Appendix M1

Geotechnical Assessment of the Non-Peat areas along the Proposed Route of the Corrib Onshore Pipeline

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1 INTRODUCTION

This report examines the geotechnical issues arising from the construction of the Corrib Onshore Pipeline in non-peatland areas along the proposed route from the landfall at Glengad to Bellanaboy Bridge Terminal Site. A geotechnical assessment report for the sections of the route within peat areas, undertaken by AGEC, is included in Appendix M2 of the EIS.

Drawing COR-25-MDR0470DG0201P03 in Appendix M1-A shows the locations and types of all the ground investigations carried out in the vicinity of the proposed route. Osiris Projects carried out a geophysical site investigation survey in Sruwaddacon Bay in 2007. Long sections of the proposed tunnel alignment in Sruwaddacon Bay (Drawing C10014-01) based on the geophysical survey are included in Appendix M1-A. Geotechnical surveys took place in Sruwaddacon Bay in 2008. Reports on these investigations are included in Appendix M1-B.

Geophysical and geotechnical data and information that is available provides sufficient level of confidence that the pipeline can be safely constructed and operated along the proposed route.

Further geotechnical investigations are planned to take place in Sruwaddacon Bay in 2010 to verify ground conditions along the tunnel route through Sruwaddacon Bay. Information gathered will be used for the final design of the TBM and tunnel.

A Geotechnical Risk Register is provided in Appendix M4. This has been compiled to identify potential risks and mitigation measures for the geotechnical elements of the proposed pipeline construction and operation along the entire route and for the various construction methods proposed. The register provides an outline description of the hazards, identifies the likely causes, describes the potential impact of the hazard and identifies the design and construction controls to be implemented in order to minimise the geotechnical risk. The register will be used actively throughout the project and will be up-dated to reflect additional data and experience as it is gained.

This report should be read in conjunction with the Corrib Onshore Pipeline EIS. Appendices referenced to in this report are appendices to the Corrib Onshore Pipeline EIS.

2 ROUTE DESCRIPTION

The proposed route is represented on the pipeline alignment sheets COR-25-MDR0470DG0301 to 0307 (see Appendix A). A site contour map is included in Appendix M1-A. For this report, the route has been divided into sections as outlined below between approximate chainages as marked on the alignment sheets (see Appendix A). Each section is described briefly below and in greater geotechnical detail in the following sections of this report.

Glengad (Chainage 83.40 – 83.90)

This section is within relatively flat improved agricultural grassland at about 10mOD (Ordnance Datum – Malin Head Datum). The proposed landfall valve location is at the west of this section and a proposed reception for the proposed tunnel under Sruwaddacon Bay to the east. A small cliff face about 3.0m to 4.0m high is exposed at the western end fronted by a beach leading to the sea. Sand dunes are located to the north of the section.

Crossing of Sruwaddacon Bay, (Glengad to Aghoos) (Chainage 83.90 – 88.77)

This section of the pipeline will be tunnelled underneath Sruwaddacon Bay. The tunnel will be approximately 4.9km long (approximately 4.6km of which will be underneath Sruwaddacon Bay) and will have an outside diameter of 4.2m.

It is proposed to tunnel from Aghoos to Glengad. The starting pit will therefore be in Aghoos and the reception pit at Glengad.

The location of the starting pit is in an area of peat with depths of up to 3.8m, averaging about 1.6m, and lies at an elevation of between about 5m and 15mOD. The reception pit at Glengad lies within agricultural grassland between about 7m and 12mOD east of the landfall valve site.

Based on site investigation works to date in the bay, overburden, deposits consist of sands/gravels and some finer silts and clays especially towards the eastern end of the bay. The overburden reaches depths of up to 25m below seabed level towards the centre of the bay. Bedrock was recorded as being variations of psammite, which have been altered due to metamorphism and foliations due to faulting in the area. Bedrock outcrops are evident on the foreshore.

The trajectory of the tunnel will be mostly within the sands and gravels which overlie bedrock in the bay. It is anticipated that approximately 80% of the length of the tunnel will be through superficial deposits (sands / gravels / clay), and the remainder (approximately 20%) will be through rock.

South of Sruwaddacon Bay to Terminal (Chainage 88.77 – 91.72)

The section of the route south of Sruwaddacon Bay traverses peat areas with peat depths of up to 5m. Some of the sections are forested. A stream crossing of the Leenamore River exists at chainage

89.15 approximately. An existing stone road (approximately 900m long) is in place along a section of the route from chainage 90.65 approximately, to the Terminal Site. The ground level varies from between 0mOD at the river crossing to a maximum of about 38mOD close to the Terminal Site.

This section contains deeper peats and is not covered further in this report. Refer to Appendix M2 of the EIS for an assessment of this section.

2.1 SITE INVESTIGATIONS

Drawing COR25MDR0470DG0201P03 in Appendix M1-A shows the historical and recent ground investigations carried out along and in the area of the pipeline route. All geotechnical investigation locations referred to in the report are indicated on this drawing.

A geophysical survey was carried out in Sruwaddacon Bay by Osiris Projects between June and August 2007. Drawing C10014-01 (see Appendix M1-A) determines the geological profile for the proposed tunnel through Sruwaddacon Bay. A more detailed summary of ground conditions encountered in each pipeline section is covered in the sections below.

3 GLENGAD (CHAINAGE 83.40 – 83.90)

Conventional pipelaying construction techniques will be used along this section as set out in Chapter 5 of the EIS. This involves laying the pipe in an open excavation approximately 2m deep but could be up to 5.5m deep approaching the landfall valve. Excavation may encounter the upper layer of bedrock due to the elevation of the landfall valve which has been lowered relative to surrounding ground level as a visual impact mitigation measure.

3.1 GROUND CONDITIONS

Boreholes in the area indicate made ground/topsoil underlain by loose to dense sands and medium to very dense gravels to depths of between about 3.85mbgl and 5.0mbgl. Weak to strong moderately weathered psammite bedrock with locally highly weathered zones exists below the overburden material. Excavations carried out in the area in 2008 / 2009 for the offshore pipeline landfall construction indicated sands/gravels/cobbles overlying rock. The soils varied in depth between about 2.5m and 3.0m with the overall excavation depth being between about 5.0m and 7.2m in depth.

3.2 GEOTECHNICAL CONSIDERATIONS

Considering that the gas pipeline and associated services will be laid in open cut trench, the main geotechnical concerns during construction would be dealing with groundwater and excavation stability. Once constructed and backfilled, the main geotechnical concern would be overall slope stability and pipe settlement. These are addressed below.

3.2.1 Groundwater

Groundwater was encountered at 3.8mbgl (medium inflow) during drilling in BH016A-07 rising to 3.1mbgl after 20 minutes. No groundwater strike was recorded in BH016-07. A standpipe was installed in BH016A-07 with a response zone of between 4.8mbgl to 24.0mbgl, which is located in bedrock. A standpipe was also installed in BH016-07 with a response zone of between 5.1mbgl to 24.4mbgl in bedrock. From intermittent groundwater readings in these boreholes groundwater varies between 2.5mbgl and 3.2mbgl in BH016A-07 and varies between 4.2mbgl and 4.5mbgl in BH016-07.

From excavations up to a depth of about 7.2mbgl carried out in late 2008 for the offshore pipeline works, no significant groundwater ingress was noted in the trench.

If groundwater is encountered during pipeline construction temporary pumping of excess water from the excavation may be required to minimise the risk of sidewall instability and to facilitate pipelaying. However, based on available information and recent construction experience in this area, ingress is not expected to be significant.

3.2.2 Stability During Construction

Laboratory testing of a sample from BH016A-07 indicated granular material. The likely range of effective angles of friction (ϕ') with depth for the soil encountered, based on SPT testing, are shown in Table 3.1 below.

A maximum trench excavation of approximately 5.5m depth in the granular material with side slopes of 1V:1.5H is required for laying the pipeline.

The stability analysis was carried out in accordance with Eurocode 7 (EC7) as follows:

1. IS EN 1997-1 Eurocode 7: Geotechnical Design - Part 1: General Rules (NSAI, 2005),
2. IS EN 1997-1 National Annex. Irish National Annex to Eurocode 7: Geotechnical Design - Part 1: General Rules (NSAI, 2007).

Local stability of the temporary trench excavation for the pipeline was examined using the SlopeW stability design computer package which implements the Method of Slices analysis technique. Traditionally an overall minimum factor of safety would be used using unfactored design parameters. Using EC7, the following inequality must be satisfied for ultimate limit state, namely:

$$E_d \leq R_d$$

where:

E_d is the design value of the effect of actions

R_d is the design value of the resistance to an action.

Stability can be expressed as the ratio of Rd/Ed, which is commonly referred to as the Factor of Safety (FOS EC7).

Using EC7, a FOS of 1 or greater is considered acceptable for stability design. Three design approaches are specified in EC7. In practice, the following is generally found to be the case for slope stability problems:

- DA1.C2 is usually more critical than DA1.C1
- DA3 usually gives the same result as DA1.C2
- It is generally not recommended to use DA2 for slope stability problems

Contour mapping for the area (COR-25-MDR0470DG0104P03, Appendix M1-A of the EIS) - indicates the maximum land slope adjacent to the pipeline along this section is about 10^{o}. The excavation cross section was therefore modelled using a worst-case slope of 10^{o} adjacent to the excavation. Figure 3.1 presents the SlopeW plot for the drained condition. A FOS of 1.011 was determined from the worst-case analysis indicating that the stability of the temporary pipeline excavation is satisfactory.

Depth (m)	SPT 'N' Range	Average SPT 'N' value	Angle of Friction Range ϕ'	Average Angle of Friction ϕ'
0 - 1	9 - 31	20	30 - 36	33
1 - 2	21	21	33	33
2 - 3	23	23	34	34
3 - 5	50	50	40	40
> 5	Bedrock			
Angle of Friction is based on a correlation with SPT (N) after Peck, Hanson and Thorburn (1974)				

Table 3.1 Variation of ϕ'

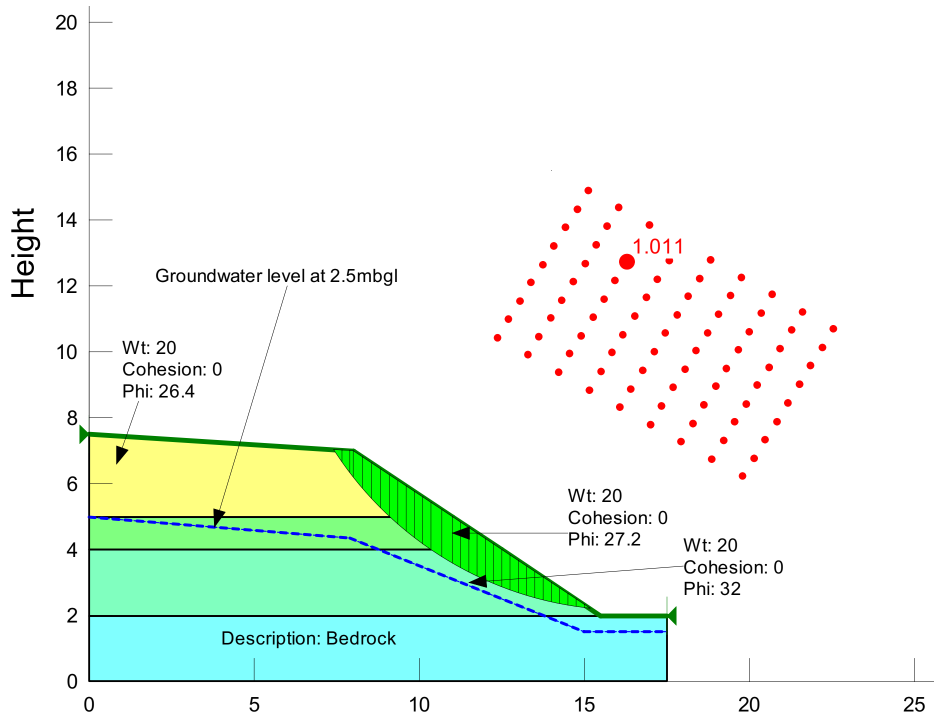

Figure 3.1 Slope Stability Analysis for Open Cut Excavation (drained)

3.2.3 Stability Post Construction

An infinite slope analysis (Skempton and DeLory, 1957) was carried out to determine a global factor of safety against sliding failure of a potential failure plane parallel to the surface of the slope above the excavation. A potential failure plane at 5.5mbgl was used. A Factor of Safety of 2.5 was determined for this section of the route based on a worst-case slope of 10°.

The potential impact on the integrity / stability of the proposed pipeline and the landfall valve installation from further landslides from Dooncarton Mountain has been assessed by AGEC. Their findings can be found in Appendix M2 of the EIS. The assessment was carried out by analysing run out distances of the failures that occurred in 2003 on the mountain. It was concluded that potential run off over an open slope would not reach the pipeline or landfall valve installation and if the flow were to become channelled in watercourses, satisfactory protection is inbuilt in the design consisting of a concrete slab above the pipe and deeper burial depth.

It is therefore highly unlikely that a debris flow will affect the pipeline however RPS have analysed this unlikely occurrence using finite element analysis for three different heights of debris flow; 1m, 2m and 3m. The analysis demonstrates that the movement determined at the locations of the buried pipe will not impact on the integrity of the pipeline.

3.2.4 Pipe Settlement

Due to the medium to very dense nature of the overburden material at the proposed excavation depths and considering that the increase in vertical stress is nominal, pipe settlement will be insignificant.

3.3 OTHER GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

Other geotechnical construction considerations would be excavatability, particularly in rock, and reuse of excavated material.

3.3.1 Excavatability

Figure 3.2 shows an assessment of the excavatability of rock encountered from BH016-07 and BH016A-07 based on Pettifer and Fookes, 1994. It appears that the rock encountered in BH016A-07 is slightly better quality than that in BH016-07. The assessment shows however, that hard to very hard ripping (medium weight hydraulic breaker) should be sufficient to remove the top layers of rock if encountered. Excavations carried out in the area in late 2008 for the offshore pipeline works indicated sands/gravels/cobbles to depths of between about 2.5m and 3.0m overlying rock. The overall excavation depth was between about 5.0m and 7.2m in depth. These excavations into rock were not problematic for conventional earthworks plant.

Cerchar abrasivity testing indicates that the rock is classed below medium abrasive to abrasive from depths of between 6.7 and 15.5mgl.

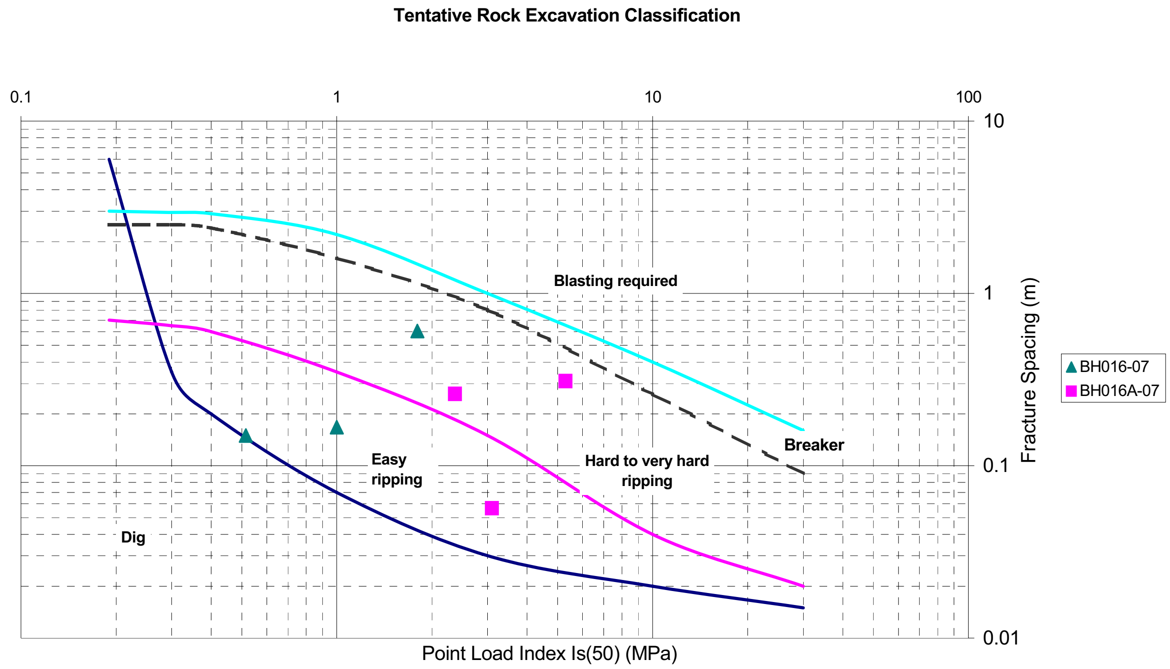

Figure 3.2 Rock Excavatability (Pettifer and Fookes, 1994)

3.3.2 Material Reuse

Site investigation gradings were split into those that met a Class 1 and those that met a Class 2 grading in terms of material classification designated within the Design Manual for Roads and Bridges Specification Series 600 Earthworks. A material with less than 15% fines (<15% passing 63μm sieve) and a maximum particle size of 125mm will be a Class 1 material. A Class 2 material is cohesive material assumed to have greater than 15% fines (>15% passing 63μm sieve) and a maximum particle size of 125mm.

Total No of PSDs	Classification
2	Class 1/Class 2

Table 3.2 Soil classification based on Gradings

The particle size distribution results indicate that the majority of superficial deposits for this section fall on the border of Class 1 and Class 2 and so with some processing will be suitable for re-use as a Class 1 or Class 2 general engineering fill.

From examination of the available laboratory test information, it appears that the moisture content of the material tested is typically between 4% and 33% with an average of about 13%. CBR testing on one sample indicated values in excess of 31% for a moisture content of about 11%, rendering the soils suitable for reuse as general engineering fill. It is also believed that the engineering properties of the soil will improve with general good handling.

3.4 OTHER CONSIDERATIONS

Consideration needs to be made for the surrounding environment at all times during construction. The area is within a Special Area of Conservation (SAC) and an Environmental Management Plan (EMP) will be implemented throughout the construction period.

4 SRUWADDACON BAY TUNNEL (GLENGAD TO AGHOOS) (CHAINAGE 83.90 – 88.77)

It is proposed to use trenchless tunnelling techniques through Sruwaddacon Bay between Glengad and Aghoos. It is proposed to tunnel from a starting pit at Aghoos to a reception pit at Glengad. The tunnel will be bored with a Tunnel Boring Machine (TBM) using segment lined tunnelling method. It is proposed that the tunnel will have an outer diameter of 4.2m and an internal diameter of 3.5m. Details on the tunnelling activities are provided in Chapter 5 of the EIS.

4.1 GROUND CONDITIONS

Geophysical survey data acquired to date indicate predominantly granular deposits in Sruwaddacon Bay up to depths of about 25m below bed level, becoming shallower at the bay edges where exposures can be seen at ground surface. The sediments are a mixture of reworked medium to fine marine sands throughout the central parts of the bay and mixed gravel deposits derived from glacial tills and weathered bedrock at the margins of the bay and in areas of stronger current.

Intrusive investigation techniques including boreholes, CPT's, televiewer surveying and trial pits have been carried out within Sruwaddacon Bay. See COR-25-MDR0470DG0201R07 in Appendix M1-A. The geotechnical information available within the bay is concentrated at either end of the bay. Further ground investigation works are proposed along the route alignment to facilitate detailed design of the tunnel boring machine.

Trial pits carried out close to the mouth of the bay (TPW1 to 6) encountered sand up to depths of 3.0mbgl overlying cobbles and boulders (weathered quartzite bedrock). The eastern shore consists of a small cliff below upwardly sloping agricultural grassland. Rockhead was encountered at or near the surface at the base of the small cliff. A borehole drilled at the base of the hillside (BH001-07) within the agricultural field showed topsoil (0.4m deep) over medium dense sand (0.8m deep) over a shallow dense gravel deposit (0.2m deep) overlying psammite bedrock. Televiewer surveying of the bedrock indicated highly weathered rock in the top 1.0m. Other highly weathered zones were evident at about 6.0mbgl and 7.3mbgl. Very close to medium spaced fracturing was evident throughout the borehole.

From boreholes BHF001-08 to BHF004-08 drilled at the mouth of Sruwaddacon Bay mainly sands with some gravel and silt layers overlying the bedrock was evident. Bedrock has been classed as psammite and quartz muscovite schist in one borehole. This may be due to foliation from faulting in the area. Bedrock was encountered between 9.3mbgl and 24.8mbgl. The shallower rock was encountered closer to the banks of the channel with the deeper overburden deposits towards the middle of the channel. This correlates with the geophysical survey data obtained in the area. CPT's were carried out to depths of between 7.6mbgl and 20.7mbgl. Deposits were classed mainly as medium to dense coarse sand and gravel with layers of silty sand.

From boreholes BHF005-08 to BHF015b-08 drilled at the upper end of Sruwaddacon Bay, mainly sands with some gravel and silt and clay layers were found overlying bedrock. Bedrock which was encountered between 3.3mbgl and 16.3mbgl was classed as variations of psammite, which have been altered due to metamorphism and foliations due to faulting in the area. The shallower rock was encountered closer to the banks of the channel with the deeper overburden deposits towards the middle of the channel. This correlates with the geophysical survey data obtained in the area. CPT's were carried out to depths of between 2.4mbgl and 10.6mbgl. Deposits were classed mainly as loose to dense coarse sand and gravel interspersed with silt and clay.

A borehole drilled close to the proposed tunnel launch pit in Aghoos encountered peat to a depth of 3.0m underlain by loose light brown slightly silty slightly gravely fine and medium sand (to a depth of 8.2mbgl) with occasional cobbles becoming medium dense at 4.5m depth and becoming dense at 6m. Weak moderately weathered Schist and very strong slightly weathered Psammite bedrock underlies the overburden.

4.2 GEOTECHNICAL CONSIDERATIONS

The main geotechnical considerations associated with tunnelling are outlined below.

4.2.1 Starting Pit / Reception Pit

The starting pit at Aghoos will be a sheet piled structure incorporating a concrete access ramp and a sealing body at the start of the tunnel. The concrete access ramp will be constructed using concrete piles, sheet piles and will be the main access to the tunnel for personnel, tunnel segments and all services associated with the TBM. The sealing body is a large block of low strength mortar and its purpose is to seal the annular space at the end of the tunnel. The starting pit will be approximately 10m deep at the tunnel entrance.

The reception pit at Glengad will be a sheet piled structure incorporating a sealing body at the end of the tunnel. Adjacent to the sealing body will be the part of the reception pit where the TBM is removed. The floor of this section will be lined with concrete to provide a seal. The reception pit will be approximately 7-10m deep.

4.2.1.1 Groundwater

Standing water was recorded between 0.3mbgl and 1.17mbgl at the location of the starting pit in Aghoos and between 0.8mbgl and 1.5mbgl at the location of the reception pit in Glengad.

Excavations are therefore likely to encounter groundwater during excavation of the launch and reception pits for the trenchless crossing. Temporary pumping of the water from the excavation will most likely be required. Pumped waters will be treated on-site prior to discharge via a settlement lagoon or disposal off-site (see Appendix M7).

4.2.1.2 Excavation Stability During Construction

The starting pit and reception pit excavations will be accommodated by sheet piles.

4.2.1.3 Excavatability

Figure 4.1 below shows an assessment of the excavatability of the rock encountered from boreholes drilled onshore and offshore close to the launch and reception pit locations based on Pettifer and Fookes, 1994. The assessment shows that hard to very hard ripping and a breaker (medium heavy hydraulic breaker) may be necessary to remove the rock.

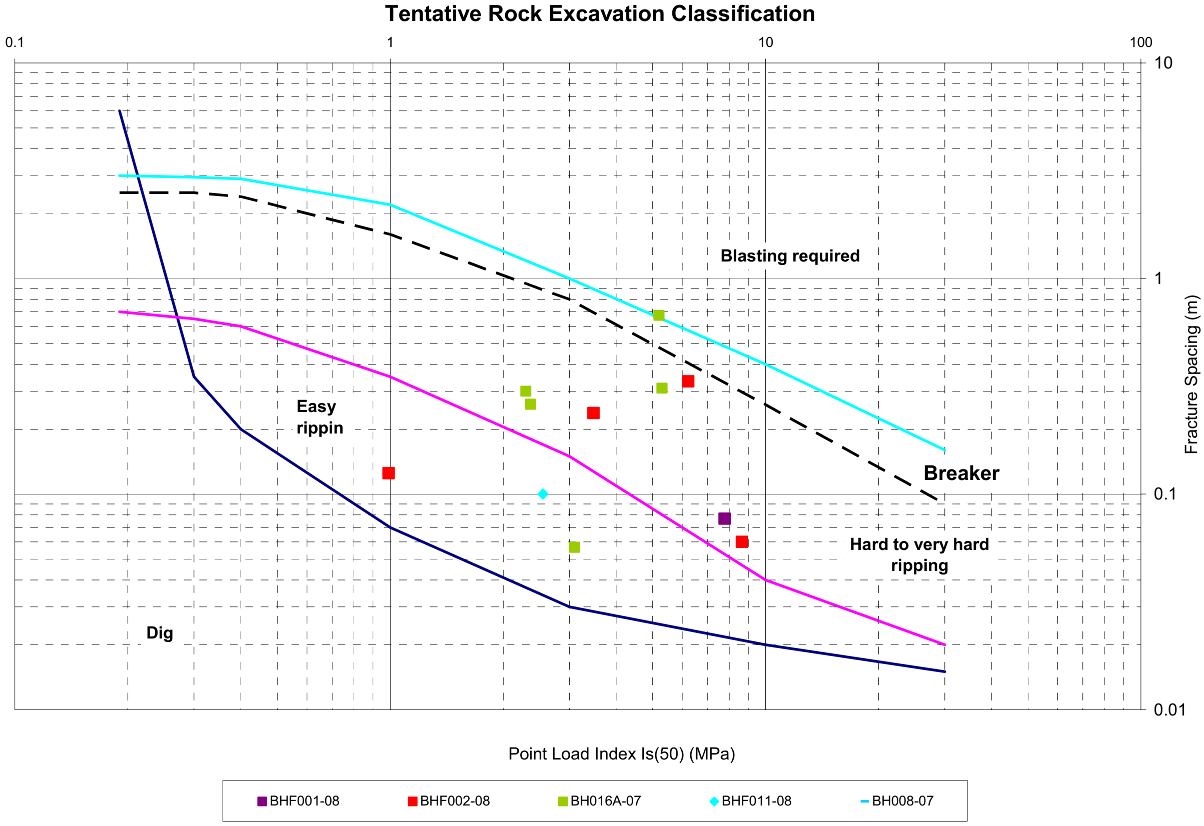

Figure 4.1 Rock Excavatability (Pettifer and Fookes, 1994)

4.2.1.4 Material Reuse

Particle size distribution grading analysis results from boreholes close to the launch and reception pit locations were split into those that met a Class 1 and those that met a Class 2 grading in terms of material classification designated within the Design Manual for Roads and Bridges Specification Series 600 Earthworks and can be seen in Table 4.1.

	Total No of PSDs	Total No Class 1	Total No Class 2
Onshore Boreholes	5	1	4

Table 4.1 Soil classification based on Gradings

Up to 3.8m of peat with an average of approximately 1.6m exists at the Aghoos location. The top layer of peat along with any other peat required for reinstatement will be stockpiled at the site. The remaining surplus peat is expected to be removed off site and transported to the Srahmore peat deposition facility (see Volume 3 of the EIS).

The particle size distribution results of other material tested indicate that it would be suitable for reuse as a Class 2 material.

Excavated rock may be reusable as a Class 6 material after some basic processing (e.g. screening).

4.2.2 Tunnel (Aghoos to Glengad)

The following presents geotechnical considerations applicable to the proposed tunnel from Aghoos to Glengad. This should be read in conjunction with Chapter 5 and Appendix M4 (Geotechnical Risk register).

4.2.2.1 TBM Cutting Head

The TBM cutting head will be designed to deal with the variable ground conditions likely to be encountered, namely overburden and/or bedrock deposits (Refer to Section 5.1) during the tunnelling works.

Face stability calculations based on the ground conditions expected will be undertaken during the tunnel detailed design. Transitions between soil / rock interfaces will be encountered and appropriate measures / protocols will be implemented to ensure that face stability and alignment are maintained and to mitigate against this risk.

Based on the available ground investigation information and the large diameter tunnel TBM selected, it is not anticipated that any obstacles will be encountered that will inhibit progress of the TBM. If such an obstacle were to be encountered, it will be dealt with from within the tunnel. The final, very unlikely, contingency measure would be to construct an intervention pit to gain access to the problem area from the surface. An intervention pit would be constructed using a sheet piled / coffer dam excavation.

Experience on similar tunnelling projects worldwide shows that a contingency intervention pit is only ever required where there are exceptional circumstances. Therefore, this is considered to be a remote possibility for the proposed tunnel.

4.2.2.2 Settlement

The annular space surrounding the tunnel as each group of segments is installed will be grouted with cement. This feature of the segment lined tunnelling process will mitigate against any potential for settlement. This risk of any settlement associated with the tunnelling works is therefore considered to be extremely low.

4.2.2.3 Bentonite

Bentonite will be used to provide many functions during tunnel advancement; to provide face support in non-cohesive soil, reduce the wear while tunnelling in rock and to assist viscosity in transporting excavated materials from the tunnel face to the surface where they can be separated. Bentonite will be recycled once separated from the tunnel cuttings and returned to the tunnel face to continue the tunnelling process.

The appropriate bentonite product must be selected to ensure its effectiveness is maintained within saline conditions likely to be encountered. Polymers may be used to improve the effectiveness of bentonite and to reduce quantities consumed.

4.2.2.4 Excavated Material Reuse

Site quality management techniques will be implemented to ensure that excavated volumes are closely monitored relative to tunnel advancement to ensure that potential for over-mining is minimised.

It is anticipated that the majority of the granular material excavated from the tunnel trajectory will be suitable for re-use subject to geotechnical classification and environmental verification testing.

Where finer clay / silt material is encountered, an additional centrifuge system will be required to supplement separation of excavated materials and recycling of slurry. It is unlikely that this finer material will be reusable on site. A Materials Management Plan is included in Appendix S4 of the EIS.

4.2.2.5 Vibration from Tunnelling

Vibration modelling has been undertaken for the tunnelling works and is detailed in Appendix H3 of the EIS.

The predicted highest root mean square (rms) vibration velocity in the ground at locations representative of the nearest residential property to the tunnelling in Aghoos is predicted to be in the 16 Hz 1/3 octave band and reaches a value of 1.8 micrometres per second. In terms of Peak Particle Velocity (PPV), taking account of the vibration throughout the 10 Hz to 100 Hz range, this corresponds to a value in the range of 0.01 to 0.02 mm per second. This is a level that is considerably below the threshold for human perception of vibration. It is no more than a fifth of the level of vibration generated by a bus or truck travelling over a bump as observed at 15m (see Appendix H2).

Appendix H2 of the EIS considers relevant standards which provide guidance on the control of noise and vibration on construction and open sites. This shows that modelled vibration levels at the nearest buildings are significantly lower than the thresholds for the possibility of minor and major damage to buildings.

The 2003 peat failures on Dooncarton Mountain are approximately 850m away from the alignment of the proposed tunnel. To put the potential vibration effects from the proposed tunnelling works into a local context, the L1202 public road is closer to Dooncarton Mountain and local dwellings than the tunnelling works. Local traffic is considered to be a more significant source of vibration at these locations. Therefore there will not be any significant negative impact from tunnelling vibration.

4.3 OTHER CONSIDERATIONS

A geophysical survey of Sruwaddacon Bay identified features running across the bay, possibly igneous dykes or power cables, in a number of locations. Enquiries with utilities companies have indicated no services in the bay. If these are igneous dykes, the Corrib TBM Specification includes the requirement for dealing with harder rock excavation when designing the TBM. The presence of the harder rock would only slow down the rate of tunnelling at that location. The TBM is designed to deal with the harder material therefore it will not cause any negative impacts to the tunnelling process.

Consideration needs to be made for the surrounding environment at all times during construction. The area is within a Special Area of Conservation (SAC) and an Environmental Management Plan (EMP) will be implemented throughout the construction period.

5 FURTHER GROUND INVESTIGATIONS

The geophysical and geotechnical data that is now available provides sufficient basis for the selection of the proposed construction method (segment lined tunnelling). The size of the TBM required to construct the proposed tunnel, and the inherent flexibility of these machines, will ensure that all ground conditions likely to be encountered can be managed.

Further ground investigations will be required along the pipeline route to verify existing geotechnical data. This will include boreholes (with extraction of samples), in-situ testing and laboratory testing of overburden and bedrock material. Information gathered during these works will serve to verify existing data with respect to ground conditions and ranges of values for various parameters. The rockhead profile will also be verified. The full set of geotechnical data gathered will be made available to the tunnelling contractor. This will enable the tunnelling contractor to optimise the detailed design of the tunnel and the head of the TBM.

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