CORRIB ONSHORE PIPELINE

ADDITIONAL INFORMATION

Prepared for:

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SUMMARY

(1) Settlement of Stone Road With Pipe

(a) A sensitivity analysis of settlement of the stone road was carried out using three Cases (1), (2) and (3) representing varying degrees of peat compression. For each case, the magnitude of settlement was expressed as a proportion of the thickness of peat (peat/stone mix) below the stone road.

(b) The predicted settlement profile along the road is shown in Figure 1.1.

(c) The results for particularly Case (1), which assumes there is no stone within the peat, are considered unrealistic. Stone will penetrate into any peat underlying the stone road as described elsewhere. Furthermore, a substantial proportion of settlement is likely to occur prior to pipeline laying.

(d) It is considered that Case (3) represents the likely settlement of the stone road and pipe.

(e) A summary of differential settlement calculated between successive points of known peat depth are given in Table 1.1.

(2) Impact of Peat Landslide on Stone Road With Pipe

(a) Assessment of landslide impact was carried out using an applied lateral load to simulate the impact of a landslide within slope stability software to determine the minimum Factor of Safety (FoS). The assessment assumes that the stone road is founded on a layer of weak sensitive clay with an undrained shear strength of 5kPa.

(b) The FoS, which applies to failure along the weak soil layer under the road, varies from in excess of 2 to 0.8 for a corresponding lateral load of 0 to 75kN (Figure 2.3). Based on this calculation the road would fail at about 84% of the passive load.

(c) The calculation was repeated using actual ground conditions found at the same location (Figure 2.3). The actual ground conditions below the peat comprise clayey gravelly sand.

(d) The FoS, again which applies to failure along the soil layer under the road, varies from in excess of 2 to 1.89 for a corresponding lateral load of 0 to 75kN (Figure 2.3). In this case the stone road can sustain the maximum passive load.

(e) The plausibility of the assumed model is discussed, which includes the potential for peat failures upslope of the pipeline and the presence or otherwise of weak sensitive soil at the base of the peat.

(f) It is noted that the presence of a weak sensitive soil layer was identified as a risk to the works and included in the Geotechnical Risk Register included with the original submission (EIS, Volume 2, Appendix M4, Hazard No. 15). The Geotechnical Risk Register provides control measures where sensitive soil is encountered.

(3) Qualitative Assessment of Relative Potential for Peat Failure

(a) A qualitative assessment of the relative potential for peat failure was carried out along the pipeline route in peat areas. This used a basket of environmental factors to determine the relative potential for peat failure with respect to sections of the proposed pipeline route.

(b) The pipeline route within the peatland areas was divided into 19 sections and a qualitative assessment, which provides a rating score, carried out for each section.

(c) The individual rating score for each section is summarised in Table 3.1. A section was assigned to a relative potential category based on its score. The categories were Category 1 (high potential), Category 2 (intermediate potential) and Category 3 (low potential).

(d) The results of the qualitative assessment are shown on plan, see Drawing No. 001.

(e) The highest score is in section 18, which is located downslope of the terminal. Section 18 has a notable large catchment upslope and relatively steep slopes. Furthermore, it has an FoS of less than 1.3. It is noted that the stone road has already been constructed through this section.

(f) In general, the higher scores are for the sections approaching the terminal. This is considered partly due to the more undulating terrain, increased upslope areas and drainage conditions. Section 2 is also given a relative potential category of 1 (high). This is notably related to a FoS of less than 1.3, which is due to a locally steeper slope at base of peat on the western end of the peatland area.

(g) In general, the qualitative assessment provides an indication of where there are potentially a number of environmental factors that may result in an increased potential for peat failure. As it is difficult to calibrate qualitative assessments its use is limited.

(h) At best the qualitative assessment can be used to inform any confirmatory investigation and to indicate to supervising geotechnical engineers during construction of possible stability issues.

(4) Brief Review of Dooncarton Mountain Landslides, 2003

(a) The distribution of landslides on the northern slopes of Dooncarton Mountain is shown on Drawing No. 001.

(b) Due to the shape of the ridges that form Dooncarton Mountain, and the susceptibility of the slopes to failure, the landslides were essentially located to the northwest and northeast and this was also the direction of the associated landslide debris trails (see Drawing No. 001).

(c) The landslides had less of an effect on the fields and area to the north of the mountain where the Landfall Valve Facility (LVI) is located. The LVI site was not affected by failure debris.

(d) The Tobin (2003) investigation and risk zoning of the area reflects the northwest and northeast landlside failure pattern. As such, the risk plan shows that the high/medium risk is on the upper slopes on the northwest and northeast of the mountainside (Drawing No. 001).

(e) The Tobin risk zoning shows that the north side of the mountain facing the LVI is a low risk area, and the area of the LVI was at such a distance from the mountain that it was not considered within the risk zoning.

(5) Confirmatory Ground Investigation

(a) The need for confirmatory investigation, which would be considered good practise, was recommended in the geotechnical reports (EIS, Volume 2, Appendix M2 and M3).

(b) Confirmatory ground investigations are to be carried out prior to construction to re-confirm the ground conditions and to identify any departures from the design ground model.

(c) Details of the confirmatory ground investigation are given and this includes:

- In situ testing and sampling (mechanical vane, and where access is possible Cone Penetration Tests (CPT), boreholes to retrieve continuous thin-wall samples from the peat and underlying mineral soils, geophysical surveying along the pipeline route using Ground Penetrating Radar (GPR) and 2-D resistivity surveying).

- Laboratory Testing (testing on undisturbed and disturbed samples retrieved from the peat and underlying mineral soil to include strength tests, index tests, compression tests)

- Instrumentation and monitoring (groundwater monitoring from piezometers, inclinometers (where installed); construction monitoring will be determined following review of construction method statements but will likely include, but not be limited to, continued monitoring of groundwater installation (and inclinometers where installed), level monuments on stone road, movement posts alongside works.)

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INTRODUCTION

This document contains additional information as requested on 9 June 2009, and other dates, by the Inspector chairing the An Bord Pleanála (ABP) Oral Hearing. The additional information request includes the following:

(1) Settlement of stone road with pipe. This includes:

(a) A sensitivity analysis of the pipeline and stone road settlement for predicted and worst case and the corresponding pipe stress.

(b) An analysis of Load Case 1 (stone road without any load) in deep peat where settlement of the basal peat layer occurs.

The assessment of the corresponding induced pipe stresses as a result of settlement is presented by J P Kenny in a separate submission.

(2) Impact of peat landslide on stone road with pipe. An analysis of Load Case 2 (stone road with 10kPa load) where a layer of very soft sensitive clay with an undrained shear strength of 5kPa is assumed below the road and where the road is subjected to upslope (passive) pressure as a result of a peat failure upslope.

(3) A plan (Drawing No. 001) of the pipeline/stone road route showing the relative potential of peat failure using a qualitative approach including environmental factors.

Environmental factors include for example:

- Ground conditions, such as presence of weak mineral soil
- Topography, such as slope of underlying mineral soil, morphology
- Water conditions, such as surface hydrology, hydrogeology
- Stability analysis assuming weak sub-peat conditions
- Peat slide history
- Land use

The plan also includes details of the risk zoning and landslide scars with associated debris run-out for the Pollathomais landslides in 2003, as given in Tobin (2003).

(4) Brief review of Dooncarton Mountain landslides in 2003*, which are shown on Drawing No. 001.

(5) Confirmatory ground investigation.

(6) Temporary discharge locations for construction are annotated on Drawing No. 001.

* Reference: Tobin (2003). Report on the Landslide at Dooncarton, Glengad, Barnachuille and Pollothomais, County Mayo. Patrick J Tobin & Co Ltd

1 SETTLEMENT OF STONE ROAD WITH PIPE

1.1 Introduction

An assessment of the settlement of the stone road with the pipe buried in the stone road has been carried out. This includes:

(1) A sensitivity analysis of the pipeline and stone road settlement for predicted and worst case and the corresponding pipe stresses.

(2) An analysis of Load Case 1 (stone road without any load) in 'deep' peat where settlement of the basal peat layer occurs.

The stone road is to be constructed with a nominal basal layer of 0.5m of peat left in situ at the base of the stone road together with, in places, a mix of peat and stone below the stone road. This will result in settlement of the road with the placed pipe as the underlying peat consolidates under the weight of the road.

The settlement of the road will result in the pipeline, which is buried in the stone road, also settling. The settlement of the pipeline will induce stresses within the pipeline.

The assessment of the corresponding induced pipe stresses as a result of settlement is provided by J P Kenny in a separate submission.

1.2 Methodology Used to Determined Settlement of Stone Road

The methodology used to determine the settlement of the road is as follows:

(1) The expected magnitude of the settlement of the stone road is required to assess the impact of the settlement on the pipeline buried within the road.

(2) The magnitude of peat settlement below the stone road will be in proportion to the peat thickness left below the road. Peat depth varies gradually along the road length therefore excessive differential settlement is not anticipated.

(3) Peat left in place below the stone road will be penetrated by stone, which will be pushed into the peat. This will result in stone within a peat matrix. It is not possible to determine the exact proportion of peat to stone therefore the settlement is determined for three cases namely; peat below the road with no stone, peat below the road with some stone, and peat with much stone below the road.

(4) Settlement due to the road loading is due to primary consolidation settlement and secondary compression of the peat and peat/stone mix. For simplicity, and based on experience of blanket peat that has been partly drained, the magnitude of settlement can be simplified to the proportion of the thickness of peat left below the road as follows:

Case (1) Peat with no stone	0.3 x peat thickness
Case (2) Peat with some stone	0.2 x peat thickness
Case (3) Peat with much stone	0.1 x peat thickness

(5) For a worst case, it is assumed that no settlement takes place before the emplacement of the pipe. In reality this is not the case, as settlement will commence relatively rapidly following initial placement of the stone road and a substantial amount of this settlement will have been completed prior to installing the pipe as a result of the self-weight of the stone road and construction traffic.

(6) It is recognised that Case (1) is unrealistic, but it is included to allow settlement and hence pipe stresses to be determine to illustrate the robustness of the pipeline.

(7) The settlement has been calculated at known peat depth locations corresponding to probe/vane locations. Anomalous peat depth probe results are excluded.

(8) The assessment includes for the stone road to be constructed with a nominal peat layer of 0.5m thickness left in situ at the base of the stone. In areas of shallow peat, the pipeline may be founded within mineral soil. Where peat depth is greater than 2.5m then stone will likely be pushed into the peat; for shallower depths all peat will be excavated except for the lower 0.5m of peat.

1.3 Findings of Settlement of Stone Road

The findings of the settlement of the road are as follows:

(1) The predicted settlement profile along the road is shown in Figure 1.1. The maximum settlement ranges from 750mm in Case (1) to 250mm in Case (3).

(2) The results for particularly Case (1), which assumes there is no stone within the peat, are considered unrealistic. Stone will penetrate into any peat underlying the stone road as described elsewhere. Furthermore, a substantial proportion of settlement is likely to occur prior to pipeline laying.

(3) It is considered that Case (3) represents the likely settlement of the stone road and pipe.

(4) Within Rossport commonage there is considered to be an anomalous reading at ch 88,134, where in an area of deep peat a single shear vane/probe shows a shallower peat thickness.

(5) A summary of differential settlement calculated between successive points of known peat depth are given in Table 1.1.

Rossport Commonage (approximate chainage 85,960 to 88,600)

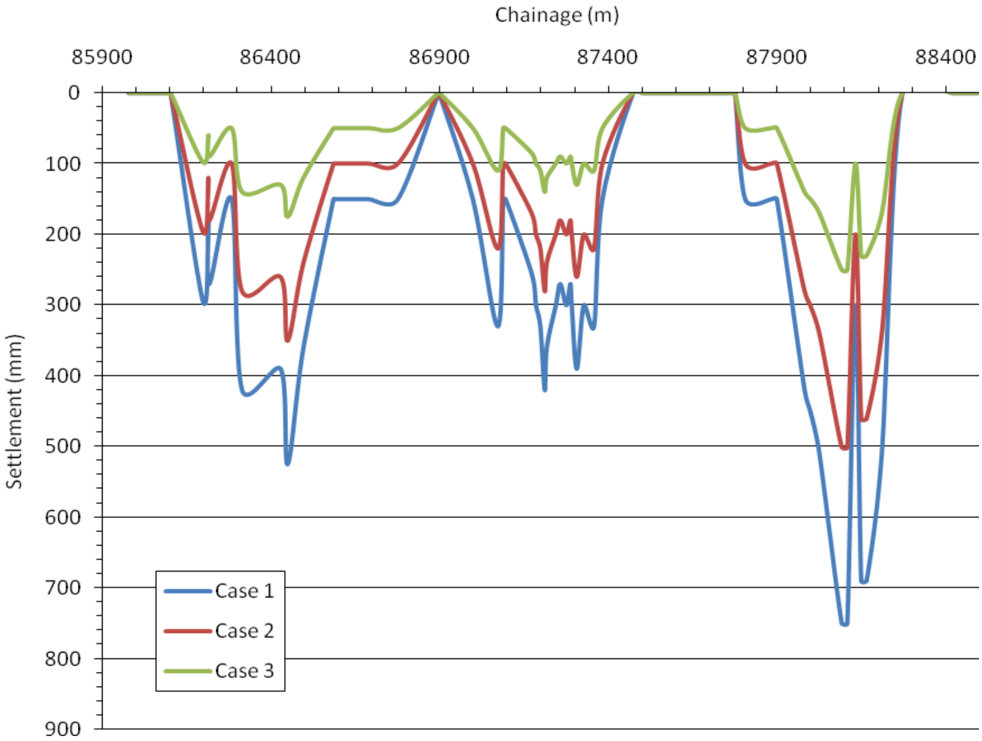
	Case (1)				Case (2)				Case (3)		
Parameter	Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)		Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)		Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)
Minimum	0	-1.07	-1.88		0	-0.72	-1.25		0	-0.36	-0.63
Maximum	750	1.72	3.00		500	1.15	2.00		250	0.57	1.00
Average	236	0.03	0.05		157	0.02	0.04		79	0.01	0.02

South of Sruwaddacon Bay to Terminal (approximate chainage 89,500 to 92,560)

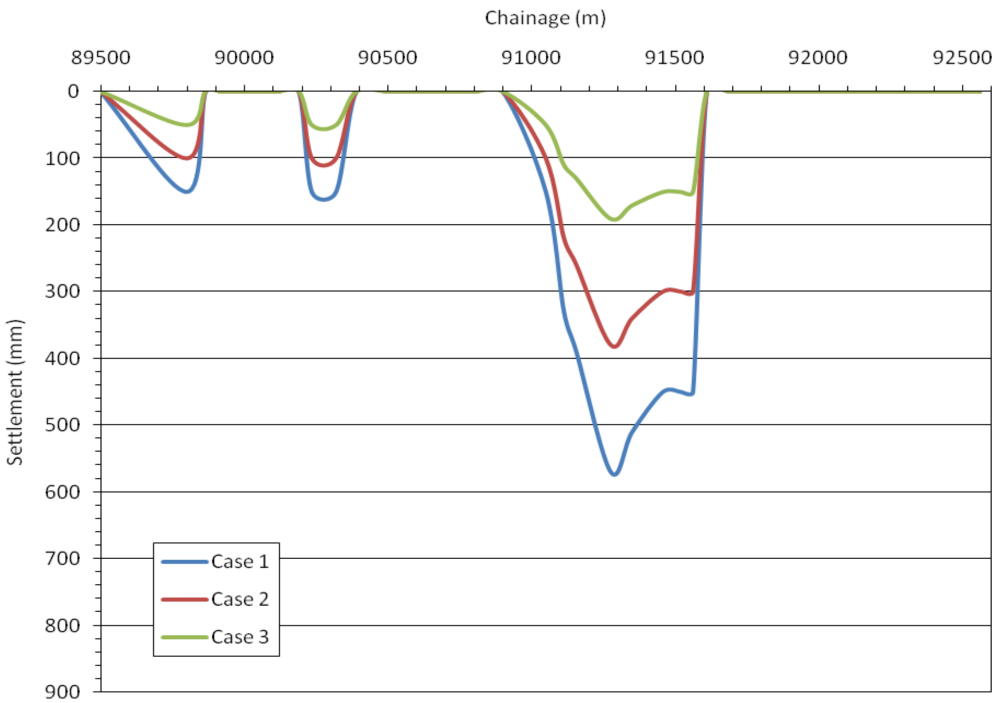
	Case (1)				Case (2)				Case (3)		
Parameter	Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)		Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)		Settlement (mm)	Differential Angular Settlement (degs)	Differential Gradient Settlement (%)
Minimum	0	-0.537	-0.938		0	-0.358	-0.625		0	-0.179	-0.313
Maximum	570	0.191	0.333		380	0.127	0.222		190	0.064	0.111
Average	107	-0.008	-0.013		71	-0.005	-0.009		36	-0.003	-0.004

Table 1.1 Summary of Predicted Settlement of Peat below Stone Road

FIGURES



(a) Predicted Settlement in Rossport Commonage



(b) Predicted Settlement in South of Sruwaddacon Bay

Figure 1.1 Predicted Settlement of Peat below Stone Road

2 IMPACT OF LANDSLIDE ON STONE ROAD WITH PIPE

2.1 Introduction

The impact of a peat landslide upslope of the stone road is examined.

The proposal was to analyse Load Case 2 (stone road with 10kPa load) where a layer of very soft sensitive clay with an undrained shear strength of 5kPa is assumed below the road. The impact of the peat landslide would subject the upslope edge of the road to lateral (passive) pressure.

2.2 Model

Two models have been examined to determine the stability of the stone road subjected to upslope lateral pressure from peat.

Model (1): Simple kinematic model of a sliding block on an inclined plane with a force applied on the upslope side. For force equilibrium the following simple model would apply:

$$\text{Factor of Safety} = \frac{\text{Resisting Force}}{\text{Destabilising Force}} = \frac{Cu.A + T'p}{W \sin\alpha + Tp}$$

Where

Cu : undrained shear strength on the sliding plane
A : area of sliding plane under road upslope of T'p
W : Weight of sliding block (road plus any load)
α : angle of inclined plane
T'p : passive force in stone road
Tp : upslope passive force due to peat landslide

Model (2): Assessment of landslide impact using an applied lateral load to simulate the impact of a landslide within slope stability software (Terrasol, 2005). This uses a slip circle analysis with an automatic search routine to determine the minimum Factor of Safety.

Following comments on models are given:

(1) The peat upslope of the stone road is assumed to be unstable and fails shortly after emplacement of the stone road.

(2) The models assume that there is a weak soil layer below the road and surrounding peat slopes.

(3) Within model (1) to resist the upslope force on the road it is assumed that the undrained shear resistance of the underlying weak soil is utilised. This may not necessarily be the case as there will also be passive resistance generated within the stone road.

(4) For model (1) there will a corresponding passive wedge generated within the stone road to initially resist the passive resistance of the peat upslope. The passive

resistance within the stone road will be significantly greater than that generated by the peat; only a portion of the passive resistance within the stone road would need to be mobilised to resist the loading from the peat.

(5) Emplacement of the stone road occurs in such a manner that the stone within the road does not embed itself within the weak soil.

(6) Failure occurs rapidly after stone emplacement as there is no material gain in strength of the weak layer due to weight of stone.

Model (1) as presented would result in the stone road resisting the applied load from the upslope peat through the development of a degree of passive pressure in the stone road itself.

Model (2) is considered to provide a more reasonable representation of the stability of the stone road subjected to upslope lateral pressure from peat. This model has been used to analyse the problem, see below.

2.3 Results of Stability Analysis

Results using model (2) are summarised below and included in Figure 2.1.

(1) Within Rossport Commonage the pipeline route/stone road is located close to the water-shed divide, that is the topographical ridgeline, which limits the potential for peat failure upslope. South of Sruwaddacon Bay the route is not close to the watershed divide and there are greater upslope areas of peat.

(2) Between ch 91.56 to 91.92 (See Drawing No 001 with this report), which is Section 18 in the Qualitative Assessment (see Chapter 3), there is the greatest upslope area.

(3) The typical peat depth along the pipeline route in Section 18 is 3m. It is considered that during construction of the road that 2.5m of peat was excavated and a nominal 0.5m depth of peat left in place with stone pushed into this peat.

(4) It is assumed that 1.0m of weak mineral soil lies beneath the peat and this material has an undrained shear strength of 5kPa.

(5) The road has a uniformerly distributed 10kPa loading on its surface (Load Case 1), in addition to a uniformerly distributed 10kPa upslope representing peat turves.

(6) A slip circle analysis has been carried out with varying lateral loads placed on the upslope face of the road to model loading from a potential peat failure upslope impacting on the road.

The Factor of Safety (FoS) has been calculated for a range of lateral loads simulating the force of landslide impact. The lateral load on the stone road was varied from 0 up to 75kN. The analysis uses a circular failure surface which passes under the stone road and along the weak soil layer.

The FoS, which applies to failure along the weak soil layer under the road, varies from in excess of 2 to 0.8 for a corresponding lateral load of 0 to 75kN (Figure 2.3). The calculated passive load for 3m of peat is given as about 77kN. Based on this calculation the road would fail at about 84% of the passive load.

The calculation was repeated using actual ground conditions found at this location (refer to table below for trial pit TP03), see Figure 2.3. The actual ground conditions below the peat comprise clayey gravelly sand (effective friction angle = 30 degrees).

The FoS, again which applies to failure along the underside of the road, varies from in excess of 2 to 1.89 for a corresponding lateral load of 0 to 75kN (Figure 2.3). In this case the stone road can sustain the maximum passive load.

Whilst the above analysis shows that based on actual soil conditions the stone road would resist landslide impact, the stone road has not been designed to act as a landslide containment barrier. The inclusion of the stone road within the peat slope provides an increase in the stability of the peat slope due to the increase in shear resistance to sliding provided by the higher shear resistance of the stone fill. As such, in the unlikely case where a peat slide occurred the stone road would provide greater resistance to movement, and offer greater security to the pipeline, than if the pipeline was buried within the peat itself.

2.4 Plausibility of Model

The plausibility of the model is examined as follows:

(1) Stability of peat upslope of the road.

Within Rossport Commonage the pipeline route/stone road is located close to the water-shed divide, that is the topographical ridgeline, which limits the potential for peat failure upslope. South of Sruwaddacon Bay the route is not close to the watershed divide.

Original stability analysis of the peat along the route used actual shear strength values derived from shear vanes. These strength values were in places as low as 1kPa to 3kPa and indicated only a limited number of places where there was a low Factor of Safety (approximate ch 87,220, 89,870 and 91,690). The use of cu = 5kPa, which is the assumed strength of the weak layer, at the base of the peat would increase the Factor of Safety.

Based on the above the likelihood of failure of the peat upslope of the road is considered remote.

(2) Presence of weak sensitive soil layer

The ground investigation of the route included examination of sub-peat soil along the route from trail pits, probes, and exposures. A weak sensitive soil would comprise a fine-grained silt/clay. The findings of these are summarised below:

Nearest Chainage/ Location	Ground Investigation	Sub-peat Soil Conditions
Nearest 84.960	From exposure in quarry face, Rossport (Exp-002 from QMEC)	Firm grey sandy gravelly SILT. Soft, light brown, silty gravelly SAND with occasional sub-angular boulders
86.125	Gouge core GC3	Very stiff brown slightly sandy SILT (micas evident) stiff refusal at 1.00m
86.315	Gouge core GC2	Very stiff, grey, sandy SILT (micas evident) stiff refusal at 2.80m
86.670	Peat cutting to mineral soil. Exposure in floor of peat cutting, refer to Geomorphological Plans	Firm grey/brown sandy gravelly silt with some tree roots
87.260	Gouge core GC1	Hard refusal at 3.54m. Assumed weathered rock/boulder
Nearest 88.420	Exposure in cliff/slope beside bay, refer to Geomorphological Plans	Firm reddish brown slightly sandy slightly gravelly silt. Firm light grey silty gravelly sand with some cobbles
88.700	Exposure in cliff/slope beside bay, refer to Geomorphological Plans	Firm light grey sandy gravelly silt
Nearest 89.500	Exposure in cliff/slope beside bay, refer to Geomorphological Plans	Firm brown silt with rootlets
89.950	Peat cutting to mineral soil. Exposure in floor of peat cutting, refer to Geomorphological Plans	Firm to stiff brown gravelly sandy silt with much angular to sub-rounded cobbles. Tree roots
90.150	Exposure in cliff/slope beside bay, refer to Geomorphological Plans	Firm dark brown slightly organic sandy gravelly silt
90.700	Trial pit TP06	Greyish brown gravelly SAND with many cobbles and occasional boulders of angular to sub-rounded quartz and quartzite. Bluish grey clayey fine SAND (liquefying/running). Gravel angular to sub-rounded quartz & quartzite and schist
91.530	Trial pit TP03	Grey slightly silty very gravelly SAND (medium) with some to many cobbles. Gravel and cobbles angular to sub-rounded quartz and quartzite. Blue grey clayey gravelly fine SAND (liquefying/running). Gravel rounded to angular quartzite & quartz and occasional mica schist

The above information shows no evidence of a weak sensitive silt/clay soil at the base of the peat. Layers of granular soil (SAND) found in TP03 and 06 were observed to be running (ie. flowed due to presence of groundwater); which was due to the release of water within the sand following removal of the normal load. Placement of the stone road onto these granular soils will greatly increase the normal load and the effective stresses.

Notwithstanding the above, the presence of a weak sensitive soil layer was identified as a risk to the works and included in the Geotechnical Risk Register included with the original submission (EIS, Volume 2, Appendix M4, Hazard No. 15). The Geotechnical Risk Register provides control measures where sensitive soil is encountered, and is reproduced below.

Hazard	Design Control	Construction Control
Presence of sensitive clay/silt below peat	1. Review of previous excavations in the locality and in similar ground 2. Detailed site investigation to include trial trenches to expose sensitive soils 3. Boreholes and trial pits to be taken below base of excavations 4. Avoidance of excessive loading and/or excessive vibration 5. Detailed method statement to be prepared in accordance with ground conditions anticipated	1. Site supervision staff to inspect trench 2. Restricted access into excavation 3. Employment of contractor/personnel familiar with soft ground conditions 4. Engineering supervision to ensure construction carried out as detailed in the method statement 5. Limited exposure of excavation base and sides to avoid potential for failure 6. Avoid excessive loading and/or vibration

The control measures would vary depending on the extent of the presence or otherwise of a weak sensitive soil. Worst case control measures (though considered unlikely) could involve for example:

(1) Removal of the weak sensitive layer where it was considered that placement of stone onto the weak soil layer would not appreciably increase the shear resistance of the weak soil.

(2) Increased burial depth of pipeline. This would involve placing the pipeline below the stone road and into the mineral soil. In this case the pipeline would be completely within the mineral soil and below the base of any potential peat failures.

It is recognised that there has been limited investigation, in particular in parts of Rossport Commonage due to access difficulties, and therefore investigation is called for in the geotechnical reports (as well as in the Geotechnical Risk Register) to confirm the ground conditions.

Details of the confirmatory investigation are included within this submission of additional information.

(3) Strength of Weak Sensitive Soil Layer

The stability model assumes that the strength of the weak soil layer is 5kPa, and that this strength is unaffected by placement of the stone road. For this to apply the underlying assumptions are as follows:

(a) Failure occurs relatively instantaneously following placement of stone road. In the unlikely event that failure should occur following placement of stone then the pipeline will not have been laid within the road.

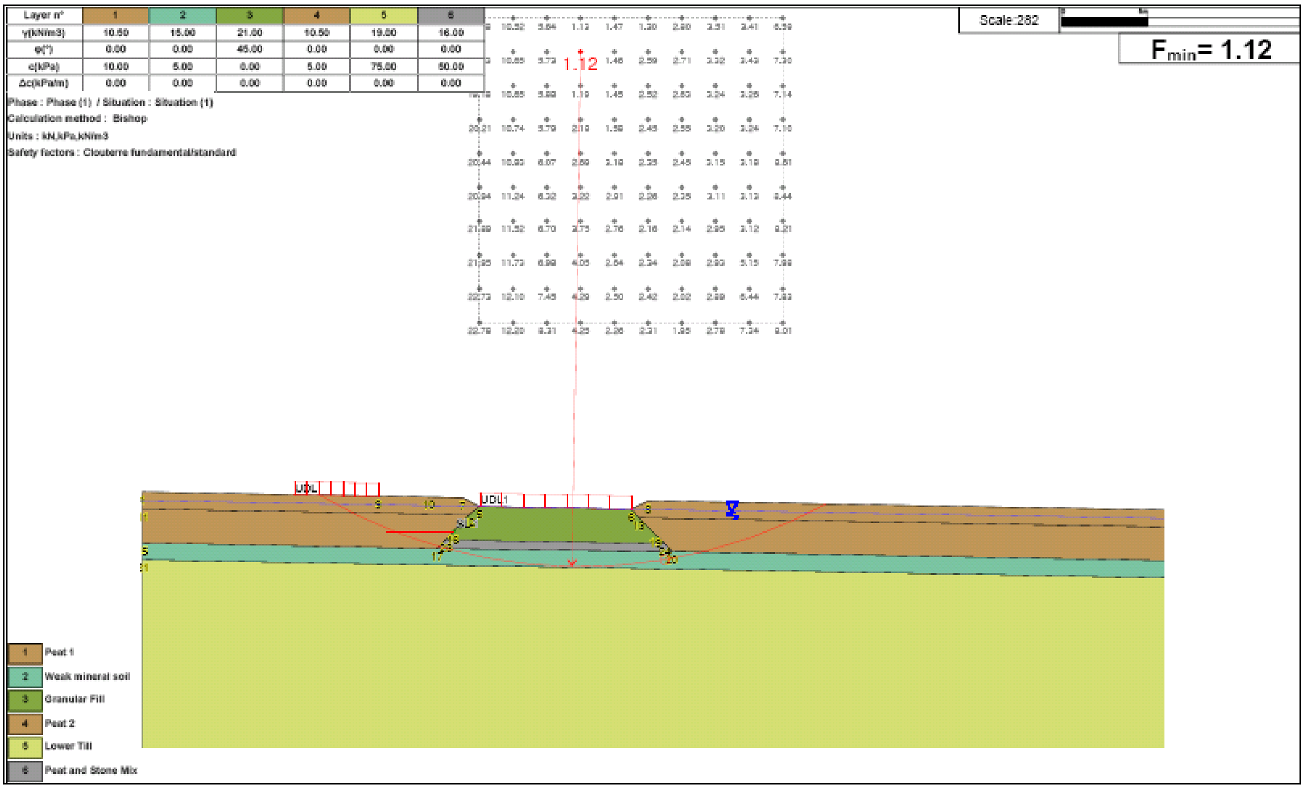
(b) There is no penetration of the placed stone into the soil layer. In most cases along the route it is considered that the stone will penetrate into the mineral soil and will therefore increase the shear resistance at the base of the peat.

(c) The placement of the stone road will result in consolidation settlement of any weak soil located below the road. The consolidation will result in strength gain in the soil.

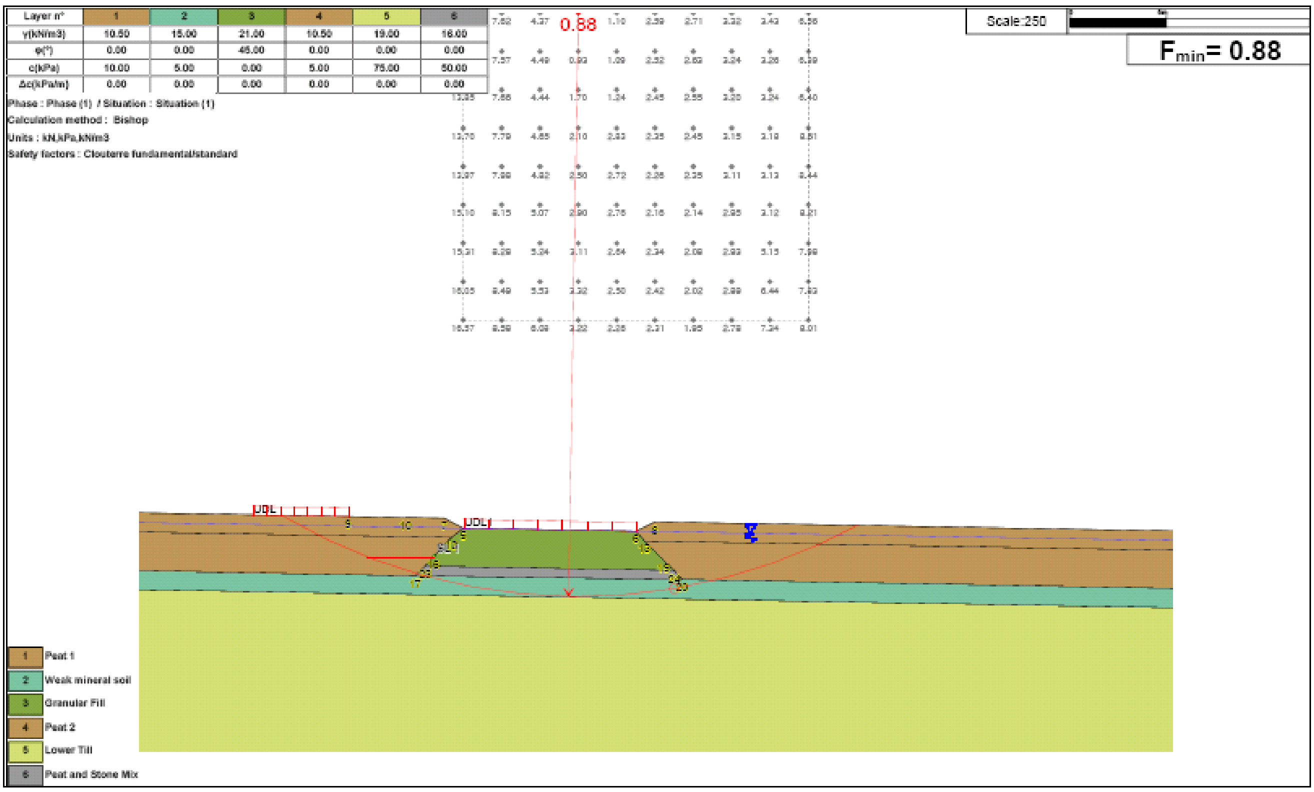
2.5 References

Terrasol (2005). Talren 4 – Stability analysis for geotechnical structures with or without reinforcement. February 2005.

FIGURES

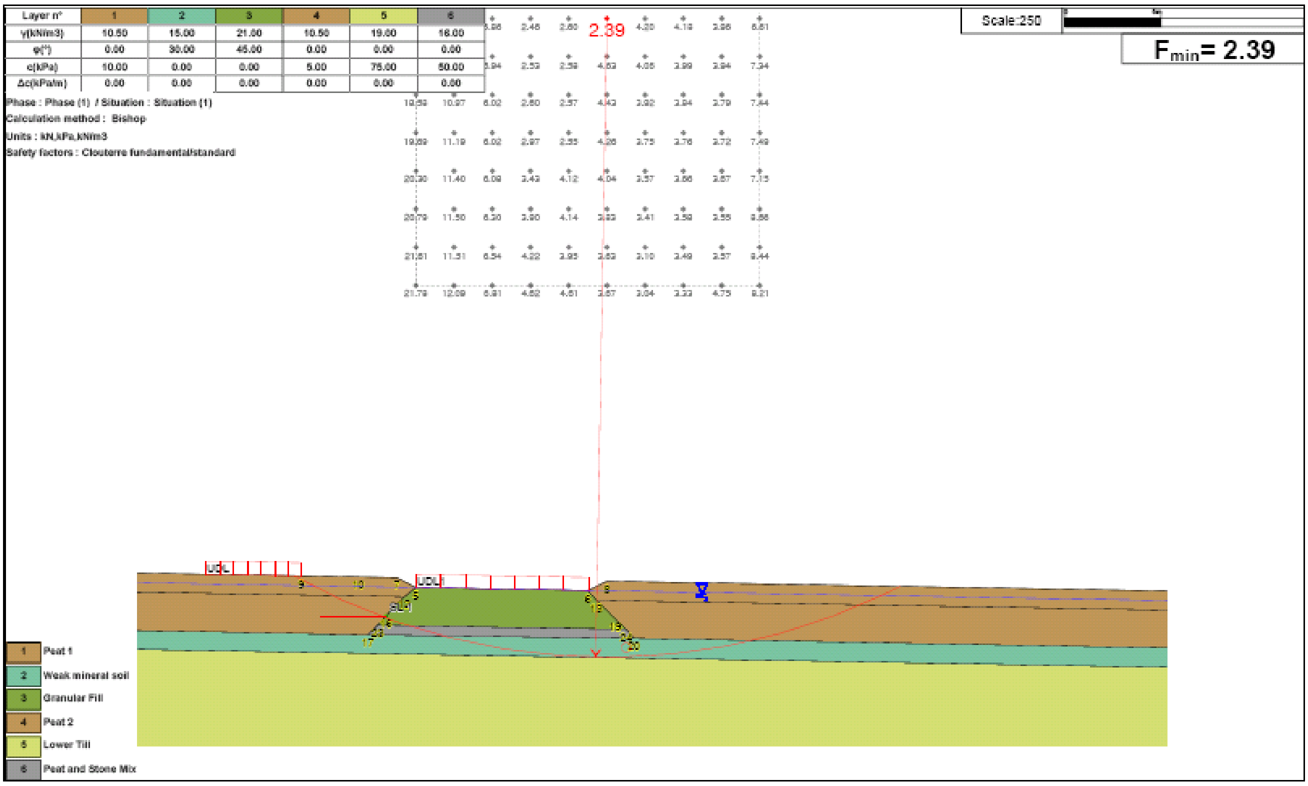


(a) Stone road on weak sensitive soil with 50kN lateral load

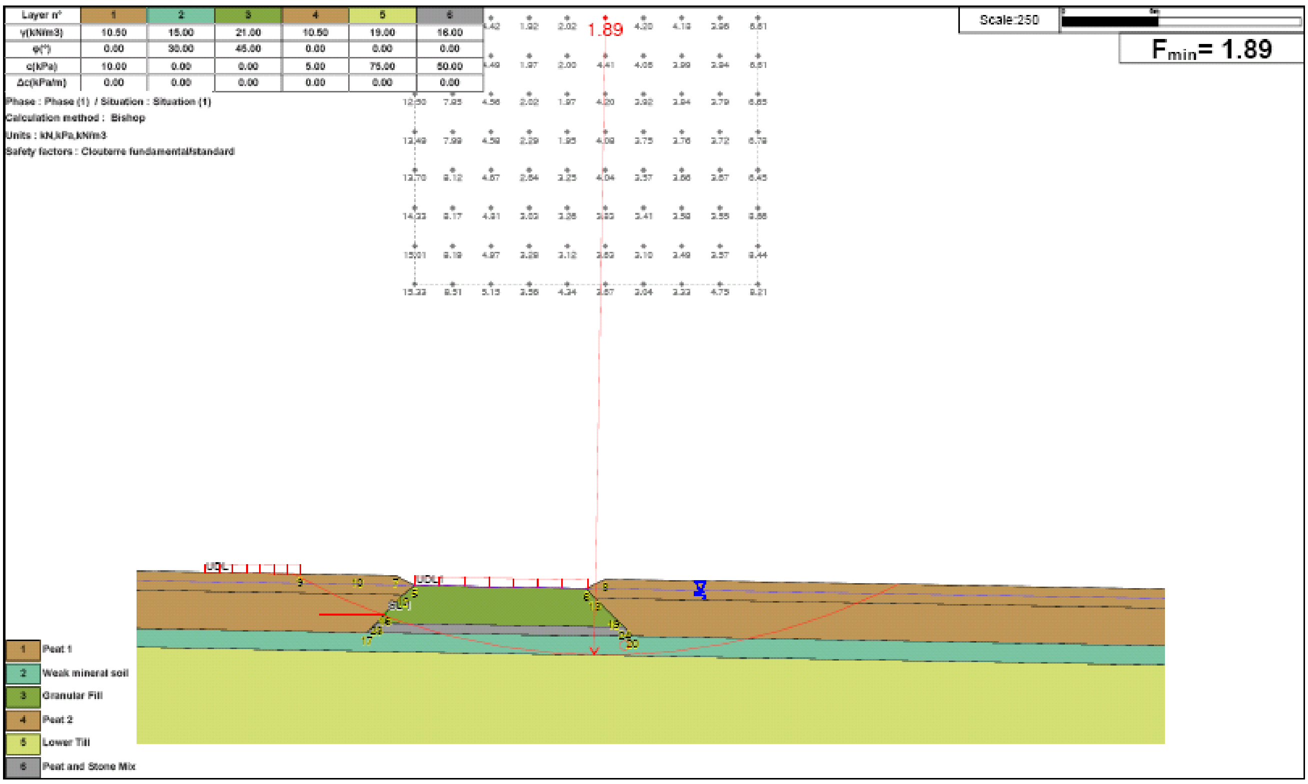


(b) Stone road on weak sensitive soil with 75kN lateral load

Figure 2.1 Stone Road on Weak Sensitive Soil



(a) Stone road on actual soil conditions with 50kN lateral load



(b) Stone road on actual soil conditions with 75kN lateral load

Figure 2.2 Stone Road on Actual Soil Conditions

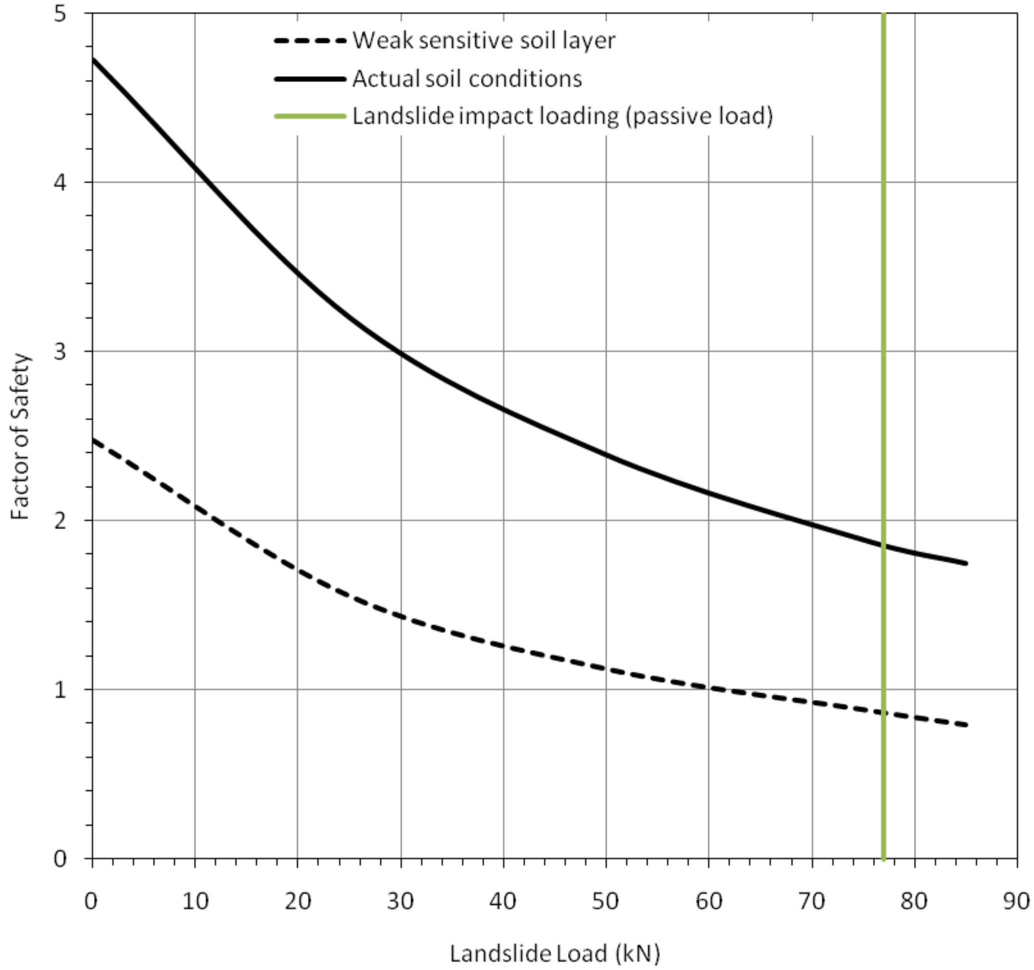


Figure 2.3 Comparison of Stability of Stone Road on Weak Sensitive Soil with Actual Soil Conditions

3 QUALITATIVE ASSESSMENT OF RELATIVE POTENTIAL FOR PEAT FAILURE

3.1 Introduction

A qualitative assessment of the relative potential for peat failure was carried out along the pipeline route in peat areas using a basket of environmental factors to determine the relative potential for peat failure with respect to sections of the proposed pipeline route.

The results of the qualitative assessment are shown on plan (Drawing No. 001) and the scoring of the environmental factors included in Appendix 3.1.

The following qualitative environmental factors were used in the assessment:

(a) Ground conditions

(b) Topography, such as peat slope, slope of underlying mineral soil, morphology

(c) Water conditions, such as surface hydrology, hydrogeology

(d) Stability analysis assuming weak sub-peat conditions

(e) Peat slide history

(f) Land use

The assessment was carried out incorporating the results of a stability analysis assuming that the stone road was constructed on a weak sub-peat soil, such as very soft sensitive soil layer, with undrained strength of 5kPa.

3.2 Methodology Used For Qualitative Assessment

The methodology used for the qualitative assessment of relative potential for peat failure is as follows:

(1) The qualitative assessment provides a measure of the relative potential for peat failure. Included within the qualitative assessment are environmental factors (such as presence of peat cuttings, drains, etc) that cannot be as easily included and quantified within a routine deterministic assessment (ie calculation of factor of safety).

(2) The qualitative assessment relies on the experience and judgement of the compiler and therefore is subjective in nature. The list of factors included in the assessment is based on a review of factors commonly considered to be discriminatory indicators of failure (eg MacCulloch, 2006).

(3) The results from the qualitative assessment are a relative measure of the potential for peat instability; this measure cannot be used as an absolute indicator of the susceptibility to failure. Instead the qualitative assessment provides an indication of areas where there is a relatively greater potential for peat failure.

(4) The qualitative assessment included the following:

(a) Compilation of a list of pertinent discriminatory factors. 32 factors were used in the assessment.

(b) Rating of these factors from 1 to 3.

(c) Dividing the pipeline route within the peatland areas into sections with similar terrain characteristics. 19 sections have been used.

(d) Rating of each factor from 1 to 3 for each individual section.

(e) Summation of all ratings for each section to provide a ratings score.

(5) Of the 32 factors up to 29 factors could be rated for any section. In all sections not all factors may be present. The maximum rating score is 87 (ie. 100%) and the minimum rating score (based on 29 factors) is 29 (ie. 33% of maximum). The minimum rating score would be terrain that showed negligible susceptibility to peat failure.

(6) The rating scores for each individual section were ranked and the highest scores given a higher relative potential category. The division between relative potential categories is based on dividing the possible range of ratings into thirds.

(7) It is considered of the many factors included within the assessment that a higher weighting, or consideration, should be given to the Factor of Safety determined from stability calculation. As such, where the Factor of Safety (FoS) determined from either ground slope or base of peat is less than 1.3 then a section is promoted to the next highest category.

(8) The relative potential category is given as follows:

Relative Potential Category	Title	Typical Description
1	High potential	Notable number of ratings of 2 and 3. Total ratings score is greater than 78%, which represents upper third.
2	Intermediate potential	High proportion of ratings of 2 and 3. Total ratings score is greater than 56% but less than 78%, which represents middle third.
3	Low potential	Significant number of ratings of 1. Total ratings score is 56% or less, which represents lower third.

3.3 Findings of Qualitative Assessment

A summary of findings of the qualitative assessment are given as follows:

(1) The individual rating score for each section is given in Table 3.1 and a detailed breakdown of the scores for each section given in Appendix 3.1.

(2) The rating scores for the sections vary from 50.6 to 66.7%. The scores occupy a relatively narrow band, and this is considered to there being similar characteristics between the sections. The highest score is in section 18, which is located downslope of the terminal.

(3) A review of the score for section 18 indicates that it has a notable number of 2 ratings. Section 18 has a notable large catchment upslope and relatively steep slopes. Furthermore, it has an FoS of less than 1.3. The relative potential category for section 18 is 1 (high). The stone road has already been constructed through this section (Drawing No. 001).

(4) In general the higher rating scores are for the sections approaching the terminal. This is considered partly due to the more undulating terrain and drainage conditions.

(5) Section 2 is also given a relative potential category of 1 (high). This is notably related to a FoS of less than 1.3, which is due to a locally steeper slope at base of peat on the western end of the peatland area.

(6) The distribution of the relative potential categories is shown in Drawing No. 001.

(7) In general, the qualitative assessment provides an indication of where there are potentially a number of environmental factors that may result in an increased potential for peat failure. As it is difficult to calibrate qualitative assessments its use is limited

(8) At best the qualitative assessment can be used to inform any confirmatory investigation and to indicate to supervising geotechnical engineers during construction of possible stability issues.

3.4 References

Macculloch, F. (2006) Guidelines for the Risk Management of Peat Slips on the Construction of Low Volume/Low Cost Roads Over Peat. EU ROADEX II Project. January 2006

Section	Rating Score	Initial Relative Potential Category	Calculated FoS		Factor of Safety < 1.3	Final Relative Potential Category	Comment
			Ground surface slope angle	Base of peat slope angle			
1	50.6	3	2.6	1.8	No	3	
2	57.5	2	2.30	1.2	Yes	1	Low Factor of Safety is due to locally steeper section of slope at base of peat on west of peatland.
3	54.6	3	1.70	2.40	No	3	
4	55.1	3	2.10	1.30	No	3	
5	55.1	3	2.10	4	No	3	
6	55.7	3	1.40	3.4	No	3	
7	56.0	3	2.20	2.8	No	3	
8	51.9	3	2.20	0.7	Yes	2	Low Factor of Safety is due to an anomalous shallow area of peat within an area of deeper peat.
9	53.6	3	2.10	2.1	No	3	
10	52.5	3	3.50	4.1	No	3	
11	52.9	3	3.10	2.8	No	3	
12	58.0	2	2.20	2.2	No	2	
13	57.7	2	2.10	3.1	No	2	
14	56.5	2	5.40	4	No	2	
15	60.7	2	2.30	6.7	No	2	
16	61.9	2	2.10	4.9	No	2	
17	63.1	2	3.60	2.7	No	2	
18	66.7	2	1.50	1.2	Yes	1	Low Factor of Safety is due to a combination of a locally steeper slope at base of peat within relatively deeper peat.
19	64.3	2	2.00	3.1	No	2	

Table 3.1 Summary of Qualitative Assessment for Individual Sections along Pipeline Route

APPENDIX 3.1

DRAWING No. 001

4 BRIEF REVIEW OF DOONCARTON MOUNTAIN LANDSLIDES, 2003

4.1 Introduction

On 19 September 2003 a cluster of 40 separate shallow landslides, including significant peat failures, occurred on Dooncarton Mountain during an exceptional rainfall event (Tobin, 2003).

The trigger for the landslides was the exceptional rainfall (estimated to be excess of 100 year return period with one estimate of in excess of 4,000 year return period) in combination with steep topography and local soil characteristics.

4.2 Distribution of Landslides

The distribution of landslides on the northern slopes of Dooncarton Mountain is shown on Drawing No. 001.

Most of the landslide scars are coincident with the upper slope area where the peat cover gives way to thin peaty soil, and the slope is at its steepest and greatest convexity. The landslides on the northern slopes of Dooncarton Mountain occurred at the following locations:

(1) On the steep Barnacuille (Barnacuillew) ridge, and

(2) On the steep slopes below the summit of Dooncarton Mountain above Glengad.

On the steep northeast-facing upper slopes of Barnacuille Mountain there are some 13 separate failure zones. Several landslide scars merged to form larger scars. The debris from these landslides involved peat, soil and boulders that passed directly downslope initially overland and in placed became channelised.

On the steep northwest facing slopes of Dooncarton Mountain above Glengad there were an estimated 10 significant landslides and several small gully washouts. The landslide debris was again of mostly peat, soil and boulders that passed directly downslope initially overland and in placed again became channelised.

Due to the shape of the ridges that form Dooncarton Mountain, and the susceptibility of the slopes to failure, the landslides were essentially located to the northwest and northeast and this was also the direction of the associated landslide debris trails (see Drawing No. 001).

Views of landslide debris trails taken by AGEC, who carried out part of the investigations into the landslides, a few days after the event, are shown in Figures 4.1 and 4.2.

The landslides had less of an effect on the fields and area to the north of the mountain where the Landfall Valve Facility (LVI) is located. The LVI site was not affected by failure debris.

4.3 Risk Zoning

The Tobin (2003) investigation and risk zoning of the area reflects the northwest and northeast failure pattern. As such, the risk plan shows that the high/medium risk is on the upper slopes on the northwest and northeast of the mountainside (Drawing No. 001).

The Tobin risk zoning shows that the north side of the mountain is a low risk area, and the area of the LVI was at such a distance from the mountain that it was not considered within the risk zoning.

The high/medium risk slopes are associated with for example where debris is still present on the slope. Mitigation measures were identified for these areas (kinetic fencing and reconstructed berm on the hillside).

Mitigation measures elsewhere on the mountain side include re-constructing of the existing berm and improved drainage. The berm proved effective at containing debris during the failure event.

4.4 Landslide Risk to the LVI and Pipeline Route

The effects of the landslides on the environment can be divided into the following:

(1) Debris impact and accumulation on the mid and upper mountain slopes.
(2) Erosion of watercourses on the lower slopes due to passage of fluidised debris.

Debris impact/accumulation is associated with mostly the upper and mid-slopes of the mountainside. Debris impact would not be considered a direct risk to the LVI taking into account the LVI's distance from the failed slopes, slope morphology, slope inclination and previous failure tracks.

Erosion of watercourses occurred on the lower slopes. The LVI is not sited on a water course.

Where the pipeline passes under watercourses there is the potential for the watercourse bed to become eroded. The pipeline would need to be protected from possible erosion of the soil cover below a watercourse. In this regard, the pipeline is placed 1.6m depth below a watercourse bed plus a concrete slab is placed over the pipeline.

The depth of burial below a watercourse will be determined on the lowest level of the bed.

If there remains a possibility that further erosion of the watercourse could occur in the lifetime of the pipeline then the pipe can be buried deeper and/or the extent of the concrete protection increased.

It is recognised that several streams pass close to the proposed tunnel works areas at the mouth of the bay. If there is significant rainfall then these streams may flood and could affect the works with a risk to the safety of workers. In siting the tunnel works, due regard to the location of these streams will be given and protective works where required, eg temporary bunding will be carried out to ensure that at all times workers are fully protected from such an event.

FIGURES

Figure 4.1 View Looking Down Landslide Debris Trail

Figure 4.2 Views Looking Up Landslide Debris Trail

5 CONFIRMATORY GROUND INVESTIGATION

5.1 Introduction

Whilst there has been extensive ground investigations carried out along the route in some areas investigation has been limited due to access difficulties. The need for confirmatory investigation, which would be considered good practise, was recommended in the geotechnical reports (EIS, Volume 2, Appendix M2 and M3):

Report on Corrib Onshore Pipeline Peat Stability Assessment, and

Report on Corrib Onshore Pipeline Geotechnical Assessment of Stone Road Construction in Peat Areas

The confirmatory ground investigation included in this submission provides further details of the confirmatory ground investigation given in the above reports.

Confirmatory ground investigations are to be carried out prior to construction to re-confirm the ground conditions and to identify any departures from the design ground model.

5.2 Confirmatory Ground Investigation

The above reports identified in situ testing, undisturbed sampling and associated laboratory testing to be carried out within the peat and underlying mineral soil. Furthermore the reports identified locations where confirmatory testing was particularly required, this included but is not limited to Rossport (Commonage) as a whole and particularly within machine cut peat areas (for example ch 86,250 to 86,600, ch 87,300 to 87,450), in the vicinity of the series of shallow bog pools (ch 87,200 to 87,300) and the area of deep peat around ch 88,100.

(1) In situ testing and sampling

This shall comprise the use of a mechanical vane (such as Geonor H-10) and where safe access is possible, and without undue disturbance to the peat, Cone Penetration Tests (CPT). It is noted that a CPT rig was able to track safely around the peat land at the gas terminal site.

In situ testing will be carried out within peat and, where penetration is possible, within the underlying mineral soil.

Boreholes will be required to retrieve continuous thin-wall samples from the peat, and where penetration is possible, in the underlying mineral soils. Within the underlying soils, Standard Penetration Tests (SPT) and U100 sampling will be carried out as appropriate.

Geophyscial surveying along the pipeline route using Ground Penetrating Radar (GPR) and 2-D resistivity surveying. The GPR would identify the peat/mineral soil interface and would also provide an indication of the presence of peat pipes.

The frequency of in situ testing and sampling is outlined as follows:

- Mechanical vanes (with CPT where access possible) at 50m spacings or less along the line including testing along cross-sections (upslope and downslope) in critical locations, such as those identified from the qualitative assessment.
- Borehole locations will be determined by access considerations. Boreholes at 100m spacings or less. With further boreholes at critical locations.
- Geophyscial surveying along the line of the pipeline with cross-sections (upslope and downslope) at 100m centres. Further survey lines off-set and parallel from the centre line will also be required at critical locations, such as those identified from the qualitative assessment.

(2) Laboratory Testing

This will involve the following, but will not be limited to, testing on undisturbed and disturbed samples retrieved from the peat and underlying mineral soil:

- Strength tests. Triaxial testing including quick and with consolidated undrained with pore water pressure measurement. Direct and simple shear box testing. Testing will be on both peat and principally weak mineral soils (where present) with shear box testing on interface surfaces between peat-mineral soil.
- Index tests. Natural moisture content, loss on ignition, Atterberg tests and particle size determination. Testing will be on both peat and mineral soils.
- Consolidation tests. 1-D consolidation or Rowe cell for peat and where appreciable thickness of relatively weak mineral soils (where present)

(3) Instrumentation and monitoring

This shall comprise groundwater monitoring from borehole (or probe) piezometer installations. Inclinometers may also be installed within boreholes in deeper peat areas. Construction monitoring will be determined following review of construction method statements but will likely include, but not be limited to, continued monitoring of groundwater installation (and inclinometers where installed), level monuments on stone road, movement posts alongside works.