without stabilizing berms.

For design purposes, the maximum total settlement of the track after 25 years has been set at about 1 metre. Therefore, any calculated values of settlement greater than 1.5 metres are not shown on Figure B.6, although calculated values greater than 1.5 metres were used to establish the shape of the curves on Figure B.6.

A set of curves, similar to Figure B.6 have been developed from the results of the FLAC analyses in which stabilizing berms were constructed on each side of the railway embankments and are presented in Appendix B, Figures B.7 and B.8 for stabilizing berms that are 1/6 and 1/3 of the embankment heights, respectively.

## 7.2 Estimated Settlements

For these estimates, track sections 5 km in length were considered. On average, this gave a total of about 10 boreholes (from the investigations prior to 2011) within each 5 km section of track, which was a sufficiently large number of boreholes to provide a good indication of the probable maximum thickness of ice or ice rich soil that might be encountered at any location within that section of track.

The maximum fill height within each 5 km section of track was determined from the profile and alignment drawings (Canarail, 2010). The maximum thickness of ice and ice rich soil was determined from an examination of all the boreholes within that particular 5 km section of track, as noted on the borehole summaries presented on the geotechnical section drawings (Knight Piesold, 2010a).

If it is assumed that the maximum thickness of ice and ice rich soil encountered in the boreholes drilled within any 5 km long section and the maximum fill height within the same 5 km section occur at the same location (which may or may not be the case), and if it is also assumed for initial estimates that the embankment sideslopes are constructed at 1.5 horizontal to 1 vertical (without stabilization berms), then the approximate maximum track settlement after 25 years within that 5 km section of track can be estimated from Figure B.6.

For example, from Appendix A, Table A.3, in the interval of track between KP 0 and KP 5, the maximum fill height was found to be 7.5 metres and the maximum thickness of ice and ice rich soil encountered in the boreholes drilled within that section of track was found to be about 6 metres. From Figure B.6, the magnitude of track settlement after 25 years, for this set of conditions is estimated to be about 0.2 metres.

The foregoing method was used to estimate the maximum track settlement that could be expected to occur after 25 years within each 5 km section of track and the results are presented in Table A.3.

### 7.3 Discussion

A review of the results presented on Table A.3 indicates that track settlements can be expected to exceed about 1 metre in four sections of the railway line, as listed for convenience on Table A.4[2]. Within these four track sections, the magnitude of track

[2] The track alignment and embankment heights may have been modified in some locations since June, 2011 when the foregoing analysis was undertaken, however since the foregoing description is only

settlements that can be expected if stabilizing berms were provided on both sides of the fill embankments have been estimated from Figures B.7 and B.8 and are also noted on Table A.4.

For example, for the track interval between KP 20 and KP 25, the maximum embankment height was found to be 13.7 metres and the maximum ice thickness encountered in the boreholes in this section was about 16 metres. From Figure B.4a, the estimated track settlement is estimated to be about 1.7 metres after 25 years[3]. From Figure B.7, if stabilizing berms 2.3 metres high were used (1/6 of 13.7 metres), track settlements could be reduced to about 0.9 metres.

If the berm height were increased to 4.6 metres (1/3 of 13.7 metres), then from Figure B.8, track settlements after 25 years would be about 0.65 metres. In this example it appears that the increase in the volume of material required for the higher stabilizing berm is not very effective.

The railway alignment and embankment heights have been modified in some locations since Tables A.3 and A.4 were prepared in June 2011; however no attempt was made to update these tables when this report was finalized, since the analysis provided here is only intended for initial design. All of the tables in Appendix A should be updated during detailed design, once the alignment and embankment heights have been finalized.

---

intended to illustrate the method of analysis, no attempt has been made to update the tables in Appendix A.

[3] Figure B.6 cannot be used in this example because that chart has a maximum track settlement of 1.5 metres.

# 8 CULVERT DEFORMATIONS

## 1.1 General

As indicated on Figures B.2 and B.3 in Appendix B, it is apparent that if a culvert is installed near the base of a high embankment, it will experience significant deformations as a result of creep of the underlying ice rich materials, even if no thaw degradation occurs in the vicinity of the culvert.

Figure B.9 presents plots of the invert elevation of a culvert constructed near the base of an embankment for several possible scenarios. The data used to plot the curves on Figure B.9 were taken from the results of the FLAC calculations for a 25 metre high embankment having sideslopes of 1.5 horizontal to 1 vertical, with no stabilization berm, a 4 metre high berm and an 8 metre high berm. In this example, the foundation was assumed to consist of 2 metres of sand and gravel soil cover overlying 20 metres of ice or ice rich soil.

The black line on Figure B.9 represents the position of the invert of the culvert immediately after construction, which was installed at an arbitrary elevation of 24 metres. After 25 years, if no stabilization berm were installed, the invert below track centreline would have settled about 3 metres, while the invert at the inlet (or outlet) would have risen about 2.6 metres.

More significantly, the culvert has been subjected to significant longitudinal compressive and tensile forces and has increased in length by about 4.7 metres, which represents an average longitudinal strain of over 10 percent. The calculated culvert deformations for the 3 stabilization berm heights shown on Figure B.9 are summarized in the following table:

| Berm Height (m) | Culvert Settlement below Track Centreline (m) | Rise at Culvert Inlet (m) | Increase in Length of Culvert (m) | Average Longitudinal Strain |
|---|---|---|---|---|
| 0 | 3.0 | 2.6 | 4.7 | 11.4% |
| 4.2 | 2.6 | 1.2 | 1.7 | 4.1% |
| 8.1 | 1.9 | 0.4 | 1.2 | 2.9% |

The culvert deformations and associated strains are significant and it can be expected that, under unfavourable circumstances (high embankments, thick ice-rich soil layers below the embankments), it is probable that culverts will eventually collapse.

## 8.1 Estimated Culvert Settlements

As demonstrated by the results presented in the foregoing table, the magnitude of vertical settlement below the centreline of an embankment provides a general indication of the amount of vertical and horizontal deformation that a culvert at that location will experience.

The estimated settlements at the base of the embankments below track centreline, as calculated by the FLAC computer program, have been plotted as a function of embankment height and the thickness of ice in the embankment foundations on Figures B.10 and B.11 in Appendix B. Figure B.10 applies to the case where the embankments have no stabilizing berms, while Figures B.11a and B.11b apply to the case where stabilizing berms, equal to 1/6 and 1/3 of the embankment heights, have been used. As illustrated in these figures, the presence of stabilizing berms reduces the expected vertical settlements and therefore the associated culvert deformations.

The list of culverts, as at July 14 2011, their locations and corresponding fill heights, as measured from the culvert invert elevation on track centreline to the top of the subgrade soil, (that is the base of the sub-ballast) was provided by Dillon Consulting and is presented in Appendix A, Table A.5. At the time of this analysis, it was expected that culverts would be required at over 180 locations along the railway line[4]. The number of culverts and their diameters will vary at each location depending on the design flows and other factors.

For the initial assessment of the culvert settlements at each location, it was assumed that all embankment sideslopes would be constructed at 1.5 horizontal to 1 vertical, as shown on Table A.5 in Appendix A.

[4] The list of culverts was updated several time after July 14, 2011, however since the foregoing description is only intended to illustrate the method of analysis, no attempt has been made to update the tables in Appendix A.

At the time of this analysis, borehole information was not generally available at the exact culvert locations. An estimate of the probable maximum thickness of ice or ice rich soil in the vicinity of each culvert was therefore obtained by recording the maximum thickness of these materials encountered in all of the boreholes drilled within a few kilometres of each culvert location. This approach is expected to result in an over-estimation of the thickness of ice rich material below some of the culverts and therefore may overestimate the culvert settlements.

The culvert settlements can be readily estimated from the fill height and probable maximum thickness of ice rich materials in the foundations soils from Figures B.10 and B.11. The creep analyses considered only a limited number of fill heights and ice rich material thicknesses, and therefore it is necessary to extrapolate between the curves presented on the figures. Nevertheless the results presented on these figures allow culvert settlements to be estimated to an accuracy that is believed to be adequate for preliminary design purposes.

For example, referring to Appendix A Table A.5, for culvert CV-14-3, which is located near KP 14.7, the fill height is 13.9 metres with assumed embankment sideslopes of 1.5 horizontal to 1 vertical. The maximum thickness of ice rich material encountered in the boreholes drilled in the vicinity of this culvert was found to be about 11 metres. From Figure B.6, the settlement at track level is expected to be about 1.4 metres after 25 years, while from Figure B.10, the settlement at the base of the embankment (culvert elevation) is expected to be about 1 metre.

A similar procedure was used to estimate the approximate magnitude of the expected settlements after 25 years, at culvert elevation, for all of the culvert locations listed on Table A.5.

As mentioned earlier, total track settlements over a period of 25 years should not exceed about 1 metre. The results presented on Table A.5 indicate that track settlements can be expected to exceed about 1 metre at 5 culvert locations which, for convenience, have been summarized on Table A.6 in Appendix A.

At each of these 5 locations, the expected track and culvert settlements with no berm (as estimated from Figures B.6 and B.10) are shown, together with the expected track and culvert settlements (as estimated from Figures B.7 and B.11) that can be expected if stabilizing berms, with a height equal to 1/6 of the total embankment height, are provided on both sides of the embankments.

As summarized on Table A.6, track settlements at these 5 culvert locations can be reduced to less than about 1 metre after 25 years by providing stabilizing berms on both sides of the embankment at these locations.

### 8.2 Discussion

A review of the results presented on Table A.5 indicates that at the majority of culvert locations, track and culvert settlements due to creep of ice rich materials in the embankment foundations soils are expected to be relatively modest.

In those locations where culvert deformations are expected to be excessive, a number of potential methods to reduce the risk of culvert failure due to creep deformations can be considered, including:

1) Undertaking more detailed subsurface investigations at the locations of culverts below high embankments to identify those locations where ice or ice rich soils are also relatively thick.

2) Planning for culvert assessment, maintenance and replacement during railway operations. The duration of the culvert design life will depend on actual embankment creep rates and the ability of the culvert to withstand the associated deformations. Replacement can be facilitated by using fill that does not contain oversize rocks in the vicinity of culverts, at those locations where there is a risk of significant creep movements, so that a replacement culvert can be installed by tunneling through the embankment without interrupting rail traffic.

3) Using heavy wall pipe rather than corrugated metal pipe for those culverts that are located where there is a high risk of significant creep movements.

4) Using a deformable coating (such as several inches of bitumen in combination with a polyethylene membrane liner) on the outside of the culvert or by using telescoping heavy wall pipe to reduce shear stresses along the culvert.

5) Placing soil reinforcement textile (such as Tensar) within the embankment fill in the vicinity of the culvert to reduce horizontal deformations.

The FLAC computer program can incorporate the presence of steel structures (such as culverts or steel pipe) and could be used to estimate bending moments, shear forces and longitudinal tensile and compressive stresses induced in the culvert due to creep. Such analyses could be used to assess the cost effectiveness of these and other potential remedial measures.

It should be noted that the list of culverts, their locations and corresponding fill heights were modified several times after Tables A.5 and A.6 were prepared in June, 2011. During detailed design, once the railway alignment, embankment heights and the locations of all large diameter culverts have been finalized, these tables should be updated as necessary.

# 9 DISCUSSION AND RECOMMENDATIONS

## 9.1 Limitations of the Evaluation

It is understood that one of the objectives of this phase of design is to refine excavation and fill quantities and update the capital cost of constructing the railway.

The assessment of the configurations of the embankment fills presented here and the associated track and culvert settlements is considered to be reasonable for initial volume estimates. The actual configurations of the embankment fills should be refined during later stages of design based on site specific conditions, including the slope of the existing ground, the requirement for sidehill cuts, properties of the blasted rock fill, fill placement procedures and, most importantly, the thickness of ice rich material that is present below the fill.

The evaluation of the probable stable cut slope angles is also expected to provide a reasonable indication of the total volume of cut excavation required to construct the railway grade. The actual cut slope angles can be expected to vary from those presented here based primarily on the local topography, surface drainage conditions, variations in the fines contents of the overburden soils and the proportion of ice rich materials exposed in the cuts.

*Availability of Subsurface Information*

The analyses described in this report were undertaken in July to allow alignment design work to proceed on schedule. At that time, very little information from the drilling investigations undertaken during 2011 was available to incorporate into the analysis. Once additional 2011 site investigation information became available, the data was reviewed to confirm that the conditions used in the analysis were still applicable.

The review of the subsurface information obtained during the 2011 field investigations indicates that subsurface conditions are substantially the same as those assumed for the analyses described in this report. The results of the 2011 field investigations are not expected to require any substantial modifications to the initial geotechnical recommendations presented in this report. However, the subsurface information obtained during 2011 in conjunction with additional subsurface data should be used to refine the design of the railway earthworks at specific locations along the railway alignment during detailed design.

*Drilling Procedure*

The earthworks assessment presented here is based primarily on subsurface conditions as encountered in boreholes drilled with a diamond drill rig. The use of chilled brine, which is required to obtain satisfactory samples of the frozen overburden soils with this type of drill, was not allowed under the terms of the drilling permit. As a result, core recovery of the overburden soils in most of the boreholes was poor and the ice and fines contents of the overburden remains uncertain.

For the purpose of this evaluation, it was assumed that the thicknesses of high ice content layers, as noted on the borehole logs, are correct. Most importantly, it was assumed that in those boreholes and sections of boreholes where core samples were not recovered, it was because the soil did not contain a significant amount of excess ice.

*Grain Size Distribution Data*

As mentioned, few representative samples of core were recovered that were suitable for the determination of the fines contents of the material at depth. Therefore, the majority of grain size analyses were carried out on near surface samples recovered from shallow test pits.

The near surface samples are believed to represent the average and range in fines contents of the various materials at depth; however this may not be the case. Of particular concern is that the near surface deposits may contain a smaller fraction of fines than the soils at depth, due to the migration of fines out of the near surface soils as a result of water infiltration into the active layer during the thaw season.

### 9.2 Summary of Recommendations

For convenience, the geotechnical recommendations that are presented in the report are summaries below. The primary focus of future geotechnical investigations should be to confirm the key assumptions on which the initial geotechnical evaluation presented in this report are based.

1) The initial embankment and cut slope configurations recommended in this report are considered to be realistic. It is recommended however, that an allowance be added to the earthworks volume estimate to account for the limitations of the existing subsurface information.

2) It is recommended that the railway operating cost estimates should include an allowance for maintenance of the cuts, embankments and culverts, which will be required during railway operations.

3) During detailed design, the initial geotechnical recommendations for the preliminary design of the embankment fills; culverts and cuts, as recommended in this report, should be reviewed and refined by an experienced geotechnical engineer, based on site specific information, including subsurface conditions, surface topography, surface drainage and other considerations.

4) It is expected that an additional 75 to 100 boreholes will need to be drilled in those locations along the railway alignment that, based on the current evaluation, appear to be of greatest concern with respect to the behaviour of the embankments, culverts and cut slopes. The exact locations and purpose of these boreholes will need to be determined during planning for future field investigations.

5) Grain size tests should be carried out on overburden samples recovered from boreholes drilled with the sonic drill, to determine if the fines contents of the near surface soils are generally representative of the soils at depth.

6) Consideration should be given to undertaking seismic refraction surveys, oriented perpendicular to track centreline, at selected locations on the valley slopes adjacent to the Cockburn Lakes to determine the depth to bedrock below the valley walls in the vicinity of the alignment.

7) Consideration should be given to excavating large test pits in locations where major cuts are expected to be excavated and at each location where the higher

fills will be constructed, particularly within the section of the railway line between the Mary River Mine and KP 35 and on the steep valley slopes adjacent to the Cockburn Lakes.

8) It is recommended that all of the borehole logs from the various geotechnical investigations and all of the geophysical survey tracks, be plotted on the terrain maps. This information should provide a more accurate indication of the extent and distribution of ground ice along the railway alignment and enable a improved assessment of potential creep deformations.

9) It is recommended that existing information from previous case histories regarding the construction of embankments on ice and ice rich soils be compiled and reviewed in an effort to provide a more reliable assessment of the effects of creep behaviour on the performance of the proposed railway.

10) The estimated track and culvert settlements due to creep of the ice rich overburden soils should be updated during detailed design, once the railway alignment, embankment heights and culvert locations have been finalized.

11) It is recommended that additional creep analyses be undertaken to verify the extrapolated design curves presented in this report, to assess the effect of embankment strength properties on creep deformations, to optimize the configuration of stabilization berms and to further assess potential culvert deformations.

12) Consideration should be given to constructing one of the proposed railway embankments as soon as possible once earthworks construction begins. The embankment should be constructed to a height of 20 metres, in a location underlain by a significant thickness of ice and should be suitably instrumented and monitored.

## 10 CLOSURE

We trust that the foregoing geotechnical evaluation and recommendations meet your current requirements. Please contact us at any time if you have questions or require additional information.

THURBER

## REFERENCES

Canarail, 2010. *Railway Plan and Profile*. Drawing numbers A3-159952-5200-121-5301-1 through 84, Revision 1, dated December 6, 2010.

EBA Engineering Consultants Ltd, 2008a. *Mary River Embankment Creep Analysis.* Technical Memo to Knight Piesold Consulting, dated November 26, 2008.

EBA Engineering Consultants Ltd, 2008b. *Geophysical Site Investigation – April 2008.* Mine, Railway Alignment and Port Facility Areas, Mary River, NU. Report to Baffinland Iron Mines Corporation, dated October, 2008.

EBA Engineering Consultants Ltd., 2011a. *Mary River Project - Geotechnical Recommendations for Design of Rock Cuts for the Mary River Railway*, (in preparation).

Knight Piesold, 2007. *Mary River Project, Seismic Design Parameters*, Memorandum submitted to Mr. M. Cambruzzi of Baffinland Iron Mines Corporation, dated April 30, 2007.

Knight Piesold, 2008. Air *Photo Interpretation, Southern Railway Alignment,* Figures 1 to 47, Revision 1, dated June 5, 2008.

Knight Piesold, 2010a. *Geotechnical Section Along Rail Alignment*, Figures 4.1 to 4.25, Revision 0, dated October 13, 2010.

Knight Piesold, 2010b. *Rail Alignment, Steensby Port Site Infrastructure, & Borrow Sources, 2008 Site Investigations Summary Report*. Report to Baffinland Iron Mines Corporation, Revision 0, dated December 31, 2010.

McRoberts, E.C., 1988. *Secondary Creep Interpretation of Ice Rich Permafrost,* Proceedings of the 5th International Conference on Permafrost, Trondheim, Norway, August, 1988.

Morgenstern, N.R., Roggensack, W.D., and Weaver, J.S., 1980. *The behaviour of friction piles in ice and ice-rich soils,* The Canadian Geotechnical Journal, 1980, Volume 17, pp. 403 to 415.

Smith, L.B., Notenboom, W.G., Campbell, M., Cheema, S., and Smith, T., *Pangnirtung water reservoir, geotechnical aspects*. The Canadian Geotechnical Journal, 1989, Volume 26, pp. 335 to 347.

Thurber Engineering Ltd., 2011a. *Mary River Project, 2011 Onshore Geotechnical Investigations, Summary of Results.* Report submitted to Hatch Ltd., dated November 10, 2011.

Thurber Engineering Ltd., 2011b. *Mary River Project, Initial Geotechnical Recommendations, Railway Bridges and Culverts.* Report submitted to Hatch Ltd., dated November 15, 2011.

THURBER

# STATEMENT OF LIMITATIONS AND CONDITIONS

## 1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

## 2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

## 3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this Report expressly addresses proposed development, design objectives and purposes, and then only to the extent there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation or to consider such representations, information and instructions.

## 4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS WE MAY EXPRESSLY APPROVE. The contents of the Report remain our copyright property. The Client may not give, lend or, sell the Report, or otherwise make the Report, or any portion thereof, available to any person without our prior written permission. Any use which a third party makes of the Report, are the sole responsibility of such third parties. Unless expressly permitted by us, no person other than the Client is entitled to rely on this Report. We accept no responsibility whatsoever for damages suffered by any third party resulting from use of the Report without our express written permission.

## 5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and this report is delivered on the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

*(see over . . . . )*

THURBER



c) Design Services: The Report may form part of the design and construction documents for information purposes even though it may have been issued prior to the final design being completed. We should be retained to review the final design, project plans and documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the report recommendations and the final design detailed in the contract documents should be reported to us immediately so that we can address potential conflicts.

d) Construction Services: During construction we must be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

## 6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

## 7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Client's benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

## 8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

## 9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

# APPENDIX A

## Tables

# A.1
# Mary River Project
# Statistical Analysis of Fines Contents of Test Pit Samples

| Terrain Type | Terrain Map Symbols | Average Fines Content | Standard Deviation | Number of Samples Tested | 67 % of Samples Had Fines Contents Less Than (%) | 95 % of Samples Had Fines Contents Less Than (%) |
|---|---|---|---|---|---|---|
| Alluvial Deposits | Ac, At, Ap, Af | 4 | 7 | 12 | 11 | 18 |
| Glaciofluvial Deposits/Terrace | GF(u), GFt | 8 | 13 | 41 | 21 | 34 |
| Kame Terrace | Kt | 14 | 32 | 9 | 46 | 78 |
| Till Blanket | Tb, Tbf | 14 | 17 | 46 | 31 | 48 |
| Till Veneer | Tv | 16 | 13 | 28 | 29 | 42 |
| Hummocky Till | Th | 26 | 9 | 28 | 35 | 44 |

# Table A.2
# Mary River Project
# Preliminary Recommendations for Railway Cut Slope Angles

| Section of Track (km) | Maximum Depth of Cut (m) | Dominant Terrain Type | Average Fines Content of Soil (%) | Recommended Average Slope Angle | Probable Maximum Ice Thickness in this Section of Track (m) |
|---|---|---|---|---|---|
| 0-5 | 4.3 | Kt | 14 | 2H:1V | 6 |
| 5-10 | 0.0 | Kt | 14 | 2H:1V | 6 |
| 10-15 | 6.7 | GFt | 8 | 2H:1V | 18 |
| 15-20 | 6.8 | GFt | 8 | 2H:1V | 23 |
| 20-25 | 0.0 | GF(u) | 8 | 2H:1V | 16 |
| 25-30 | 1.8 | Tb | 14 | 2H:1V | 6 |
| 30-35 | 8.5 | Tb | 14 | 2H:1V | 0 |
| 35-40 | 3.8 | At | 4 | 2H:1V | 20 |
| 40-45 | 0.7 | Th | 26 | 3H:1V | 8 |
| 45-50 | 2.8 | Tb | 14 | 2H:1V | 8 |
| 50-55 | 1.1 | Tb | 14 | 2H:1V | 10 |
| 55-60 | 0.0 | Tb | 14 | 2H:1V | 5 |
| 60-65 | 0.0 | Tv | 16 | 3H:1V | 7 |
| 65-70 | 0.0 | Tb | 14 | 2H:1V | 20 |
| 70-75 | 3.6 | Tb | 14 | 2H:1V | 5 |
| 75-80 | 5.1 | Tb | 14 | 3H:1V | 4 |
| 80-85 | 4.2 | Tv | 16 | 3H:1V | 0 |
| 85-90 | 14.0 | Tv | 16 | 3H:1V | 0 |
| 90-95 | 11.2 | Tv | 16 | 3H:1V | 0 |
| 95-100 | 15.8 | Tv | 16 | 3H:1V | 0 |
| 100-105 | 64.8 | Tv | 16 | 3H:1V | 0 |
| 105-110 | 53.6 | Tv | 16 | 3H:1V | 0 |
| 110-115 | 7.6 | Tv | 16 | 3H:1V | 5 |
| 115-120 | 12.0 | GFt | 8 | 2H:1V | 2 |
| 120-125 | 27.8 | GFt | 8 | 2H:1V | 0 |
| 125-130 | 6.9 | GFt | 8 | 2H:1V | 18 |
| 130-135 | 23.0 | Ap | 4 | 2H:1V | 0 |
| 135-140 | 26.8 | Tv | 16 | 3H:1V | 0 |
| 140-145 | 17.2 | Tv | 16 | 3H:1V | 0 |

**Table A.3**
**Mary River Project**
**Preliminary Estimate of Track Settlements After 25 Years**
**Due to Creep of Embankment Foundation**

| Track Section (km) | Maximum Height of Fill in Track Section (m) | Probable Maximum Thickness of Ice Rich Soil in Track Section (m) | Embankment Side Slope Angle | Total Track Settlement with No Berm |
|---|---|---|---|---|
| 0-5 | 7.5 | 6 | 1.5 | 0.2 |
| 5-10 | 14.1 | 6 | 1.5 | 0.7 |
| 10-15 | 19.9 | 18 | 1.5 | 4.3 |
| 15-20 | 11.2 | 23 | 1.5 | 0.9 |
| 20-25 | 13.7 | 16 | 1.5 | 1.7 |
| 25-30 | 13.9 | 6 | 1.5 | 0.7 |
| 30-35 | 5.5 | 0 | 1.5 | 0.1 |
| 35-40 | 15.0 | 20 | 1.5 | 2.4 |
| 40-45 | 7.2 | 8 | 1.5 | 0.2 |
| 45-50 | 9.0 | 8 | 1.5 | 0.4 |
| 50-55 | 5.2 | 10 | 1.5 | 0.1 |
| 55-60 | 9.8 | 5 | 1.5 | 0.4 |
| 60-65 | 6.3 | 7 | 1.5 | 0.2 |
| 65-70 | 3.5 | 20 | 1.5 | 0.2 |
| 70-75 | 3.8 | 5 | 1.5 | 0.1 |
| 75-80 | 11.2 | 4 | 1.5 | 0.4 |
| 80-85 | 9.9 | 0 | 1.5 | 0.1 |
| 85-90 | 14.7 | 0 | 1.5 | 0.1 |
| 90-95 | 8.0 | 0 | 1.5 | 0.1 |
| 95-100 | 10.0 | 0 | 1.5 | 0.1 |
| 100-105 | 13.7 | 0 | 1.5 | 0.1 |
| 105-110 | 12.7 | 0 | 1.5 | 0.1 |
| 110-115 | 8.4 | 5 | 1.5 | 0.2 |
| 115-120 | 14.6 | 2 | 1.5 | 0.5 |
| 120-125 | 9.1 | 0 | 1.5 | 0.1 |
| 125-130 | 14.6 | 18 | 1.5 | 2.1 |
| 130-135 | 8.4 | 0 | 1.5 | 0.1 |
| 135-140 | 13.5 | 0 | 1.5 | 0.1 |
| 140-145 | 6.6 | 0 | 1.5 | 0.1 |

**Table A.4**

**Mary River Project**

**Preliminary List of Track Sections Requiring Stabilizing Berms**

| Track Section (km) | Maximum Height of Fill in Track Section (m) | Probable Maximum Thickness of Ice Rich Soil in Track Section (m) | Embankment Side Slope Angle | Total Track Settlement with No Berm | Total Track Settlement with Berms 1/6 Height of Embankment | Total Track Settlement with Berms 1/3 Height of Embankment |
|---|---|---|---|---|---|---|
| 10-15 | 19.9 | 18 | 1.5H:1V | 4.3 | 2.2 | 1.5 |
| 20-25 | 13.7 | 16 | 1.5H:1V | 1.7 | 0.9 | 0.7 |
| 35-40 | 15.0 | 20 | 1.5H:1V | 2.4 | 1.3 | 1.0 |
| 125-130 | 14.6 | 18 | 1.5H:1V | 2.1 | 1.2 | 0.9 |

**Mary River Project**

**Prelliminary Estimate of Track and Culvert Settlements at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| CV-1-1 | 1.5 | 4.2 | 7 | 1.5H:1V | 0.1 | 0.1 |
| CV-1-2 | 1.5 | 4.5 | 7 | 1.5H:1V | 0.1 | 0.1 |
| CV-2-1 | 2.4 | 7 | 7 | 1.5H:1V | 0.2 | 0.2 |
| CV-2-2 | 2.8 | 6.3 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-3-1 | 3.1 | 8.1 | 4 | 1.5H:1V | 0.2 | 0.1 |
| CV-3-2 | 3.9 | 6.3 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-4-1 | 4.3 | 9.4 | 4 | 1.5H:1V | 0.3 | 0.2 |
| CV-4-2 | 4.6 | 3.4 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-4-3 | 4.8 | 5.4 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-5-1 | 5.2 | 4.7 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-5-2 | 5.5 | 3.6 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-5-3 | 5.6 | 7.6 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-6-1 | 6.2 | 7.4 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-6-2 | 6.4 | 5.2 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-6-3 | 6.7 | 9.2 | 3 | 1.5H:1V | 0.2 | 0.2 |
| CV-7-1 | 7.1 | 9.2 | 3 | 1.5H:1V | 0.2 | 0.2 |
| CV-7-2 | 7.3 | 3.8 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-7-3 | 7.6 | 8.4 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-8-1 | 8.1 | 8.8 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-8-2 | 9.0 | 8.8 | 6 | 1.5H:1V | 0.3 | 0.2 |
| CV-9-1 | 9.8 | 10.1 | 6 | 1.5H:1V | 0.4 | 0.2 |
| CV-10-1 | 10.8 | 7.5 | 18 | 1.5H:1V | 0.4 | 0.4 |
| CV-12-1 | 12.6 | 4.6 | 18 | 1.5H:1V | 0.1 | 0.1 |
| CV-12-2 | 12.9 | 7.6 | 13 | 1.5H:1V | 0.3 | 0.3 |
| CV-13-1 | 13.7 | 6.6 | 10 | 1.5H:1V | 0.2 | 0.2 |
| CV-13-2 | 14.0 | 5.3 | 10 | 1.5H:1V | 0.1 | 0.1 |
| CV-14-1 | 14.3 | 5.9 | 10 | 1.5H:1V | 0.2 | 0.2 |
| CV-14-2 | 14.5 | 5.3 | 11 | 1.5H:1V | 0.1 | 0.1 |
| CV-14-3 | 14.7 | 13.9 | 11 | 1.5H:1V | 1.4 | 1 |
| CV-15-1 | 15.9 | 6.5 | 11 | 1.5H:1V | 0.2 | 0.2 |
| CV-16-1 | 16.1 | 6.6 | 11 | 1.5H:1V | 0.2 | 0.2 |

**Table A.5** (Page 2 of 6)

**Mary River Project**

**Prelliminary Estimate of Track and Culvert Settlements at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| CV-16-2 | 16.4 | 6.4 | 11 | 1.5H:1V | 0.2 | 0.2 |
| CV-17-1 | 17.2 | 9 | 11 | 1.5H:1V | 0.5 | 0.4 |
| CV-17-2 | 17.9 | 4.2 | 22 | 1.5H:1V | 0.1 | 0.1 |
| CV-18-1 | 18.0 | 4.1 | 22 | 1.5H:1V | 0.1 | 0.1 |
| CV-18-2 | 18.3 | 5.4 | 22 | 1.5H:1V | 0.2 | 0.1 |
| CV-18-3 | 18.6 | 4.3 | 22 | 1.5H:1V | 0.1 | 0.1 |
| CV-19-1 | 19.1 | 11.4 | 22 | 1.5H:1V | 1.3 | 1.2 |
| CV-19-2 | 19.7 | 10.1 | 22 | 1.5H:1V | 0.9 | 0.8 |
| CV-20-1 | 20.5 | 10.3 | 13 | 1.5H:1V | 0.7 | 0.6 |
| CV-20-2 | 20.7 | 7.9 | 13 | 1.5H:1V | 0.4 | 0.4 |
| CV-21-1 | 21.0 | 7.5 | 16 | 1.5H:1V | 0.4 | 0.4 |
| CV-21-2 | 21.2 | 7.8 | 16 | 1.5H:1V | 0.4 | 0.4 |
| CV-21-3 | 22.0 | 4.6 | 16 | 1.5H:1V | 0.1 | 0.1 |
| CV-22-1 | 22.3 | 6.7 | 16 | 1.5H:1V | 0.3 | 0.2 |
| CV-22-2 | 22.5 | 4.4 | 16 | 1.5H:1V | 0.1 | 0.1 |
| CV-22-3 | 22.7 | 7 | 16 | 1.5H:1V | 0.3 | 0.3 |
| CV-22-4 | 22.8 | 6.9 | 16 | 1.5H:1V | 0.3 | 0.3 |
| CV-22-5 | 23.0 | 14.2 | 16 | 1.5H:1V | 1.7 | 1.2 |
| CV-23-1 | 23.5 | 9.3 | 16 | 1.5H:1V | 0.6 | 0.5 |
| CV-24-1 | 24.4 | 10.2 | 15 | 1.5H:1V | 0.8 | 0.7 |
| CV-24-2 | 24.9 | 14.3 | 15 | 1.5H:1V | 1.7 | 1.2 |
| CV-25-1 | 25.6 | 9.9 | 15 | 1.5H:1V | 0.7 | 0.7 |
| CV-25-2 | 26.0 | 5.6 | 15 | 1.5H:1V | 0.2 | 0.2 |
| NCV-26-1 | 26.7 | 2.9 | 15 | 1.5H:1V | 0.1 | 0.1 |
| NCV-26-2 | 26.8 | 3.3 | 15 | 1.5H:1V | 0.1 | 0.1 |
| NCV-26-3 | 27.0 | 3 | 15 | 1.5H:1V | 0.1 | 0.1 |
| NCV-27-1 | 27.3 | 3 | 7 | 1.5H:1V | 0.1 | 0.1 |
| NCV-27-2 | 27.6 | 4.5 | 7 | 1.5H:1V | 0.1 | 0.1 |
| NCV-27-3 | 27.7 | 3.1 | 7 | 1.5H:1V | 0.1 | 0.1 |
| NCV-27-4 | 27.9 | 3.8 | 7 | 1.5H:1V | 0.1 | 0.1 |
| NCV-28-1 | 28.4 | 4.3 | 6 | 1.5H:1V | 0.1 | 0.1 |
| | | | | | | |

**Table A.5** (Page 3 of 6)
**Mary River Project**
**Prelliminary Estimate of Track and Culvert Settlements**
**at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| NCV-28-2 | 28.8 | 3 | 6 | 1.5H:1V | 0.1 | 0.1 |
| NCV-29-1 | 29.9 | 5.1 | 6 | 1.5H:1V | 0.1 | 0.1 |
| NCV-30-1 | 30.6 | 4.2 | 6 | 1.5H:1V | 0.1 | 0.1 |
| NCV-31-1 | 31.3 | 4.3 | 6 | 1.5H:1V | 0.1 | 0.1 |
| NCV-31-2 | 31.8 | 4.5 | 0 | 1.5H:1V | 0.1 | 0.1 |
| NCV-32-1 | 33.0 | 4.5 | 0 | 1.5H:1V | 0.1 | 0.1 |
| NCV-33-1 | 33.3 | 4.5 | 0 | 1.5H:1V | 0.1 | 0.1 |
| NCV-34-1 | 34.4 | 4.5 | 0 | 1.5H:1V | 0.1 | 0.1 |
| NCV-34-2 | 34.9 | 2.9 | 0 | 1.5H:1V | 0.1 | 0.1 |
| NCV-36-1 | 36.4 | 2.3 | 10 | 1.5H:1V | 0.1 | 0.1 |
| NCV-36-2 | 36.7 | 3 | 10 | 1.5H:1V | 0.1 | 0.1 |
| NCV-38-1 | 38.1 | 9.6 | 10 | 1.5H:1V | 0.6 | 0.4 |
| NCV-38-2 | 38.5 | 4.9 | 10 | 1.5H:1V | 0.1 | 0.1 |
| NCV-39-1 | 39.0 | 6.6 | 10 | 1.5H:1V | 0.2 | 0.2 |
| NCV-39-2 | 39.4 | 1.8 | 10 | 1.5H:1V | 0.1 | 0.1 |
| NCV-40-1 | 40.3 | 2.6 | 10 | 1.5H:1V | 0.1 | 0.1 |
| NCV-40-2 | 40.9 | 4.8 | 2 | 1.5H:1V | 0.1 | 0.1 |
| NCV-41-1 | 41.8 | 1.9 | 2 | 1.5H:1V | 0.1 | 0.1 |
| NCV-42-1 | 42.5 | 4.6 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-43-1 | 43.4 | 4.3 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-44-1 | 44.9 | 4.4 | 8 | 1.5H:1V | 0.1 | 0.1 |
| CV-45-1 | 46.0 | 4.3 | 8 | 1.5H:1V | 0.1 | 0.1 |
| CV-47-1 | 47.8 | 2.6 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-49-1 | 49.1 | 2.8 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-49-2 | 49.4 | 3.8 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-49-3 | 50.0 | 2.8 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-50-1 | 50.2 | 3.1 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-50-2 | 50.8 | 2.7 | 3 | 1.5H:1V | 0.1 | 0.1 |
| CV-51-1 | 51.3 | 2.7 | 10 | 1.5H:1V | 0.1 | 0.1 |
| CV-52-1 | 52.5 | 2.6 | 10 | 1.5H:1V | 0.1 | 0.1 |
| CV-52-2 | 52.9 | 4.9 | 10 | 1.5H:1V | 0.1 | 0.1 |

**Table A.5** (Page 4 of 6)
**Mary River Project**
**Prelliminary Estimate of Track and Culvert Settlements**
**at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| CV-53-1 | 53.4 | 5.2 | 10 | 1.5H:1V | 0.1 | 0.1 |
| CV-56-1 | 56.3 | 9.7 | 5 | 1.5H:1V | 0.4 | 0.2 |
| CV-57-1 | 57.6 | 3.6 | 5 | 1.5H:1V | 0.1 | 0.1 |
| CV-60-1 | 60.6 | 4.7 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-62-1 | 62.9 | 5.4 | 5 | 1.5H:1V | 0.1 | 0.1 |
| CV-63-1 | 63.9 | 5.3 | 5 | 1.5H:1V | 0.1 | 0.1 |
| CV-64-1 | 64.8 | 4.2 | 6 | 1.5H:1V | 0.1 | 0.1 |
| CV-65-1 | 65.1 | 2.9 | 6 | 1.5H:1V | 0.1 | 0.1 |
| CV-65-2 | 65.4 | 2.7 | 6 | 1.5H:1V | 0.1 | 0.1 |
| CV-67-1 | 67.1 | 2.6 | 20 | 1.5H:1V | 0.1 | 0.1 |
| CV-68-1 | 68.1 | 3.2 | 20 | 1.5H:1V | 0.1 | 0.1 |
| CV-69-2 | 69.6 | 4.3 | 20 | 1.5H:1V | 0.1 | 0.1 |
| CV-71-1 | 71.7 | 4.3 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-73-1 | 73.2 | 4.4 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-74-1 | 74.6 | 4.1 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-75-1 | 75.1 | 3.4 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-77-1 | 77.1 | 5.2 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-77-2 | 77.2 | 2.6 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-78-1 | 78.6 | 13.1 | 4 | 1.5H:1V | 0.6 | 0.3 |
| CV-78-2 | 78.9 | 10.7 | 4 | 1.5H:1V | 0.4 | 0.2 |
| CV-78-3 | 79.0 | 8.2 | 4 | 1.5H:1V | 0.2 | 0.1 |
| CV-79-1 | 79.2 | 8.0 | 4 | 1.5H:1V | 0.2 | 0.2 |
| CV-79-2 | 79.4 | 3.3 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-79-3 | 79.6 | 6.2 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-79-4 | 79.8 | 5.4 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-80-1 | 80.2 | 10.2 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-80-3 | 80.6 | 7.4 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-80-4 | 80.7 | 2.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-80-5 | 80.9 | 2.8 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-81-1 | 81.1 | 4.3 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-81-2 | 81.2 | 4.8 | 0 | 1.5H:1V | 0.1 | 0.1 |

**Table A.5** (Page 5 of 6)
**Mary River Project**
**Prelliminary Estimate of Track and Culvert Settlements**
**at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| CV-81-3 | 81.4 | 2.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-81-5 | 81.9 | 4.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-81-6 | 82.0 | 2.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-82-1 | 82.0 | 3.1 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-84-1 | 84.1 | 9.3 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-84-2 | 84.5 | 4.8 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-85-1 | 85.1 | 5.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-85-2 | 85.3 | 7.5 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-93-1 | 93.3 | 2.9 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-93-2 | 93.6 | 4.3 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-98-1 | 98.3 | 3.1 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-99-1 | 99.3 | 3.6 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-101-1 | 101.4 | 4.4 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-105-1 | 106 | 5.6 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-107-1 | 107.9 | 10.7 | 2 | 1.5H:1V | 0.3 | 0.1 |
| CV-109-1 | 109.4 | 4.4 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-110-1 | 110.8 | 4.6 | 2 | 1.5H:1V | 0.1 | 0.1 |
| CV-111-1 | 111.8 | 3 | 4 | 1.5H:1V | 0.1 | 0.1 |
| CV-117-1 | 117.4 | 6.9 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-117-2 | 117.7 | 4.9 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-117-3 | 117.8 | 4.3 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-118-1 | 118 | 3.7 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-118-2 | 118.8 | 7.1 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-119-1 | 119 | 8.6 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-119-2 | 119.4 | 4.8 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-119-3 | 120 | 8.3 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-120-1 | 120.6 | 6 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-121-1 | 121.1 | 5.1 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-121-2 | 121.2 | 5.1 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-121-3 | 121.4 | 4.3 | 1 | 1.5H:1V | 0.1 | 0.1 |
| CV-121-4 | 121.5 | 5.2 | 1 | 1.5H:1V | 0.1 | 0.1 |

**Table A.5** (Page 6 of 6)
**Mary River Project**
**Prelliminary Estimate of Track and Culvert Settlements**
**at Railway Culvert Locations**

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) |
|---|---|---|---|---|---|---|
| CV-121-5 | 122 | 5.1 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-123-1 | 123.2 | 7.3 | 14 | 1.5H:1V | 0.3 | 0.3 |
| CV-123-2 | 123.3 | 4.9 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-123-3 | 123.6 | 2.6 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-123-4 | 123.8 | 2.5 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-124-1 | 124.1 | 4.8 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-124-2 | 124.6 | 7.7 | 14 | 1.5H:1V | 0.4 | 0.4 |
| CV-124-3 | 124.9 | 6.7 | 14 | 1.5H:1V | 0.3 | 0.3 |
| CV-125-1 | 125 | 6.5 | 14 | 1.5H:1V | 0.3 | 0.3 |
| CV-125-2 | 125.2 | 8 | 14 | 1.5H:1V | 0.4 | 0.4 |
| CV-125-3 | 125.9 | 6.7 | 14 | 1.5H:1V | 0.3 | 0.3 |
| CV-126-1 | 126.2 | 14.6 | 14 | 1.5H:1V | 1.7 | 1.3 |
| CV-127-1 | 127.4 | 3.5 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-127-2 | 127.6 | 3.5 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-127-3 | 127.8 | 3.5 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-129-1 | 129.4 | 4.1 | 14 | 1.5H:1V | 0.1 | 0.1 |
| CV-131-1 | 131.7 | 8.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-133-1 | 133.2 | 3.2 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-133-2 | 133.9 | 9.1 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-134-1 | 134.3 | 6.7 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-134-2 | 134.7 | 2.9 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-136-1 | 136.2 | 7.4 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-136-2 | 136.8 | 8.8 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-138-1 | 139 | 4.8 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-140-1 | 140.6 | 3.3 | 0 | 1.5H:1V | 0.1 | 0.1 |
| CV-142-1 | 142.1 | 6.7 | 0 | 1.5H:1V | 0.1 | 0.1 |

# Table A.6
# Mary River Project
# Preliminary List of Culvert Locations Requiring Stabilizing Berms

| Culvert Number | Location (KP) | Fill Height Above Culvert Invert (m) | Maximum Probable Thickness of Ice Rich Soil Near this Location (m) | Embankment Sideslope Angle | Estimated Track Settlement Without Berms (m) | Estimated Culvert Settlement Without Berms (m) | Estimated Track Settlement With Berms 1/6 of Fill Height (m) | Estimated Culvert Settlement With Berms 1/6 of Fill Height (m) |
|---|---|---|---|---|---|---|---|---|
| CV-14-3 | 14.7 | 13.9 | 11 | 1.5H:1V | 1.4 | 1.0 | 0.6 | 0.2 |
| CV-19-1 | 19.1 | 11.4 | 22 | 1.5H:1V | 1.3 | 1.2 | 0.8 | 0.5 |
| CV-22-5 | 23.0 | 14.2 | 16 | 1.5H:1V | 1.7 | 1.3 | 0.9 | 0.4 |
| CV-24-2 | 24.9 | 14.3 | 15 | 1.5H:1V | 1.7 | 1.2 | 0.6 | 0.4 |
| CV-126-1 | 126.2 | 14.6 | 14 | 1.5H:1V | 1.7 | 1.3 | 0.9 | 0.4 |