Corrib natural gas

Corrib Onshore Pipeline

Environmental Impact Statement

Volume 2 of 3

Book 4 of 5 - Appendix M (M1)

RPS

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Appendix M

Soils and Geology

M1: Geotechnical Assessment of the Non-Peat areas along the Proposed Route of the Corrib Onshore Pipeline

Appendix M1

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

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

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AGEC 2004 Final Report on the Derrybrien Windfarm post landslide site appraisal

AGEC 2004 Final Report on onshore gas pipeline Glenamoy River Estuary to Corrib Gas Terminal site investigation factual report;

AGEC 2005 Final Report on Onshore Gas Pipeline Geotechnical Interpretative Report;

Geotechnical & Environmental Services 2007 Corrib Onshore Pipeline Route Selection Geotechnical Ground Investigation Factual Report.

Irish Drilling Limited 2008 Site Investigations Factual Report

1 INTRODUCTION

This report examines the geotechnical issues relating to the non peat areas along the route of the proposed Corrib Onshore Pipeline as indicated on the Route Alignment Sheets provided in Appendix A(1) of the EIS.

A number of ground investigations have been carried out to date along the route as shown on the location plan in Appendix (M)1-A (COR-25-MDR0470DG0201R07). Osiris Projects carried out a geophysical investigation in Sruwaddacon Bay in 2007. Further investigations are required to confirm ground conditions at certain locations including the launch pits for the tunnelling process under Sruwaddacon Bay. However, the quantity of data and information that is available provides sufficient level of confidence that the pipeline can be safely constructed and operated along the proposed route. This report outlines the specific geotechnical challenges in each area outside of the peat areas and proposes appropriate construction methodologies to meet these challenges. Reports on the Stability of Peat and the Stone Road Construction are included in Appendix (M)2 and Appendix (M)3 of the EIS.

A Geotechnical Risk Register is provided in Appendix (M)4 of the EIS. The register has been compiled to show the degree of risk attached to various elements of the proposed pipeline construction and operation. The purpose of the Geotechnical Risk Register is to provide an outline description of the hazards, identify the likely causes, describe the potential impact of the hazard and identify the design and construction controls to be implemented in order to minimise the geotechnical risk.

The Geotechnical Risk Register will be used actively throughout the project and will be up-dated to reflect additional data and experience as it is gained.

2 SOURCES OF INFORMATION

The following sources of information were used in the compilation of this report.

a) BKS, Aerial Photography, 2006 & 2008

b) Applied Ground Engineering Consultants, July 2004. Final Report on Onshore Gas Pipeline – Glenamoy River Estuary to Corrib Gas Terminal – Site Investigation Factual Report;

c) Applied Ground Engineering Consultants, Sept 2004. Final Report on Onshore Gas Pipeline – Glenamoy River Estuary Crossing – Site Investigation Factual Report;

d) Geological Survey of Ireland, 1992. Geology of North Mayo. Sheet 6;

e) Geological Survey of Ireland, 2004. Geology of Galway Bay. A Geological Description to accompany the bedrock geology 1: 100,000 scale Map Series, Sheet 6, North Mayo;

f) Geotechnical and Environmental Services, 2007. Corrib Onshore Pipeline Onshore pipeline Route Selection Geotechnical Ground Investigation Factual Report;

g) Irish Drilling Limited, 2002. Bellanaboy Bridge Co. Mayo Factual Site Investigation;

h) Irish Drilling Limited, 2008. Corrib Foreshore Pipeline Site Investigation Factual Report;

i) Ordnance Survey of Ireland, Sheet 89;

j) Osiris Projects, 2007. Corrib Gas Pipeline Landfall Sruwaddacon Bay Geophysical Survey;

k) Tobin, 2003. Report on the Landslides at Dooncarton, Glengad, Barnachuille and Pollathomais, County Mayo.

This report should be read in conjunction with the Corrib Onshore Pipeline EIS.

3 ROUTE DESCRIPTION

The proposed route is represented on the Route Alignment Sheets presented in Appendix A(1) of the EIS. For this report, the route has been divided into sections as outlined below between approximate chainages. Each section is described briefly below and in greater geotechnical detail in the following sections of this report.

Glengad (Chainage 83.40 – 83.91)

This section is within relatively flat improved agricultural grassland at about 10mOD (Ordnance Datum). The proposed landfall valve location is at the west of this section and a proposed launch/reception pit for the tunnelling process 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, (Lower Crossing, Glengad to Rossport) (Chainage 83.91 – 84.51)

This section of the pipeline will cross beneath the mouth of Sruwaddacon Bay using a micro-tunnelling trenchless technique. The western shore of this crossing is an intertidal, flat, low-lying sandy foreshore area. The eastern shore is upwardly sloping agricultural grassland. A proposed launch/reception pit for the trenchless crossing is located within the agricultural grassland.

Rossport landfall to Rossport commonage (Chainage 84.51 – 85.99)

This section is within agricultural grassland. The land rises from about 14mOD to 20mOD for the first 200m approximately, and then falls again to about 10mOD. The pipeline route then runs along sloping agricultural land at about 10mOD for approximately 800m where it veers to the northeast climbing to about 25mOD where it crosses a public road (L52453-25) before entering Rossport commonage.

Rossport Commonage (Chainage 85.99 – 88.52)

Rossport commonage consists of areas of cutover, intact and eroding blanket bog designated as non Special Area of Conservation (SAC) and SAC further to the east. Extensive turf extraction has occurred especially at the edges of the bog and adjacent to access tracks and roads. Peat to depths of between about 0.25m and 5m are evident along the pipeline route.

As this section contains deeper peat it is not covered further in this report.

Crossing of Sruwaddacon Bay (Upper Crossing) (Chainage 88.52 – 89.55)

This section consists of about a 1km trenchless crossing of Sruwaddacon Bay using a micro-tunnelling trenchless technique. The northern shore of this crossing is located close to the boundary of Rossport commonage and consists of peats to depths of between about 0.5m and 2m. Turf cutting has occurred extensively in this area. The southern shore consists of peat to a depth of about 3m according to a borehole located to the west of the landing point.

South of Sruwaddacon Bay to Terminal (Chainage 89.55 – 92.56)

The section of the route south of Sruwaddacon Bay to the forested area at about chainage 90.40 crosses peat land, with peat to depths of between about 1m and 4m, and a small valley associated with a stream entering Sruwaddacon Bay.

From the forested section to the Terminal the route runs south east crossing commercially forested land before changing direction where it crosses a public road (L1202). The pipeline route changes direction again to run south to south west, falling towards a small valley and rising again to the Terminal site. Peat probes indicate peat depths of between about 2m and 5m in this area.

As this section contains deeper peats it is not covered further in this report.

3.1 SITE INVESTIGATIONS

Drawing COR25MDR0470DG0201R07 in Appendix (M)1-A shows the historical and recent ground investigations carried out as part of preparation work for the original pipeline route and also during more recent work on the modified 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 for the Developer between June and August, 2007. Osiris drawings C7009a-08 and C7009a-09 in Appendix (M)1-A illustrate the geological profile at the proposed crossings of Sruwaddacon Bay.

As constructed boreholes in Sruwaddacon Bay that were carried out during 2008 are indicated on drawings COR25MDR0470DG0082 – COR25MDR0470DG 0084 (see Appendix (M)1-A).

A more detailed summary of ground conditions encountered in each pipeline section can be found in the following sections, based on ground investigations in the area.

4 GLENGAD (CHAINAGE 83.40 – 83.91)

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. In the vicinity of the landfall valve, excavations may be up to 5.5m deep. Excavation may encounter the upper layer of bedrock due to the elevation of the landfall valve.

4.1 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 late 2008 for the offshore pipeline works 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.

4.2 GEOTECHNICAL CONSIDERATIONS

Considering that the pipe will be laid in open cut, 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 in the following sections.

4.2.1 Groundwater

Groundwater was struck at 3.8mbgl (medium inflow) during drilling in BH016A-07 rising to 3.1mbgl after 20 mins. 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 carried out in late 2008, up to a depth of about 7.2mbgl, 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, however ingress is not expected to be significant.

4.2.2 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 4.1 below.

A maximum trench excavation of approximately 5.5m in the granular material with side slopes of 1V:1.5H is required for laying the pipeline. Local stability of the temporary excavation for the pipeline was examined using the SlopeW stability design computer package. A factor of safety of 1.25 is acceptable for the temporary excavation in accordance with Eurocode 7.

Contour mapping for the area (see Appendix M(1)-A) - indicates the maximum land slope adjacent to the pipeline along this section is about 10°. The excavation cross section was therefore modelled using a worst-case slope of 10° adjacent to the excavation. Figure 4.1 presents the SlopeW plot for the drained condition. A FOS of 1.28 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 4.1 Variation of ϕ'

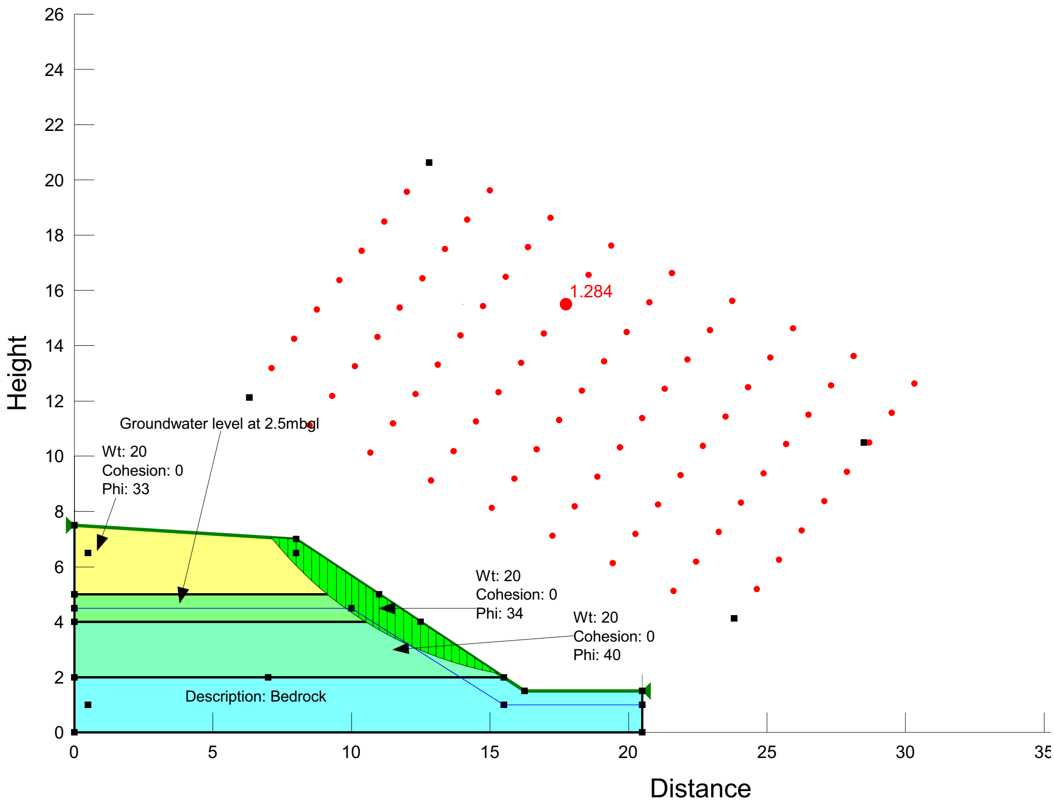

Figure 4.1 Slope Stability Analysis for Open Cut Excavation (drained)

4.2.3 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°.

A potential debris flow failure due to construction activities or other causes from Dooncarton Hill towards the pipeline has been assessed empirically from information gained from Tobin (2003) for Mayo County Council regarding the landslides at Dooncarton, Glengad, Barnachuille and Pollathomais in September 2003. These landslides occurred following a 1 in 100 year rainfall event (i.e. the event is expected to occur only once in 100 years) and were initiated on the steep (slope angle greater than 20^{o}) upper slopes of the mountains. The debris once mobilised behaved as a fluid due to the amount of entrained water. Drainage channels in the area came into play as conduits for the debris material. The topography of the area also affected the flow path of the debris. A pre-existing berm between commonage lands and privately owned properties on the higher ground above the LP1202 road absorbed a significant portion of the energy of the flow. No debris material reached the area where the proposed pipeline is to be laid. This area is on the relatively flat lying coastal strip. Since the failures of September 2003, Mayo County Council, with the assistance of the Office of Public Works have reinstated the berm between the commonage lands and the privately owned properties, improved/repaired drainage systems and installed landslide barriers. These measures will further reduce the prospect of a landslide reaching Glengad headland where the pipeline is proposed.

The effect of an unlikely debris flow failure from Doooncarton on the pipeline has been assessed for three different heights of debris failure; 1m, 2m and 3m. The analysis demonstrates that the movement determined at the locations of the buried pipe will not jeopardise the integrity of the pipeline.

4.2.4 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.

4.3 OTHER GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

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

4.3.1 Excavatability

Figure 4.2 shows an assessment of the ripability 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.

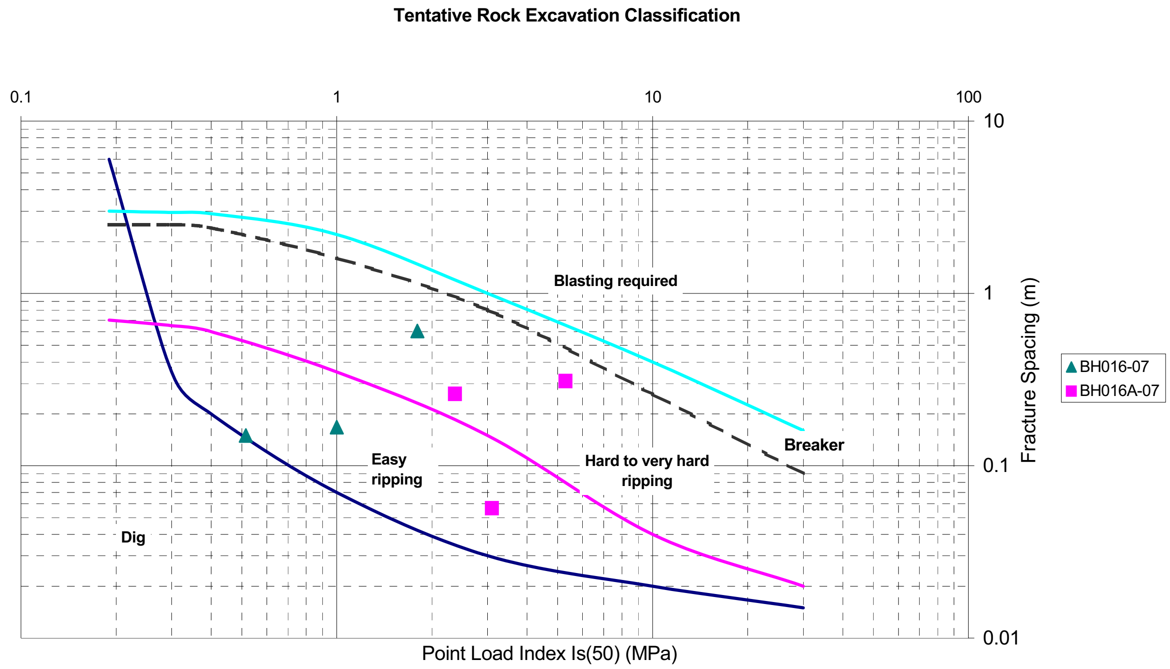

Figure 4.2 Rock Excavatability (Pettifer and Fookes, 1994)

4.3.2 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. A Class 2 is cohesive material assumed to have greater than 15% fines (>15% passing 63µm sieve) and a maximum particle size of 125mm.

Section	Total No of PSDs	Classification
Section 1	2	Class 1/Class 2

Table 4.2 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 re-use as general engineering fill. It is also believed that the engineering properties of the soil will improve with general handling.

4.4 OTHER CONSIDERATIONS

Consideration should be taken 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 CROSSING OF SRUWADDACON BAY, (LOWER CROSSING, GLENGAD TO ROSSPORT) (CHAINAGE 83.91 – 84.51)

It is proposed to use micro-tunnelling trenchless technology to cross the mouth of Sruwaddacon bay. It is expected that the launch pit for this trenchless crossing will be within improved agricultural grassland on the western side of the crossing. The reception pit will be in sloping agricultural grassland on the eastern side of the crossing. However, it is possible that the successful contractor may decide to launch from the eastern side.

The assessment of excavation stability for the pits assumes that they are both are approximately the same depth. Therefore, the direction of micro-tunnelling is not relevant in terms of the geotechnical assessment of this section of the pipeline route.

A cofferdam located in the bay may be required if obstructions are encountered during tunnelling. This will be confirmed during construction.

5.1 GROUND CONDITIONS

Trial pits in the flat low-lying sandy foreshore area (TPW-1 and TPW-2) indicate sand up to depths of 3.0mbgl, however, the trial pits were terminated due to instability of the excavations and running sands. Trial pits carried out in the estuary (TPW-3 to TPW-6) indicated sand to a depth of 3.0mbgl (depth of deposit unproven) and sand overlying cobbles and boulders (weathered quartzite bedrock) closer to the eastern foreshore. 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 indicated 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 shallower highly weathered zones were evident at about 6.0mbgl and 7.3mbgl. Very close to medium spaced fracturing is evident throughout the borehole.

Geophysical surveying indicated granular deposits consisting of mainly gravels with occasional cobbles and small boulders to depths of about 25.0m in the middle of the channel overlying bedrock. The rock slopes sharply at the sides of the channel to be exposed at the surface on the eastern shore.

Four boreholes were drilled within the lower bay crossing either side of the proposed route. Cone penetration testing (CPT) was also carried out at each location. Televiewer surveying was carried out in two of the boreholes.

Mainly sands with some gravel and silt layers were found overlying the bedrock. Bedrock has been classed as psammite and quartz muscovite schist in one borehole that 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.

5.2 GEOTECHNICAL CONSIDERATIONS

Considering that this section of pipe will be laid using trenchless technology beneath the bay mouth, between a launch pit and reception pit constructed on land, other than the tunnelling considerations, the main geotechnical concerns during construction would be dealing with groundwater in the pit excavations and the stability of same. Once construction is completed and the area is reinstated, the main geotechnical concern would be long-term stability of the tunnel bore. These are addressed in the following sections.

5.2.1 Groundwater

Groundwater was struck at 1.1mbgl (medium inflow) in BH001-07, drilled on the eastern shore. A standpipe was installed in this borehole with a response zone of between 1.3mbgl to 20.0mbgl, which is located in bedrock. A groundwater monitoring device was installed in the borehole and indicated an average water depth of about 2.7mbgl with a rise of about 1.6m and a fall of about 0.9m due to tidal influence. Excavations may occur below groundwater during excavation of the access pit for the trenchless crossing. Temporary pumping of the water from the excavation may be required.

5.2.2 Stability During Construction

Stability of soils and weak rock within the micro-tunnel will be aided by the use of bentonite. The risk of collapse during micro-tunnelling is reduced by the use of a sleeve pipe.

If an obstacle is encountered and the cutter-head fails to progress efficiently, it may be necessary to sink a shaft directly over the obstacle, or alternatively dig a pit, in order to remove it.

The entry and exit points of the trenchless operation may be several metres deep. The excavation can be accommodated by temporary supports or battered slopes where appropriate. For example a battered excavation with side slopes of 1V:1.5H can be safely achieved in the ground expected.

The pipe thrusters will be anchored into the ground in the launch pit and will be designed to take the maximum thrust expected. If shallow rock is assumed at the location, the pipe thrusters will be anchored into the rock and will be designed at detailed design stage.

5.2.3 Stability Post Construction

The long-term stability of the tunnel bore will be aided by the presence of a permanent sleeve pipe that is installed during the tunnelling operation (see EIS, Chapter 5). The sleeve pipe will be installed at a sufficient depth below the channel to avoid exposure through erosion of the bed.

5.3 OTHER GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

Other geotechnical construction considerations would be excavatability, particularly in rock at the launch/reception pit locations, and reuse of excavated material.

5.3.1 Excavatability

Figure 5.1 below shows an assessment of the ripability of the rock encountered on the eastern shore (BH001-07) and within the bay (BHF001-08 to BHF004-08) 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, if required.

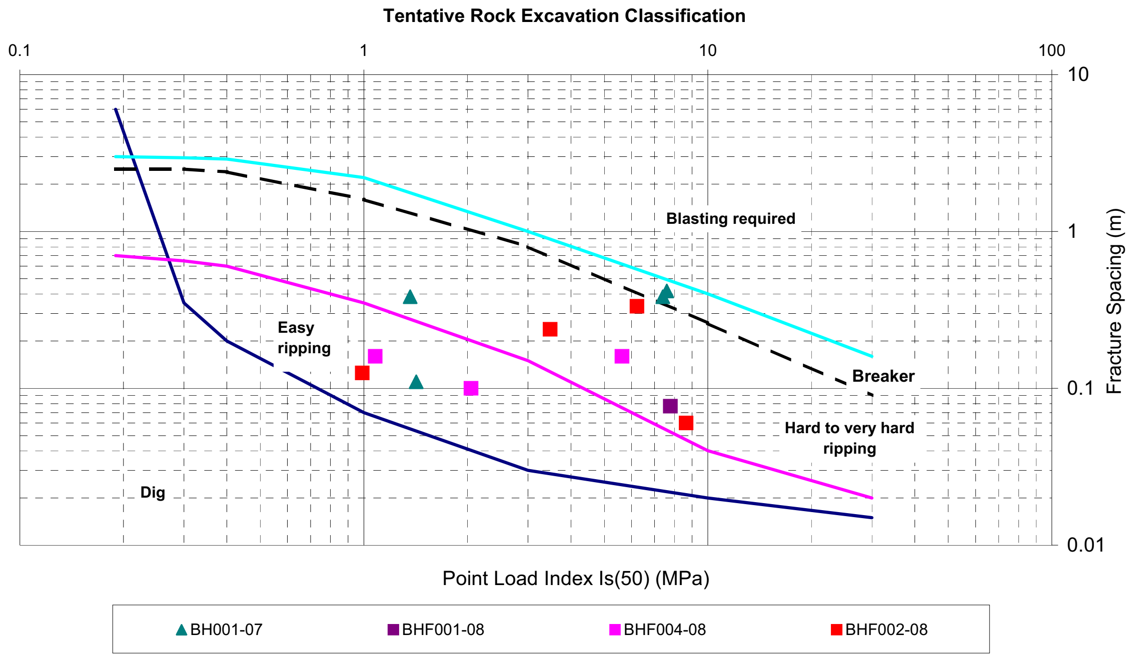

Figure 5.1 Rock Excavatability (Pettifer and Fookes, 1994)

5.3.2 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 and can be seen in Table 5.1.

Section		Total No of PSDs	Total No Class 1	Total No Class 2
Section 2	Onshore Boreholes	1	0	1
	Foreshore Boreholes	16	16	0

Table 5.1 Soil classification based on Gradings

The particle size distribution results indicate that the superficial deposits excavated from the eastern shore along this section will be suitable for re-use as a Class 2 material.

From examination of the available lab test information a moisture content of 28% was determined for the very silty gravelly fine to coarse sand material tested. It is believed that the engineering properties of the soil will improve with general good handling.

Bentonite will be used in the trenchless operation for the following reasons; lubrication and cooling, spoil removal, stability of bore and grouting. The material excavated from the operations will be mixed with bentonite during excavation. This material will be separated from the bentonite and disposed of at a licensed disposal/ recovery facility. Some bentonite will be lost with excavated spoil, but most of the bentonite will be recycled in the process. Where material can be recovered it may be re-used as a Class 1 material.

5.4 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.

Consideration should be taken 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.

6 ROSSPORT LANDFALL TO ROSSPORT COMMONAGE (CHAINAGE 84.51 – 85.99)

Conventional pipelaying construction techniques will be used along this section. This involves laying the pipe in an open excavation approximately 2m deep but could be up to 2.5m deep approaching the road crossing.

6.1 GROUND CONDITIONS

The general ground conditions expected along this section of the pipeline route are as follows; topsoil from 0.0mbgl to 0.4mbgl overlying medium dense to very dense sand and/or gravel deposits from 0.4mbgl to 8.1mbgl. A very stiff clay layer exists in BH002-07 from 4.8mbgl to 10.1mbgl. The other boreholes along this section have shallow clay or silt layers underlying the topsoil. Psammite bedrock exists below the overburden with a discontinuous band of pelitic schist observed in BH002-07 and BH003-07 between 12.5mbgl and 18.8mbgl.

6.2 GEOTECHNICAL CONSIDERATIONS

Considering that the pipe will be laid in open cut, the main geotechnical concerns during construction is 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 in the following sections.

6.2.1 Groundwater

Groundwater was struck at between 2.4mbgl and 3.5mbgl (slow to medium inflow) during drilling of boreholes along this section of the pipeline route. Standpipes were installed in boreholes as shown in Table 6.1 with response zones as indicated, located mainly in the bedrock.

Groundwater monitoring devices installed in three no. boreholes indicated groundwater responses as shown below in Table 6.2.

Groundwater may be struck during excavation of the pipeline trench. Temporary pumping of the water from the excavation may therefore be required.

Borehole	Date	Water Strike (mbgl)	Standpipe Response Zone (mbgl)
BH001A-07	-	-	5.0 to 39.9
BH002-07	01/10/07	3.5 – slow flow	10.5 to 19.5
BH003-07	27/09/07	2.4 – medium flow	6.6 to 19.9
BH004-07	-	-	4.5 to 23.0

Table 6.1 Groundwater Standpipe Details

Borehole	Average water depth in BH (mbgl)	Significant tidal influence (m)	
		Plus	Minus
BH001A-07	9.0	-	-
BH003-07	1.9	1.1	0.4
BH004-07	3.6	1.3	0.6

Table 6.2 Groundwater Responses

6.2.2 Stability During Construction

Laboratory testing of samples from BH002-07 and BH003-07 indicated granular material. The likely range of ϕ' values with depth based on SPT testing are shown in Table 6.3 below:

Depth (m)	SPT 'N' Range	Average SPT 'N' value	Angle of Friction Range ϕ'	Average Angle of Friction ϕ'
0 - 1	20 - 50	35	33 - 41	37 limited to 36*
1 - 2	6 - 50	30	39 - 41	36
2 - 5	19 - 50	42	33 - 41	39 limited to 36*
Angle of Friction is based on a correlation with SPT (N) after Peck, Hanson and Thorburn (1974)				
*ϕ' has been limited to a more conservative / realistic value				

Table 6.3 Variation of ϕ'

A 2.5m deep excavation with side slopes of 1V:1.5H is assumed in the granular material. Local stability of the excavation for the pipeline was examined using the SlopeW stability design computer package. This section was analysed under drained conditions only. Contour mapping of the area (see, Appendix M(1)-A) indicated a slope of about 7° behind the excavation. Figure 6.1 presents the SlopeW plot for the drained condition. From the results obtained, FOS 1.33, it is evident that stability for the temporary pipeline excavation is satisfactory.

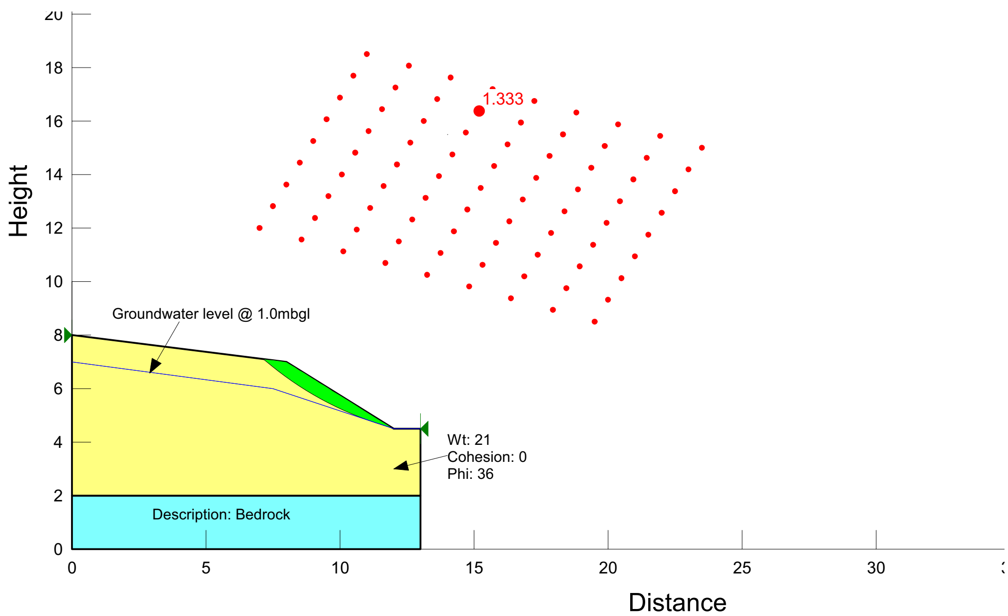

Figure 6.1 Slope Stability Analysis for Open Cut Excavation (drained)

6.2.3 Stability Post Construction

An infinite slope analysis (Skempton and DeLory, 1957) was carried out to determine a factor of safety against sliding failure along the pipeline route. A Factor of Safety of 4.0 was determined for this section of the route based on a slope of 7^{o} adjacent to the pipeline excavation.

6.2.4 Pipe Settlement

A pipe bearing test (BT1) was carried out along this section of the pipeline route in 2002 in a trench 2.1m deep. Settlements up to a maximum of 6mm were recorded. Due to the medium to dense nature of the overburden material at the proposed excavation depths pipe movements should be insignificant as indicated by the pipe bearing test. The pipe will also be bedded in an appropriately compacted material, which will mitigate against excess movements.

6.3 OTHER GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

Other geotechnical construction considerations would be excavatability of the overburden material and its reuse potential.

6.3.1 Excavatability

The overburden material along this section is mainly medium dense to very dense sand and/or gravel and/or very soft to very stiff clay with SPT's ranging from 6 to refusal, with the majority of values greater than 27. Some cobbles and boulders are noted in the borehole logs. Despite the presence of medium dense to very dense sands and gravels, it is not envisaged that materials will be problematic for conventional earthworks plant.

6.3.2 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 and can be seen in Table 6.4.

Section	Total No of PSDs	Total No Class 1	Total No Class 2
Section 3	5	0	5

Table 6.4 Soil classification based on Gradings

The particle size distribution results indicate that the superficial deposits excavated from this section of the pipeline route will be suitable for re-use as a Class 2 material.

From examination of the available lab test information, it appears that the moisture content of the material tested is typically between 2% and 15% with an average of about 10%. It is believed that the engineering properties of the soil will improve with general good handling.

6.4 OTHER CONSIDERATIONS

The presence of underground services such as ESB, water pipes etc., in the vicinity of the excavations should be confirmed prior to any construction works commencing. These may be of most concern near road crossings. The location of any overhead power lines should also be noted prior to construction and measures taken to avoid any accidental damage.

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

7 CROSSING OF SRUWADDACON BAY (UPPER CROSSING) (CHAINAGE 88.52 – 89.55)

It is proposed to use micro-tunnelling trenchless technology to cross the bay at this location. A proposed launch/reception pit may be located close to the boundary of Rossport commonage. Another launch pit may be located in the peat area south of the bay. A cofferdam in the centre of the crossing, in the bay, may be required if an obstruction is encountered during tunnelling. This will be confirmed during construction.

7.1 GROUND CONDITIONS

Geophysical surveying indicated probable interbedded sand and gravel with occasional cobbles and small boulders to depths of about 10.0m in the deepest part of the channel rising to about 2.0m at the edges of the bay.

Ten boreholes were drilled within the bay crossing either side of the proposed route. Cone penetration testing (CPT) was also carried out at each location.

Mainly sands with some gravel and silt and clay layers were found overlying the bedrock. Bedrock was classed as variations of psammite, which have been altered due to metamorphism and foliations due to faulting in the area. Bedrock was encountered between 3.3mbgl and 16.3mbgl. 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 on the southern shore indicated peat to a depth of 3.0m underlain by medium to dense sand, loose gravel and stiff clay deposits to a depth of 9.3mbgl. Very weak psammite rock underlies the overburden deposits with a thin band of weak to moderately strong pelite between 26.0 and 26.7mbgl. The northern shore of this crossing in located close to the boundary of Rossport commonage. Turf cutting has occurred extensively in this area and peat depths of between 0.5m and 2m have been found.

7.2 GEOTECHNICAL CONSIDERATIONS

Considering that this section of pipe will be laid using trenchless technology beneath the bay, between a launch pit and reception pit constructed on land, other than the tunnelling considerations, the main geotechnical concerns during construction would be dealing with groundwater in the pit excavations and the stability of same. Once construction is completed and the area is reinstated, the main geotechnical concern would be long-term stability of the tunnel bore. These are addressed in the following sections.

7.2.1 Groundwater

Groundwater was struck at ground level in the borehole on the southern shore. A standpipe with a response zone of between 6.7mbgl to 27.7mbgl was installed. Monitoring of the groundwater indicated an average level of 2.1mbgl with a small tidal influence.

7.2.2 Stability During Construction

Stability of soils and weak rock within the micro-tunnel will be aided by the use of bentonite. The risk of collapse during micro-tunnelling is reduced by the use of a sleeve pipe.

If an obstacle is encountered and the cutter-head fails to progress efficiently, it may be necessary to sink a shaft directly over the obstacle, or alternatively dig a pit, in order to remove it.

The proposed launch pit in the peat area south of the bay may be a sheet-piled pit with a retained height of about 4.0m. The likely range of ϕ' and c_u values with depth, based on SPT testing, and typical values for peats are shown in Table 7.1 below:

Depth (m)	SPT 'N' value	Angle of Friction ϕ'	c_u (kN/m^2) with f_1 = 4.8 where SPT available
0 - 3	-	15	10
3 - 5	14	31	67
5 - 6	8	29	38
6 - 7	39	38	130
7 - 8	18	33	86
8 - 9	40	39	130
> 9	Bedrock		
Angle of Friction is based on a correlation with SPT (N) after Peck, Hanson and Thorburn (1974)			
$c_u = f_1 \times N$ (kN/m^2)			

Table 7.1 Variation of ϕ' and c_u

A braced sheet-piled excavation may be suitable with a nominal embedment depth. Detailed design of the sheet-piles will be carried out prior to construction.

The pipe thrusters will be anchored into the ground in the launch pit and will be designed to take the maximum thrust expected. If shallow rock is assumed at the location, the pipe thrusters will be anchored into the rock and will be designed at detailed design stage.

A similar pit will be constructed on the northern side of this crossing in Rossport Commonage. Peat depths of between about 0.5m and 1.5m have been recorded here. This pit may be a sheet-piled pit with a retained height of about 4.0m. More detailed information on the underlying mineral soil deposits should be gathered prior to construction and the excavation designed at detailed design stage.

7.2.3 Stability Post Construction

The long-term stability of the tunnel bore will be aided by the presence of a permanent sleeve pipe that is installed during the tunnelling operation (see EIS, Chapter 5). The sleeve pipe will be installed at a sufficient depth below the channel to avoid exposure through erosion of the bed.

7.3 OTHER GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

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

7.3.1 Excavatability

The psammite rock samples extracted from a borehole on the southern shore of the crossing west of the landing point were very weak and broke down to a silty/silty clayey sand or a stiff slightly sandy clay. They fall within the dig category based on Pettifer and Fookes, 1994. Figure 7.1 below shows an assessment of the ripability of the rock encountered within the bay (BHF005-08 to BHF014b-08). The assessment shows that “dig” to “easy ripping” would remove the rock, if required.

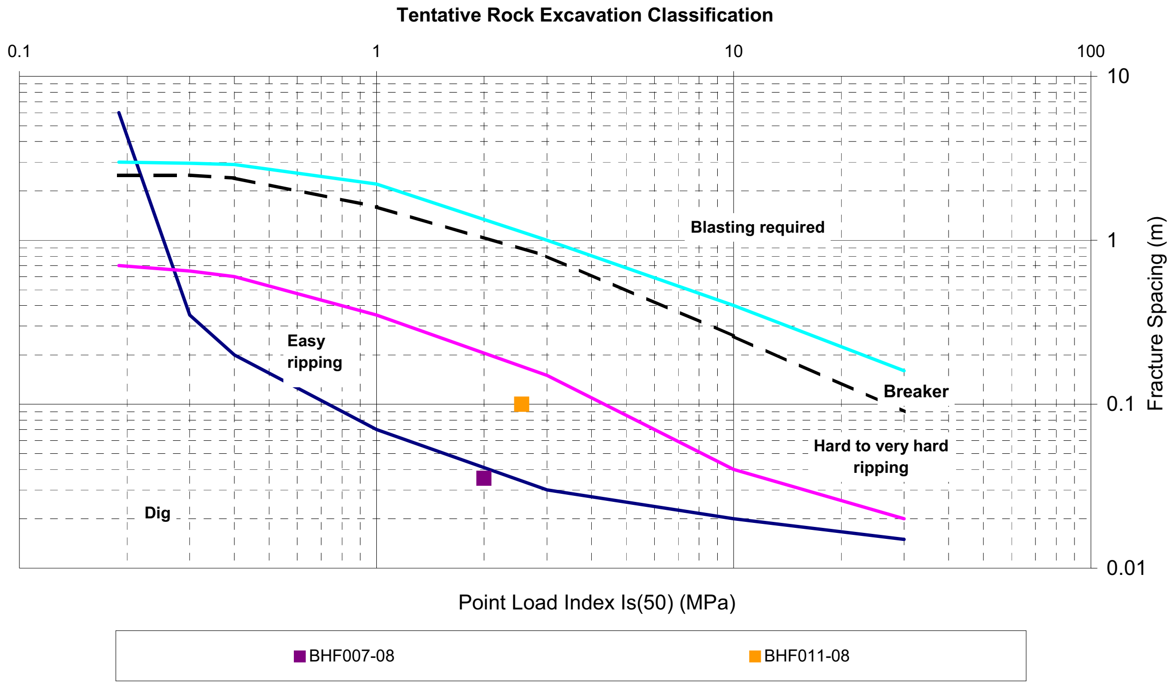

Figure 7.1 Rock Excavatability - Upper Sruwaddacon Bay Crossing (Pettifer and Fookes, 1994)

Cerchar abrasivity testing indicated rock classified below the not very abrasive category.

7.3.2 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 and can be seen in Table 7.2.

Section		Total No of PSDs	Total No Class 1	Total No Class 2
Section 5	Onshore Boreholes	4	1	3
	Foreshore Boreholes	35	20	15

Table 7.2 Soil classification based on Gradings

An excavation about 4.0m deep may be required for the launch pit. The top 3.0m approximately may be peat and may be reused as a backfill in the excavation. Backfilling with peat only, can lead to very long term soft spots and so it is recommended that any peat being used for backfill be mixed with an appropriate material to avoid long term soft spots developing. Below the peat layer, one particle size distribution test carried out at 5.0mbgl indicated Class 1 material and it is therefore expected that any material excavated below the peat may be used as Class 1 general engineering fill. Material at greater depths is Class 2, however it is not expected that excavation will occur at these depths.

Bentonite may be used in the trenchless operation for one or more of the following reasons; lubrication and cooling, spoil removal, stability of bore and grouting. The material excavated from the operations will be mixed with bentonite once removed. This material will be separated from the bentonite and disposed of at a licensed disposal/ recovery facility. Some bentonite will be lost with excavated spoil, but most of the bentonite will be recycled in the process.

7.4 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.

Consideration should be taken 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.

8 RECOMMENDATIONS FOR FURTHER GROUND INVESTIGATION

Further ground investigations will be required along the pipeline route to supplement the existing information. This should include boreholes and trial pits (with extraction of samples), in-situ testing and laboratory testing of overburden and bedrock material where necessary. The locations for further ground investigation are the launch and reception pit locations and peat areas.

9 REFERENCES

Design Manual for Roads and Bridges (NRADMRB). Specification Series 600 Earthworks.

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APPENDIX (M)1-A

Drawings