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ENNISCORTHY FLOOD DEFENCE SCHEME

GROUND INVESTIGATION CONTRACT

INTERPRETATIVE REPORT

NO. P16087

Client: Wexford County Council
County Hall,
Carricklawn,
Wexford,
Co. Wexford.

Engineer: Mott MacDonald Consulting Engineers,
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21/12/2017

REPORT CONTROL SHEET

Employer	Wexford County Council					
Employer's Representative	Mott MacDonald Consulting Engineers					
Project Name	Enniscorthy Flood Defence Scheme – Ground Investigation Contract					
Report Name	Enniscorthy Flood Defence Scheme – Ground Investigation Contract – Interpretative Report					
Project Number	P16087					
This Report Comprises of	RCS	TOC	Text	No. of Appendices	Drawings	Electronic data
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D01	Draft	GH	NL	19.04.2017
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1 INTRODUCTION

1.1 SCOPE OF WORKS

In June 2016, Mott MacDonald, Consulting Engineers acting as the Employer's Representative and lead Consultant and on behalf of the Client, Wexford County Council commissioned Priority Geotechnical (PGL), to carry out a ground investigation contract at the locations of proposed works for Enniscorthy Flood Defence Scheme (FDS). Roughan & O'Donovan Ltd. are acting Consultant Engineers on behalf of Wexford County Council for the design of a new bridge structure and associated roadworks that will be constructed on the southern approach to the Town.

The purpose of the investigation is to provide detailed geotechnical and hydrogeological information for the provision and design of a flood relief scheme.

The scope of the ground investigation (Specification and Related Documents for Ground Investigation in Ireland (Engineers Ireland, October 2006), Eurocode 7- Geotechnical Design Part 2, ground investigation and testing (BS EN 1997-2: 2007) and the relevant British Standards (BS 5930 (1999) Code of Practice for Site Investigation +A2:2010 and BS 1377, Method of Tests for Soil for Civil Engineering Purposes, *in situ* Tests Parts 1 to 9)) specified by Mott MacDonald, as Tendered (initially) comprised of the following:

- 35Nr. Trial pits excavations;
- 27Nr. Slit trench excavations;
- 70Nr. Cable percussion boreholes;
- 51Nr. Rotary core boreholes;
- 2Nr. Dynamic probes;
- 11Nr. pavement cores;
- All associated sampling;
- *In-situ* testing, but not limited to, standard penetration testing, California bearing ratio (CBR), vane testing, plate loading tests and variable head permeability tests;
- Geophysical investigation; Seismic Refraction and Electrical Resistivity;
- *In situ* Piezo-cone penetrometer tests;
- 25Nr. groundwater monitoring wells;
- Laboratory testing of soil and rock samples;
- Chemical analysis and contaminant testing and
- Factual reporting, together with the production of digital *.ags data.

The scope of the works was amended to include for additional investigation works. The fieldworks were carried out between 10th June, 2016 and 28th September, 2016 and again between the 09th August and the 08th September, 2017. The final works as completed and reported on are detailed in Section 3.2.

1.2 REPORTING

This geotechnical interpretative report (GIR ref: P16087_RP_INT_F01) has been produced by Mr. G. Hayes, B.E. M.Eng.Sc. CEng, Geotechnical Specialist on behalf of PGL. This geotechnical interpretative report presents a summary of the factual records of the fieldwork with respect to the site investigation works contract for the proposed Enniscorthy Flood Defence Scheme and the geotechnical interpretation of same. This report should be read in conjunction with the factual report and its appendices.

Static cone penetration testing, CPT was carried out by In Situ Site Investigations Ltd (UK) on behalf of PGL. A report; “Enniscorthy Flood Relief Static Cone Penetration Testing Factual Report” (ref: 1163055 November, 2016) is presented in APPENDIX B of the factual report. This geotechnical interpretative report shall be read in conjunction with the factual report ref: P16087_RP_F01 and its appendices.

A non-intrusive geophysical survey was undertaken by Minerex Geophysics Ltd (MGX). A report titled “Enniscorthy Flood Defence Scheme, Ground Investigation, Geophysical Survey” (ref: MGX *6092f-005, January 2017)* is presented in APPENDIX C of the factual report.

A second non-intrusive geophysical survey was undertaken by PGL along 3.36km of the Slaney River through Enniscorthy town. A report titled “Enniscorthy River Geophysical Survey” is presented in APPENDIX C of the factual report.

No responsibility can be held by PGL for ground conditions between exploratory locations. The exploratory logs provide for ground profiles and configuration of strata relevant to the investigation depths achieved during the fieldworks. Caution shall be taken when extrapolating between such exploratory locations. No liability is accepted for ground conditions extraneous to the exploratory locations.

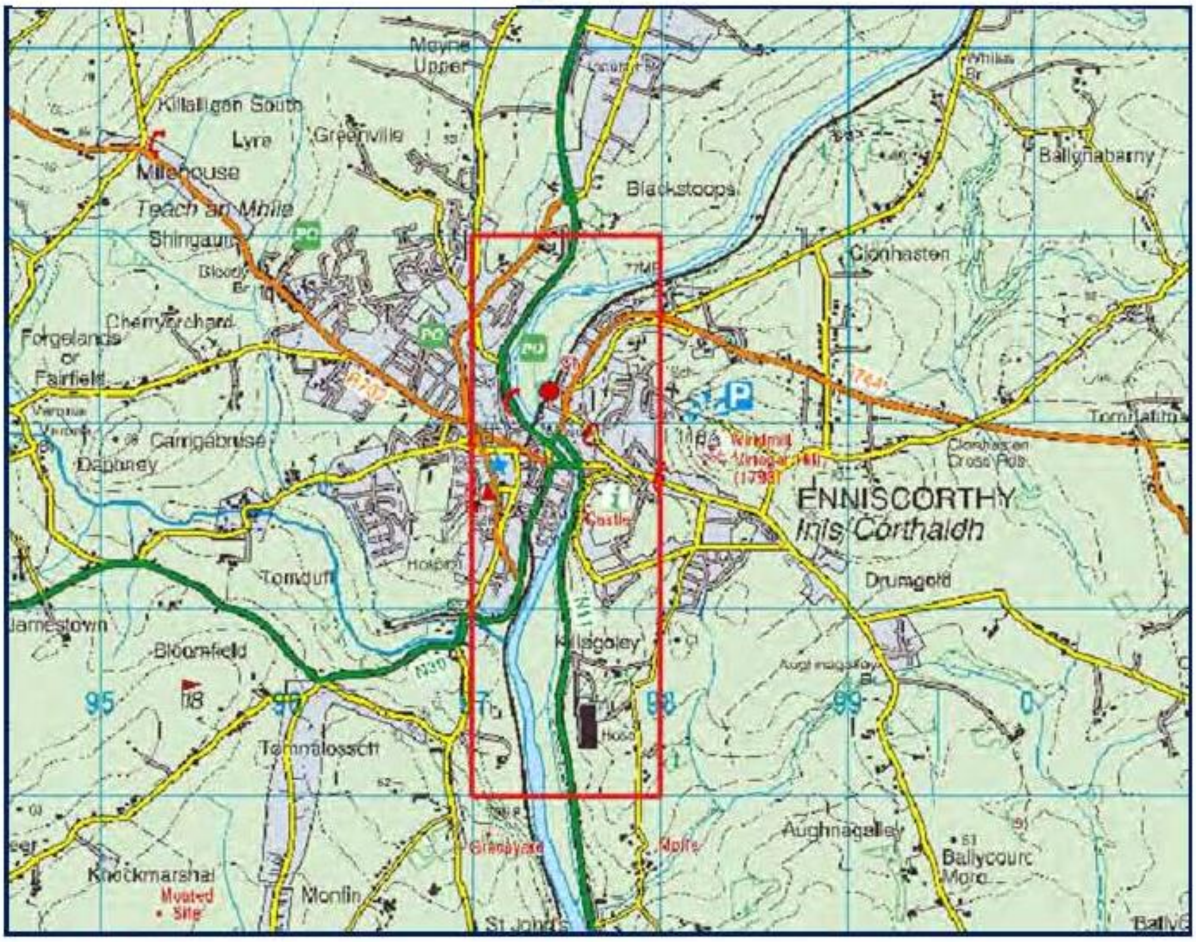
This report may be subject to change where further information becomes available.

No account has been taken of potential subsidence or ground movement due to mineral extraction, mining works or karstification below or in proximity to the site, unless specifically addressed.

This report has been prepared for the Employer and their Representative as outline, herein. The information should not be used without their prior written permission. PGL accepts no responsibility or liability for this document being used other than for the purposes for which it was intended.

1.3 SITE LOCATION & DESCRIPTION

The town of Enniscorthy is located on the banks of the River Slaney. The locations of the exploratory investigations are on both public and private lands, in and around the town of Enniscorthy, County Wexford.



It is noted that access may be restricted due to the presence of invasive alien species; Himalayan balsam and Japanese knotweed, within the site.

1.4 PUBLISHED GEOLOGY

The published Geology maps Geological Society of Ireland, GSI, Sheet 19 1:100,000 shows the site to be underlain by Campile Formation (CA, CAiv and CAfv) described as Rhyolitic Volcanics [1] and intermediate[2] and felsic [3] volcanics (pale coloured Rhyolites and Rhyolitic Tuffs or Agglomerates) and grey/ brown Slate (Slatey Mudstones with occasional Andesitic Tuffs or Agglomerates). Dolerite (D) was also noted north of Enniscorthy being described as Basalt and Gabbro. Structural geology indicated faulting in the study area being in an N-S direction with dip 60^o to 70^o. A small intrusion of Blackstairs Type 2 Equigranular Granite (Bs2E) described as pale, fine to coarse-grained granite was also identified NE of Enniscorthy of the bend of the River Slaney. The GSI well data base indicated bedrock at depth 2m (well ref: 2913NWW013) to 10m (well ref: 2913NWW006). Historical ground investigation associated with Enniscorthy Main Drainage (ref: 1917 and 1718) identified bedrock at 0.5m to 6.0m. Historical ground investigation associated with Barrack St. Commercial development (ref: 6214) identified bedrock was at 1.9m to 3.7m. The bedrock profile was varied.

The Teagasc mapping on the GSI website shows this area of the site to be underlain by Alluvial deposits, Glacio-fluvial sands and gravels, Glacial till from lower Palaeozoic shales, Bedrock outcropping and Made ground. Historical ground investigation associated with Enniscorthy Main Drainage (ref: 1917 and 1718) indicated soft to firm sandy gravelly CLAY deposits to depths of 4.5m underlain by Gravels. Historical ground investigation associated with Barrack St. Commercial development (ref: 6214) identified made ground 1.5m to 3.3m thick underlain by firm sandy gravely Silt. The superficial deposits were variable.

[1] Fine-grained igneous.

[2] Low quartz content

[3] High Silica content.

2 FIELDWORK

2.1 GENERAL

The fieldwork was carried out in general accordance with British Standards (BS 5930 (1999) Code of Practice for Site Investigation +A2:2010 and BS 1377, Method of Tests for Soil for Civil Engineering Purposes, *in situ* Tests Parts 1 to 9). Details of the equipment and plant used are presented below.

Operation	**Equipment**	**Nominal diameter, mm**	**Flush**	**Comments**
Cable Percussion Boring	Dando 2000	200mm	N/A	Visual observations of ground and groundwater conditions. Standard Penetration Test, N values obtained, bulk disturbed and undisturbed sampling. *In-situ* permeability testing. Standpipe monitoring well construction
Rotary Coring and Open Hole Drilling	Delta Base 520 6t tracked rig	Symmetrix 131mm diameter open hole 76mm diameter core	Compressed Air mist	Standard Penetration Test, N values obtained in overburden. Visual observations of ground and groundwater conditions. Installation of standpipe monitoring wells and *in situ* permeability testing.
Trial pit and Slit trench excavations	JCB backhoe excavator; 3t (ST TPBH), 20t and 22t tracked excavator(s)	N/A	N/A	Visual observations of ground and groundwater conditions. In-situ soakaway test. Bulk disturbed sampling, Observation of pavement construction and presence of utilities; Plate loading tests.
Pavement cores	Hilti drill	100mm	Water	Pavement coring, assessment of pavement construction
Static cone testing (In Situ Ltd.)	CPT003 3.5t rig Cone S10CFIP.768	$10cm^2$ rate 2cm/s	N/A	Cone tip resistance and sleeve friction measurements and dissipation testing.
Dynamic probe	TRL	8kg/ 575mm drop ht.	N/A	Pen rate mm/blow recorded & CBR estimated.

The ‘as constructed’ exploratory locations are provided to the Ordinance Survey, Irish National Grid (ING) system of co-ordinates and elevations to Malin Head datum. These locations are shown on the Exploration Location Plans (dwg. No.: P16087-SI-0A, P16087-SI-01 to P16087-SI-05) presented in APPENDIX D of the factual report and presented in **APPENDIX 1** herein.

2.2 EXPLORATORY HOLES

The exploratory holes as completed during the ground investigation are listed in the following table(s):

SUMMARY OF EXPLORATORY HOLES

Type	**Quantity, Nr.**	**Depth Range, m bgl**	**Comments**
Trial Pit Excavations	61	0.80 – 4.50	TP001, TP002,TP002A, TP003, TP004, TP005, TP006, TP007, TP008, TP009, TP010, TP011, TP012, TP013, TP014, TP018, TP019, TP021, TP022, TP023, TP025, TP026, TP027, TP028, TP029, TP030, TP031, TP032, TP033, TP034, TP035, TP036, TP037 and TP038. TPA01, TPA02, TPA03, TPA04, TPA05, TPA06, TPA07, TPA08, TPA09, TPAB01 and TPAB02. TPBH015, TPBH016, TPBH057, TPBH058, TPBH071, TPBH106, TPBH107 and TPBH108. TPP1, TPP5 and TPP6. TP701, TP702 and TP703. SKA01 and SKA02.

Type	**Quantity, Nr.**	**Depth Range, m bgl**	**Comments**
Cable Percussion Boreholes	105	0.50 – 10.20	BH001, BH002, BH003, BH004, BH005, BH006, BH007, BH007A, BH008, BH009, BH010, BH011, BH012, BH013, BH014, BH014A, BH015, BH016, BH017, BH018, BH018A, BH019, BH019A, BH020, BH021, BH022, BH023, BH023A, BH025, BH025A, BH026, BH027, BH028, BH029, BH031, BH032, BH033, BH033A, BH034, BH035, BH036, BH037, BH038, BH039A, BH040, BH041, BH042, BH043, BH044, BH045, BH046, BH046A, BH047, BH048, BH049, BH050, BH051, BH052, BH053, BH054, BH055, BH056, BH057, BH057A, BH058, BH059, BH060, BH061, BH062, BH063, BH063A, BH064, BH065, BH066, BH067, BH068, BH071 and BH072 BH101, BH103, BH104, BH105, BH106, BH107, BH108, BH109 and BH110; BH201 and BH202; BH301, BH302, BH303, BH304, BH305, BH306 and BH307; BHA01, BHA01A, BHA02, BHA03, BHA04, BHA05 and BHA06, BHA111 and BHA112.

Type	Quantity, Nr.	Depth Range, m bgl	Comments
Rotary Boreholes	74	1.2– 25.10	RC001, RC004, RC006, RC007A, RC009, RC010, RC012, RC014, RC014A, RC015, RC017, RC018A, RC020, RC022, RC023A, RC025, RC026, RC028, RC030, RC032, RC033, RC033A, RC033B, RC035, RC036, RC037, RC039A, RC041, RC043, RC045, RC046A, RC047, RC048, RC049, RC050, RC051, RC052, RC053, RC054, RC055, RC056, RC057A, RC058, RC059, RC060, RC061, RC062, RC063A, RC064, RC065, RC066, RC067, RC068, RC069, RC070 and RC072. RC101, RC103, RC104, RC105, RC106, RC107, RC108, RC109 and RC110; RC201 and RC202; RC301, RC303 and RC305; RC401, RC402, RC403 and RC404. RCA111 and RCA112. RCP1, RCP3, RCP5 and RCP6.
Slit Trenches	31	0.10 – 1.8	ST001A, ST001B, ST002, ST003, ST003A, ST004, ST005, ST006, ST007, ST008, ST009, ST010, ST011, ST012, ST013, ST014, ST015, ST016, ST017, ST018, ST019, ST020A, ST020B, ST021, ST022, ST022A, ST023, ST024, ST025, ST026 and ST027.
Pavement cores	11	0.10 to 0.30	PC01, PC02, PC03, PC04, PC05, PC06, PC07, PC08, PC09, PC10 and PC11.

2.3 SAMPLING

Six hundred and sixty two (662) bulk disturbed samples (B), two hundred and ninety one (291) small disturbed samples (D), eighteen (18) undisturbed 100mm dia. samples (U/U100), eleven (11) pavement cores, thirty three (33) grab samples (GB00) in the bed of the River Slaney at fifteen (15) locations at depths 0.0m, 0.5m, 1.0m and 1.5m and 587.6lin.m rotary core were recovered from the exploratory holes in accordance with Geotechnical Investigation and Sampling – Sampling Methods and Groundwater Measurements (EN ISO 22475-1:2006).

Thirty one (31) environmental soil samples (ES, WAC and ENV) were recovered from exploratory holes. Environmental samples were placed immediately in air-tight containers, which were filled to the top of the sample container. The sample suite consisted of: 2No. small disturbed samples (D) not less than 1.0kg, 2No. 250g amber glass sample containers and 2No. 60g amber glass sample containers.

2.4 GROUNDWATER MONITORING

Groundwater was recorded when encountered/ observed during boring and trial pit excavations over a period of 20 minutes, noting any changes that may occur. Groundwater levels were also monitored at start and end of drilling shifts. Groundwater is presented in a table in Section 5.

It should be noted that the normal rate of boring may not permit the recording of equilibrium groundwater levels for any one groundwater water strike where casing may exclude low volume flows as the borehole progresses. Groundwater conditions observed in the borings or pits are those appertaining to the period of the investigation. Groundwater levels may be subject to diurnal, seasonal and climatic variations and can also be affected by drainage conditions, tidal variations etc. The groundwater regime should be assessed from standpipe well installations, where available.

Arisings backfill to borehole

GRAVEL backfill to installation/ rotary borehole

BENTONITE grout backfill to rotary boreholes/ installation

uPVC slotted pipe

2.5 IN SITU TESTING

Standard Penetration Tests, N values, were typically carried out in the boreholes using the 60^{o} solid cone (CPT) in place of the standard split barrel sampler. The Standard Penetration Test was carried out in accordance with Geotechnical Investigation and Testing, Part 3 Standard penetration test, BS EN ISO 22476-3:2005+A1:2011. The data was presented on the relevant logs in APPENDIX A of the factual report.

Static cone penetration tests, CPT were carried out by In Situ, Site Investigations Ltd (UK) on behalf of PGL using CPT003, 3.5t tracked rig with cone $10cm^2$ at a rate of $2cms^{-1}$ to a maximum depth of 7.38m. The report "Enniscorthy Flood Relief Static Cone Penetration Testing Factual Report" (ref: 1163055 November, 2016) is presented in APPENDIX B of the factual report.

The non-intrusive geophysical survey comprised of continuous 2D Electrical Resistivity (herein referred to as ERT), Seismic Refraction profiling. The survey fieldwork was carried out by Minerex (MGX) on behalf of PGL. A separate report has been produced and is presented in APPENDIX C of the factual report.

In situ variable falling head permeability tests were carried out in 115mm, 131mm and 200mm diameter borehole casing(s). *In-situ* permeability tests were carried out in accordance with BS5930: 1999, Section 4: Cl. 25.4, within the superficial deposits over a duration of up to one (1) hour, as detailed on the borehole logs, APPENDIX A of the factual report. The processed test data was presented on the relevant borehole logs presented in APPENDIX A of the factual report. The shape or intake factor, *f* was derived from the condition at the base of the borehole at the test depth and test geometry as per Hvorslev (1951).

$$k = \frac{A}{fd} \frac{\log_e (H_0 / H_t)}{t}$$

Generally for the tests the casing was withdrawn 0.5m giving an L/d between 1 to 13; so a value of mean k ($k_H = k_V$) was measured providing a shape factor, *f* of 7 to 20. In some instances the casing was not withdrawn (L/d =0) and so a shape factor of 2.75 was provided. It shall be noted in some cases the permeability did not allow a hydraulic head to be developed and so no assessment of permeability was made.

In situ hand vanes were carried out in trial pit excavations using 33mm vane. The data was presented on the relevant logs (TP005 and TP007) in APPENDIX A of the factual report.

Infiltration tests were carried out in general accordance with the BRE Digest 365, 2007 Soakaway Design Standards. A single cycle of infiltration was undertaken and not the three cycles outlined in the standard. Soakaway pits failed to drain in full over the test duration up to 7hrs. The data from the testing was presented in APPENDIX A of the factual report**,** accompanying the relevant exploratory records.

TRL dynamic probes (8kg drop weight, 575mm drop height) were carried out in pit excavations to establish *in situ* California bearing ratio, CBR to depths between 0.19m bgl to 1.88m bgl. The data from the testing was presented in APPENDIX A of the factual report, with the relevant trial pit records.

SUMMARY OF IN-SITU TESTING

Type	Quantity	Remarks
Standard penetration test, N_{SPT} value	629Nr. BH (476) RC (176)	Uncorrected Nspt 0 – 139 (including refusals N presented a numerical value 50); APPENDIX A of the factual report
In situ permeability test	30Nr.	$2.7 \times 10^{-3} ms^{-1}$ to $2.4 \times 10^{-7} ms^{-1}$ APPENDIX A of the factual report
Infiltration/ Soakaway tests	6Nr.	$f = 2.5 \times 10^{-6} ms^{-1}$ to $2.1 \times 10^{-4} ms^{-1}$ APPENDIX A of the factual report
Geophysical survey	5628lin.m 2D res 5995lin.m Seismic	MGX *6092f-005, January 2017* APPENDIX C of the factual report
Plate loading tests	3Nr.	APPENDIX A of the factual report
Dynamic probing (TRL)	6Nr.	TP003, TP004, TP018, TP019, TP021 and TP023; APPENDIX A of the factual report
In situ hand vane, undrained shear strength	4Nr.	TP005 (33mm) and TP007 (33mm). 10kPa to 29kPa; APPENDIX A of the factual report
In situ CPT tests (static cone penetration test)	24Nr.	Report Ref: 1163055 November, 2016; APPENDIX B of the factual report
Dissipation tests	3Nr.	

3 LABORATORY TESTING

All samples were transported to Priority Geotechnical's laboratory in Midleton, Co. Cork examined, logged and prepared for scheduled testing. Laboratory testing was proposed by PGL, being reviewed and approved by Mott MacDonald. Testing was carried out by PGL, in accordance with BS1377 (1990), Methods of test for soils for civil engineering purposes and the ISRM suggested methods for rock characterisation, testing and monitoring. Specialist chemical testing was undertaken by Chemtest Ltd. (UK) on behalf of PGL. Specialist soil and rock testing was carried out by GSTL Ltd (UK) on behalf of PGL. The laboratory test results were presented in APPENDIX C of the factual report. A summary of tests undertaken were detailed below.

SUMMARY OF LABORATORY TESTING UNDERTAKEN – SUPERFICIAL DEPOSITS

SOILS		
Type	**No.**	**Remarks**
Natural Moisture Content	241	4.4% to 209%
Atterberg Limit	173	Liquid limit, LL 26% to 163% Plastic limit, PL 19% to 106% and NP non-plastic soils Plasticity index, PI 7 to 72
Particle Size Distribution (grading)	326	See APPENDIX D of the factual report
Grading by hydrometer	97	
pH SO_4 water soluble SO_4 acid soluble	63 59 38	5.5 to 11.2 <0.01g/l to 0.65g/l <0.01% to .03%
Chloride (extractable)	22	<20mg/kg to 34mg/kg
Chloride water soluble	39	<0.01g/l to 0.046g/l
Loss on ignition	93	0.4% to 59%
Organic content	04	<0.4% to 2.1%
Proctor compaction (Moisture content/dry density relationship)	41	Optimum moisture content 6% to 29% Maximum dry density 1.21Mg/m^3 to 2.01Mg/m^3
Moisture condition value, MCV	70	0 – 10.4
MCV moisture content relationship	26	APPENDIX D of the factual report BH101 0.0m; TP004 0.5m, 1.5m; TP006 0.5m; TP007 1.5m; TP008 0.5m; TP012 0.5m; TP013 0.5m; TP014 0.5m, 1.5m; TP027 1.0m; TP028 0.5m; TP029 0.5m, 2.0m, 3.0m; TP031 2.6m; TP033 1.5m; TP035 0.5m; TPA03 0.5m, 1.6m; TPA04 0.3m, 1.0m; TPA05 0.5m, 2.0m; TPA06 1.5m; TPA08 1.5m.
California Bearing Ratio, CBR	23	<1% to 28%

SOILS		
Type	**No.**	**Remarks**
CBR Moisture content/dry density relationship	01	TPA002 0.5m
Direct shear (60mm sq shear box)	04	14^{o} to 40^{o} 0kPa to 31kPa
Direct shear (100mm sq shear box)	02	11^{o} to 13^{o} 1kPa to 9kPa
1D consolidation, oedometer	01	BHA06 1.0m, APPENDIX D of the factual report
Permeability – remoulded sample in Triaxial cell by back pressure	14	9.6×10^{-10} ms^{-1} to 2.4×10^{-8} ms^{-1} BHA051.0m, 2.0m; TPA01 1.2m; TPA02 1.5m; TPA05 2.0m; TP029 0.5m, 1.3m, 2.0m 3.0m; TP032 0.5m, 1.5m; TP035 0.5m; TPA03 1.6m; TPA06 0.6m
Permeability – undisturbed sample in Triaxial cell by back pressure	01	5×10^{-10} ms^{-1} BHA06 1.0m
Permeability –remoulded sample – Constant head	01	9×10^{-6} ms^{-1} TP008 1.5m
Pyrite suite	02	APPENDIX D of the factual report
Waste Acceptance Criteria, WAC	47	APPENDIX D of the factual report

SUMMARY OF LABORATORY TESTING UNDERTAKEN – SOLID GEOLOGY

ROCK		
Type	**No.**	**Remarks**
Uniaxial Compressive Strength (UCS)	43	4.7MPa to 84.1MPa
UCS with Young's Modulus and Poisson's Ratio	03	77.7GPa to 81.8GPa 0.21 to 0.24
Point Load Test (I_{P50})	194	(0.11MPa) 0.54MPa to 9.9MPa Values <0.5MPa considered unrealistically low)
L.A. Abrasion	02	27 and 31

4 GROUND CONDITIONS

The full details of the ground conditions encountered are provided for on the exploratory records accompanying this report. The records provide descriptions, in accordance with BS 5930 (1999) +A2: 2010 and Eurocode 7, Geotechnical Investigation and Testing, Identification and classification of soils, Part 1, Identification and description (EN ISO 14688-1: 2002),– Identification and Classification of Soil, Part 2: Classification Principles (EN ISO 14688-2:2004) and Identification and Classification of Rock, Part 1: Identification & Description (EN ISO 14689-1:2004) of the materials encountered, in situ testing and details of the samples taken, together with any observations made during the ground investigation.

5 GROUNDWATER CONDITIONS

Groundwater was encountered during cable percussion drilling, during rotary drilling and excavations between 1.6m bgl and 9.20m bgl. Details of the ground water and installations are presented graphically on the relevant exploratory logs within APPENDIX A of the factual report and are summarised below. See also section 3.4 for general details.

Eighteen (16) number 50mm diameter standpipe well, installations were constructed to allow for groundwater monitoring.

SUMMARY OF GROUNDWATER INFORMATION OBTAINED DURING SITE WORKS

Location	Depth Strike, m bgl	Depth Water after 20min.	Comments	Standpipe installation Y/N
BH003	6.00	5.00	-	N
BH005	2.80	2.80	-	N
BH006	5.40	5.40	-	N
BH007A	4.10	3.50	-	N
BH008	3.00	2.00	-	N
BH009	3.20	3.20	-	N
BH014	2.50	2.30	-	N
BH028	6.50	5.80	-	N
BH031	3.20	3.20	-	N
BH032	4.70	3.70	-	N
BH034	3.80	3.80	-	N
BH035	2.80	2.00	-	N
BH036	3.20	2.70	-	N
BH037	3.80	2.95	-	N
BH038	4.20	4.00	-	N
BH039A	3.20	3.20	-	N
BH041	5.00	5.00	-	N
BH045	4.50	3.00	-	N
BH050	4.50	4.15	-	N
BH051	5.20	5.00	-	N
BH053	1.60	1.30	-	N
BH054	3.80	3.50	-	N
BH055	3.70	3.70	-	N
BH056	4.00	3.65	-	N
BH062	4.00	4.00	-	N
BH064	3.75	3.75	-	N
BH065	3.10	2.70	-	N
BH066	3.50	3.30	-	N
BH067	4.00	3.80	-	N
BH067	6.10	2.70	-	N
BH072	6.10	4.90	-	N
BH103	4.00	3.30	-	N
BH104	5.00	3.70	-	N
BH105	3.50	3.30	-	N
BHA03	-	-	-	**Y**

Location	Depth Strike, m bgl	Depth Water after 20min.	Comments	Standpipe installation Y/N
BHA05	2.10	1.65	-	**Y**
BHA06	2.50	2.50	-	**Y**

Location	Depth Strike, m bgl	Comments	Standpipe installation Y/N
RC004	2.6	See shift data.	N
RC006	4.7	See shift data.	N
RC007A	6.1	See shift data.	N
RC009	6.2	See shift data.	N
RC010	5.7	See shift data.	**Y**
RC012	2.7	See shift data.	**Y**
RC014	5.7	See shift data.	N
RC014A	-	See shift data.	N
RC015	7.2	See shift data.	N
RC017	5.9	See shift data.	N
RC018A	8.4	See shift data.	N
RC020	4.2	See shift data.	N
RC022	2.4	See shift data.	**Y**
RC023A	5.5	See shift data.	N
RC025	3.9	See shift data.	N
RC026	9.2	See shift data.	N
RC028	-	None encountered.	N
RC030	4.5	See shift data.	N
RC032	2.9	See shift data.	N
RC033	-	See shift data.	N
RC033A	-	None encountered.	N
RC033B	-	None encountered.	N
RC035	2.8	See shift data.	N
RC036	-	See shift data.	N
RC037	2.8	See shift data.	**Y**
RC039A	6.1	See shift data.	N
RC041	3.1	See shift data.	**Y**
RC043	2.3	See shift data.	N
RC046	-	None encountered.	N
RC046A	-	None encountered.	N
RC047	7.6	See shift data.	
RC048	7.9	See shift data.	**Y**
RC049	1.8	See shift data.	**Y**
RC050	4.2	See shift data.	N
RC051	5.5	See shift data.	N
RC052	4.5	See shift data.	N
RC053	4.5	See shift data.	N

Location	Depth Strike, m bgl	Comments	Standpipe installation Y/N
RC054	4.2	See shift data.	N
RC055	4.5	See shift data.	**Y**
RC056	4.3	See shift data.	N
RC057	6.5	See shift data.	N
RC058	7.8	See shift data.	N
RC059	-	None encountered.	Y
RC060	-	None encountered.	N
RC061	-	None encountered.	N
RC062	2.7	See shift data.	N
RC063A	--	None encountered.	**Y**
RC064	3.9	See shift data.	N
RC065	3.98	See shift data.	N
RC066	4.4	See shift data.	**Y**
RC067	3.4	See shift data.	N
RC068	-	None encountered.	N
RC069	-	None encountered.	N
RC070	-	None encountered.	N
RC072	9.1	See shift data.	N
RC101	4.3	See shift data.	N
RC101	5.7	See shift data.	N
RC103	5.2	See shift data.	N
RC104	4.5	See shift data.	N
RC105	4.5	See shift data.	**Y**
RC106	6.9	See shift data.	N
RC107	4.2	See shift data.	N
RC108	4.6	See shift data.	N
RC109	-	None encountered.	N
RC110	-	None encountered.	N
RC201	6.9	See shift data.	N
RC202	7.4	See shift data.	N
RC301	3.8	See shift data.	**Y**
RC303	3.8	See shift data.	N
RC305	-	None encountered.	N
RC401	7.6	See shift data.	N
RC402	7.5	See shift data.	N
RC403	7.5	See shift data.	N
RC404	7.9	See shift data.	N
RCA111	8.2	See shift data.	N
RCA112	3.75	See shift data.	Y
RCP1	-	Drilled over water.	N
RCP3	-	Drilled over water.	N
RCP5	-	Drilled over water.	N
RCP6	-	Drilled over water.	N

Location	Depth water strike, m bgl	Comments
SKA01	3.0	-
SKA02	2.2	-
ST001A	-	None encountered.
ST001B	-	None encountered.
ST002	-	None encountered.
ST003	-	None encountered.
ST003A	-	None encountered.
ST004	-	None encountered.
ST005	-	None encountered.
ST006	-	None encountered.
ST007	-	None encountered.
ST008	-	None encountered.
ST009	-	None encountered.
ST010	-	None encountered.
ST011	-	None encountered.
ST012	-	None encountered.
ST013	-	None encountered.
ST014	-	None encountered.
ST015	-	None encountered.
ST016	-	None encountered.
ST017	-	None encountered.
ST018	-	None encountered.
ST019	-	None encountered.
ST020A	-	None encountered.
ST020B	-	None encountered.
ST021	-	None encountered.
ST022	-	None encountered.
ST022A	-	None encountered.
ST023	-	None encountered.
ST024	-	None encountered.
ST025	-	None encountered.
ST026	-	None encountered.
ST027	-	None encountered.
TP001	-	None encountered.
TP002	-	None encountered.
TP002A	-	None encountered.
TP003	2.0	2.00m: Slow flow rate.
TP004	-	None encountered.
TP005	3.5	3.50m: Steady flow rate.
TP006	3.3	3.30m: Steady flow rate.
TP007	1.8	1.80m: Steady flow rate.
TP008	1.9	1.90m: Steady flow rate.
TP009	1.8	1.80m: Steady flow rate.
TP010	1.8	1.80m: Steady flow rate. Rising to 1.40m (10min).
TP011	1.6	1.60m: Steady flow rate.
TP012	2.8	2.80m: Steady flow rate.
TP013	3.0	3.00m: Steady flow rate.
TP014	3.0	3.00m: Steady flow rate.
TP018	-	None encountered.
TP019	-	None encountered.

Location	Depth water strike, m bgl	Comments
TP021	-	None encountered.
TP022	-	None encountered.
TP023	-	None encountered.
TP025	-	None encountered.
TP026	-	None encountered.
TP027	2.85	2.85m: Steady flow rate.
TP028	2.8	2.80m: Steady ingress.
TP029	3.3	3.30m: Steady flow rate.
TP030	2.5	2.50m: Trickle flow rate.
TP031	3.3	3.30m: Steady flow rate.
TP032	2.6	2.60m: Steady flow rate.
TP033	3.3	3.30m: Slow flow rate.
TP034	1.7	1.70m: Slow flow rate.
TP035	3.0	3.00m: Fast flow rate.
TP036	-	None encountered.
TP037	-	None encountered.
TP701	2.2	2.2m: Trickle flow rate.
TP701	2.4	2.4m: Slow flow rate.
TP702	2.3	2.3m: Slow flow rate.
TP703	-	None encountered.
TPA001	2.0	2.00m: Slow flow rate.
TPA002	3.3	3.30m: Steady flow rate.
TPA003	2.5	2.50m: Steady flow rate.
TPA004	2.5	2.50m: Steady flow rate.
TPA005	2.9	2.90m: Steady flow rate.
TPA006	2.9	2.90m: Steady flow rate.
TPA007	2.4	2.40m: Slow flow rate.
TPA007	3.2	3.20m: Steady flow rate
TPA008	3.3	3.30m: Slow flow rate.
TPA009	3.3	3.30m: Slow flow rate.
TPAB01	-	None encountered.
TPAB02	-	None encountered.
TPBH015	-	None encountered.
TPBH016	-	None encountered.
TPBH057	-	None encountered.
TPBH058	-	None encountered.
TPBH071	-	None encountered.
TPBH106	-	None encountered.
TPBH107	-	None encountered.
TPBH108	-	None encountered.
TPP1	-	None encountered.
TPP5	-	None encountered.
TPP6	-	None encountered.

SUMMARY OF GROUNDWATER MONITORING

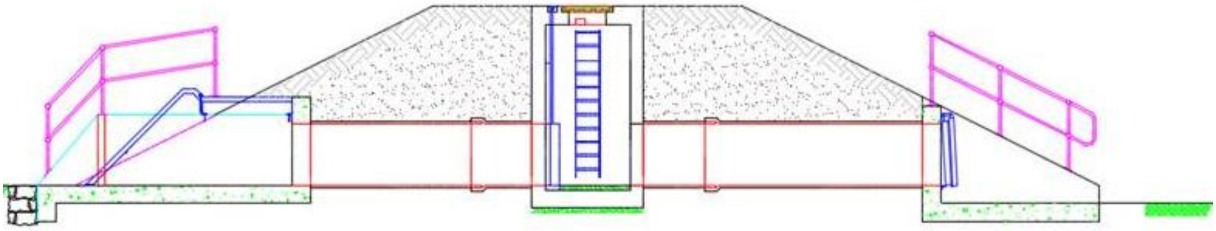
Location	Groundwater strike, m bgl	Response zone		Date constructed, dd/mm/yy	Groundwater monitoring, m bgl			
		from	*to*		11/08/16	24/08/16	13/09/16	dd/mm/yy
BHA01	none			10/08/2016	-	2.05	2.02	
BHA02	none			09/08/2016	-	-	2.30	
BHA03	none	4.8	6.8	08/08/2016	-	-	-	
BHA04	none			05/08/2016	-	-	1.90	
BHA05	2.1	2.0	4.0	17/08/2016	-	1.20	-	
BHA06	2.5	2.0	7.0	12/08/2016	-	0.95	-	
RC010	5.7	2.0	7.0	11/08/2016	-	-	2.00	
RC012	2.7	5.0	14.0	23/08/2016	-	-	2.57	
RC022	2.4	2.0	7.0	16/08/2016	-	-	2.74	
RC037	2.8	2.0	6.0	26/08/2016	-	-	2.38	
RC041	3.1	3.0	6.0	13/09/2016	-	-	-	
RC048	7.9	5.0	8.0	26/09/2016	-	-	-	
RC049	1.8	2.0	8.0	06/09/2016	-	-	2.22	
RC055	4.5	8.6	18.6	01/07/2016	2.42	2.20	-	
RC063A	8.4	2.0	8.0	09/09/2016	-	-	5.7	
RC066	4.4	3.0	5.0	07/07/2016	2.2	1.85	-	
RC105	4.5	2.6	4.6	12/07/2016	2.08	1.92	-	
RC301	3.8	3.0	6.0	15/09/2016	-	-	-	

6 GEOTECHNICAL REVIEW

The flood defences are primarily located along the banks of the River Slaney. Drawings have been provided for reference, being those presented by Mott MacDonald (MMD) as specimen design details. In the case of the Bridge Crossing drawings have been presented, being design option 7A, provided by Roughan O'Donovan (RO'D), 2015.

The Enniscorthy Flood Defence Scheme is envisaged to comprise the following non-exhaustive list of works:

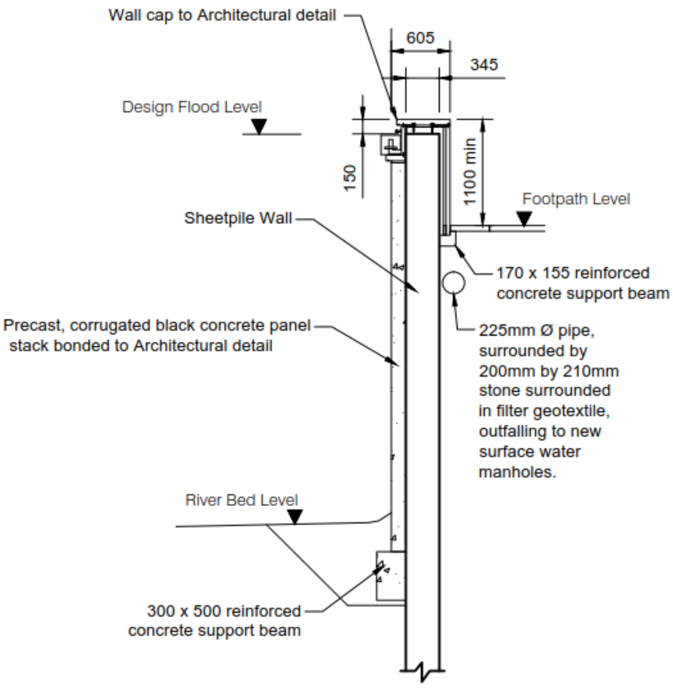
- Channel dredging and realignment;
- Construction of flow control structures;



MMD detail, 2017, pipe 450mm to 600m diameter, central manhole chamber

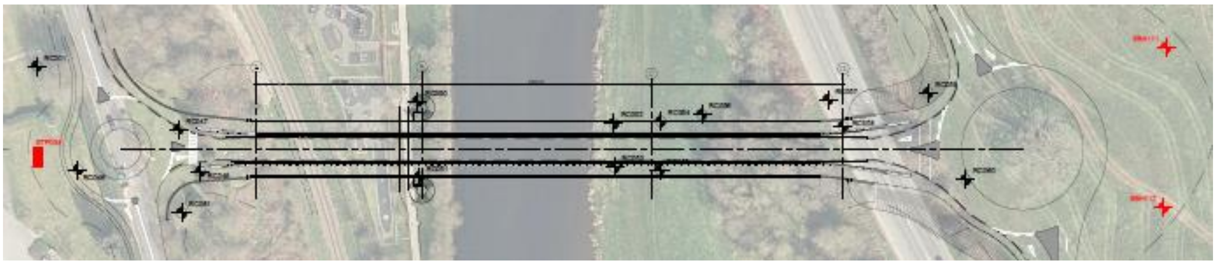
- Construction of flood walls and embankments;

Preliminary design details were reviewed and are referenced. Corrugated concrete panelling and cut-off sheet pile walls are proposed along the river bank(s).



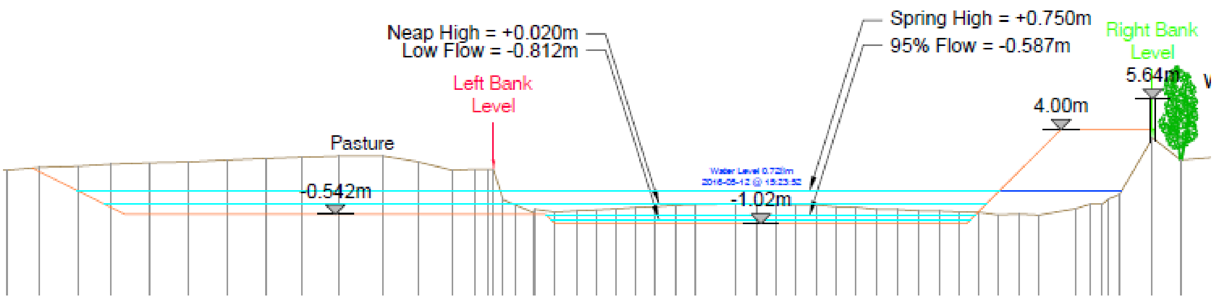
MMD Wall Type 3, 2017

- Underpinning of the existing Railway and Enniscorthy bridge structures;
- Raising of roads and relocation of services;
- Removal of the Seamus Rafter Bridge;
- Construction of new bridge structures, one (1) road and one (1) pedestrian bridge; approach junctions, roundabouts and ancillary works.

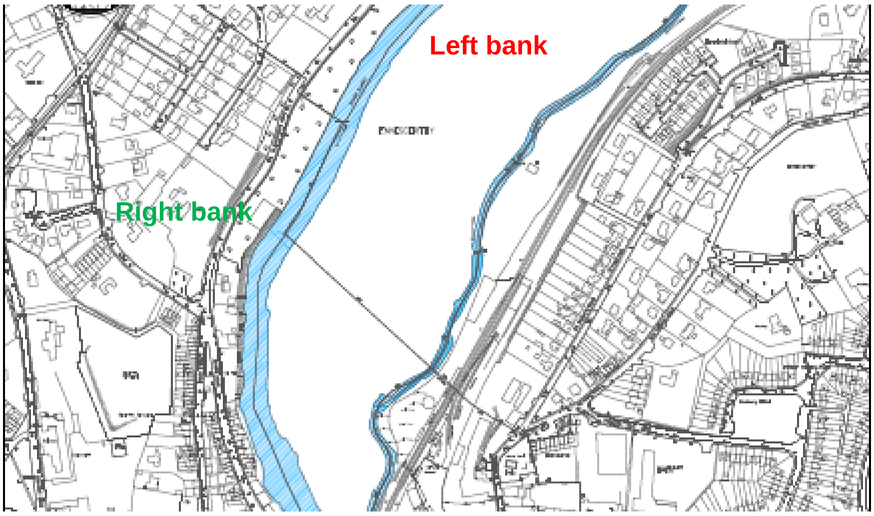


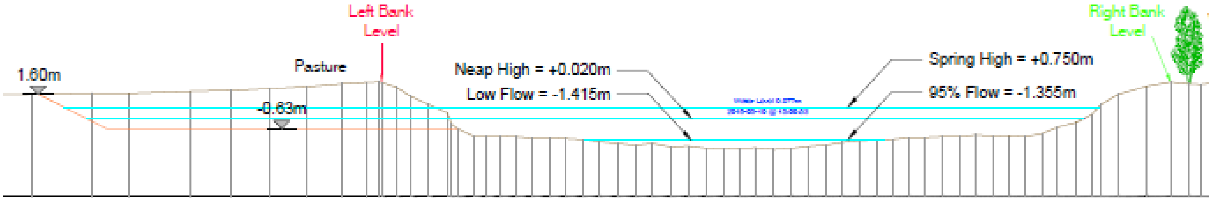
3 Span Bridge crossing (RO'D Drawing ref: Option 7A 008)

It is proposed to widen the river channel, left bank (left hand side looking in the downstream direction) by lowering the adjacent lands by approximately 2.5m to 3m.

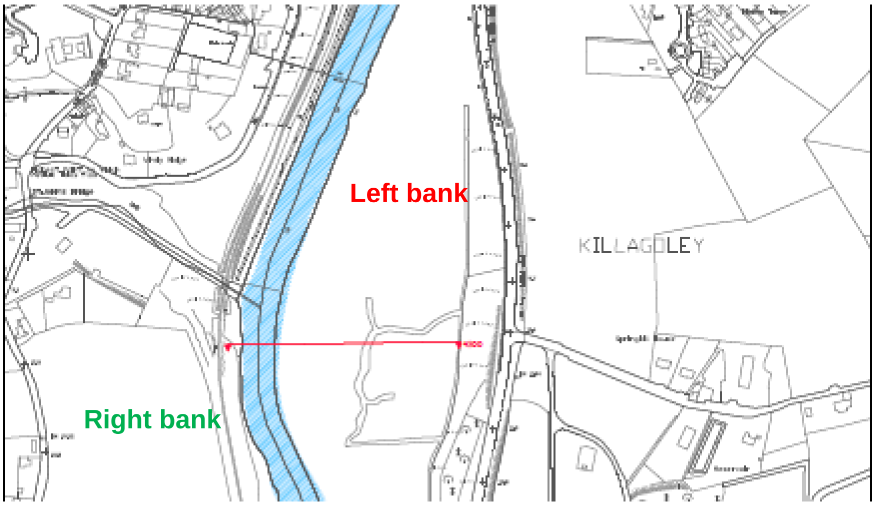


Channel Cross sections (MMD, January 2017, information only P01.1)

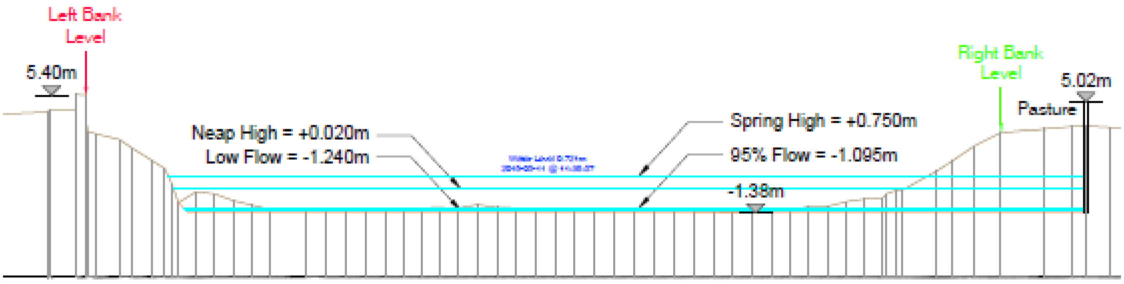




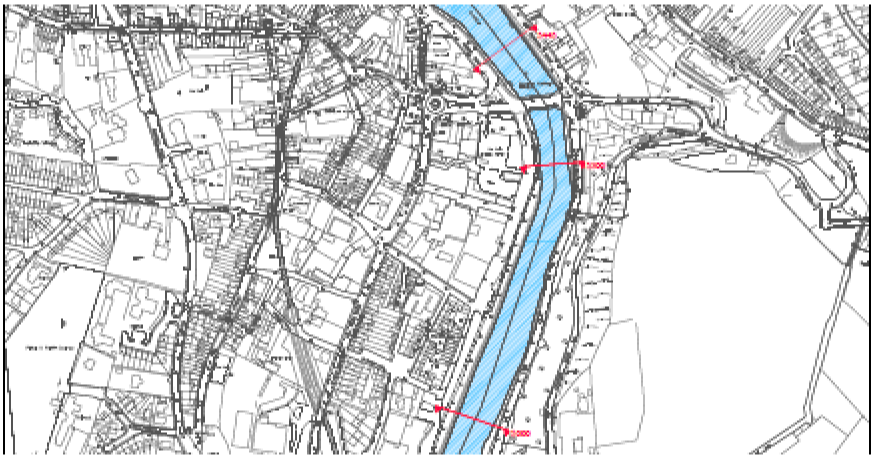
Channel Cross sections (MMD, January 2017, information only P01.1)



It is proposed to widen the river channel by lowering the adjacent lands by approximately 4m to 6m.



Channel Cross sections (MMD, January 2017, information only P01.1)



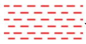
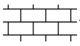
6.1 GROUND MODEL(S) AND CHARACTERISTIC PROPERTIES

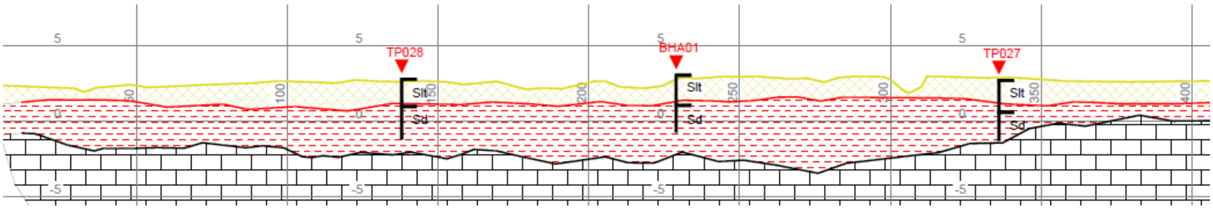
6.1.1 Right bank (SKA01 – TPA01) ch6+900m to ch6+500m

Channel, CH1[4] dredging and realignment, EX1[4]/ EX2[4], along with some embankment works are proposed along this section.

Topsoil, 200mm to 500mm thick was encountered. This was underlain by mixed alluvial deposits. Soft to firm slightly sandy SILT deposits to a depth between 2.3m below existing ground level (bgl) to 2.7m bgl. PEAT and peaty deposits were encountered 600mm thick at TP027, TP028 and SKA01. The SILT and PEAT deposits were underlain by granular deposits; silty very gravelly SAND and (slightly) silty very sandy GRAVEL with varied Cobble and low Boulder content to a depth 4.0m bgl to 4.6m bgl.

Standard penetration test, Nspt data indicated firm SILT and medium dense granular deposits. BHA01A extended to 4.6m bgl (-1.57mOD) terminating after one (1) hour chiselling. RC001 in the vicinity (left bank) indicated weak highly weathered/ fractured Slatey-Mudstone at 4.3m bgl. Boulders were noted below 4.0m bgl.

The geophysical survey (R4, S2 and S3) indicated approximately 4m to 5m of superficial deposit; soft (Silt) overlying medium dense (Granular deposits ). Bedrock (Slatey-Mudstone ) is assumed between -0.02mOD (SKA01) to -2.5mOD.



Groundwater was encountered at depths 2.2m bgl to 3.3m bgl (-0.33mOD to 0.35mOD). The static groundwater level was measured at BHA01 at 2.02m bgl (1.01mOD). Groundwater is assumed confined within the granular deposits below 2.2m bgl.

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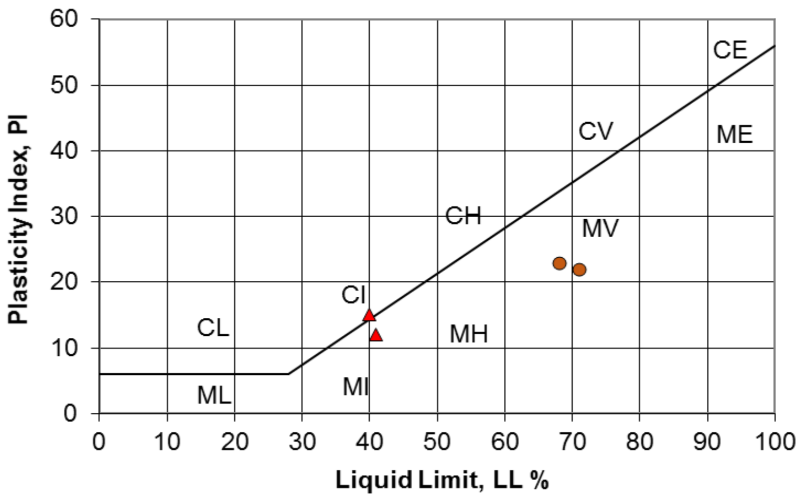
[4] CH1 etc.- schedule reference- Mott MacDonald Scheme Proposal, 2425/DR/001 to 2425/DR/007, 2012.

Seasonal fluctuations are anticipated but have not been defined. *In situ* groundwater data loggers are recommended to monitor variations in groundwater levels.

The SILT deposits were of mixed and varied plasticity; intermediate plasticity (MI) and 'high to very high' plasticity (MH-MV). The SILT was of low organic content (loss on ignition 3.0% and 3.8%). The sand fraction was 18% to 57% with 1% to 2% gravels and 33% to 73% silt fraction.

Summary of plasticity data



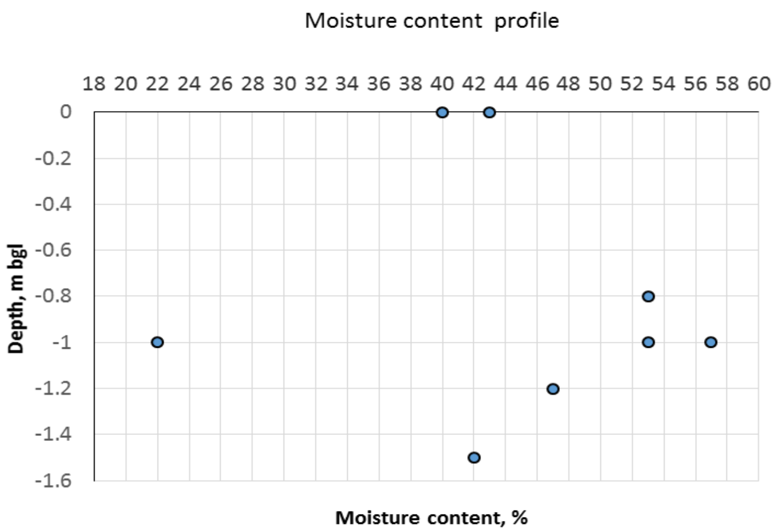
Natural moisture content, w ranged between 22% and 57%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.9 to 1.9 indicative of very soft to firm deposits (C504 Engineering in glacial deposits). This correlated with the *in situ* tactile assesment.

In situ plate loading tests indicated an undrained shear strength of the order 50kPa, correlating with Nspt data (10 – 11) at 1.0m bgl. With a plasticity index, PI 12 – 23; a factor f_1 = 5 to 6 was such that:

$$Cu(kPa) = Nspt \times f_1 \text{ (Stroud, 1975).}$$

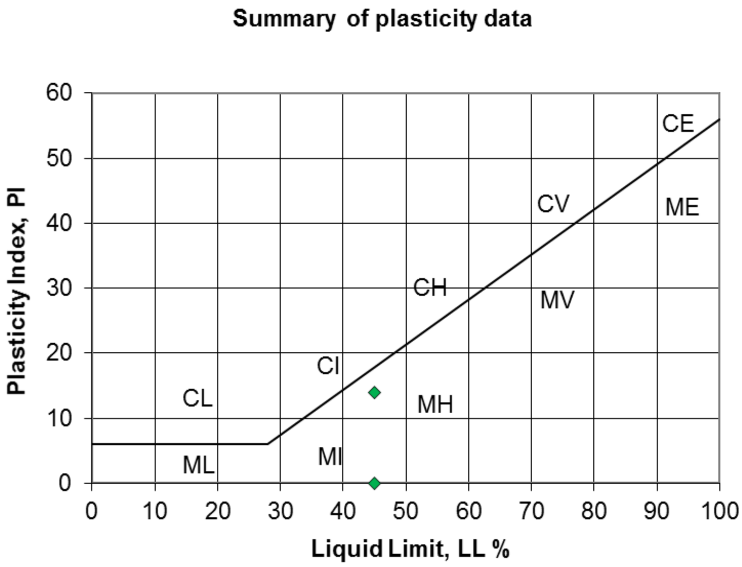
This yielded undrained shear strengths of 50kPa to 66kPa, describing firm deposits (BS5930 1999). With Nspt values 18 to 24 (values 38 and refusal attributed to coarse particles and transition to bedrock), allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^o is expected of the granular deposits. Plasticiy data, PI 12 – 23 suggested an angle of friction of 28° for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

Taking a characteristic Nspt value of 10 in the sandy SILT, an unfactored stiffness modulus (Young's modulus, E) of 10MPa to 15MPa is expected of the firm sandy SILT (PI 12 – 23, Stroud, 1975). In the absence of further data on the soft SILT a stiffness modulus of 5MPa is proposed. Compressibility of the SILT is expected to be variable being low compressibility (MI) to high compressibility (MH-MV).



Permeability in the SILT and PEAT was measured as 2.7 x 10^{-4} ms^{-1} to 2.7 x 10^{-5} ms^{-1} (SKA01 and SKA02) being indicative of medium to low permeability (C113, Control of groundwater for temporary works). Particle size d_{10} has not been determined (<0.063mm) but would be suggestive of a lower permeability. Permeability tests on recompacted deposits (TPA01) yielded a value 1.6x 10^{-9} ms^{-1}.

The SAND desposit were characterised by 6% to 26%, silt fraction, 0% to 54% Gravel and 39% to 74% sand fractions. The GRAVEL was characterised by 1% to 4% silt 17% to 35%, sand and 61% to 69% gravels and a medium Cobble content (6% to 15%). The Silt fraction was of intermediate plasticity (MI).

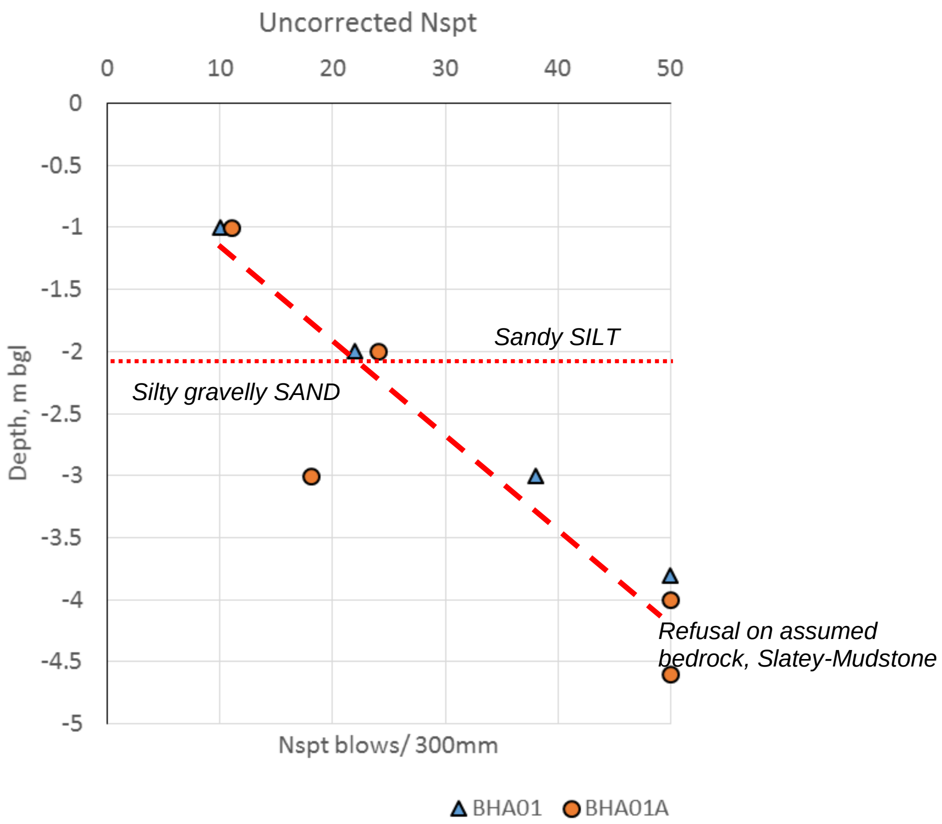


Taking a characteristic Nspt value of 20 in the SAND/ GRAVEL deposits a stiffness modulus of 50MPa is expected of the silty very gravelly SAND/ silty very sandy GRAVEL (Menzenbach, 1967).

Particle size(s) d_{10} of 1.18mm to 0.212mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-4} ms^{-1}. This described medium to high permeability (C113). Particle size(s) d_{10} of 0.150mm was measured in the gravelly SAND indicative of a permeability of the order 10^{-4} ms^{-1}. This described medium permeability (C113).

The PEAT deposits were not presently characterised.

The bedrock, assumed Slatey-Mudstone was not presently characterised.



The geotechnical hazard in this area is considered to be the (long-term) erosion of the fine SILT and Sandy deposits. Appropriate erosion control along the realigned river bank/ embankment shall be provided. The form of the bank (erosion) protection shall be determined by the re-profiled river bank cross-section, EX1/ EX2. Hard (riprap or other) and soft (geotextiles and planting) options are considered suitable.

> *It is recommended that further trial pit excavations or window sampling be undertaken so as the PEAT can be sampled and characterised. It is also recommended to assess d_{10} for the SILT where additional samples may be taken. Additional sampling will also allow for further environmental assessment of excavated deposits. Three (3) locations are proposed in the vicinity of existing exploratory locations; TP028, SK01 and SK02.*

> *Seasonal fluctuations in groundwater levels are anticipated but have not been defined. In situ groundwater data loggers are recommended to monitor variations in groundwater level at BHA01.*

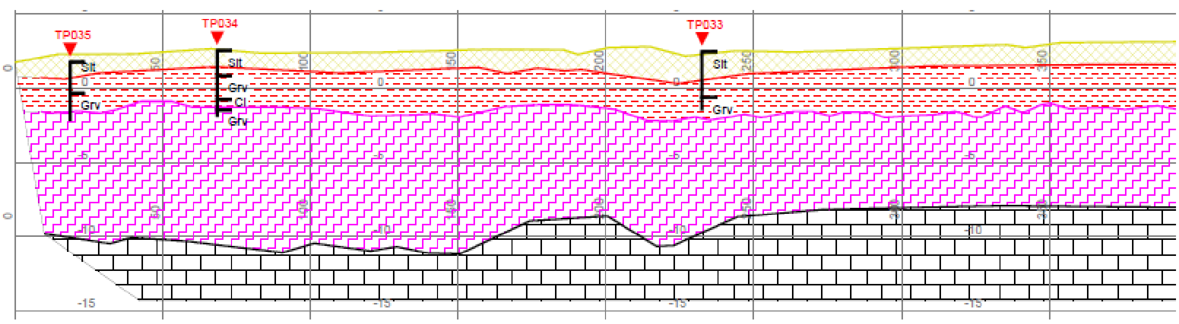
6.1.2 Left bank (TPA02 – TP35) ch6+500 to ch5+750

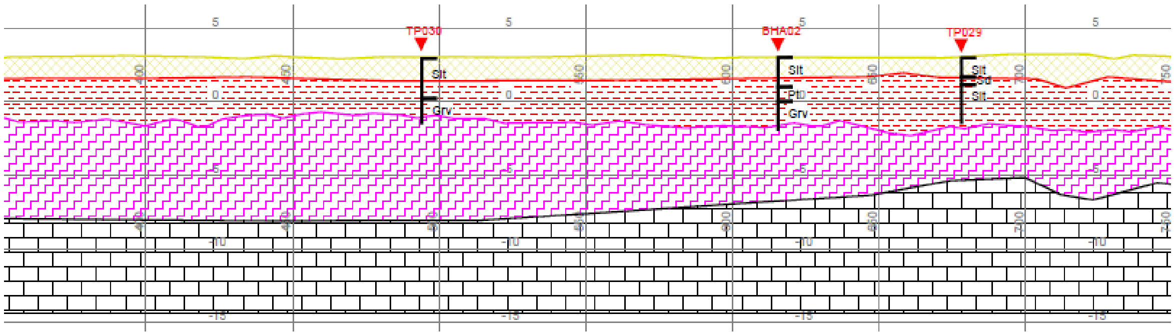
Channel, CH1 realignment and dredging along with embankment works, EX4 are proposed along this section.

This section of the left bank of the River Slaney above Enniscorthy Bridge was such that Topsoil, 400mm to 500mm thick was encountered. This was underlain by mixed alluvial deposits. Soft to firm slightly sandy SILT/ slightly gravelly sandy SILT deposits to a depth between 1.5m bgl to 3.1m bgl. The SILT deposits were underlain by granular deposits; silty (very) sandy GRAVEL and (very) silty (very) gravelly SAND with low to medium Cobble content and low Boulder content to a depth 4.5m bgl. A thin PEAT layer 100mm thick was encountered at TPA008 between 3.7m bgl to 3.8m bgl separating the SILT and GRAVEL deposits. The SILT increased in thickness locally at TPA008 being encountered to a depth 3.7m bgl. A PEAT layer was also encountered at BHA02 between 2.0m bgl to 3.0m bgl.

Standard penetration test, Nspt data indicated firm SILT (Nspt 10) and medium dense granular deposits (Nspt 18 – 28). BH004 extended to 4.8m bgl and BH005 to 5.8m bgl, both terminating after one (1) hour chiselling. RC004 indicated weak highly weathered/ fractured Slatey-Mudstone at 5.2m bgl (-3.38mOD). Boulders were noted below 2.0m bgl and 4.0m bgl within the granular deposits.

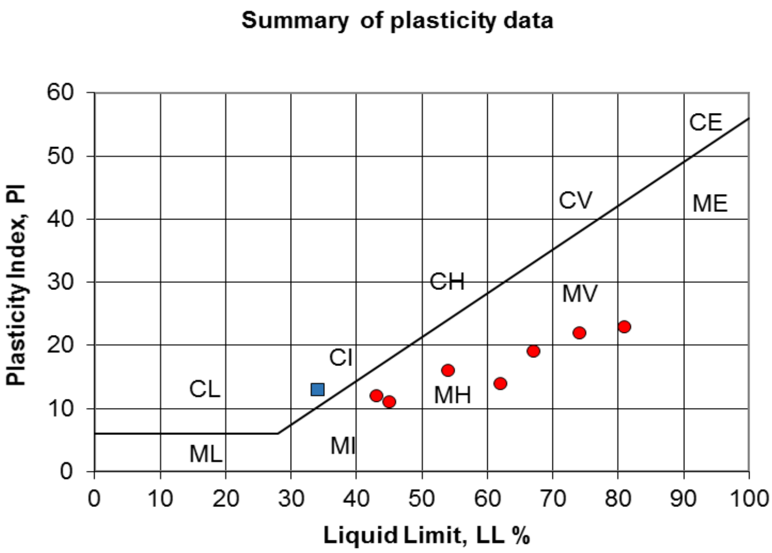
The geophysical survey (R3[I] and S6) indicated approximately 4m to 5m of superficial deposit; soft (Silt) overlying medium dense (Granular deposits). A zone of dense, weak weathered rock ( ) 4m to 7m thick overlay bedrock (Slatey-Mudstone) below – 10.0mOD.





The geophysical survey data indicated the variable weathering profile of the Slatey-Mudstone further highlighted by the low RQD, 0%. The weathered bedrock was below -5.0mOD correlating with RC004.

Groundwater was encountered at depths 1.7m bgl to 3.3m bgl (0.85mOD to -1.18mOD). The static groundwater level was measured at BHA02 at 2.3m bgl (0.73mOD). Groundwater is assumed confined within the granular deposits below. Seasonal fluctuations are anticipated but have not been defined. *In situ* groundwater data loggers are recommended to monitor variations in groundwater levels.

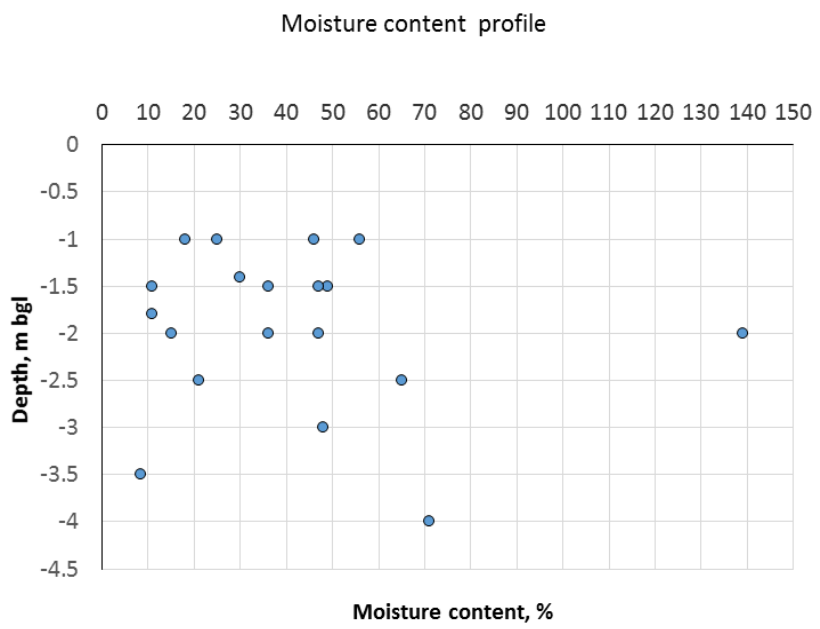


The SILT deposits were of mixed and varied plasticity; intermediate to very high (MI to MV). CLAY of intermediate plasticity (CI) was encountered at TP034 at 3.5m bgl. The SILT was of low organic content (loss on ignition 0.7% and 4.4%; organic content 1.4% and 2.1%). The sand fraction was 24% to 51% with 1% to 22% gravels and 42% to 76% silt fraction.

Natural moisture content, w ranged between 9% and 71% and a single value of w = 139% at BHA02, 2.0m bgl were measured. The ratio of natural moisture content to plastic limit (w/PL) was 0.2 to 1.6 indicative of stiff and very soft deposits (C504). This correlated with the *in situ* tactile assesment and where the ratio was <1.0 high the low moisture content (11% - 18%) were atributed to sand fractions.

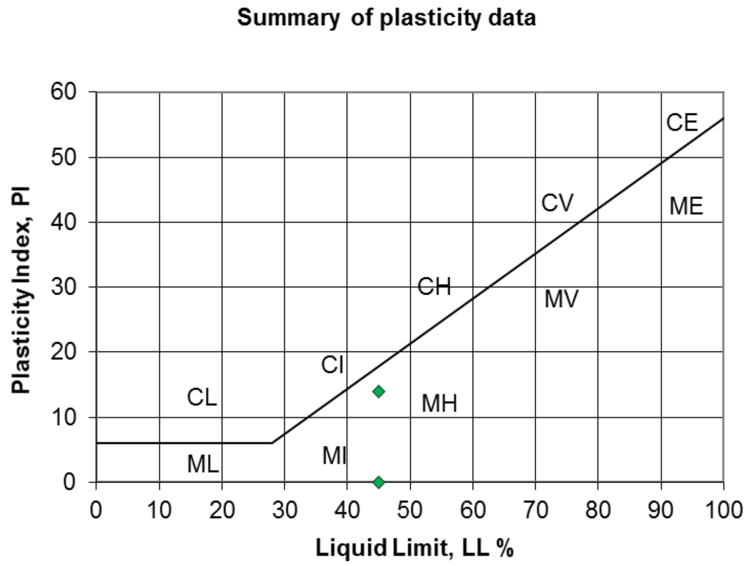
In situ plate loading tests indicated an undrained shear strength of the order >50kPa correlating with Nspt data (10) at 1.0m bgl. With a plasticity index, PI 11 – 23; a factor f_1 = 5 to 6 (Stroud, 1975) was such to yield undrained shear strengths of 50kPa to 60kPa, describing firm deposits (BS5930 1999). Plasticiy data, PI 11 – 23 suggested an angle of friction of 28^o for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

Compressibility of the SILT is expected to be variable being low compressibility (MI) to high compressibility (MH-MV).



Taking a characteristic Nspt value of 10 in the sandy SILT, an unfactored stiffness modulus (Young's modulus, E) of 10MPa to 15MPa is expected of the firm sandy SILT (PI 11 – 23, Stroud, 1975). In the absence of further data on the soft SILT a stiffness modulus of 5MPa is proposed. Dynamic probing may be considered to further characterise the soft/ loose deposits.

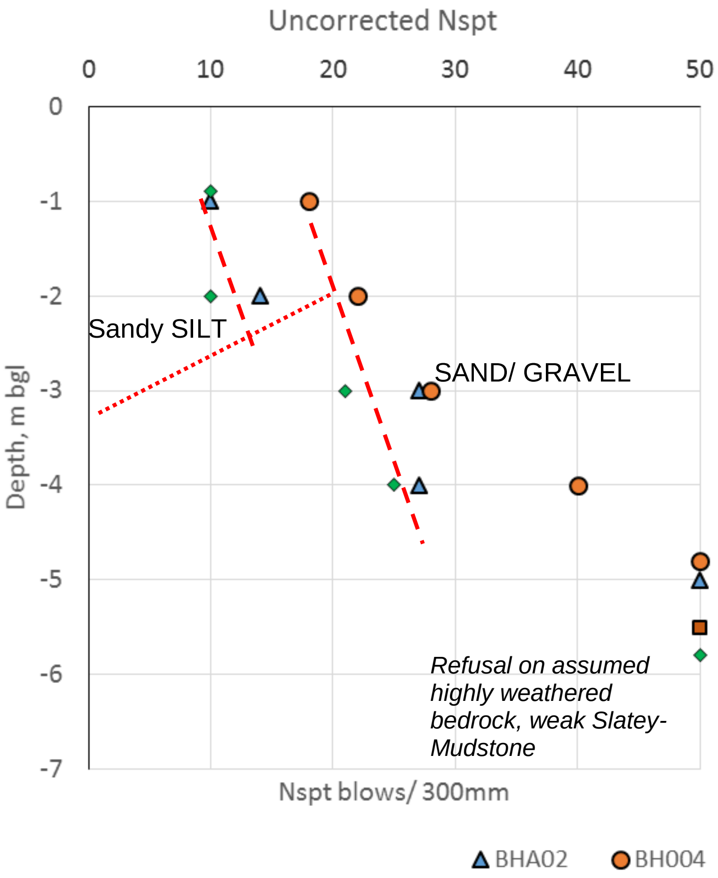
Permeability in the SILT was determined based on particle size d_{10} being <0.002mm giving a value of the order $10^{-8}ms^{-1}$, indicative of very low permeability (C113, Control of groundwater for temporary works). Permeability tests on recompacted deposits (TP029,TP033 and TPA06) yielded values 2.68x 10^{-8} ms^{-1} to 8.7 x 10^{-10} ms^{-1}.



The silt fraction in the granular deposits was of intermediate plasticity (MI). Natural moisture content, w for the granular deposits typically ranged between 8% and 9%. A value of 32% was measured at 2.9m in TP029 and is considered to characterise the Silt where 2.9m was the transition from the Silt to the Gravel. Similarly a value of 18% was measured at 1.0m in TP033 and is considered to characterise the Silt. The SAND desposit were characterised by 6% to 33% silt fraction, 2% to 22% Gravel and 53% to 85% sand fractions an dmedium Cobble content (19%). The GRAVEL was characterised by 1% to 8% silt, 10% to 43%, sand and 41% to 73% gravels and a medium to high Cobble content (15% to 31%).

With Nspt values 18 to 28 (values 40 and refusal attributed to coarse particles), allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^{o} to 36^{o} is expected of the granular deposits.

Taking a characteristic Nspt value of 18 in the SAND/ GRAVEL deposits a stiffness modulus of 45MPa is expected of the silty very gravelly SAND/ silty very sandy GRAVEL (Menzenbach, 1967).



Particle size(s) d_{10} of 2.0mm to 0.15mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-4} ms^{-1}. This described medium to high permeability (C113). Particle size(s) d_{10} of 0.150mm to 0.007mm was measured in the gravelly SAND indicative of a permeability of the order 10^{-4} ms^{-1} to 10^{-7} ms^{-1}. This described medium to low permeability (C113). The permeability in the SAND was controlled by the silt fraction.

The PEAT deposits were characterised by extremely high plasticity (LL 163% and PI 58) and a natural moisture content 139%. A loss on ignition of 1.1% and 9.6% was measured. A single Nspt value of 14 was measured indicting some level of historical compression in the PEAT at BHA02.

The Slatey-Mudstone rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (medium strong to strong I_{P50}, 0.5MPa – 0.9MPa), fracture spacing (NI) and condition (highly weathered), Rock Quality Designation

(RQD non-intact/ 0%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 21- 25, describing Class V-IV very poor to poor Slatey -Mudstone. A geological strength index, GSI (Hoek and Brown) of 15 is assumed for the poorly interlocked highly broken rockmass. An angle of friction, $\phi = 16^o$ and cohesion 110kPa are recommended for the CV-IV, weak, highly weathered Slatey-Mudstone.

RC004 Slatey -Mudstone

The geotechnical hazard in this area is considered to be the (long-term) erosion of the fine SILT and Sandy deposits. Appropriate erosion control along the realigned river bank shall be provided. The form of the bank (erosion) protection shall be determined by the re-profiled river bank cross-section. Hard (riprap or other) and soft (geotextiles and planting) options are considered suitable.

It is recommended that further trial pit excavations or window sampling be undertaken to allow for further environmental assessment of excavated deposits. Three (3) locations are proposed in the vicinity of existing exploratory locations; TPA02, TP030 and TP034.

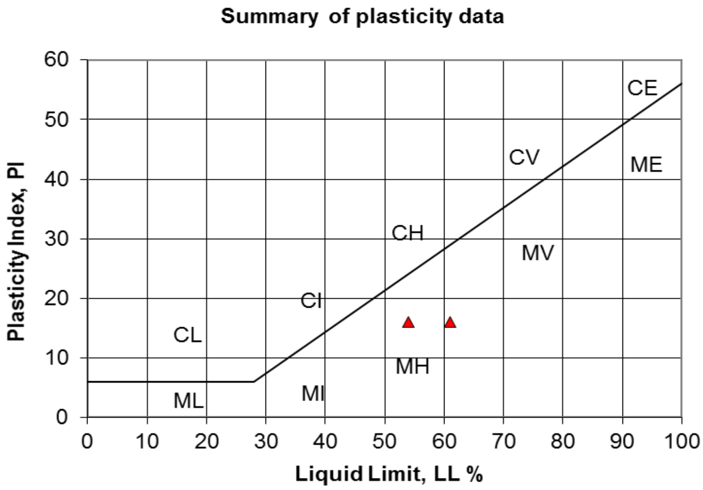
Seasonal fluctuations in groundwater levels are anticipated but have not been defined. In situ groundwater data loggers are recommended to monitor variations in groundwater level at BHA02.

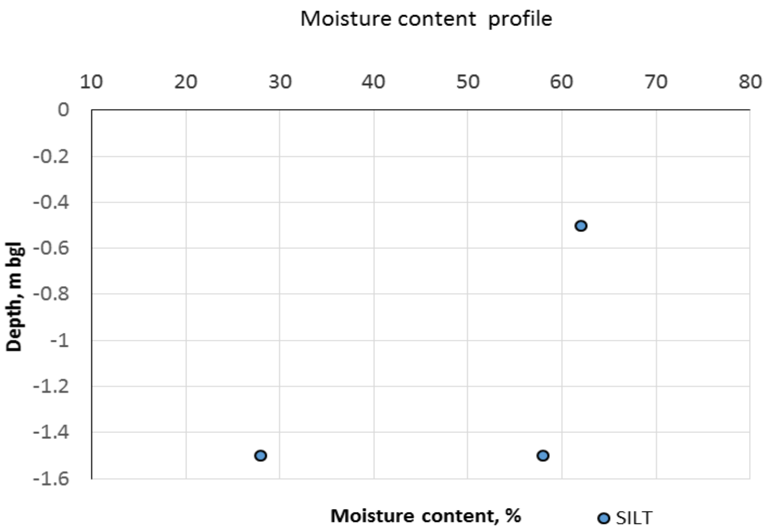
6.1.3 Deposition area DA1 (TPA03 – TPA05) ch6+700m

Topsoil was 300mm to 500mm thick underlain by slightly gravelly sandy SILT deposits to a depths between 1.7m bgl and 2.9m bgl. The SILT was underlain by sandy very silty GRAVEL deposits. This correlated with CPT piezocone data highlighting fine grained deposits to depths up to 2.2m bgl (CPT PZ001) in the area. These fine grained deposits were underlain by medium dense to dense granular deposits. CPT data indicated very soft deposits 1.0m bgl to 2.0m bgl (Nspt estimated 1 – 6).

Groundwater was at a depth 2.5m bgl to 2.9m bgl within the GRAVEL deposits.

The Silt was characterised by high plasticity (MH) and natural moisture content 28% to 62%. A loss on ignition of 59% was measured in the upper 1.0m indicating high organic content. The ratio *w*/PL (C504) indicated soft to very soft deposits to a deposits 1.4m bgl cu <20kPa – 40kPa (BS5930; 1999). Below this the deposits became stiff. Taking estimated Nspt values 1 to 6 and plasticity index PI = 16, f_1 = 6 (Stroud, 1975) such that cu = 6kPa to 36kPa.





The sand fraction was variable 48% to 68%. Particle size d_{10} was 0.001mm indicative of permeability 10^{-8} ms^{-1}, low permeability. Plate loading test PLTA01 indicated settlement up to 15mm under imposed loading associated with the deposition of excavated material up to 5m high.

The GRAVEL was characterised by low silt fraction, 2% to 3% being of a high plasticity (MH). Particle size d_{10} was 0.6mm indicative of permeability 10^{-3} ms^{-1}, medium to high permeability (C118).

The geotechnical hazard in this area is considered to be the shallow slip or shear failure at the toe of fill material. It is recommended to allow for dissipation of pore pressure in the fine grained SILT that the deposition of excavated material be staged, subject to the design top height.

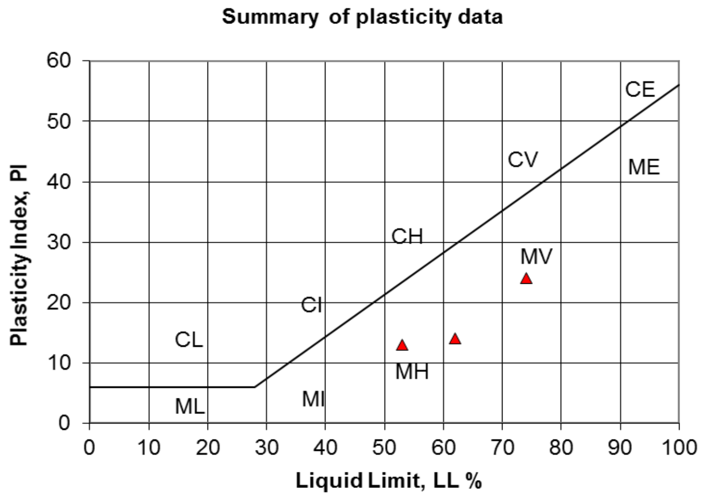
6.1.4 Deposition area DA2 (TPA08 – TP035) ch6+000m

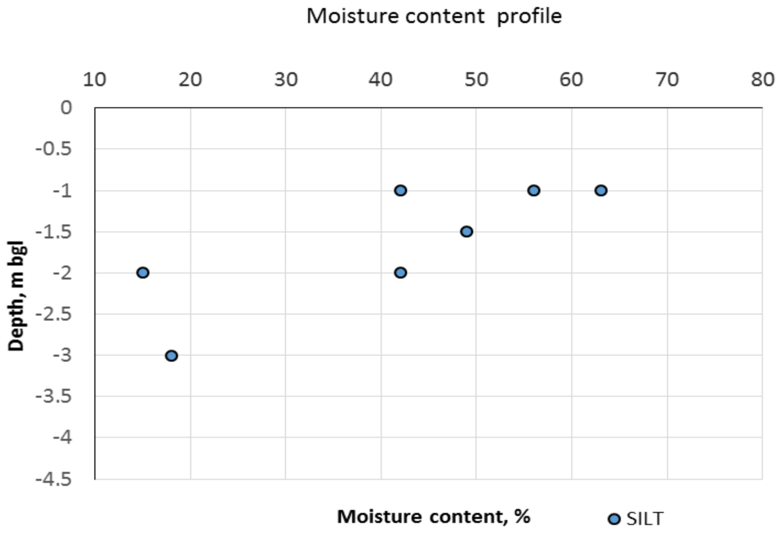
Topsoil was 500mm thick underlain by slightly gravelly slightly sandy SILT deposits to a depths between 1.0m bgl and 3.8m bgl. The SILT was underlain by slightly silty gravelly SAND/silty very sandy GRAVEL with medium Cobble content deposits to a depth 4.8m bgl to 5.8m bgl. MUDSTONE was encountered at this depth. This correlated with CPT piezocone data, CPT PZ02B in the area. These fine grained deposits were underlain by medium dense to dense granular deposits. CPT data indicated very soft to firm deposits 1.0m bgl to 2.2m bgl (Nspt estimated 2 – 10). The granular deposits were characterised by Nspt >20.

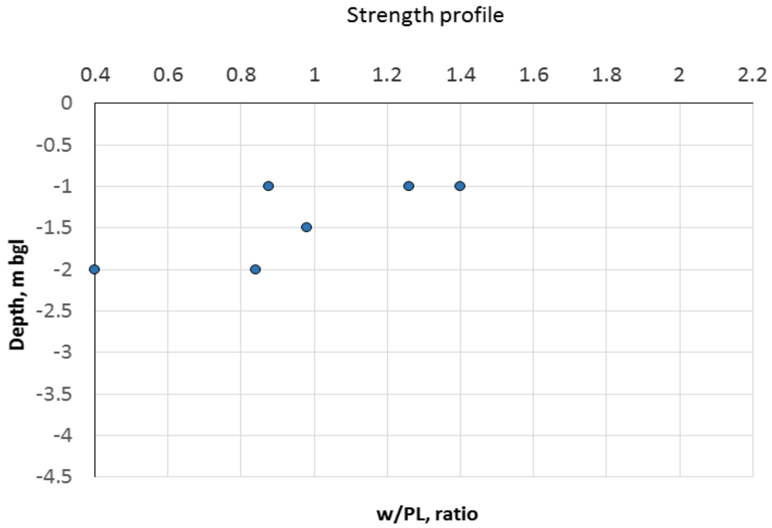
Groundwater was at a depth 2.6m bgl to 3.0m bgl within the granular deposits.

The Silt was characterised by high to very high plasticity (MH- MV) and natural moisture content 15% to 63%. A loss on ignition of 1.1 to 5.1% was measured in the upper 1.0m indicating low organic content. The ratio *w*/PL (C504) indicated soft to very soft deposits to a deposits 1.4m bgl cu <20kPa – 40kPa (BS5930; 1999). Below this the deposits became stiff. Taking estimated Nspt values 2 to 10 and plasticity index PI = 13 - 25, f_1 = 6 and 4.5 (Stroud, 1975) such that cu = 12kPa to 60kPa. Boreholes indicated Nspt 10 in the SILT deposits. The sand fraction was variable 25% to 60%. Standard penetration test, N value of 10 was measured indicating firm deposits. Particle size d_{10} was <0.001mm indicative of permeability 10^{-8} ms^{-1}, low permeability. Plate loading test PLTA02 indicated settlement up to 30mm under imposed loading associated with the deposition of excavated material up to 5m high.

The GRAVEL was characterised by low silt fraction, 5% to 15%. Particle size d_{10} was 2.0mm to 0.425mm indicative of permeability 10^{-2} ms^{-1} to 10^{-3} ms^{-1}, medium permeability (C118). Standard penetration test, N values ranged from 18 to 28 indicating medium dense deposits.


Summary of plasticity data
Plasticity Index, PI
Liquid Limit, LL %
CE
CV
ME
CH
MV
CI
CL
MH
ML
MI
0
10
20
30
40
50
60
70
80
90
100


Moisture content profile
Depth, m bgl
Moisture content, %
SILT
10
20
30
40
50
60
70
80
0
-0.5
-1
-1.5
-2
-2.5
-3
-3.5
-4
-4.5


Strength profile
Depth, m bgl
w/PL, ratio
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2

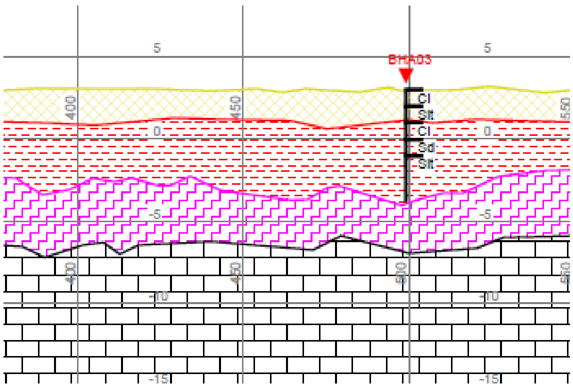
The geotechnical hazard in this area is considered to be the shallow slip or shear failure at the toe of fill material. It is recommended to allow for dissipation of pore pressure in the fine grained SILT that the deposition of excavated material be staged, subject to the design top height. It is also recommended to keep the limit of the area below TPA08 where the SILT deposits were up to 3.3m thick.

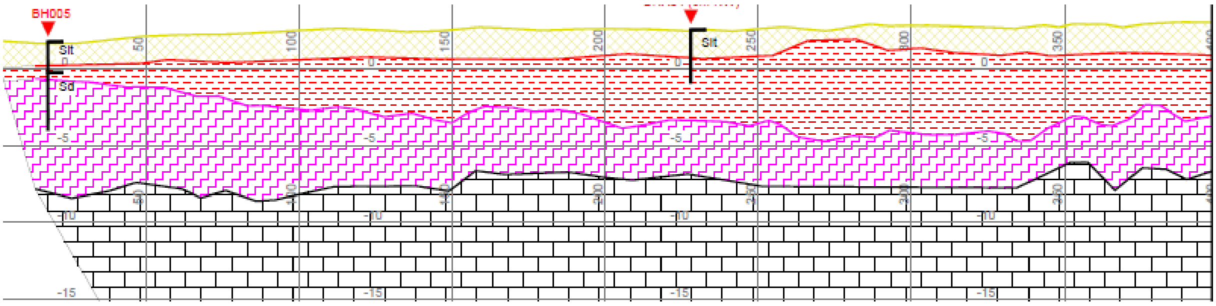
6.1.5 Railway line (BH301 – BH307) ch0+100m to ch0+500m

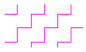
Construction of flood walls and embankments are proposed along this section along with channel refurbishment, CH2/ G02

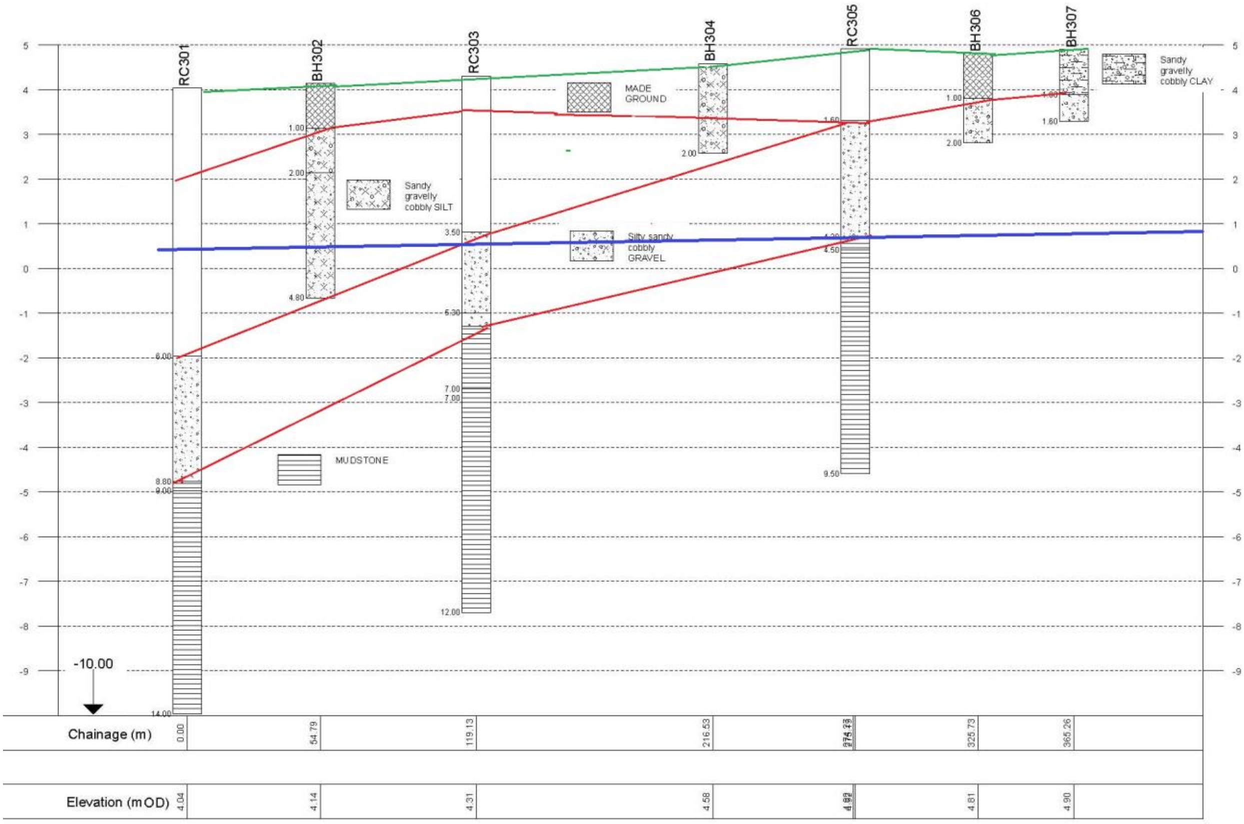
The ground adjacent to the railway line was characterised by Made ground to depths between 1.0m bgl to 2.0m bgl. The Made ground was described as slightly sandy slightly gravelly CLAY/SILT with medium Cobble content, Boulder content and concrete, brick and timber inclusions. This was underlain by stiff slightly sandy slightly gravelly SILT with low Cobble content to depths between 1.6m bgl to 6.0m bgl. The thickness of the SILT varied. The SILT was underlain by glacial deposits, dense (slightly) silty (very) sandy GRAVEL to depths 4.2m bgl to 8.8m bgl being 2.0m to 3.0m thick. Below this highly weathered, weak SLATEY-MUDSTONE was encountered (0.72mOD to -4.76mOD).

The geophysical survey (R3 and S5) indicated approximately 5m to 7m of superficial deposit; soft (Silt) overlying medium dense (Granular deposits). A zone of dense, weak weathered rock ( ) 3m to 4m thick overlay bedrock (Slatey-Mudstone) below – 5.5mOD to -8.0mOD.



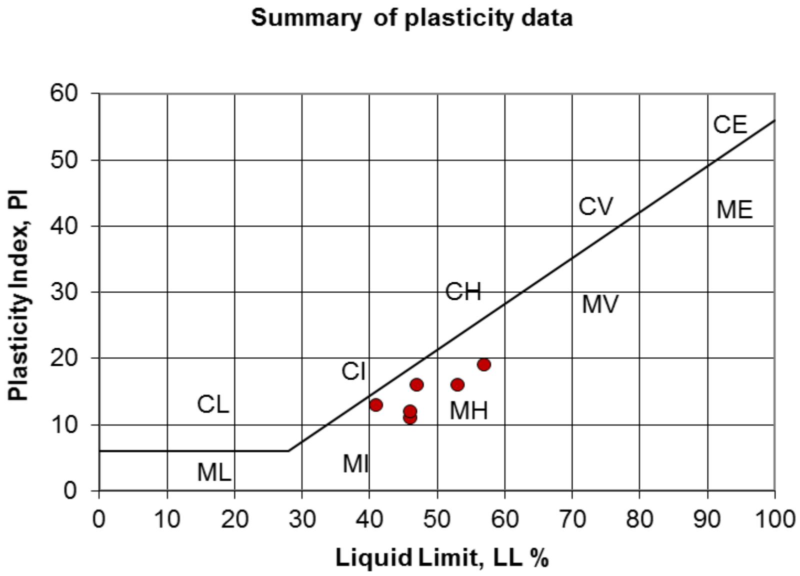


The geophysical survey layer  contoured the top of the weathered Slatey-Mudstone rockmass. The geotechnical section is presented below.



Groundwater (----) was encountered at depths 3.8m bgl (0.24mOD to 0.51mOD). Groundwater is assumed confined within the granular deposits below. Seasonal fluctuations are anticipated but have not been defined. Installation of an *in situ* groundwater data logger is recommended to monitor variations in groundwater levels.

> *Ongoing groundwater monitoring is reocmmended at RC301 to define groundwater regime and likely variations in groundwater levels.*

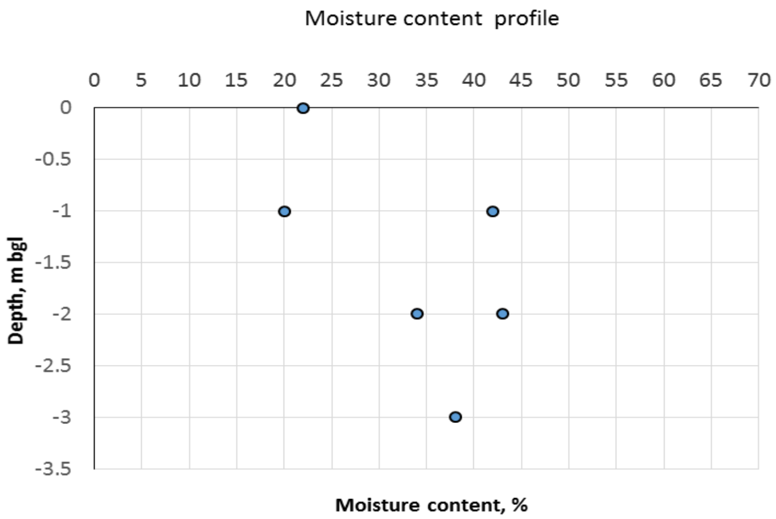


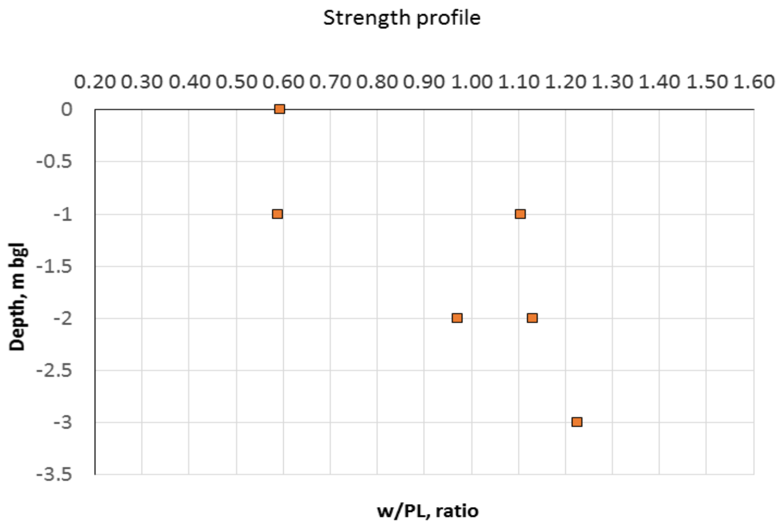
The SILT deposits were of mixed and varied plasticity intermediate to very high (MI to MH). The SILT was of low to medium organic content (loss on ignition 0.8% and 6.7%). The silt fraction was 30% to 65% with 3% to 30% gravels and 29% to 46% sand fractions. Cobble content was between 0% and 9%, medium.

Natural moisture content, w ranged between 20% and 43%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.6 to 1.2 indicative of stiff (<1.0) and firm deposits, 1.0<*w*/PL <1.2 (C504). Undrained shear strength of 75kPa to 150kPa are expected (BS5930; 1999)

Standard penetration tests Nspt values ranged between 18 to 34. With a plasticity index, PI 11 – 19; a factor f_1 = 5 to 6 (Stroud, 1975) was such to yield undrained shear strengths of 90kPa to 138kPa, describing stiff deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 30^{o} for the sandy gravelly SILT (C504; Terzaghi, Peck & Mesri, 1996). Elevated Nspt (refusals >50) are attributed to coarse particles and inclusions within the Silt deposits.

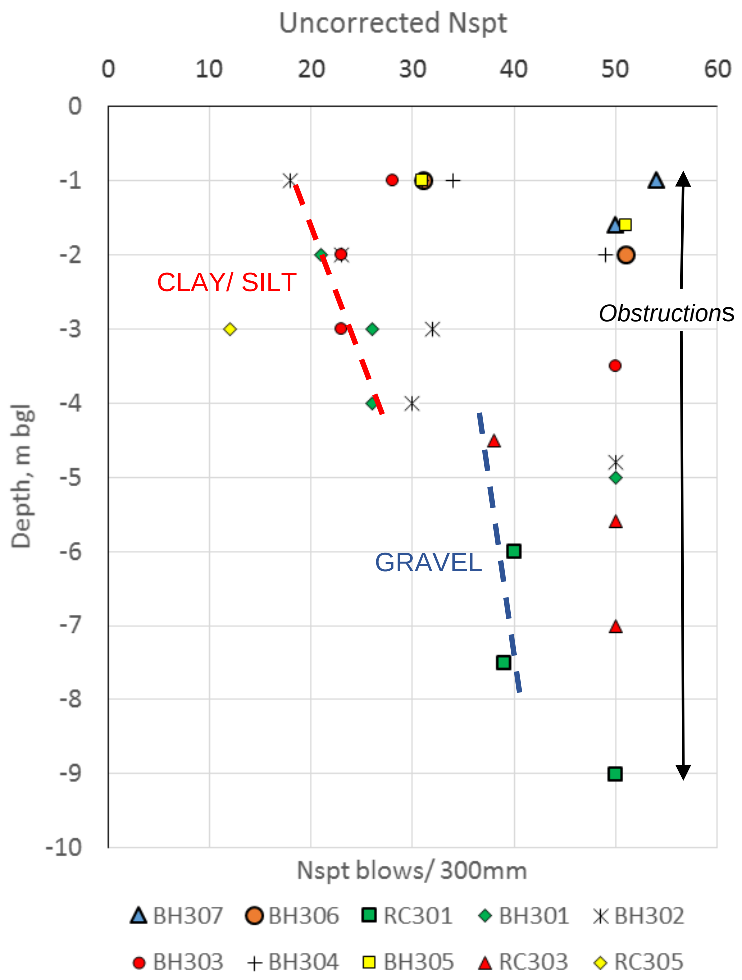
Compressibility of the SILT is expected to be variable being moderately compressibility (MI-MH).





Taking a characteristic Nspt value of 18 in the sandy SILT, an unfactored stiffness modulus (Young's modulus, E) of 21MPa to 27MPa is expected of the stiff sandy SILT (PI 11 – 19, Stroud, 1975).

Permeability in the SILT was determined based on particle size d_{10} being 0.003mm to 0.001mm, giving a value of the order $10^{-8}ms^{-1}$, indicative of very low permeability (C113, Control of groundwater for temporary works).



The GRAVEL was characterised by 2% to 15% silt, 31%, sand and 40% to 51% gravels and high Cobble content (28%).

With Nspt values 12 to 39 (refusal are attributed to coarse particles), allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 36^{o} to 38^{o} is expected of the medium dense to dense granular deposits. The value of Nspt 12 is possibly associated with the presence of groundwater. Taking a characteristic Nspt value of 38 in the dense GRAVEL deposits a stiffness modulus of 35MPa to 40MPa is expected of the dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

Particle size(s) d_{10} of 0.6mm to 0.006mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-3} ms^{-1} to 10^{-7} ms^{-1}. This described medium to low permeability (C113). The permeability in the GRAVEL was controlled by the silt fraction.

The (Slatey) Mudstone rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (medium strong to strong I_{P50}, 0.24MPa – 1.6MPa), fracture spacing (NI) and condition (highly weathered), Rock Quality Designation (RQD non-intact/ 0%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 27- 30, describing Class IV poor Slatey-Mudstone. A geological strength index, GSI (Hoek and Brown) of 15 is assumed for the poorly interlocked highly broken rockmass. An angle of friction, $\phi = 18^o$ and cohesion 200kPa are recommended for the CIV, weak, highly weathered Slatey-Mudstone.

RC301 (Slatey) Mudstone

RC303 (Slatey) Mudstone

RC305 (Slatey) Mudstone

The geotechnical hazard in this area is considered to be coarse Cobble and Boulder particles within the superficial deposits. Cobble and Boulder content may obstruct sheetpiles. Similarly inclusions within the Made ground may obstruct sheet piles. The chiselling records are summarised as follows:

Location	Depth, m bgl		Duration; hh:mm	Strata
	Start	End		
BH301	0.8	1.0	00:40	Made ground
	5.0	5.0	01:00	GRAVEL
BH302	3.3	3.4	00:30	SILT
	4.8	4.8	01:00	
BH303	1.2	1.3	00:40	SILT
	3.5	3.5	01:00	GRAVEL
BH304	2.0	2.0	01:00	SILT
BH305	1.6	1.6	01:00	Made ground
BH306	1.3	1.4	00:30	GRAVEL
	2.0	2.0	01:00	
BH307	1.2	1.3	00:45	
	1.6	1.6	01:00	

It can be seen that boreholes were advanced by chiselling within the Made ground, stiff SILT and obstructed in the dense GRAVELS. Further obstructions may be present below the depth of the cable percussion boreholes.

Caution needs to be exercised not to overdrive and damage sheet piles. A ‘toe-hold’ only is expected in the weathered rock mass. The level of groundwater cut-off shall be assessed. Pre-boring may be required to advance the sheetpiles achieve the cut-off at bedrock level where both stiff cohesive and dense saturated granular deposits were encountered. Water-jetting may also be considered. It may be considered that jetting may offer a means of grouting the base of the sheetpile in the weathered rock improving cut-off and fixing the toe. A Specialist sheetpiling contractor shall be consulted with regard the driving system, driven or vibratory or hybrid system best suited to the ground model presented. Vibration and noise shall be considered.

In the absence of a construction detail, a presumed bearing capacity of 150kNm^{-2} (kPa) is expected (BS8004; 1986 Code of practice for foundations) for foundations constructed within the stiff SILT deposits underlying the Made ground below 1.6m bgl.

> *It is recommended to carry out three (3) number plate loading tests to verify the design bearing capacity at a depth 1.0m bgl within the Made ground at BH302, BH304 and BH305.*

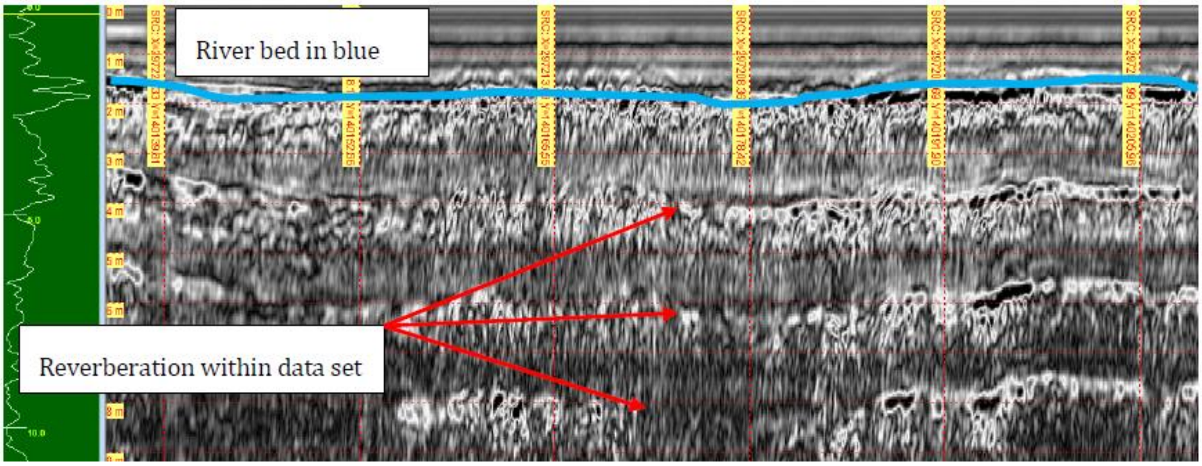
To complete the hydrogeological model in situ permeability tests (falling and constant head tests) are recommended at locations; BH301 and BH305. The stiff SILT, medium dense GRAVEL and bedrock strata should be assessed.

6.1.6 Railway Bridge (RCP1, RCP3, RCP5 and RCP06) ch5+690m

Underpinning of existing railway masonry bridge piers

The River Slaney, river bed was investigated at the bridge pier locations. Cobbles with sandy and gravel were noted to a depth 1.5m bgl to 1.8m bgl (-1.48mOD to -1.62mOD) below this clayey/silty sandy GRAVEL with Cobble content was noted overlying SHALE 3.9m bgl to 4.5m bgl (-3.69mOD to -4.32mOD).

A geophysical survey in the river bed returned poor data from the sub-bottom profiling as a result of background noise associated with shallow water flow. No viable interpretation was available.



No samples were recovered in the GRAVEL deposits. With Nspt values 32 and 36 (refusal and elevated values >50, were attributed to coarse Cobble particles), allowing for the silt fractions an angle of friction, ϕ of 35^o to 38^o is expected of the dense granular deposits. Taking a characteristic Nspt value of 32 in the dense GRAVEL deposits a stiffness modulus of 50MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

The Shale rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to medium strong), fracture spacing (20mm – 150mm) and condition (highly weathered), Rock Quality Designation (RQD 0% to 11%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 26- 29, describing Class IV poor Shale. A geological strength index, GSI (Hoek and Brown) of 20 is assumed for the heavily broken rockmass. An angle of friction, $\phi = 20^{o}$ and cohesion 150kPa are recommended for the upper CIV, Shale.

RCP1 Shale

RCP3 Shale

RCP5 Shale

RCP6 Shale

The geotechnical hazard in this area is considered to be; coarse Cobble and Boulder particles within the superficial granular deposits overlying the rock mass and the poor quality rock mass itself.

In the absence of a construction detail, a presumed bearing capacity of 200kNm^{-2} to 600kNm^{-2} (kPa) is expected (BS8004; 1986 Code of practice for foundations) for foundations constructed within the dense GRAVEL deposits. Standard penetration test N values suggested an allowable bearing pressure of 300kPa (Terzaghi and Peck, 1967) for settlements not exceeding 25mm.

Where foundations are to be constructed within the weathered rock mass, BS8004 (1986) identified a presumed bearing value of 2,000 kN/m^2 for non-weathered strong Sand/ Siltstones (Group 4). In accordance with *Figure 1 — Allowable bearing pressures for square pad foundations bearing on rock (for settlement not exceeding 0.5 % of foundation width*) this should be reduced to a value within the range of 100kPa to 250kPa for end bearing piled foundations in the bedrock with non-intact fractures fracture spacing <200mm.

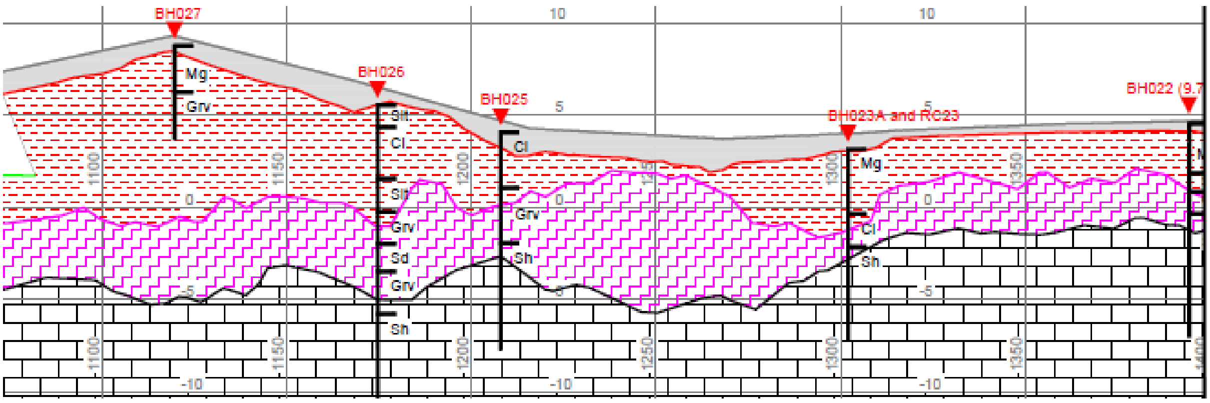
6.1.7 Right Bank (BH018 – BH027) ch6+100m to ch5+690m

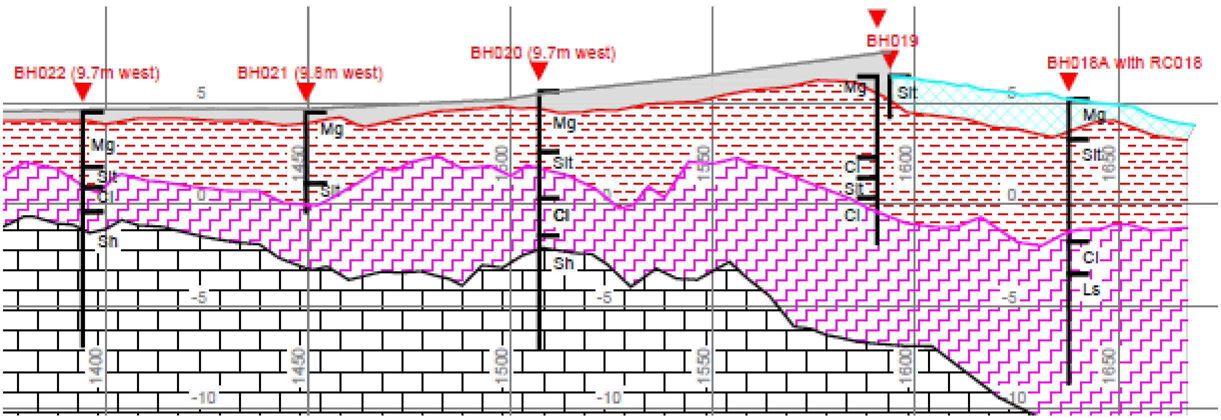
Construction of flood walls flood wall L01 (6+100m to 5+750m) and embankment works E01 are proposed along this section.

This section of the right bank of the Slaney River above Enniscorthy Bridge was characterised by Made ground slightly silty gravelly SILT/ CLAY/ silty very sandy GRAVEL with Cobble and Boulder content and brick and concrete inclusions to depths between 2.0m bgl and 3.5m bgl. This was typically underlain by alluvial deposits of soft slightly sandy slightly gravelly (peaty) SILT deposits with medium Cobble content and some wood inclusions (below 4.0m bgl; BH18A) to depths between 3.0m bgl and 7.0m bgl. The SILT was underlain by stiff glacial deposits; slightly sandy slightly gravelly CLAY; 2.0m thick to 2.3m and medium dense and dense slightly silty very sandy GRAVEL with Cobble content, overlying DOLERITE, 8.6m bgl (RC18A, -3.48mOD) and SHALE, 5.1m bgl to 11.3 m bgl (-0.52mOD to -5.75mOD).

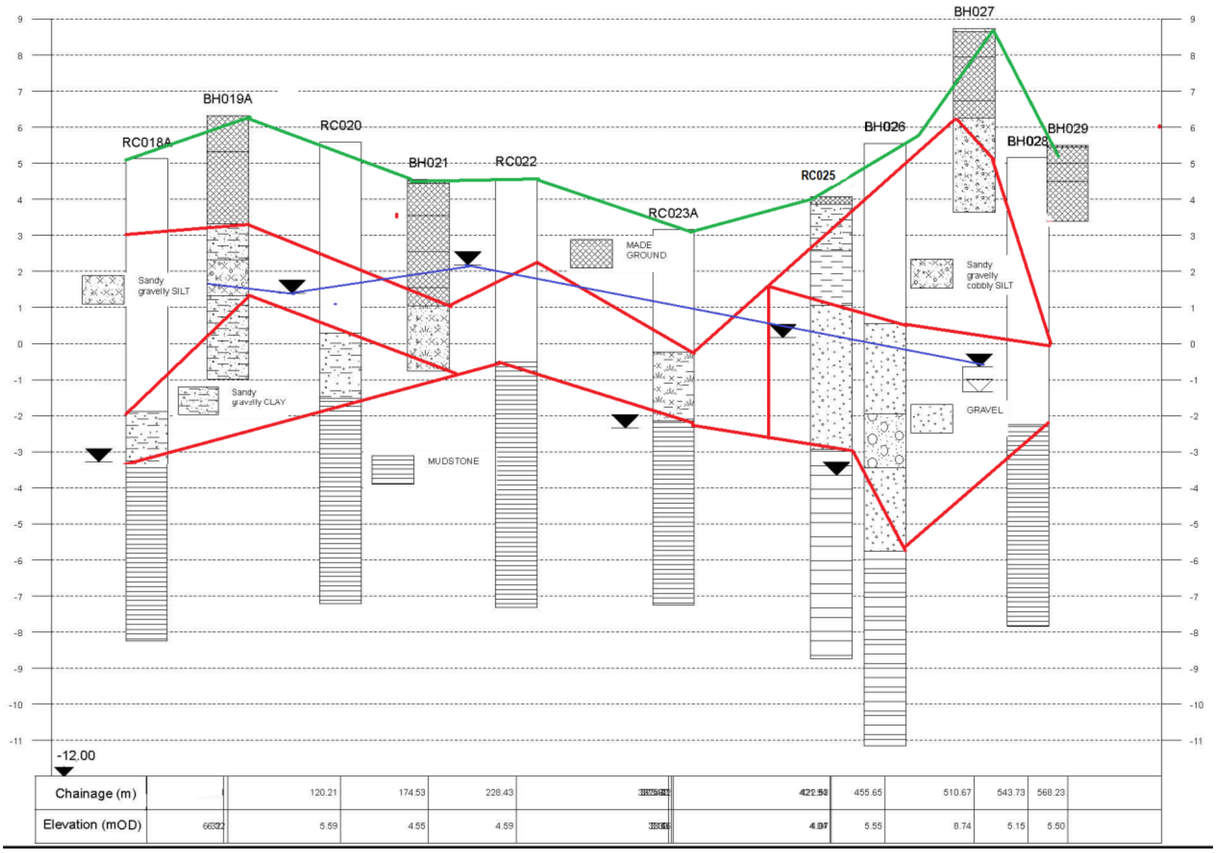
Hardstanding was encountered beyond BH025 100mm to 200mm of bituminous surfacing with 700mm sub-base cl.804 or similar; overlying made ground described as CLAY with Boulder content.

The geophysical survey (R9, R8, R13 and S10, S15, S13 and S16) indicated approximately 4m to 9m of superficial deposit; (Hardstanding) overlying medium dense (Granular deposits). Below –1.0mOD to 0.0mOD, a zone of dense Gravel, weak weathered rock ( ) 2.5m to 7m thick overlay bedrock (Slatey-Mudstone/ Shale) -1mOD to -5mOD. The thickness of the weathered bedrock, increased at BH18. Geological mapping (1:100k) indicated a number of N-S faults in the area.

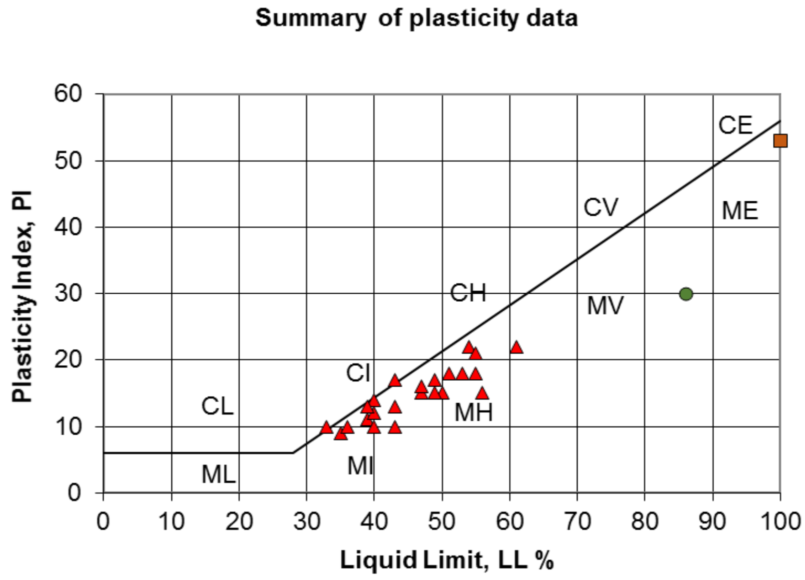




The geophysical survey layer contoured the top of the dense GRAVEL or weathered rockmass. The bedrock and weathering profile is variable. The geotechnical section is presented as follows;



Groundwater (----) was encountered at depths 2.4m bgl to 9.2m bgl (2.2mOD to -3.7mOD). The static groundwater level was measured at RC022 at 2.74m bgl (1.84mOD). Installation of an *in situ* groundwater data logger is recommended to monitor variations in groundwater levels.

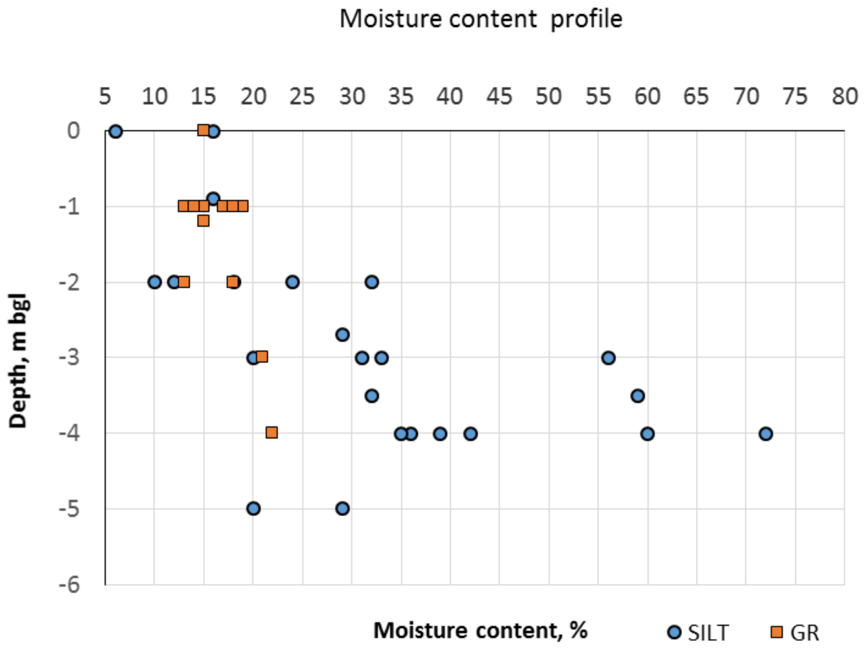


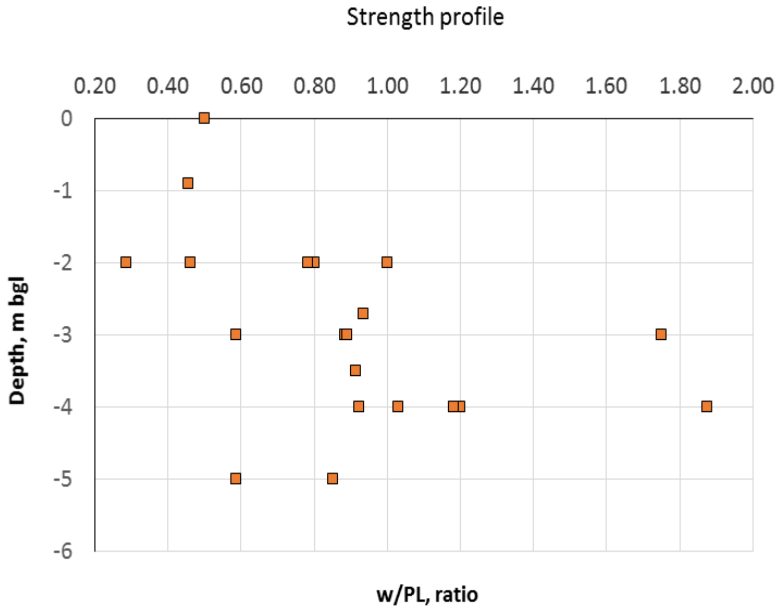
The SILT deposits were of mixed and varied plasticity intermediate to very high (MI to MH). The SILT was of low to medium organic content (loss on ignition 1.6% to 5.6%). Very high to extremely high plasticity were measured corresponding to peaty Silt deposits (loss on ignition 6.1% and 12%). The silt fraction was 28% to 58% with 16% to 43% gravels and 12% to 33% sand fractions. Cobble content was between 0% and 25%, low to high.

Natural moisture content, w ranged between 6% and 72%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.3 to 1.8 indicative of stiff (<1.0) and very soft deposits, *w*/PI >1.2 (C504).

Standard penetration tests Nspt values ranged between 3 to 17. With a plasticity index, PI 9 – 22; a factor f_1 = 5 to 6 (Stroud, 1975) was such to yield undrained shear strengths of 15kPa to 102kPa, describing soft to stiff deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 25^o to 30^o for the sandy gravelly SILT (C504; Terzaghi, Peck & Mesri, 1996). Elevated Nspt (refusals >50) are attributed to coarse particles and inclusions within the Silt deposits.

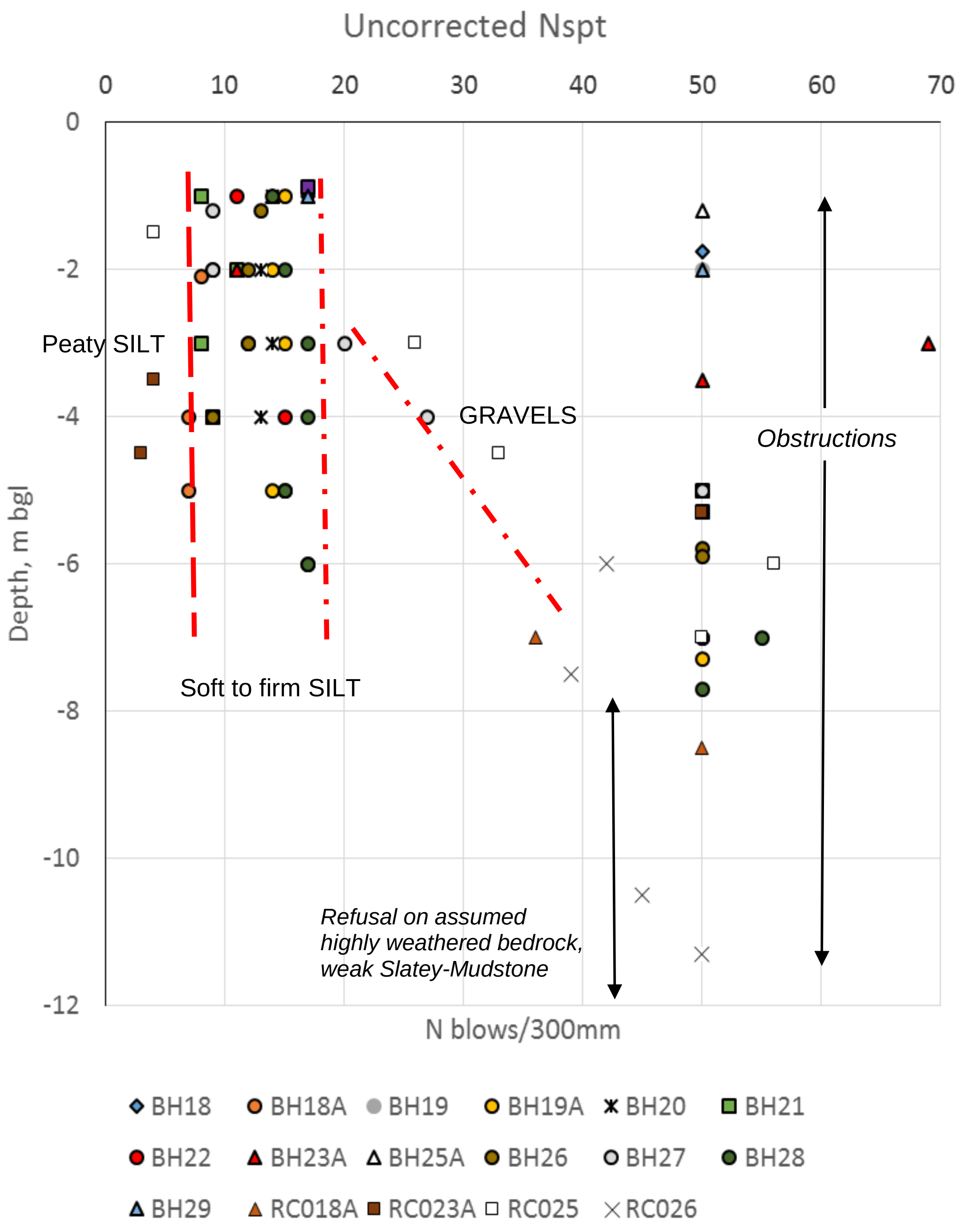
Compressibility of the SILT is expected to be variable being moderately compressibility (MI-MH). The peaty deposits are considered compressible.





Taking a characteristic Nspt value of 3 in the soft peaty SILT, an unfactored stiffness modulus (Young's modulus, E) of 1.5MPa is expected (PI 30 and 53, Stroud, 1975). Taking a characteristic Nspt value of 7 in the soft sandy SILT, an unfactored stiffness modulus (Young's modulus, E) of 8MPa is expected of the stiff sandy SILT (PI 9 - 22, Stroud, 1975). Taking a characteristic Nspt value of 12 in the sandy gravelly SILT, an unfactored stiffness modulus (Young's modulus, E) of 18MPa is expected of the firm to stiff sandy gravelly SILT (PI 9 - 22, Stroud, 1975).

Permeability in the SILT was determined based on particle size d_{10} being 0.002mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works). An *in situ* (falling head) permability of 2.4 x 10^{-7} ms^{-1} was measured at BH22.



The GRAVEL was characterised by 0% to 26% silt, 15% to 35% sand and 39% to 85% gravels and variable Cobble content (1% to 52%).

With Nspt values 13 to 56 (refusal and elevated values >50, were attributed to coarse particles), allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^o to 38^o is expected of the medium dense to dense granular deposits. Taking a characteristic Nspt value of 23 in the dense GRAVEL deposits a stiffness modulus of 45MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

Particle size(s) d_{10} of 0.3mm to 0.003mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-4} ms^{-1} to 10^{-6} ms^{-1}. This described medium permeability (C113). The permeability in the GRAVEL was controlled by the silt fraction noting particle size(s) d_{10} of 2mm to 3mm indicating permeabilities of the order 10^{-2} ms^{-1} were measured at BH28 and BH29 describing medium to high permeability deposits. Permeability was not determined in situ where it was not possible to develop a head above the groudnwatre level in GRAVELS. This was indicative of 'high' permeability.

The Made ground was variable with Nspt 8 to 17, a characteristic value of 12 is recommended, indicative of an undrained shear strength 72kPa.

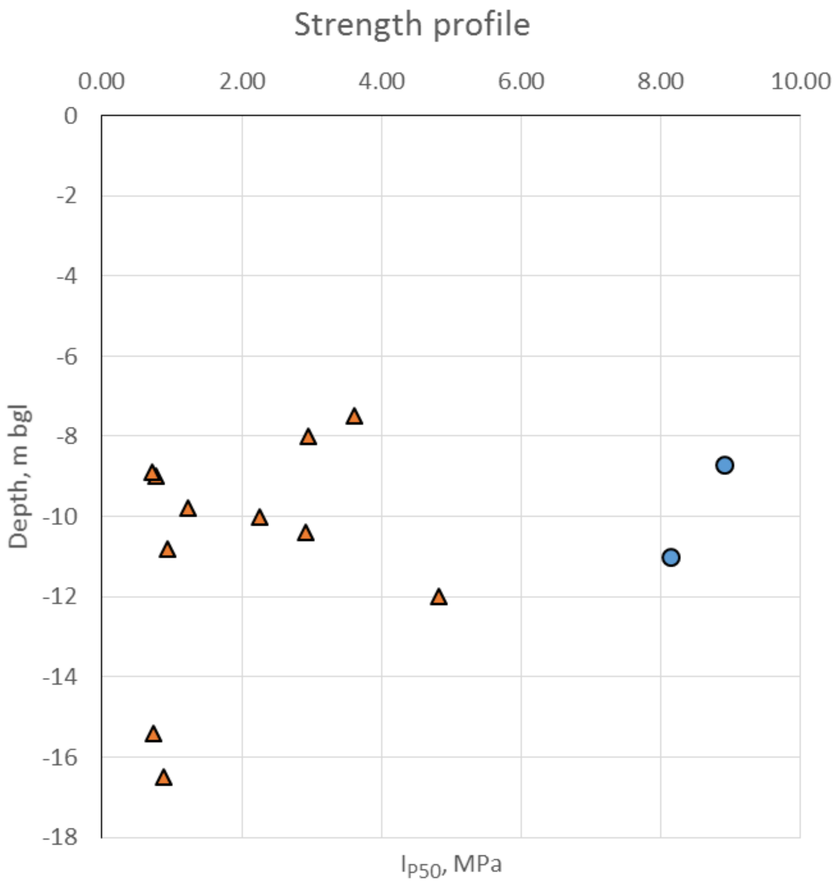
The Dolerite rock mass characterization at RC018A, has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (medium strong to very strong I_{P50}, 8MPa – 9MPa; UCS 34MPa), fracture spacing (130mm – 520mm) and condition (slightly weathered), Rock Quality Designation (RQD 18% to 66%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 46- 81, describing Class III - II fair becoming good Dolerite. A geological strength index, GSI (Hoek and Brown) of 50 is assumed for the partially disturbed blocky rockmass. An angle of friction, $\phi = 32^o$ and cohesion 250kPa are recommended for the upper CIII, Dolerite.

RC018A Dolerite

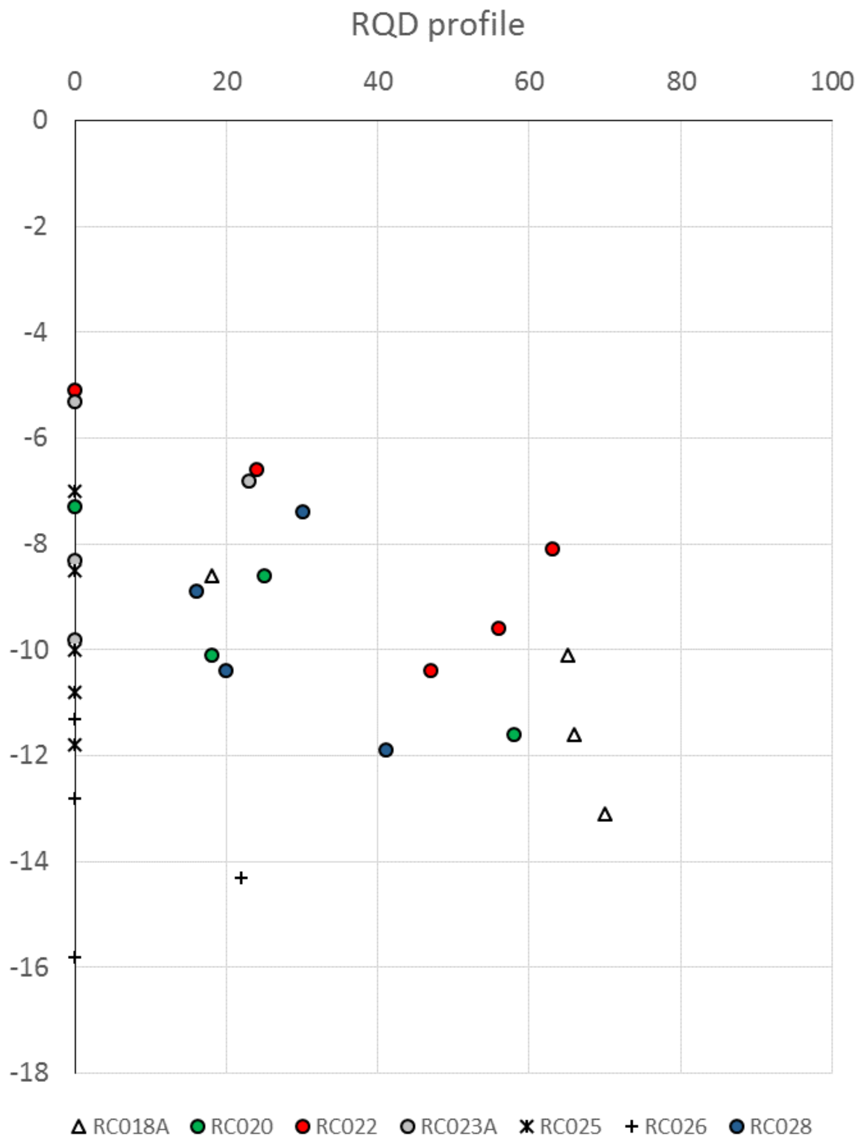
The Slatey- Mudstone/ Shale rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to strong I_{P50}, 0.7MPa – 4.8MPa; UCS 4MPa), fracture spacing (NI, 120mm to 370mm) and condition (highly weathered), Rock Quality Designation (RQD non-intact/ 0% to 63%) and groundwater (assumed ‘wet’ within the zone of influence) was undertaken. The rock mass rating, RMR range was 35- 45, describing Class IV poor Slatey-Mudstone. A geological strength index, GSI (Hoek and Brown) of 20 is assumed for the poorly interlocked highly broken rockmass. An angle of friction, $\phi = 20^{o}$ and cohesion 150kPa are recommended for the CIV, weak to medium strong, highly weathered Slatey-Mudstone/ Shale.

RC020, Slatey- Mudstone/ Shale

RC028 Slatey- Mudstone/ Shale



Strength profile
0.00
2.00
4.00
6.00
8.00
10.00
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
Depth, m bgl
I_{P50}, MPa

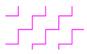


The geotechnical hazard in this area is considered to be; obstructions within the made ground and coarse Cobble and Boulder particles within the superficial deposits. Cobble and Boulder content may obstruct sheetpiles. Similarly inclusions within the Made ground may obstruct sheetpiles. The chiselling records are summarised as follows:

Location	Depth, m bgl		Duration; hh:mm	Strata
	Start	End		
BH018	1.70	1.75	01:00	Made ground
BH018A	2.85	2.90	01:00	Soft SILT
BH018A	7.00	7.00	01:00	Stiff SILT
BH019	2.00	2.10	01:30	Made ground
BH019A	4.60	4.70	01:00	Stiff SILT
BH019A	7.20	7.30	01:00	

Location	Depth, m bgl		Duration; hh:mm	Strata
BH020	5.20	5.30	01:00	GRAVEL
BH021	5.30	5.40	01:00	Bedrock
BH022	5.10	5.20	01:00	
BH023A	3.40	3.50	01:00	Made ground
BH025A	1.10	1.20	01:00	
BH026	5.80	5.90	01:00	Dense GRAVEL
BH027	5.00	5.10	02:00	
BH028	4.90	5.00	01:00	Made ground
BH028	6.50	6.60	01:00	SILT
BH028	7.70	7.80	01:00	Bedrock
BH029	2.05	2.10	01:00	Made ground

The variable weathering profile in bedrock is also considered a geo-hazard with regard to depth of sheetpile installation.

It can be seen that boreholes were advanced by chiselling within the Made ground, soft SILT and obstructed in the stiff SILT and dense GRAVEL. Boreholes terminated after one (1) hour chiselling on weathered bedrock. Further obstructions may be present below the depth of the cable percussion boreholes. The geophysical survey layer  shall be considered a dense strata which may arrest the progress of sheetpiles.

Caution needs to be exercised not to overdrive and damage sheetpiles given the presence of coarse particles and variability in the rock type and quality of the rock mass. A ‘toe-hold’ only is expected in the weathered rock mass. The level of groundwater cut-off shall be assessed. Pre-boring may be required to advance the sheetpiles achieve the cut-off at bedrock level where both stiff cohesive and dense saturated granular deposits were encountered. Water-jetting may also be considered. It may be considered that jetting may offer a means of grouting the base of the sheetpile in the weathered rock improving cut-off and fixing the toe. A Specialist sheetpiling contractor shall be consulted with regard the driving system, driven or vibratory or hybrid system best suited to the varied ground conditions. Vibration and noise shall be considered.

In the absence of a construction detail, a presumed bearing capacity of $75kNm^{-2}$ to $150kNm^{-2}$ (kPa) is expected (BS8004; 1986 Code of practice for foundations) for foundations constructed within the firm SILT/ Made ground deposits 1.0m bgl. Standard penetration test N values suggested an allowable bearing pressure of 80kPa (Terzaghi and Peck, 1967) for settlements of 5mm to 15mm. Differential settlement of 10mm can be expected where the made ground was of varied thickness and the underlying SILT of varied stiffness. Where the peaty SILT was present (BH021 and BH023), an allowable bearing pressure of 40kPa is recommended with settlement 25mm, subject to a review of foundation width. A retaining wall type geometry may be more appropriate at these locations. It shall be considered compacting the made ground to provide for a more uniform deposits in which to construct foundations where made ground by its nature is variable.

It is recommended to carry out four (4) number plate loading tests to verify the design bearing capacity at a depth 1.0m bgl within the Made ground and soft SILT at BH018, BH020, BH023 and BH025. Dynamic probing (40 to 75number, 5m to 10m spacing) shall also be considered to a depth 5.0m along the alignment of wall L01 to assess the variability in made ground and delineate peaty deposits in the vicinity of BH021 and BH023.

High permeability in the GRAVEL deposits is considered a geo-hazard. To complete the hydrogeological model it is recommended that in situ permeability tests (falling and constant head tests) are recommended at location; BH018, BH023 and BH025. The soft SILT/ peaty SILT, Made ground, dense GRAVEL and bedrock strata should be assessed.

6.1.8 Left Bank (TP001- BH003)

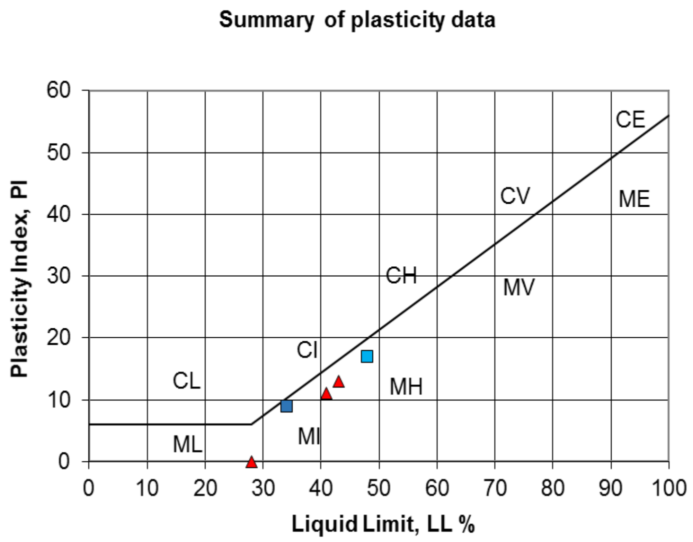
Strengthening of flood wall, L02 and raising the carpark (RC01) level 0.3m are proposed along this section.

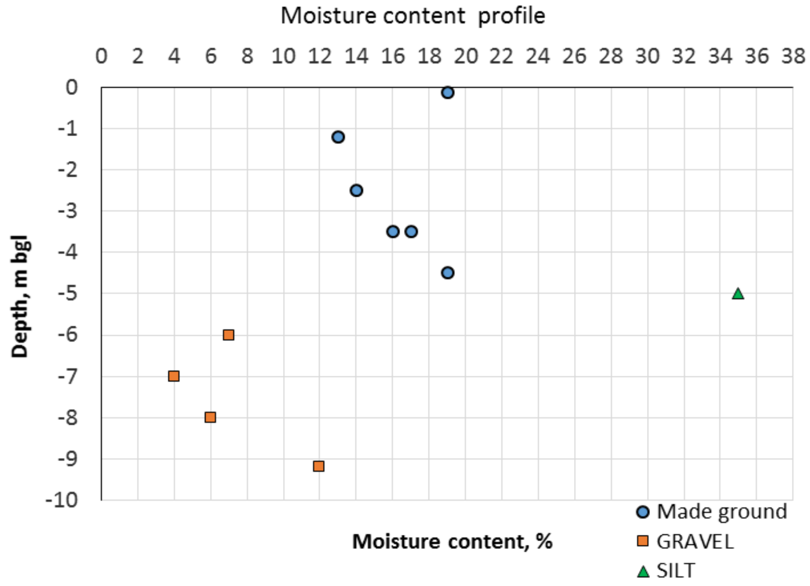
This section was characterised by Made ground; loose silty very gravelly SAND and silty very sandy GRAVEL with inclusions of plastic, glass and bitumen inclusions 1.6m to 5.0m thick overlying a thin very soft slightly sandy gravelly SILT/ silty SAND strata 500mm to 600mm thick (Nspt = 0). This was underlain by very loose becoming medium denes sandy very silty GRAVEL deposits to a depth 9.5m bgl. Bedrock is assumed below this depth where the borehole terminated after one(1) hour chiselling without progress. There was olfactory evidence of hydrocarbon contamination at 1.8m bgl in BH003.

Groundwater was encountered 5.0m bgl (0.37mOD) confined within the GRAVEL deposits.

The Made ground was characterised by intermediate plasticity and natural moisture content, *w* 13% to 19%. Loss on ignition, LoI of 2% and 2.2% was measured indicating a low organic content. Standard penetration test Nspt value ranged between 2 to 8 indicating very loose to loose deposits. Particle size d_{10} ranged between 0.043mm to 0.017mm indicating a permeability of the order 10^{-6} ms^{-1}, describing low permeability (C113).

The SILT was characterised by intermediate plasticity and natural moisture content, *w* 35% and sand fraction 28%. Standard penetration test Nspt value ranged between 0 indicating very soft deposits. The ratio w/PL was 1.17 indicative of soft deposits Cu <50kPa (C504). Particle size d_{10} <0.002mm indicating a permeability of the order 10^{-8} ms^{-1}, describing very low permeability (C113).

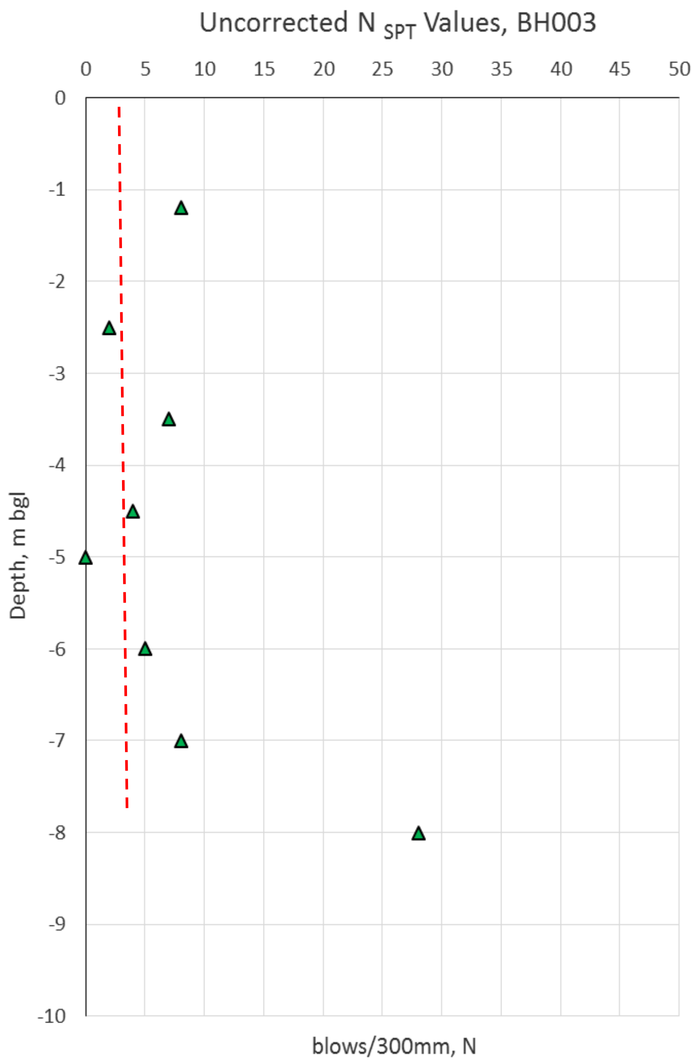




The GRAVEL was characterised by low Nspt values 5 and 8 and silt fraction 13% to 27%. Particle size d_{10} 0.063mm indicative of permeability the order $10^{-5}ms^{-1}$, describing low permeability. Below 9m particle size d_{10} was 0.6mm indicative of permeability the order 10^{-3} ms^{-1}, describing medium permeability.

In the absence of a construction detail, a presumed bearing capacity of <100kNm^{-2} (kPa) is expected (BS8004; 1986 Code of practice for foundations) for foundations constructed within the very loose to loose Made ground deposits 1.0m bgl. Standard penetration test N values suggested an allowable bearing pressure of 40kPa (Terzaghi and Peck, 1967) for settlements of 10mm. Under loading up to 100kPa settlement of 25mm is expected. Differential settlement can be expected where the made ground was of varied thickness and the underlying SILT of varied stiffness.

It may be required to underpin the existing foundations. Micropiling is considered an effective method where the micropiles would be end bearing below 9.5m bgl. Other pile forms such as mini augered piles or screw piles are also considered suitable subject to confirming the geometry of the existing foundations.



Nspt values suggested a California bearing ratio 1% at a formation depth 1.0m bgl. The existing bituminous construction was 200mm (TP002) with 150mm to 600mm sub-base cl.804 or similar. Where it is proposed to raise the car park 300mm is it provisionally proposed that 150mm sub-base cl.804 overlay is placed with a further 150mm bituminous construction.

It is recommended to undertake a slit trench excavations to assess the existing wall foundations and undertaken two(2) plate loading tests to assess the bearing capacity of the made ground where standard penetration test N value of 2 to 8 were measured.

A shallow trial pit excavation is recommended to verify the construction the car park and assess bearing for overlay construction. Environmental PAH samples are recommended in the carpark to assess the existing surfacing.

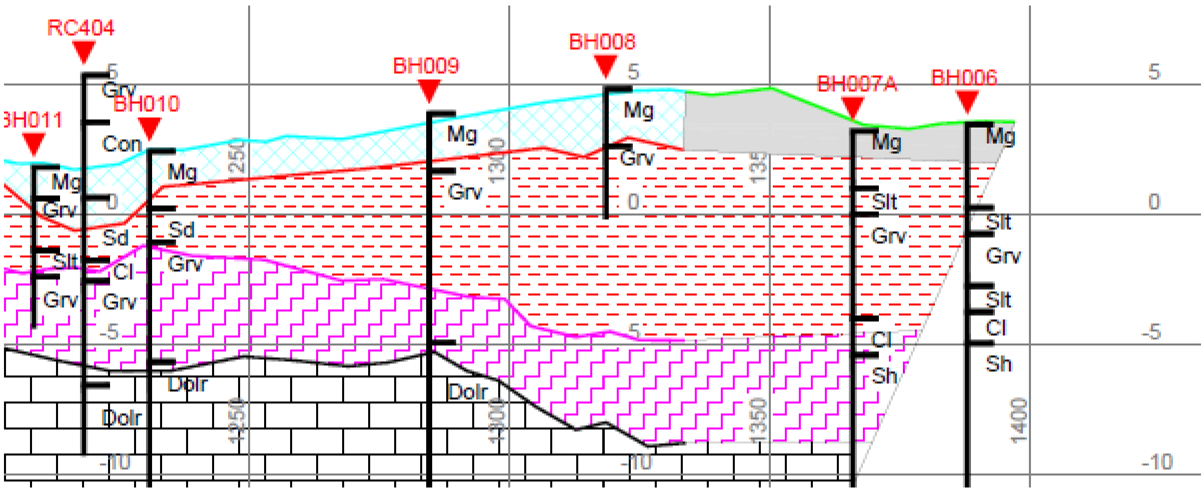
6.1.9 Left Bank (BH006 – BH010) ch5+690m to 5+535m

Channel, CH1 widening EX5 and construction of flood wall, L03 1.4m to 2.5m high is proposed along this section.

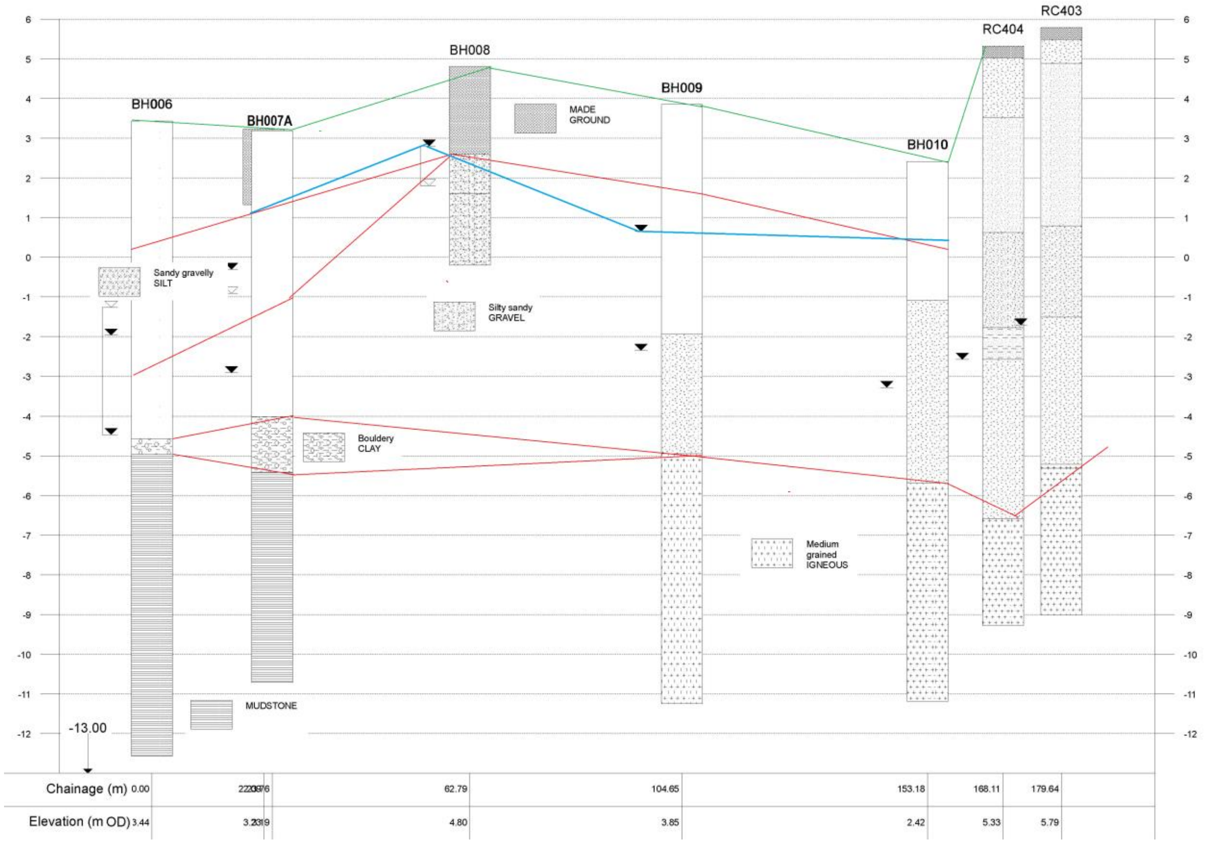
This section of the left bank of the Slaney River, between the Railway Bridge and Enniscorthy Bridge, was characterised by Made ground; slightly sandy slightly gravelly SILT/ silty very sandy GRAVEL with low Cobble content and brick/ concrete inclusions to depths between 2.2m bgl to 3.2m bgl. This was underlain by alluvial deposits of firm slightly sandy slightly gravelly SILT to depths 4.1m bgl to 5.0m bgl, 1.8m to 1.9m thick. The SILT in turn was underlain by glacial deposits of medium dense slightly sandy silty GRAVEL with medium Cobble content to depths 7.2m bgl. Below the GRAVEL a layer of stiff 'Boulder Clay', slightly sandy gravelly SILT with Boulder content overlay weak SHALE below 8.4m bgl to 8.6m bgl (-4.96mOD to -5.41mOD).

At BH008 to BH010 the made ground was underlain by medium dense silty sandy GRAVEL with medium to high Cobble content below 2.2m bgl to a depths 8.1m bgl to 8.8m bgl. Below this depth strong to very strong DOLERITE was encountered.

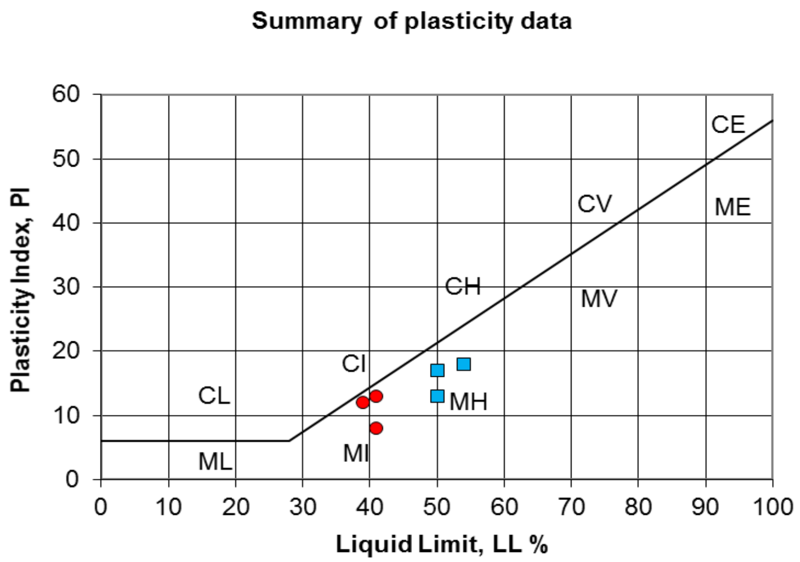
The geophysical survey (R10 and S11) indicated approximately 4m to 10m of superficial deposit; (Hardstanding) overlying medium dense (Granular deposits). Below –-2.0mOD to -5.0mOD, a zone of dense Gravel ( ) 4m thick overlay bedrock (Dolerite,) -5.5mOD to -9mOD.



The geophysical survey layer contoured the top of the weathered Shale rockmass or dense GRAVEL. The bedrock and weathering profile is variable. The geotechnical section is presented as follows;



Groundwater (----) was encountered at depths 2.0m bgl to 5.4m bgl (2.8mOD to -1.96mOD). The static groundwater level was measured at RC010 at 2.0m bgl (0.42mOD). Installation of an *in situ* groundwater data logger is recommended to monitor variations in groundwater levels.



The slightly sandy slightly gravelly SILT was of intermediate and high plasticity (MI – MH). The intermediate plasticity deposits were at lower depths corrssponding to glacial (Boulder Caly) deposits. The SILT was of low to medium organic content (loss on ignition 4.2%). The silt fraction was 37% to 66% with 10% to 40% gravels and 16% to 32% sand fractions. Cobble content was between 0% and 6%, low to medium.

Natural moisture content, w ranged between 17% and 41%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.56 to 1.14 indicative of stiff (<1.0) and firm deposits, 1.0< *w*/PL <1.2 (C504).

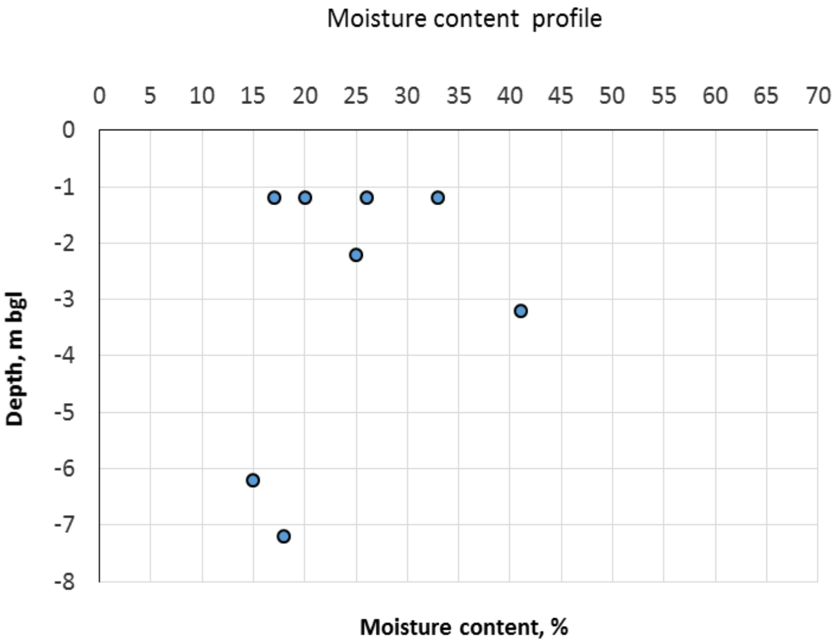
Standard penetration tests Nspt values ranged between 12 to 20. With a plasticity index, PI 8 – 17; a factor f_1 = 5.5 to 6 (Stroud, 1975) was such to yield undrained shear strengths of 66kPa to 120kPa, describing firm to stiff deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 28^o for the sandy gravelly SILT (C504; Terzaghi, Peck & Mesri, 1996). Elevated Nspt (refusals >50) are attributed to coarse particles and inclusions within the stiff Silt deposits.

Compressibility of the SILT is expected to be variable being moderately compressibility (MI-MH).

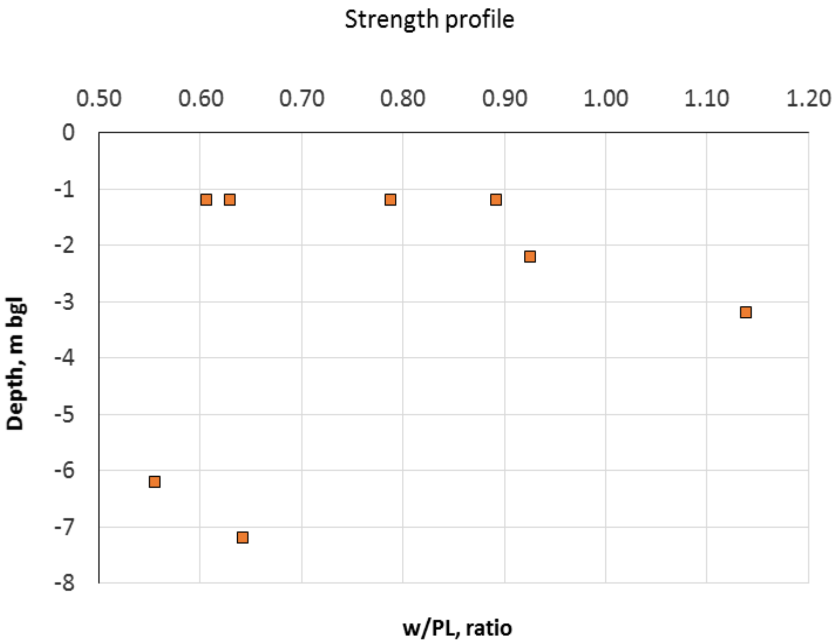
Taking a characteristic Nspt value of 14 in the upper SILT, an unfactored stiffness modulus (Young’s modulus, E) of 14MPa to 21MPa is expected (PI 8 and 17, Stroud, 1975).

Permeability in the SILT was determined based on particle size d_{10} being 0.002mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works).

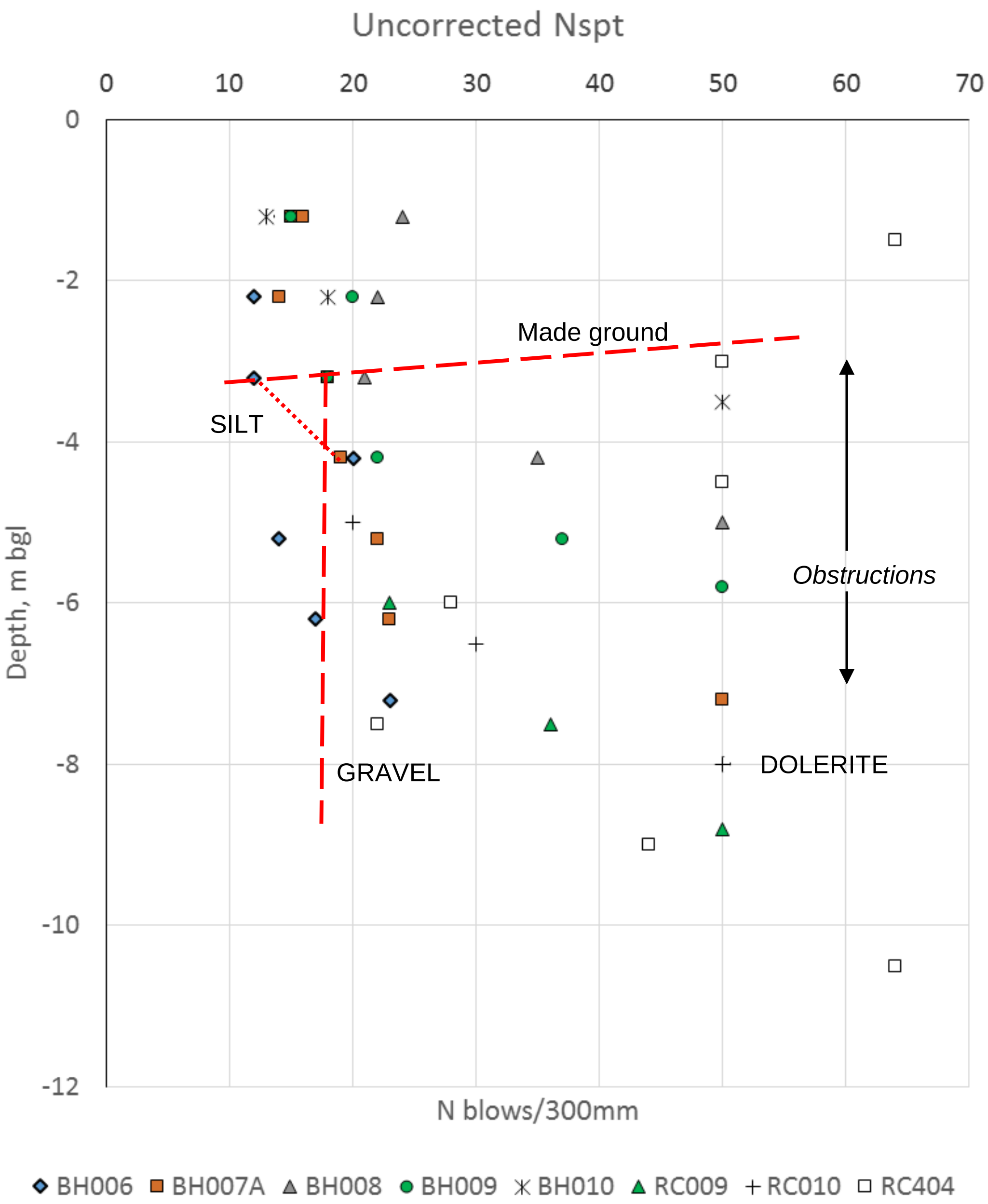
Within the granular Made ground deposits, noting inclusions of brick and slate; standard penetration test N values of 13 to 16 were measured describing medium dense deposits, indicative of an angle of friction of 30^o to 32^o.



Moisture content profile
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70
0 -1 -2 -3 -4 -5 -6 -7 -8
Depth, m bgl
Moisture content, %



Strength profile
0.50 0.60 0.70 0.80 0.90 1.00 1.10 1.20
0 -1 -2 -3 -4 -5 -6 -7 -8
Depth, m bgl
w/PL, ratio



The GRAVEL was characterised by 1% to 27% silt, 16% to 39% sand and 41% to 78% gravels and variable Cobble content (6% to 42%).

With Nspt values 14 to 36, allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^{o} to 36^{o} is expected of the medium dense to dense granular deposits. Taking a characteristic Nspt value of 18 in the dense GRAVEL deposits a stiffness modulus of 40MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

Particle size(s) d_{10} of 0.8mm to 0.063mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-3} ms^{-1} to 10^{-5} ms^{-1}. This described medium permeability (C113). Permeability was not determined in situ where it was not possible to develop a head above the groundwater level in the GRAVEL. This was indicative of 'high' permeability.

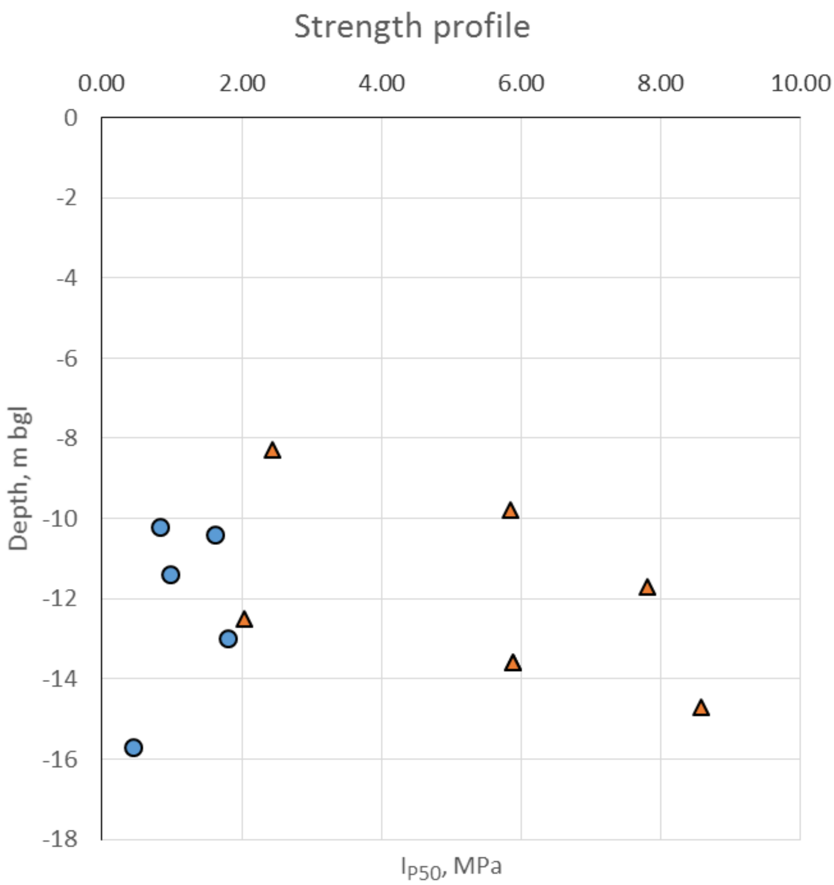
The Made ground was variable with Nspt 12 to 24, a characteristic value of 12 is recommended, indicative of an undrained shear strength 72kPa.

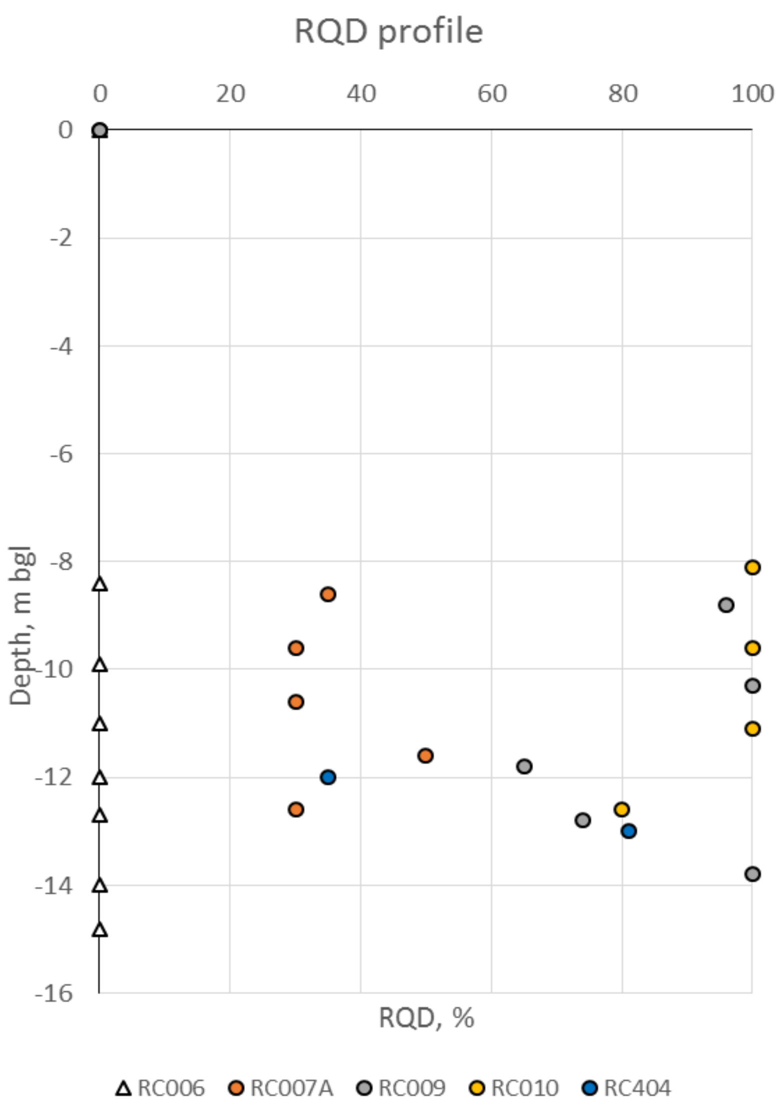
The Dolerite rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (medium strong to very strong I_{P50}, 2MPa – 8.6MPa; UCS 61MPa and 84MPa), fracture spacing (150mm – 650mm) and condition (slightly weathered), Rock Quality Designation (RQD 30% to 100%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 61, describing Class III - II fair to good Dolerite. A geological strength index, GSI (Hoek and Brown) of 50 is assumed for the partially disturbed blocky rockmass. An angle of friction, ϕ = 35^o and cohesion 300kPa are recommended for the upper CIII-II, Dolerite.

RC009 Dolerite

The Shale (BH006 and BH007A) rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to strong I_{P50}, 0.4MPa – 1.8MPa), fracture spacing (NI) and condition (highly weathered), Rock Quality Designation (RQD non-intact/ 0%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 31, describing Class IV poor Slatey-Mudstone. A geological strength index, GSI (Hoek and Brown) of 15 is assumed for the poorly interlocked highly broken rockmass. An angle of friction, ϕ = 20^o and cohesion 150kPa are recommended for the CIV, weak to medium strong, highly weathered Shale at BH006.

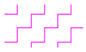
RC006 Shale


Strength profile
0.00
2.00
4.00
6.00
8.00
10.00
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
Depth, m bgl
I_{P50}, MPa



The geotechnical hazard in this area is considered to be; obstructions within the made ground and coarse Cobble and Boulder particles within the superficial deposits. Cobble and Boulder content may obstruct sheetpiles. Similarly inclusions within the Made ground may obstruct sheetpiles. The chiselling records are summarised as follows:

Location	Depth, m bgl Start	Depth, m bgl End	Duration; hh:mm	Strata
BH006	4.70	4.85	00:40	SILT
BH006	8.00	8.00	01:00	Bedrock
BH007A	7.20	7.20	01:00	Boulder Clay
BH008	2.35	2.45	00:40	
BH008	5.00	5.00	01:00	GRAVEL
BH009	5.80	5.80	01:00	GRAVEL
BH010	3.35	3.50	01:00	GRAVEL

The variable weathering profile in bedrock is also considered a geo-hazard with regard to depth of sheetpile installation. The geophysical survey layer  shall be considered a dense strata which may arrest the progress of sheetpiles.

It can be seen that boreholes were advanced by chiselling within the Made ground, firm to stiff SILT and obstructed in the stiff Boulder Clay and medium dense to dense GRAVEL. Boreholes terminated after one (1) hour chiselling on weathered bedrock.

Caution needs to be exercised not to overdrive and damage sheetpiles given the presence of coarse particles and variability in the rock type and quality of the rock mass. A ‘toe-hold’ only is expected in the weathered rock mass. The level of groundwater cut-off shall be assessed. Pre-boring may be required to advance the sheetpiles achieve the cut-off at bedrock level where both stiff cohesive and dense saturated granular deposits were encountered. Water-jetting may also be considered. It may be considered that jetting may offer a means of grouting the base of the sheetpiles in the weathered rock improving cut-off and fixing the toe. A Specialist sheetpiling contractor shall be consulted with regard the driving system, driven or vibratory or hybrid system best suited to the varied ground conditions. Vibration and noise shall be considered.

Made ground by its nature is variable, inclusions of brick, concrete and slate were encountered. In the absence of a construction detail, a presumed bearing capacity of 75kNm^{-2} to 150kNm^{-2} (kPa) is expected (BS8004; 1986) for foundations constructed within the firm SILT/ Made ground deposits 1.0m bgl. Standard penetration test N values suggested an allowable bearing pressure of 130kPa (Terzaghi and Peck, 1967) for settlements of 5mm to 10mm. Differential settlement of 10mm can be expected where the made ground was of varied thickness.

> *It is recommended to carry out two (2) number plate loading tests to verify the design bearing capacity at a depth 1.0m bgl within the Made ground at BH007 and BH009. Dynamic probing (20number, 15m spacing) shall also be considered to a depth 4.0m along the alignment of wall L03 to assess the variability in made ground.*
>
> *High permeability in the GRAVEL deposits is considered a geo-hazard. To complete the hydrogeological model it is recommended that in situ permeability tests (falling and constant head tests) are recommended at within the GRAVEL deposits at a location between existing exploratory locations; BH008 and BH009.*

6.1.10 Left Bank (BH011 – BH072) ch5+535m to ch4+900m

Construction of flood walls, L06/L07 and raising footway and/or pavement 0.1m to 1.2m, R06, are proposed along this section.

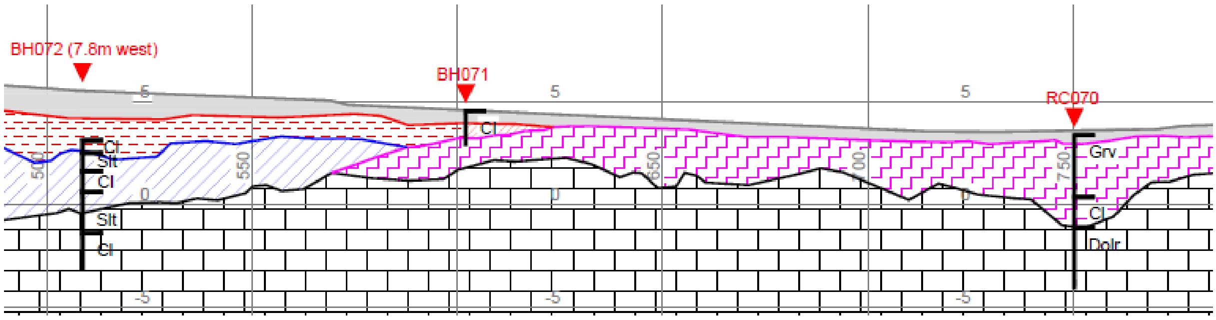
This section of the left bank of the Slaney River, downstream of the Enniscorthy Bridge, was characterised by Made ground; gravelly SILT/ silty sandy GRAVEL with varied Cobble content and brick/ slate inclusions to depths between 1.2m bgl to 2.5m bgl. This was underlain by mixed glacial deposits medium dense slightly silty very sandy GRAVEL and stiff slightly sandy slightly gravelly SILT with low Cobble content to depths 6.2m bgl to 6.6m bgl. Cobbles and Boulders were noted below 6.0m bgl. The granular deposits were locally loose at BH012 between 2.0m bgl to 4.0m bgl at between 3.0m bgl and 4.5m bgl at RC014. A zone of PEAT was encountered and assumed to be localised at BH014/ RC014, between 4.0m bgl and 6.0m bgl. RC014A indicated PEAT from 1.5m bgl to 7.5m bgl. Very soft to soft alluvial deposits; slightly sandy SILT underlay the Made ground below 2.7m bgl to 6.8m bgl between RC015 and BH072.

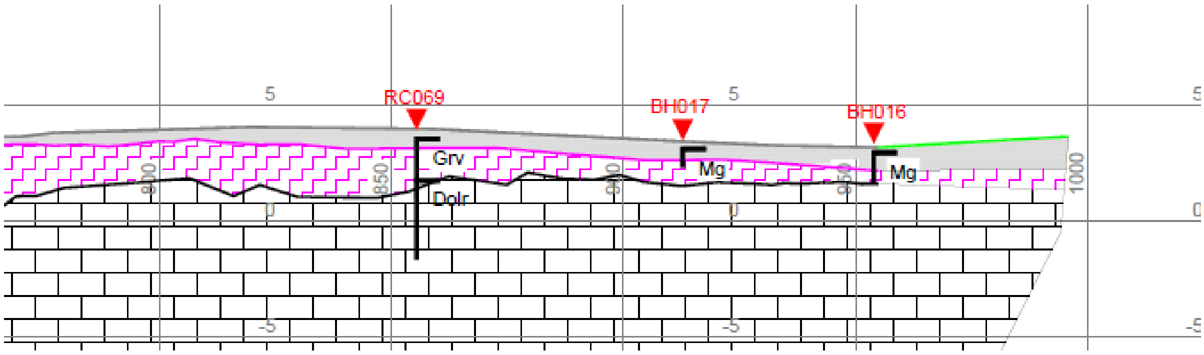
Stiff ‘Boulder Clay’ 1.3m thick overlay DOLERITE 8.1m bgl at RC015. The glacial deposits were underlain by weak to very strong DOLERITE 5.9m bgl (-2.8mOD) to 8.6m bgl (-6.15mOD). Bedrock became shallow at RC069, 1.5m bgl (2.06mOD) to 4.3m bgl (-0.93mOD). The depth to bedrock increased at RC072, 12.0m bgl (-8.9mOD). The bedrock profile was variable.

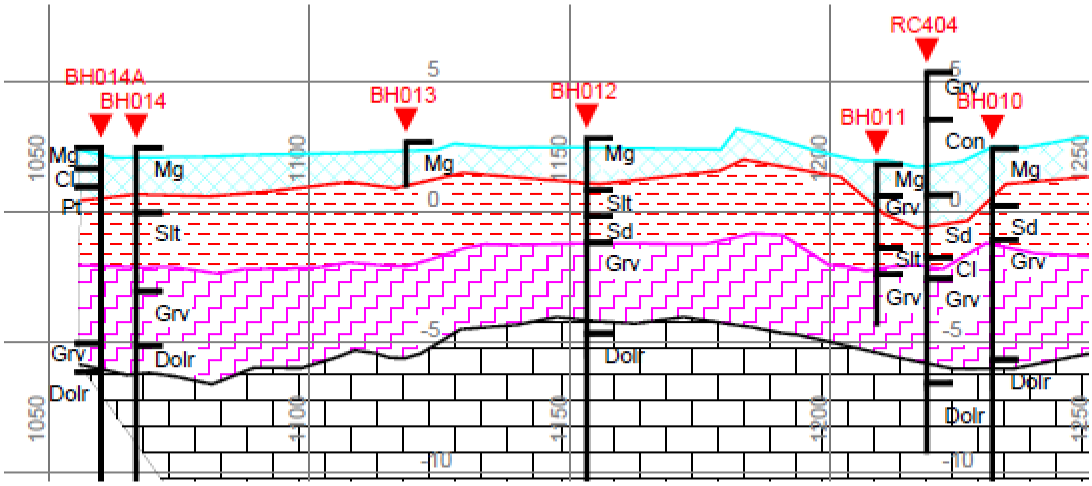
Slit trench ST003 indicated the foundation of the existing wall, at a single location, to be 0.25m bgl, 330mm thick and an assumed width, B 1050mm. The wall construction is considered poorly defined along its length and may be of varied construction.

Pavement construction consisted of 300mm to 400mm bituminous construction, 100mm to 300mm sub-base cl.804 or similar and formation (Fill) very gravelly SAND with Cobble content.

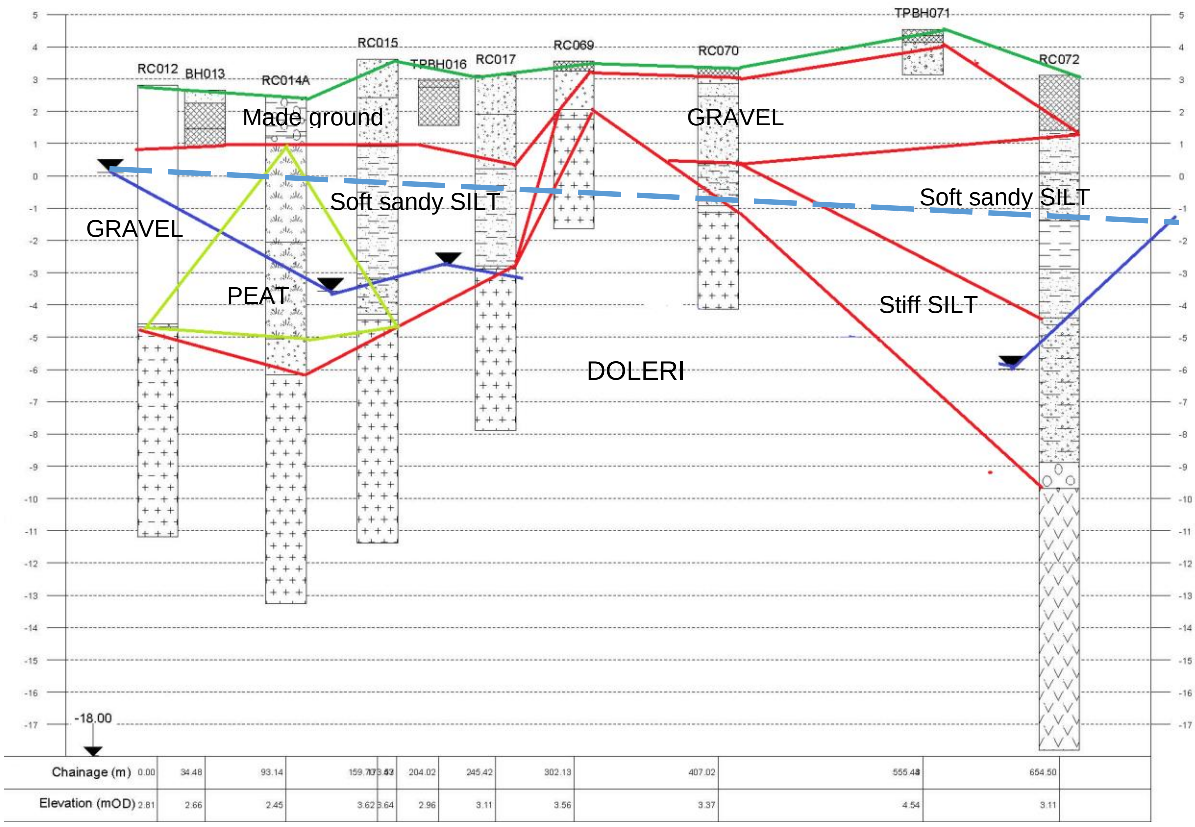
The geophysical survey (R12 and S12) indicated approximately 1m to 5m of superficial deposit; (Made ground) overlying medium dense (Granular deposits). Below 2.0mOD to -2.0mOD, a zone of dense Gravel (/) overlay shallow bedrock (Dolerite,) 3.0mOD to -6mOD.







The geophysical survey layer contoured the top of the dense GRAVEL. The ground conditions along this section were variable. The geotechnical section is presented as follows;



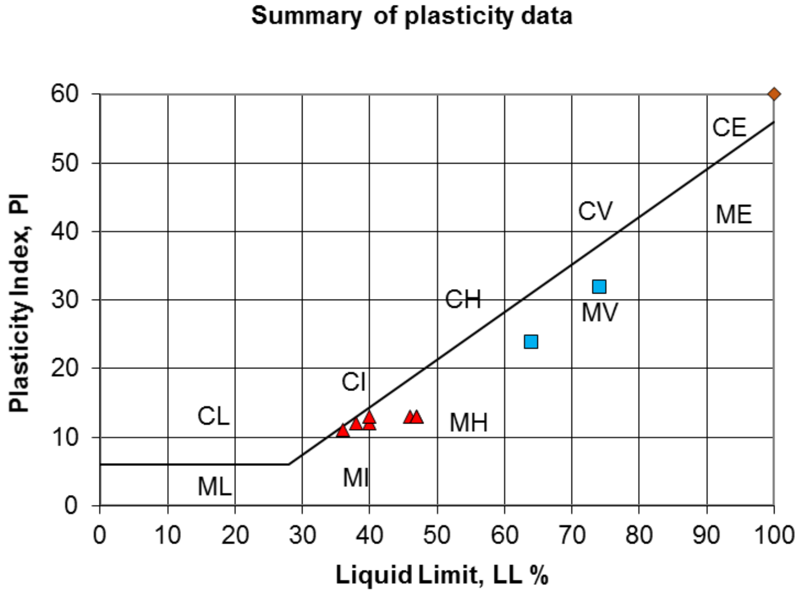
Groundwater (----/ ----) was encountered at depths 1.2m bgl to 7.2m bgl (1.6mOD to -3.6mOD). The static groundwater level was measured at RC022 at 2.6m bgl (0.24mOD). Installation of an *in situ* groundwater data logger is recommended to monitor variations in groundwater levels.

Further infill standpipe wells should be installed between RC017 and BH072 to fully assess the groundwater regime.

The Made ground, Gravels was characteristed by standard penetartion test N values 16 to 34. No further data was available. Based on a characteristic Nspt = 16 an angle of friction of 32^o is recommended. It is not recommended to found within the Made ground where these deposits were variable. The made groud Silt has not been characterised where there was insufficient data availale.

It is recommended that dynamic probing beundertaken to assess the shallwo made ground deposits. Window sampling shall also be considerd to recover samples to further characterise the shallow Made ground deposits.

The slightly sandy slightly gravelly SILT was of intermediate plasticity (MI). The SILT was of low organic content (loss on ignition 1.3% to 4.0%). The silt fraction was 61% to 67% with 1% to 25% gravels and 12% to 37% sand fractions. Cobble content was between 0% and 21%, low to high.



The high and very high plasticities (MH – MV) corresponded to BH014 and BH072. The elevated plasticity ha sbeen assumed indicative peaty, organic fine grained deposits. The silt fraction was 85% with 3% gravels and 5% sand fractions. Cobble content was 6%, medium.

Natural moisture content, w ranged between 28% and 78%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.9 to 1.8 indicative of firm (1.0 < *w*/PL < 1.2) and very soft deposits, *w*/PL >1.2 (C504).

Standard penetration tests Nspt values ranged between 3 to 7 in the alluvial SILT deposits. With a plasticity index, PI 11 – 13; a factor $f_1 = 6$ (Stroud, 1975) was such to yield undrained shear strengths of 18kPa to 42kPa, describing soft to firm alluvial deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 28^o for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

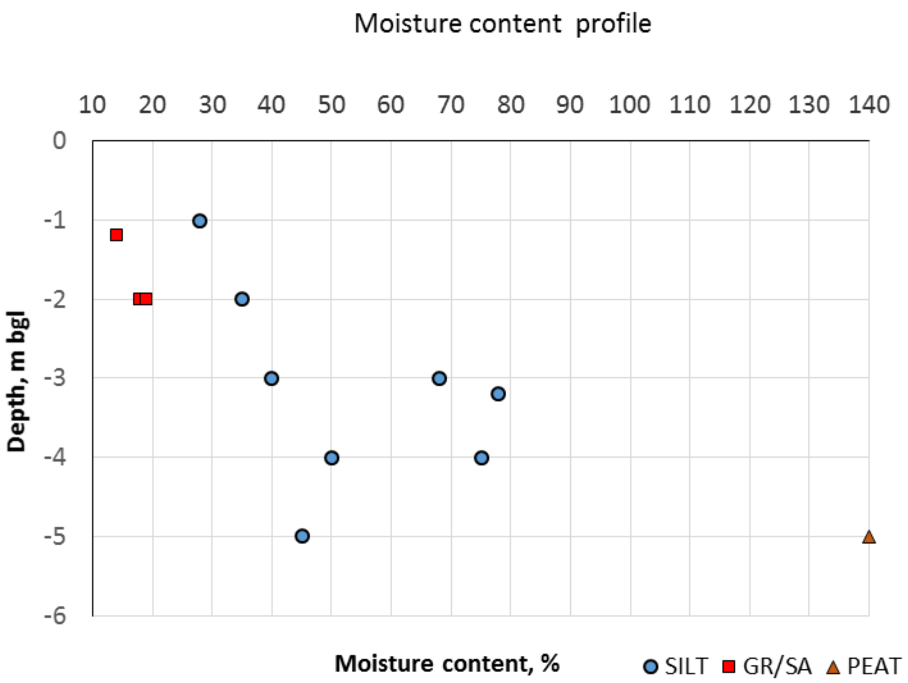
The lower glacial SILT deposits yielded Nspt values 12 to 17. This was indicative of undrained shear strenght 72kPa to 102kPa being stiff (BS5930, 1999). These deposits were characterised by Nspt alone.

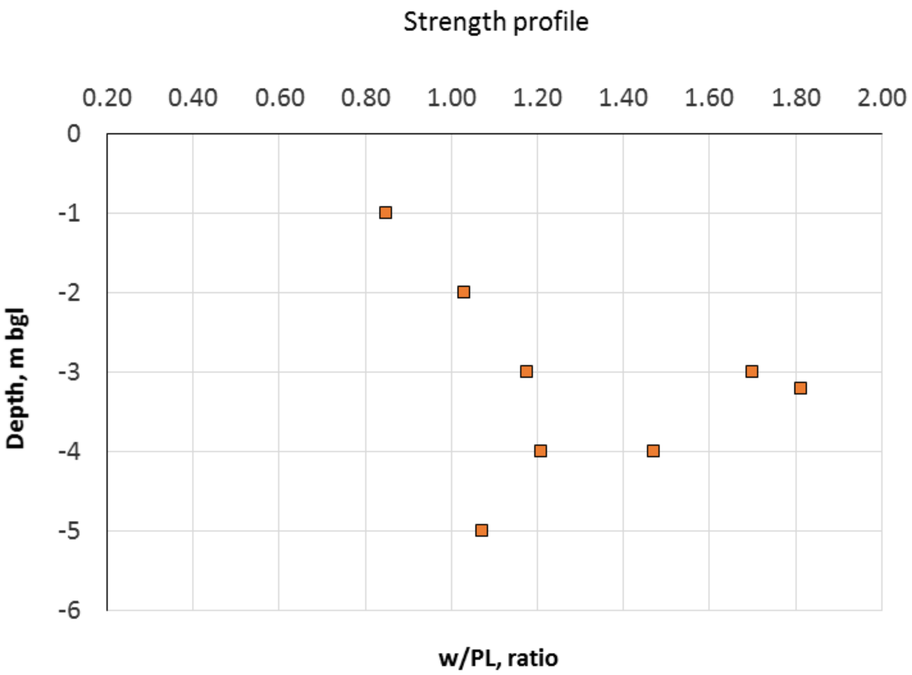
Compressibility of the alluvial deposits is expected to be variable being moderately compressibility (MI-MH).

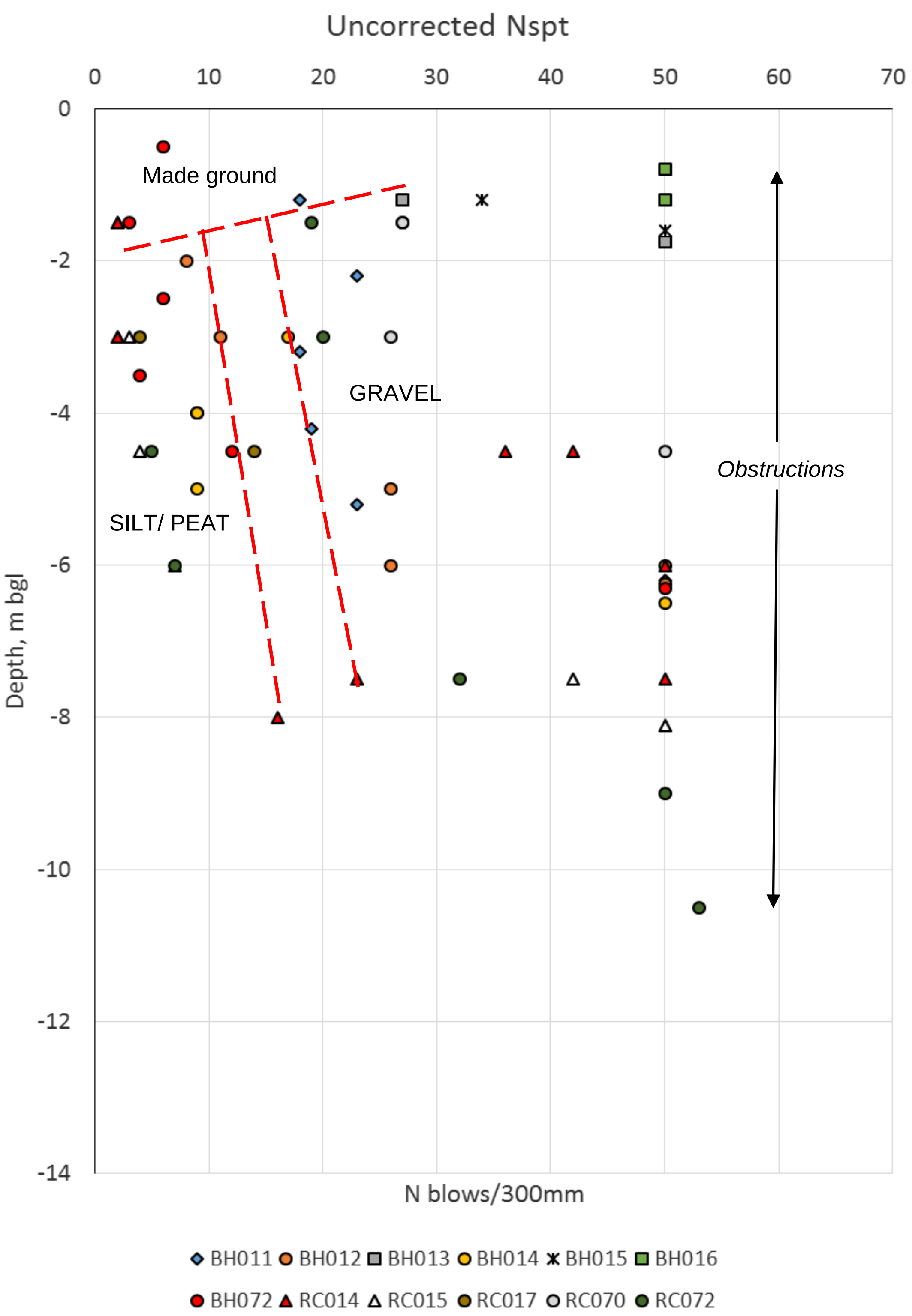
Taking a characteristic Nspt value of 4 in the upper SILT, an unfactored stiffness modulus (Young's modulus, E) of 4MPa is expected (PI 11 and 13, Stroud, 1975). Taking a characteristic Nspt value of 12 in the upper SILT, an unfactored stiffness modulus (Young's modulus, E) of 14MPa is expected of the glacial SILT deposits (Stroud, 1975).

Permeability in the SILT was determined based on particle size d_{10} being 0.002mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works).

Within the granular Made ground deposits, noting inclusions of brick and slate; standard penetration test N values of 27 and 34 were measured describing medium dense to dense deposits. The N values are considered elevated due to brick inclusions and so no friction angle is where there is no data above a depth 1.2m bgl.




Strength profile
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
0
-1
-2
-3
-4
-5
-6
Depth, m bgl
w/PL, ratio



The GRAVEL was characterised by 1% to 12% silt, 18% to 54% sand and 37% to 70% gravels and variable Cobble content (0% to 32%).

With Nspt values 16 to 42, allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32º to 36º is expected of the medium dense to dense granular deposits. Taking a characteristic Nspt value of 26 in the medium dense GRAVEL deposits a stiffness modulus of 60MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

Particle size(s) d_{10} of 0.7mm to 0.063mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-3} ms^{-1} to 10^{-5} ms^{-1}. This described medium permeability (C113). *In situ* permeability was not assessed in the GRAVELS.

The Made ground was variable with Nspt 8 to 34, a characteristic value of 25 is recommended.

The PEAT was characterised by extremely high plasticity (ME) and natural moisture content 140%. Standard penetration test Nspt values of 2 to 9 were measured. Permeability was measured in situ in the yielding a value of 4.04×10^{-7} ms^{-1} (RC014A.)

The Dolerite rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to very strong I_{P50}, 0.24MPa – 7.9MPa; UCS 18MPa to 49MPa), fracture spacing (80mm – 840mm) and condition (slightly to moderately weathered), Rock Quality Designation (RQD 0% to 100%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 33 to 69, describing Class IV and Class - III poor and fair Dolerite. A geological strength index, GSI (Hoek and Brown) of 30 to 50 is assumed for the partially disturbed blocky rockmass.

	RC012	**RC014**	**RC014A**	**RC015**	**RC017**	**RC069**	**RC070**	**RC072**
	42	38	33	42	47	57	45	52
RMR	59	45	40	61	69	57	58	62
Class	III	IV-III	IV-III	III	III-II	III	III	III
GSI	45	30	30	45	50	45	45	45
Friction ϕ, º	33	28	28	33	35	33	33	33
Cohesion, kPa	250	200	200	250	350	250	250	325

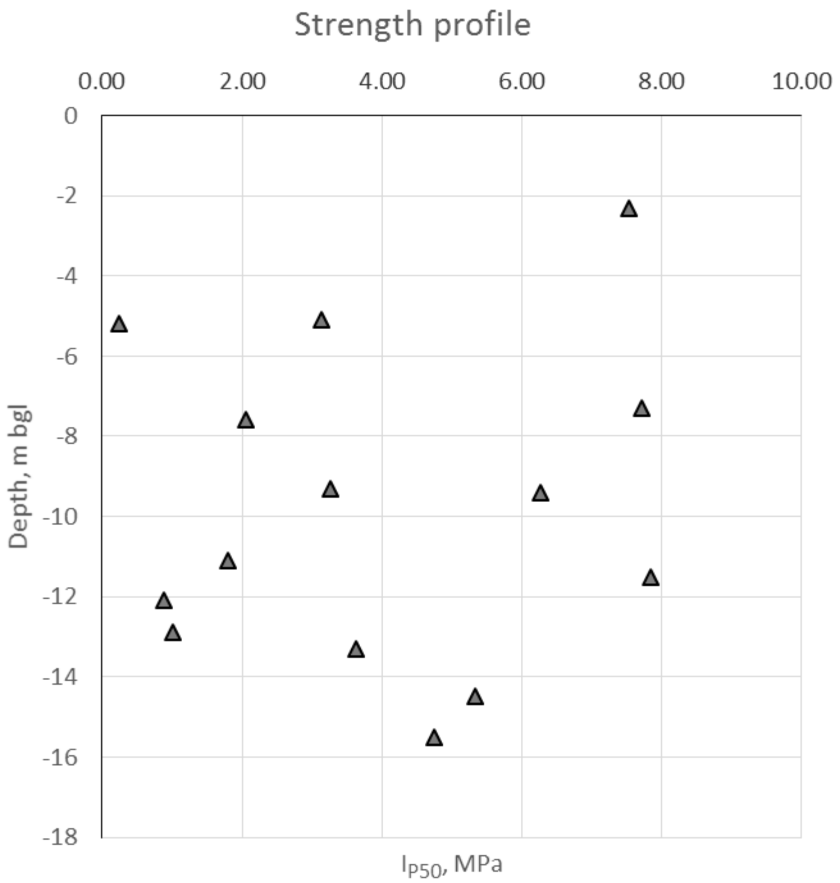
RC012 Dolerite

RC014 Dolerite

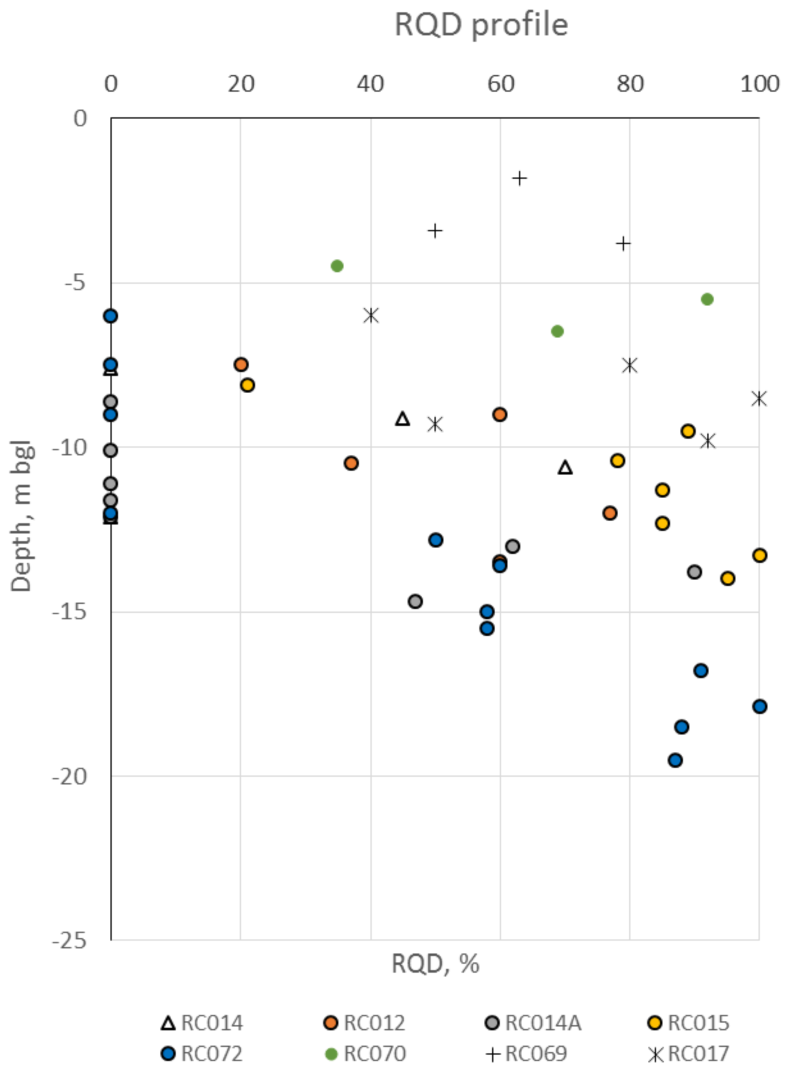
RC017 Dolerite

RC070 Dolerite

Permeability was measured in situ in the Dolerite yielding a value of 5.04 $x10^{-7}$ ms^{-1} (RC012). It must be noted that at permeabilty wil be a function of the rock quality which was variable. A review of the RQD data indicated RC012 to be characteristic of the rock mass in this area with th eexceptionof RC014 and RC072 where th eupper rock mass was more fractured and expected to have a higher (undetermined) permeability.



Strength profile
0.00
2.00
4.00
6.00
8.00
10.00
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
Depth, m bgl
I P50, MPa



The geotechnical hazard in this area is considered to be; obstructions within the made ground and coarse Cobble and Boulder particles within the superficial deposits. Cobble and Boulder content may obstruct sheetpiles. Similarly inclusions within the Made ground may obstruct sheetpiles. The chiselling records are summarised as follows:

Location	Depth, m bgl		Duration; hh:mm	Strata
	Start	End		
BH012	6.1	6.25	01:00	GRAVEL
BH013	1.7	1.75	01:00	Made ground
BH014	1.4	1.7	01:00	Made ground
BH014	6.5	6.6	01:00	GRAVEL
BH014A	0.6	0.65	00:30	Made ground
BH015	1.6	1.6	01:00	Made ground
BH016	1.3	1.3	01:00	Made ground
BH017	0.8	0.8	01:00	Made ground

Location	Depth, m bgl		Duration; hh:mm	Strata
BH068	0.5	0.6	01:00	GRAVEL
BH068	1.1	1.3	01:00	Bedrock
BH071	1.6	1.7	01:00	SILT/ Made ground
BH072	0.4	0.5	00:30	SILT/ Made ground
BH072	6.3	6.3	01:00	GRAVEL

The variable rock profile (shallow bedrock) is also considered a geo-hazard with regard to depth of sheetpile installation. The geophysical survey layer shall be considered a dense strata which may arrest the progress of sheetpiles.

It can be seen that boreholes were advanced by chiselling within the Made ground, firm to stiff SILT and obstructed in the stiff Boulder Clay and medium dense to dense GRAVEL. Boreholes terminated after one (1) hour chiselling on weathered bedrock. It is recommended pre-boring and advancing cable percussion boreholes BH014 to BH016 to fully assess the superficial deposits where shallow Coble/ Boulder obstructions resulted in boreholes being terminated. Groundwater monitoring and in situ permeability testing shall be carried out to further inform the ground model in these areas.

Caution needs to be exercised not to overdrive and damage sheetpiles given the presence of coarse particles and variability in the rock type and quality of the rock mass. A 'toe-hold' only is expected in the weathered rock mass. The level of groundwater cut-off shall be assessed. Pre-boring may be required to advance the sheetpiles achieve the cut-off at bedrock level where both stiff cohesive and dense saturated granular deposits were encountered. Water-jetting may also be considered. It may be considered that jetting may offer a means of grouting the base of the sheetpiles in the weathered rock improving cut-off and fixing the toe. A Specialist sheetpiling contractor shall be consulted with regard the driving system, driven or vibratory or hybrid system best suited to the varied ground conditions. Vibration and noise shall be considered.

Made ground by its nature is variable. In the absence of a construction detail, a presumed bearing capacity of 100kNm^{-2} to 600kNm^{-2} (kPa) is expected (BS8004; 1986) for foundations constructed within the medium dense GRAVEL/ loose SAND deposits below 1.2m bgl (L06). Standard penetration test N values suggested an allowable bearing pressure of 80kPa to 200kPa (Terzaghi and Peck, 1967) for settlements of 25mm. Differential settlement of 10mm can be expected where the made ground was of varied thickness and stiffness/ relative density.

Location BH014 is considered separately where very loose GRAVEL and very soft PEAT was encountered below 3.0m bgl. It is recommended keep foundations at a depth 0.5m bgl in the Made ground and reduce bearing to 30kPa for 1.5m wide retaining wall type construction. Settlement up to 10mm is expected.

> *It is recommended to undertake three (3) additional boreholes or dynamic probing to address this variation in ground condition between BH013 and BH014.*

From ch5+350 to ch4+820m (L07) the Made ground was underlain by very soft to soft SILT deposits (Nspt = 4). It is recommended keep foundations at a depth 0.5m bgl in the made ground and reduce bearing to 30kPa for 1.5m wide retaining wall type construction. Settlement up to 10mm to 15mm is expected.

> *It is recommended to undertake plate loading test within the made ground at 0.5m bgl at a number of locations to asses bearing capacity for shallow foundations for the proposed wall construction L07.*

Pavement construction consisted of 300mm to 400mm bituminous construction, 100mm to 300mm sub-base cl.804 or similar and formation (Fill) very gravelly SAND with Cobble content. With overlay proposed 100mm to 1200mm (R06) it is expected for thicknesses up to 150m a bituminous surfacing be laid. Where the depth of overlay exceeds 150mm it is recommended to construct the pavement with subbase cl.804or similar with bituminous construction to match the design traffic loading in accordance with the relevant pavement design standard. Where PAH has not been assessed, planning has not been considered.

For the foothway is will be appropriate to overlay the surfacing or provide for granular fill and sub-base in accordance with the relevant design detail.

> *It is recommended to further assess the pavement construction using TRL cone penetration tests to establish the California bearing ratio, CBR of the formation deposits. A pavement condition survey is also recommended to identify area of defect to allow for appropriate remediation. Falling weight deflectometer, FWD stage and stage 2 analysis shall also be considered with regard to pavement overlay design.*

High permeability in the GRAVEL deposits is considered a geo-hazard. In situ permeability should be assessed by means of constant head testing in a suitably sized test well in the vicinity of BH012.

6.1.11 Right Bank (RC030 – BH101) ch5+690m to ch4+900m

Construction of flood wall L05 and embankment works EX6/ EX7 are proposed along this section.

This section of the right bank of the Slaney River, downstream of the Enniscorthy Bridge, was characterised by hardstanding 300mm to 400mm thick, bituminous surfacing. This was underlain by Made ground described as medium dense sandy GRAVEL with Cobble content, 1.15m to 4.2m thick to depths between 1.2m bgl to 4.2m bgl. The Made ground was underlain by medium dense (slightly) silty very sand GRAVEL to a depth 6.9m bgl to 10.5m bgl. The GRAVEL overlay medium strong to very strong DOLERITE 6.9m bgl (-4.3mOD) to 10.5m bgl (-6.1mOD).

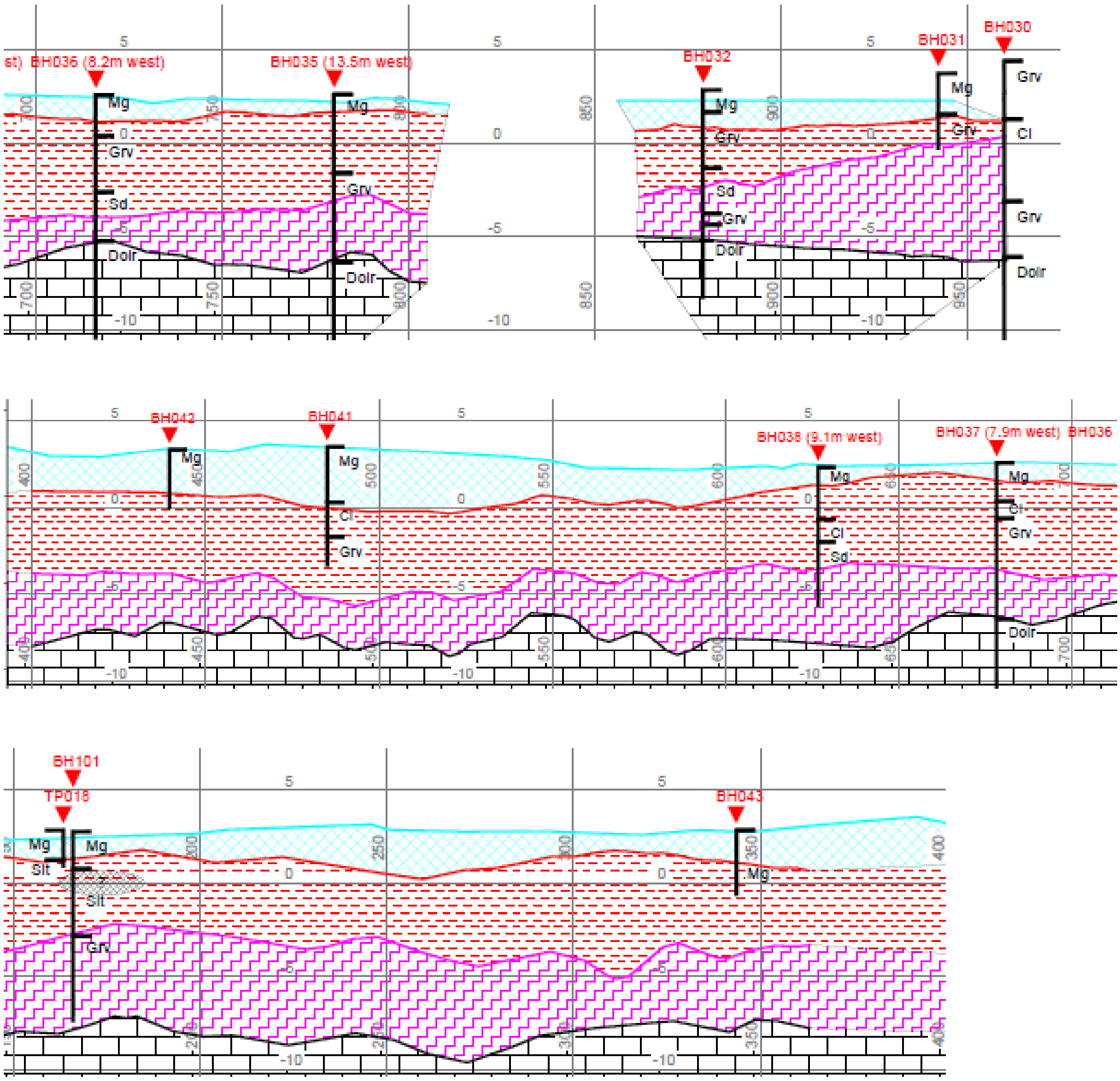
At location BH033 there was evidence of historical fill with concrete encountered 2.0m bgl to 2.2m bgl. Made ground was 3.5 thick being described as very dense GRAVEL with concrete inclusions. The Made ground was underlain by loose silty sandy GRAVEL to a depth 5.5m bgl becoming medium dense below this with increased Gravel content to a depth 8.7m bgl.

Topsoil 100mm thick was encountered at BH034 overlying made ground 2.4m thick; soft slightly sandy gravelly SILT with glass inclusions. The Made ground was underlain by slightly sandy GRAVEL to a depth 5.0m bgl. The relative density was varied being loose and dense. A layer of PEAT 800mm thick was present 5.0m bgl to 5.8m bgl. Being underlain by GRAVEL to a depth 6.5m bgl. At BH38 a layer of soft slightly sandy gravelly peaty SILT underlay the made ground 1.3m thick to a depth 4.3m bgl. The SILT was underlain by GRAVEL. A layer of medium dense slightly silty very gravelly SAND 2.4m thick overlay the bedrock at RC036.

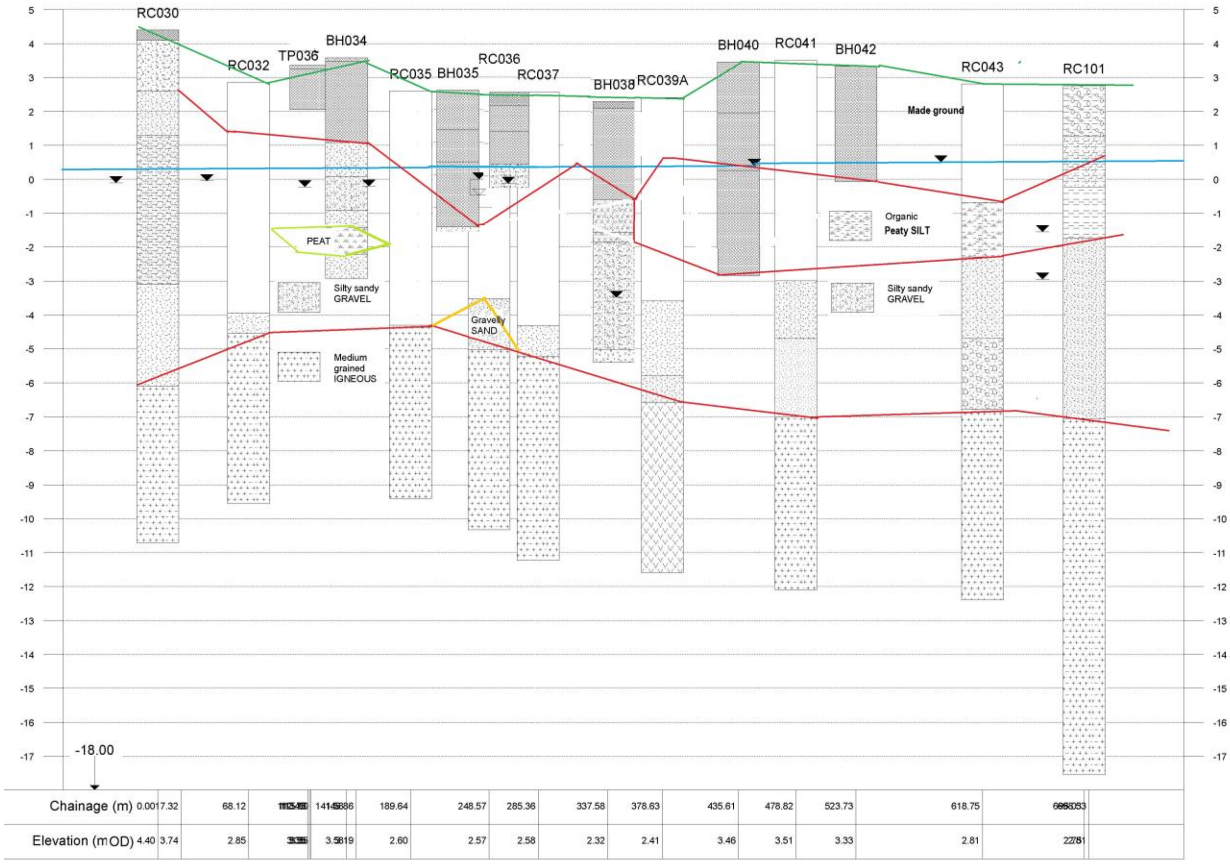
Beyond BH40 Made ground; slightly sandy gravelly SILT with low Cobble content, Boulder content (below 3.0m bgl) and with brick and glass inclusions was present 2.0m to 3.5m thick underlain by very soft and firm slightly sandy slightly gravelly peaty SILT to depths between 2.0m bgl to 5.0m bgl. The SILT was underlain by medium dense slightly silty very sandy

GRAVEL with medium Cobble content to 9.6m bgl to 10.5m bgl. Weak to very strong DOLERITE underlay the Gravel deposits (-6.79mOD to -7.05mOD).

The geophysical survey (R14, R6, R6f, S17, S8 and S8') indicated approximately 1.5m to 9m of superficial deposit; (Made ground) overlying medium dense (Granular deposits). Below 0.0mOD to -4.0mOD, a zone of dense, Gravel ( ) 2m to 4m thick overlay bedrock (Dolerite,) -5.5mOD to -8.5mOD.



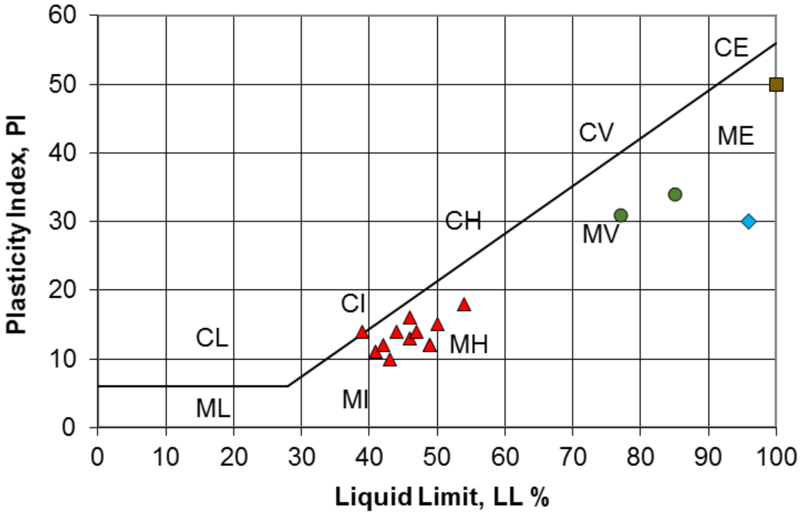
The geophysical survey layer contoured the top of the dense GRAVEL. The bedrock and weathering profile is variable. The geotechnical section is presented as follows;



Groundwater (----) was encountered at depths 2.0m bgl to 6.1m bgl (0.6mOD to -1.6mOD). The static groundwater level was measured at RC037 at 2.38m bgl (0.2mOD). Installation of an *in situ* groundwater data logger is recommended to monitor variations in groundwater levels.

The Made ground, Gravels was characteristed by standard penetartion test N values 7 to 37. Based on a characteristic Nspt = 15 an angle of friction of 32° is recommended. Natural moisture content ranged between 6% and 27%. The Made groud Silt was characterised by standard penetartion test N values 7 to 23. Based on a characteristic Nspt = 11 an undrained shear strength 55kPa is expected of the Silt with a range of 35kPa to 85kPa is recommended. Natural moisture content ranged between 17% and 35%. It is not recommended to found within the Made ground where these deposits were variable.

Summary of plasticity data



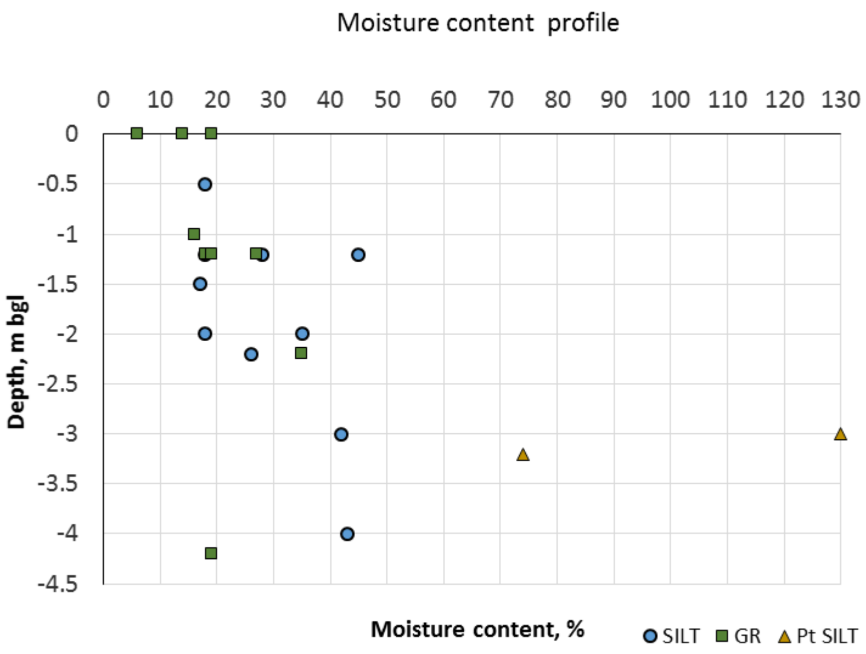
The slightly sandy slightly gravelly SILT was of intermediate and high plasticity (MI – MH). The SILT was of low to medium organic content (loss on ignition 0.97% to 7.5%). At BH041 extremely high plasticity (ME) deposits were found to underlie the made ground below 3.2m bgl. The SILT was of low to medium organic content (loss on ignition 4.7%; BH041 and 11%, BH038). The silt fraction was 28% to 76% with 1% to 42% gravels and 23% to 40% sand fractions.

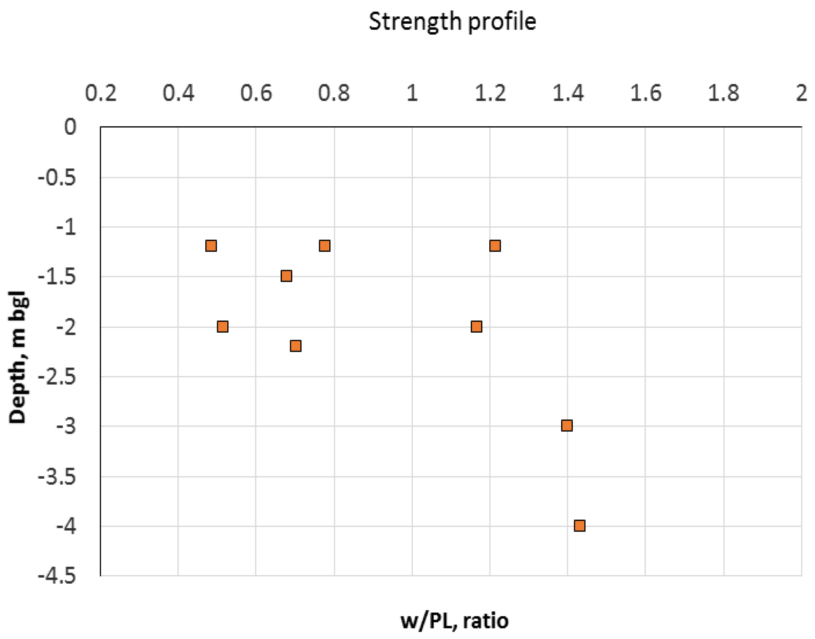
Natural moisture content, w ranged between 18% and 45%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.5 to 1.4 indicative of stiff (<1.0) and soft deposits, *w*/PL >1.2 (C504).

Standard penetration tests Nspt values ranged between 3 to 14 were measured in the alluvuial SILT deposits. With a plasticity index, PI 10 – 18; a factor f_1 = 5.5 to 6 (Stroud, 1975) was such to yield undrained shear strengths of 17kPa to 84kPa, describing very soft to stiff deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 26^o to 30^o for the sandy gravelly SILT (C504; Terzaghi, Peck & Mesri, 1996).

Taking a characteristic Nspt value of 5 in the alluvial SILT, an unfactored stiffness modulus (Young's modulus, E) of 7MPa to 9MPa is expected (PI 10 and 18, Stroud, 1975). Taking a characteristic Nspt value of 14 in the glacial SILT, an unfactored stiffness modulus (Young's modulus, E) of 20MPa to 25MPa is expected (Stroud, 1975). Compressibility of the SILT is expected to be variable being moderately compressibility (MI-MH).

Permeability in the SILT was determined based on particle size d_{10} being 0.002mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works).





The GRAVEL was characterised by 4% to 28% silt, 5% to 47% sand and 50% to 89% gravels and variable Cobble content (0% to 35%).

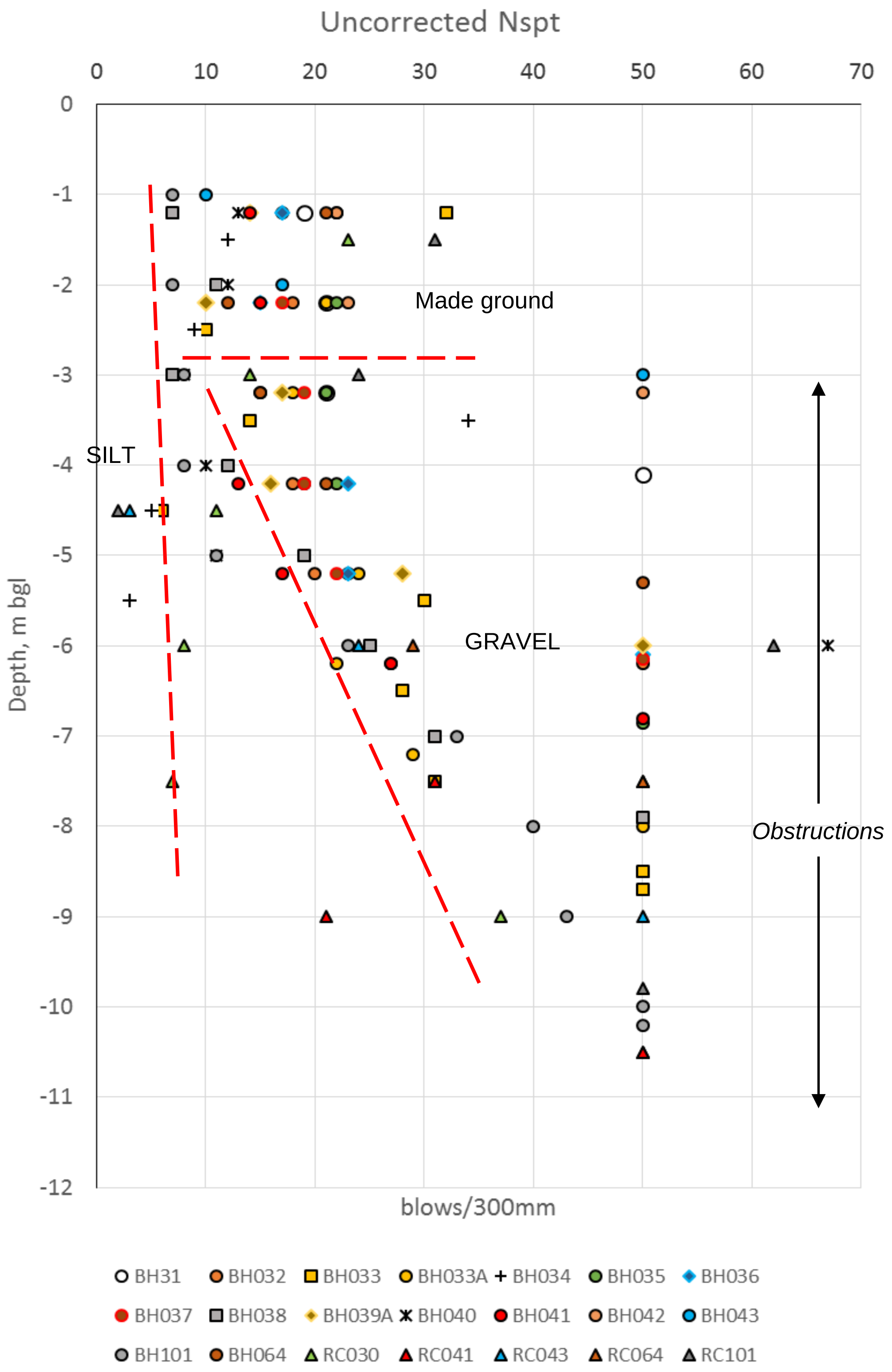
With Nspt values 13 to 43, allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^o to 38^o is expected of the medium dense to dense granular deposits. Some localised loose deposits (Nspt 6) were encountered at BH33 these have been interpreted as having organic content and may represent a peaty lense identified at BH34. Taking a characteristic Nspt value of 23 in the medium dense GRAVEL deposits a stiffness modulus of 45MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967).

Particle size d_{10} of 0.6mm to 1mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-3} ms^{-1}. This described medium to high permeability (C113). It was noted that where the Gravel fraction increased (83% to 89%) particle size d_{10} of 2mm to 5mm was measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-1} ms^{-1}, high permeability. Furthermore it was noted that where the Silt fraction increased (17% to 28%) particle size(s) d_{10} of 0.005mm to 0.001mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-7} ms^{-1}, low permeability. *In situ* permeability in the GRAVEL immediately above the Dolerite at RC035 and BH038 yielded permeabilities of 1.6×10^{-6} ms^{-1} and 5.3×10^{-6} ms^{-1}, indicative of low permeability. Particle sizes suggested higher values. In some instances permeability was not determined *in situ* where it was not possible to develop a head above the groundwater level in GRAVELS (BH032 and BH037). This was indicative of 'high' permeability noting particle size d_{10} of 1mm to 2mm was measured.

The Made ground was variable with Nspt 7 to 32, a characteristic value of 15 is recommended. At BH37 and BH041 very to extremely high plasticity was noted (MV-ME) in the cohesive fills. This was indicative of organic content and a high level of compressibility.

Soft peaty deposits were encountered typically 1.0m thick characterised by Nspt 3 to 9 (BH33 and BH34). Natural moisture content of 74% to 130% was measured with extremely high plasticity (ME). Compressibility of the Peaty deposits is expected to have a high level of compressiblity.

Organic SILT deposits (PI 34; f_1 = 4) yielded Nspt values 10 to 15. Undrained shear strengths of 40kPa to 60kPa, described soft to firm deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 26º for the sandy gravelly SILT (C504; Terzaghi, Peck & Mesri, 1996).



The Dolerite rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to very strong I_{P50}, 0.2MPa – 9.8MPa; UCS 13MPa to 41MPa), fracture spacing (80mm – 750mm) and condition (slightly to moderately weathered), Rock Quality Designation (RQD 0% to 100%) and groundwater (assumed 'wet' to 'damp' within the zone of influence) was undertaken. The rock mass rating, RMR range was 34 to 64, describing a variable rock mass; Class IV and Class - II poor to good Dolerite. The variation was primarily associated with non-intact weathered zones in the upper sections of the rock mass. A geological strength index, GSI (Hoek and Brown) of 30 to 55 is assumed for the partially disturbed blocky rockmass.

	RC030	**RC032**	**RC033**	**RC035**	**RC036**	**RC037**	**RC039A**	**RC041**	**RC043**	**RC064**	**RC101**
	36	47	56	45	44	42	58	37	37	42	34
RMR	39	58	64	62	54	61	61	52	49	54	34
Class	IV	III	II-III	III	III	III	II	IV-III	IV-III	III	IV
GSI	30	40	45	40	40	40	55	30	30	40	30
Friction ϕ, °	22	33	35	32	32	32	35	26	26	30	20
Cohesion, kPa	150	250	350	200	250	250	300	150	150	250	150

RC030 Dolerite

RC032 Dolerite


RC033 Dolerite

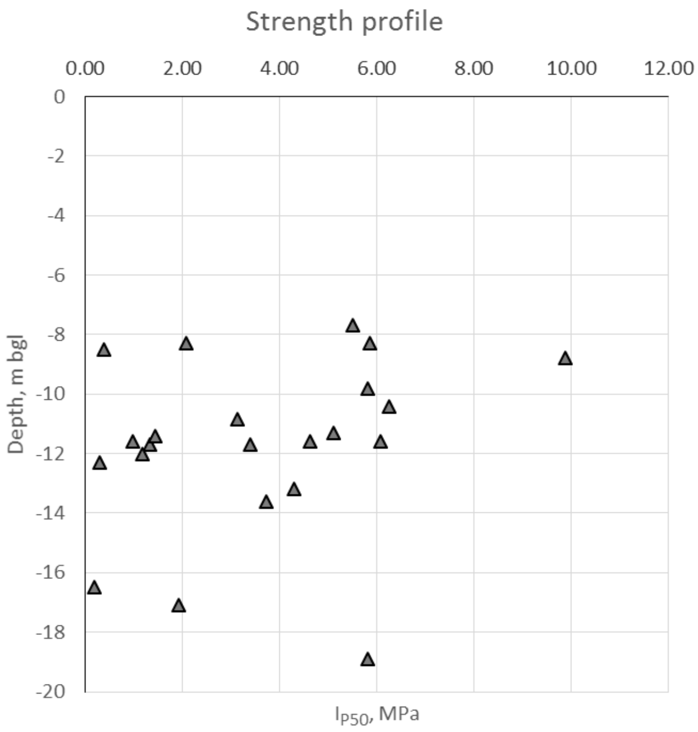

RC036 Dolerite

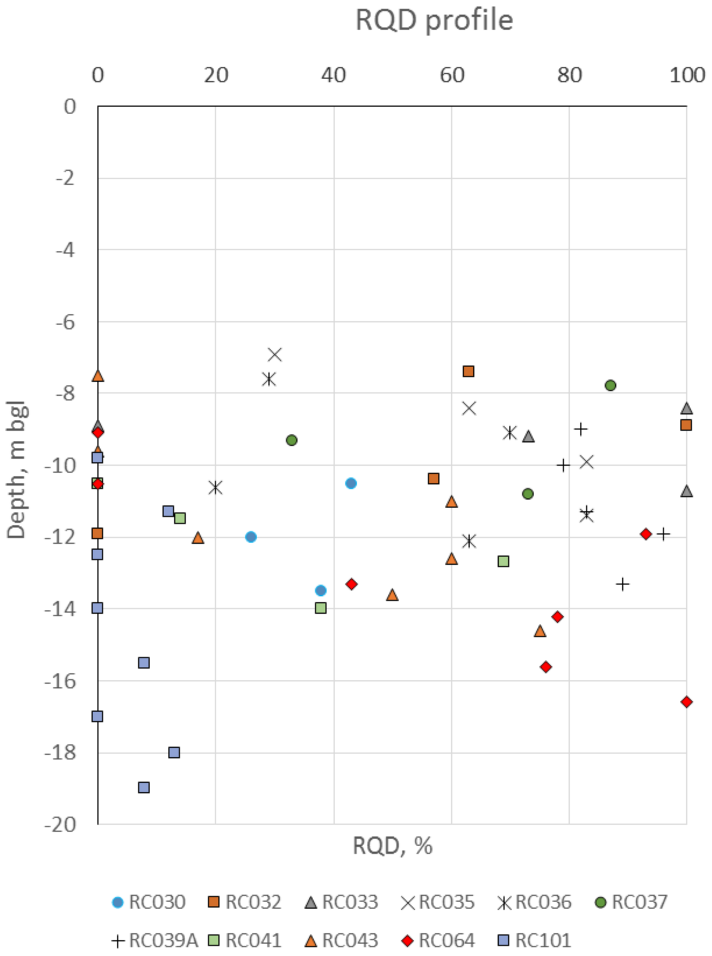
RC039A Dolerite


RC041 Dolerite

RC064 Dolerite

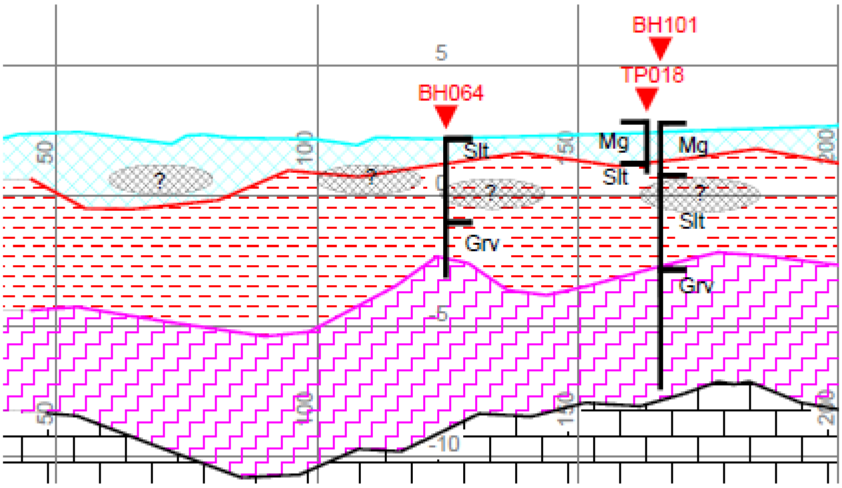
Permeability of the Dolerite over this section has not been assessed. Permeability is expected to be variabll based on the quality of the rock encountered, particularly RC041, RC043, RC064 and RC101.


Strength profile
0.00
2.00
4.00
6.00
8.00
10.00
12.00
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth, m bgl
I_{P50}, MPa


RQD profile
0
20
40
60
80
100
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth, m bgl
RQD, %
RC030
RC032
RC033
RC035
RC036
RC037
RC039A
RC041
RC043
RC064
RC101

The geotechnical hazards in this area are considered to be obstructions within the Made ground and coarse Cobble and Boulder particles within the superficial deposits. Cobble and Boulder content may obstruct sheetpiles. Similarly inclusions within the Made ground may obstruct sheetpiles. Cable percussion boring was advanced at BH33 using water to aid drilling in very dense Made ground; GRAVEL. At location BH033 concrete encountered 2.0m bgl to 2.2m bgl. RC033A terminated at 1.2m bgl on concrete obstruction. It will be required to pre-excavate along this section to remove the concrete obstruction. The extent of this in plan is not defined and represents a geotechnical hazard. The chiselling records are summarised as follows:

	Depth, m bgl			Strata
Location	**Start**	**End**	**Duration; hh:mm**	
BH031	2.8	2.9	00:30	GRAVEL
BH032	6.2	6.2	01:00	
BH033	2.0	2.2	01:00	Made ground
BH033	8.7	8.7	01:00	DOLERITE
BH033A	5.5	5.65	00:30	
BH033A	7.9	8.0	01:00	
BH034	6.2	6.5	01:00	GRAVEL/ DOLERITE
BH035	2.3	2.5	00:40	Made ground
BH035	6.8	6.85	01:00	DOLERITE
BH036	6.1	6.1	01:00	GRAVEL
BH037	6.1	6.15	01:00	
BH038	6.5	6.7	01:00	
BH038	7.9	8.0	01:00	
BH039A	6.0	6.0	01:00	
BH040	6.3	6.3	01:00	
BH041	6.9	6.9	01:00	
BH042	2.9	3.0	00:50	Made ground
BH042	3.4	3.4	01:00	
BH043	3.4	3.5	01:00	
BH064	5.2	5.3	01:00	GRAVEL
BH101	6.8	7.0	00:30	
BH101	7.6	7.7	00:30	
BH101	10.1	10.2	00:30	DOLERITE



A series of sub-surface anomalies were noted which may represented possible buried structures. These are obstructions and a geotechnical risk with regard sheetpiling. Trial excavations are recommended to assess the nature of the geophysical anomaly.

The variable profile in bedrock is also considered a geo-hazard with regard to depth of sheetpile installation. The geophysical survey layer shall be considered a dense strata which may arrest the progress of sheetpiles.

It can be seen that boreholes were advanced by chiselling within the Made ground, medium dense to dense GRAVEL and bedrock, Dolerite.

Caution needs to be exercised not to overdrive and damage sheetpiles given the presence of coarse particles and variability in the rock type and quality of the rock mass. A 'toe-hold' only is expected in the weathered rock mass. The level of groundwater cut-off shall be assessed. Pre-boring may be required to advance the sheetpiles achieve the cut-off at bedrock level where both stiff cohesive and dense saturated granular deposits were encountered. Water-jetting may also be considered. It may be considered that jetting may offer a means of grouting the base of the sheetpiles in the weathered rock improving cut-off and fixing the toe. A Specialist sheetpiling contractor shall be consulted with regard the driving system, driven or vibratory or hybrid system best suited to the varied ground conditions. Vibration and noise shall be considered.

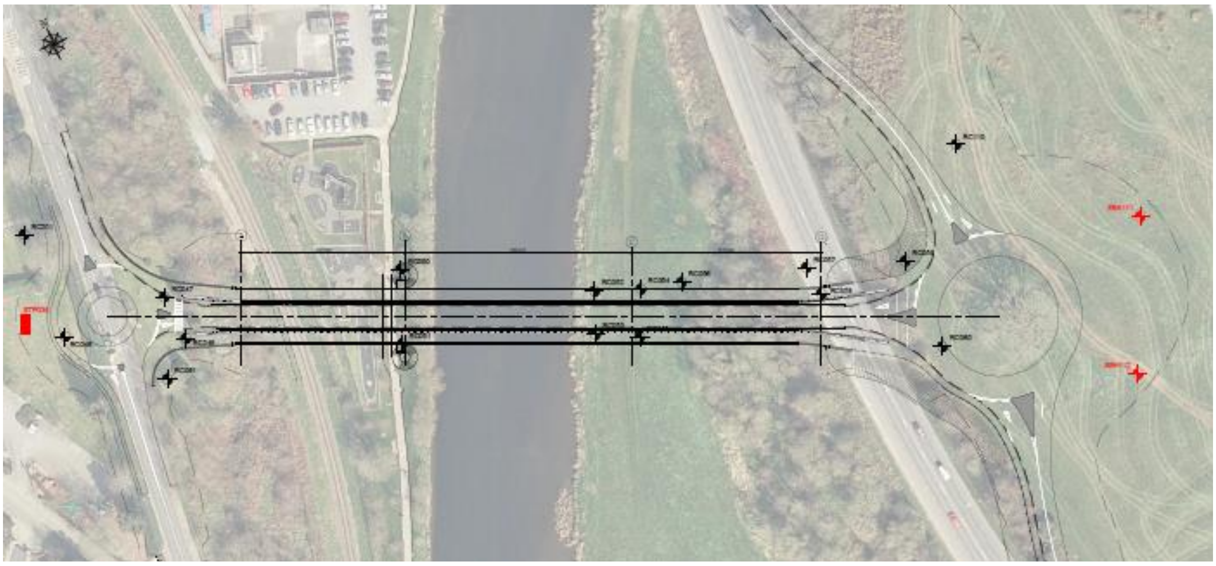
Made ground by its nature is variable with glass, plastic and brick inclusions encountered. In the absence of a construction detail, a presumed bearing capacity of 300kNm^{-2} to 600kNm^{-2} (kPa) is expected (BS8004; 1986) for foundations constructed within the medium dense GRAVEL deposits below 1.5m bgl (L06). Standard penetration test N values suggested an allowable bearing pressure of 90kPa to 140kPa (Terzaghi and Peck, 1967) for settlements up to 25mm.

It is recommended carrying out plate bearing tests to assess the bearing capacity of the made ground along wall L05 and L08 at BH032, BH036, BH038 and BH040.

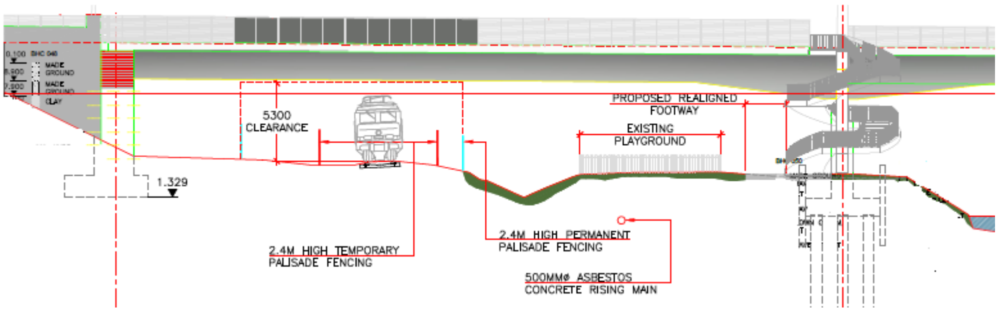
High permeability in the GRAVEL deposits is considered a geo-hazard. In situ permeability should be assessed by means of constant head testing in suitably sized test well at BH033, BH035, BH037, BH041 and BH043. Permeability of the made ground and peaty SILT deposits should also be assessed.

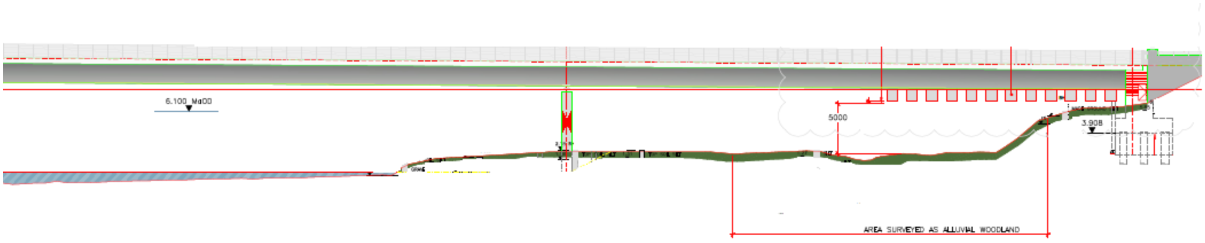
6.1.12 New bridge structure B05; RC201 – BH109, ch4+700m

Construction of a proposed new bridge structures, one (1) road and one (1) pedestrian bridge; approach junctions, roundabouts and ancillary works.



3 Span Bridge crossing (RO'D Drawing ref: Option 7A 008)



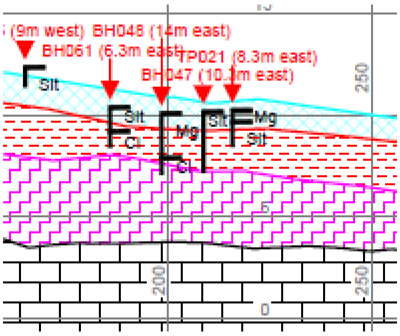


3 Span Bridge crossing (RO'D Drawing ref: Option 7A 008)

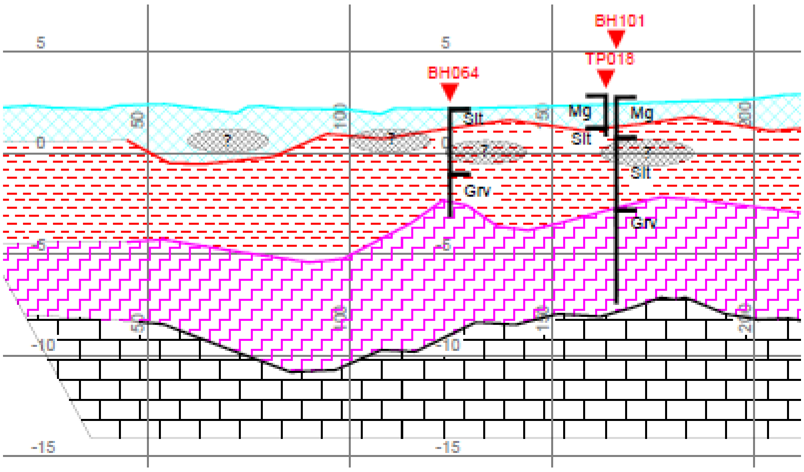
The ground conditions at the site of the propose new bridge crossing were such that;

On the western bank, Made ground described as soft slightly silty slightly sandy SILT 1.2m to 2.8m thick overlay medium dense very silty very sandy GRAVEL to depths between 2.2m bgl to 3.1m bgl. DOLERITE was encountered below this 10.99mOD (RC201) to 11.19mOD (RC047). Assumed infilled fractures, 900mm to 2000mm were noted at RC046. The infill was very soft (Nspt = 3 and 4) 3.7m bgl and 8.2m bgl.

On the western abutment, Made ground was present to a depth 2.2m bgl, described as lightly sandy gravelly SILT. A layer 0.8m thick of stiff slightly sandy slightly gravel SILT overlay the bedrock. Below the SILT, DOLERITE was encountered, 8.4mOD (RC061) to 7.1mOD (RC048).

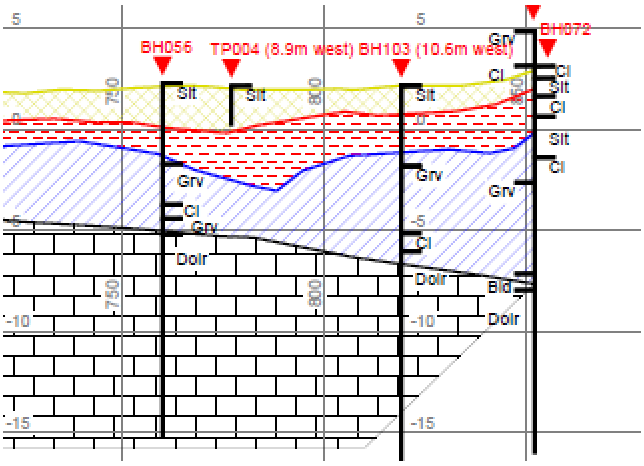


On the western piers and pedestrian access, RC101 indicated made ground to a depth 1.0m bgl (1.7mOD, BH050) to 2.0m bgl (0.75mOD) underlain by soft to firm alluvial deposits slightly gravelly sandy SILT to a depth 4.5m bgl to 5.0m bgl. BH051 identified a thin PEAT deposit 200m thick (-2.15mOD – 2.35mOD). The SILT was undertaken by medium dense GRAVEL 8.7m bgl to 9.8m bgl. Below this depth (-6.00mOD to -7.05mOD) DOLERITE was encountered.

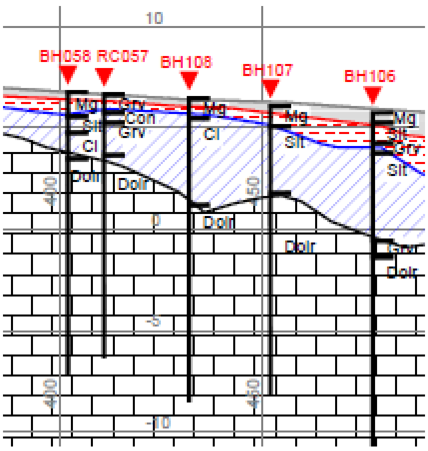


On the eastern bank, Made ground was encountered 0.7m to 2.0m thick underlain by medium dense and dense silty sandy GRAVEL with Cobble content to 3.0m bg (3.65mOD) and firm becoming stiff slightly sandy gravelly SILT to 3.8m bgl (3.05mOD). Below this DOLERITE was encountered 3.0m bgl (RC057, 3.65mOD) to 4.3m bg (RC059, 8.03mOD/ 1.78mOD RC107). RC072 North of the alignment encountered Dolerite 12.8m bgl (-9.96mOD). |The rock profile in the area was varied.

On the eastern piers, very soft to firm slightly sandy slightly gravelly SILT 1.5m to 4.0m thick overlay medium dense to dense silty very sandy GRAVEL to a depth 6.2m bgl to 8.2m bgl. Below this DOLERITE was encountered -4.63mOD (RC052)/ -4.77mOD (RC053) to -5.23mOD (RC056). A thin layer of peaty SILT was encountered 500mm thick (-1.35mOD to -1.85mOD) at BH103.



On the eastern abutment Made ground bituminous surfacing and sub-base was encountered to a depth 0.8m bgl to 1.2m bgl, underlain by GRAVEL to a depth 3.5m bgl. DOLERITE was encountered 3.0m bgl to 3.2m bgl; 3.85mOD (RC057) to 3.46mOD (RC058).

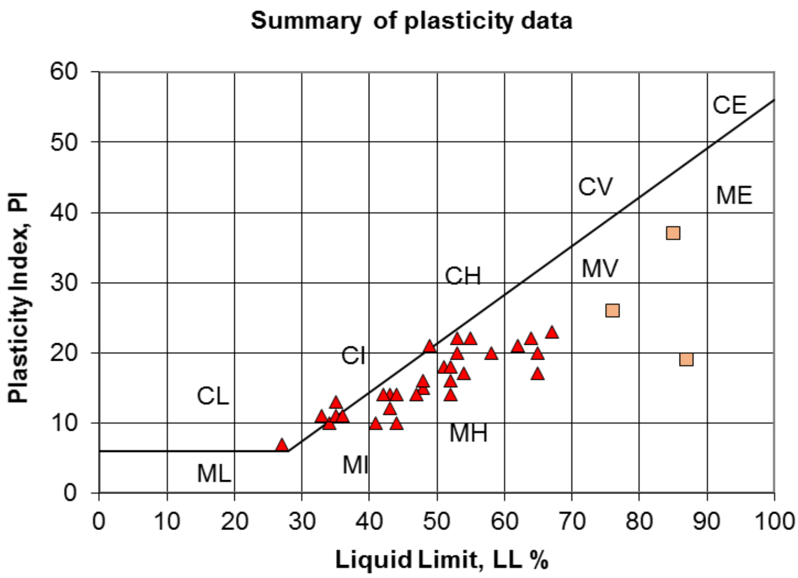


Groundwater (----) was encountered at depths 1.6m bgl to 7.9m bgl (2.58mOD to -3.07mOD).

The Made ground was characterised by Nspt within the range 7 to 15 (Silt) and 27 (Gravel). Nspt values were considered elevated due to Cobble content, brick and other inclusions within the granular Made ground deposits. The granular made ground was considered medium dnese with an expected angle fo friction, ϕ 30°. The silt fraction was of intermediate plasticity (LL 43% and 44%). Natural moisture content was 19% to 16% for the Gravel. Undrained shear strength of the order 31kPa to 67kPa are expected in the cohesive Made ground, describing soft to firm deposits. The Silt was of intermediate to high plasaticity (LL 36% to 60%). Natural moisture content was 19% to 23% and 50%, where measured. It is not recommended to found within the Made ground due to its variable nature.

The slightly sandy slightly gravelly SILT was of intermediate to high plasticity (MI-MH). The SILT was of low to medium organic content (loss on ignition 2.9% to 6.1%). The silt fraction was 14% to 70% with 8% to 56% gravels and 10% to 49% sand fractions. Cobble content was between 0% and 35%, low to high. Natural moisture content, w ranged between 11% and 57%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.4 to 1.7 indicative of stiff (*w*/PL < 1.0) and very soft deposits, *w*/PL >1.2 (C504).

The high and very high plasticities (MV) corresponded to peaty SILT deposits with natural moisture content 76% and 77% and *w*/PL =1.65, very soft deposits.



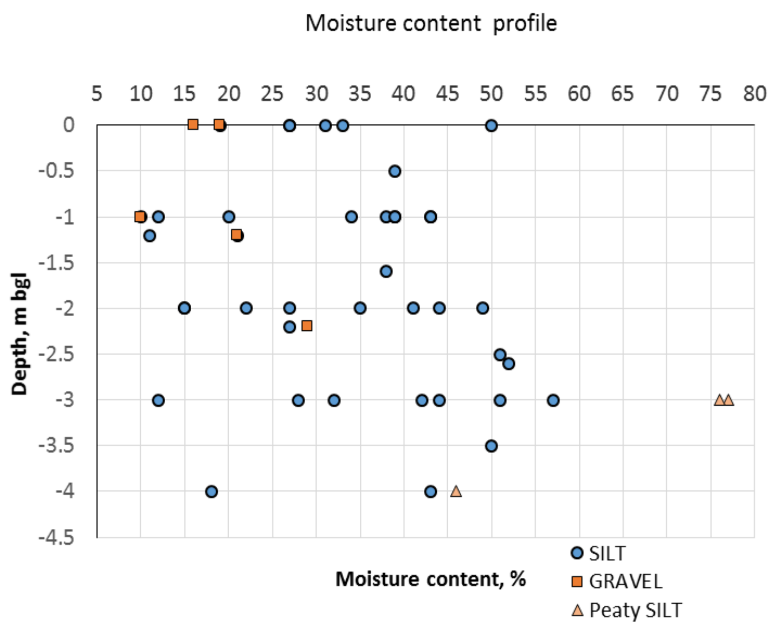
Standard penetration tests Nspt values ranged between 4 to 12 in the alluvial SILT deposits. With a plasticity index, PI 10 – 23; a factor f_1 = 6 and 5 (Stroud, 1975) was such to yield undrained shear strengths of 20kPa to 72kPa, describing soft to stiff alluvial deposits (BS5930 1999). Plasticiy data, PI, suggested an angle of friction of 26^o to 30^o for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

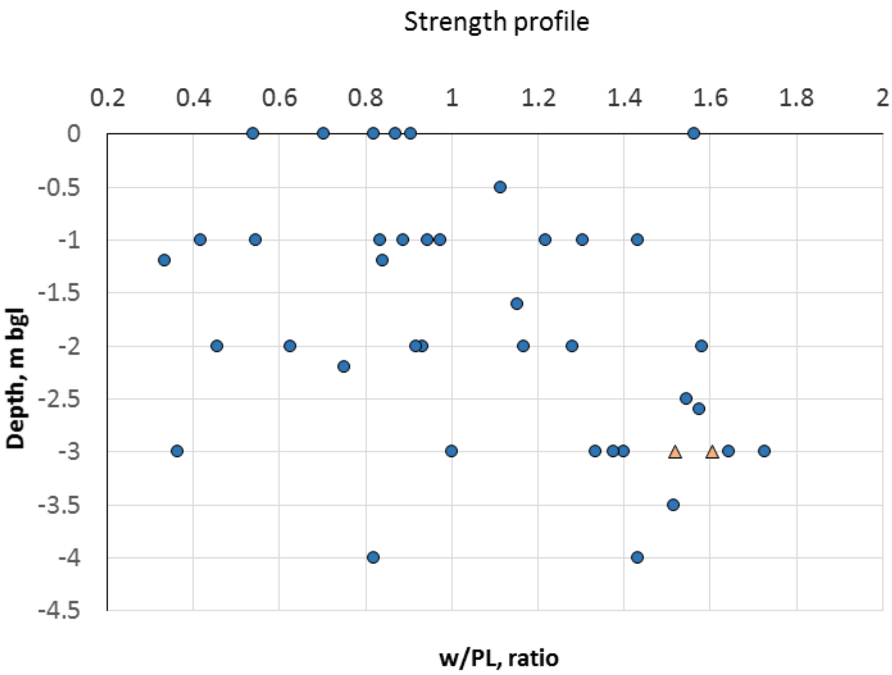
The lower glacial SILT deposits yielded Nspt values 11 to 21. This was indicative of undrained shear strenght 55kPa to 126kPa being firm to stiff (BS5930, 1999).

Compressibility of the alluvial deposits is expected to be variable being moderately compressibility (MI-MV).

Taking a characteristic Nspt value of 4 in the upper SILT, an unfactored stiffness modulus (Young's modulus, E) of 4MPa is expected (PI 10 and 23, Stroud, 1975). Taking a characteristic Nspt value of 11 in the lower SILT, an unfactored stiffness E of 12MPa is expected of the glacial SILT deposits (Stroud, 1975). Taking a characteristic Nspt value of 4 in the peaty SILT, an unfactored stiffness modulus E of 2.5MPa is expected (PI 26 and 37, Stroud, 1975).

Permeability in the SILT was determined based on particle size d_{10} being 0.002mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works).





The GRAVEL was characterised by 1% to 35% silt, 19% to 49% sand and 35% to 85% gravels and variable Cobble content (0% to 35%).

With Nspt values 12 to 63, allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 32^{o} to 38^{o} is expected of the medium dense to dense granular deposits. Taking a characteristic Nspt value of 20 in the medium dense GRAVEL deposits a stiffness modulus of 20MPa to 30MPa is expected of the medium dense slightly silty very sandy GRAVEL (Menzenbach, 1967, Boeles 1988).

Particle size d_{10} of 0.06mm to 1.18mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-7} ms^{-1}. This described low to high permeability (C113). The silt fraction control permeability.


Uncorected Nspt data
0
10
20
30
40
50
60
70
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
BH046
BH046A
BH047
BH048
BH050
BH051
BH052
BH053
BH054
BH055
BH056
BH058
BH060
BH061
BH062
BH101
BH103
BH104
BH105
BH106
BH107
BH108
BH109
BH110
BH202
RC046
RC062
RC101
RC103
RC105
RC109

The Dolerite rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to very strong I_{P50}, 0.33MPa – 9.4MPa; UCS 18MPa to 50MPa), fracture spacing (40mm – 1000mm) and condition (slightly to moderately weathered), Rock Quality Designation (RQD 0% to 100%) and groundwater (assumed 'wet' within the zone of influence) was undertaken. The rock mass rating, RMR range was 27 to 66, describing Class IV to Class - II poor to good Dolerite. A geological strength index, GSI (Hoek and Brown) of 15 to 50 is assumed for the poorly interlocked heavily broken to partially disturbed, blocky rockmass.

	RC202	**RC047**	**RC048**	**RC061**
	37	42	32	27
RMR	44	52	35	50
Class	IV	III	IV &III	IV & III
GSI	15 - 20	35	15 &35	15 &35
Friction ϕ, º	28	30	28	20/ 30
Cohesion, kPa	200	200	150	100/200

RC047 Dolerite


RC048 Dolerite

	RC101	RC050	RC051	RC062
	32	40	52	37
RMR	32	45	52	41
Class	IV	III	III	IV
GSI	15	40	40	30
Friction ϕ, º	25	28	30	26
Cohesion, kPa	200	200	200	200


RC101 Dolerite


RC050 Dolerite


RC062 Dolerite

	RC103	RC104	RC052	RC053
	37	53	53	42
RMR	47	61	53	61
Class	IV-III	III	III	III-II
GSI	15-30	30-40		
Friction ϕ, º	28	35	35	35
Cohesion, kPa	200	300	200	250

RC103 Dolerite

RC104 Dolerite

	RC054	RC055	RC056	RC057	RC106	RC107	RC108
	37	54	47	40	47	37	27
RMR	42	66	52	40	61	52	52
Class	IV-III	III-II	III	IV-III	III	IV-III	IV-III
GSI	15-30	50	40	15-30	40	15-40	15-40
Friction ϕ, º	28	36	30	28	33	30	25/35
Cohesion, kPa	200	300	200	150	250	200	100/250

RC054 Dole rite

RC055 Dolerite

RC056 Dolerite

RC057 Dolerite

RC106 Dolerite

RC107 Dolerite

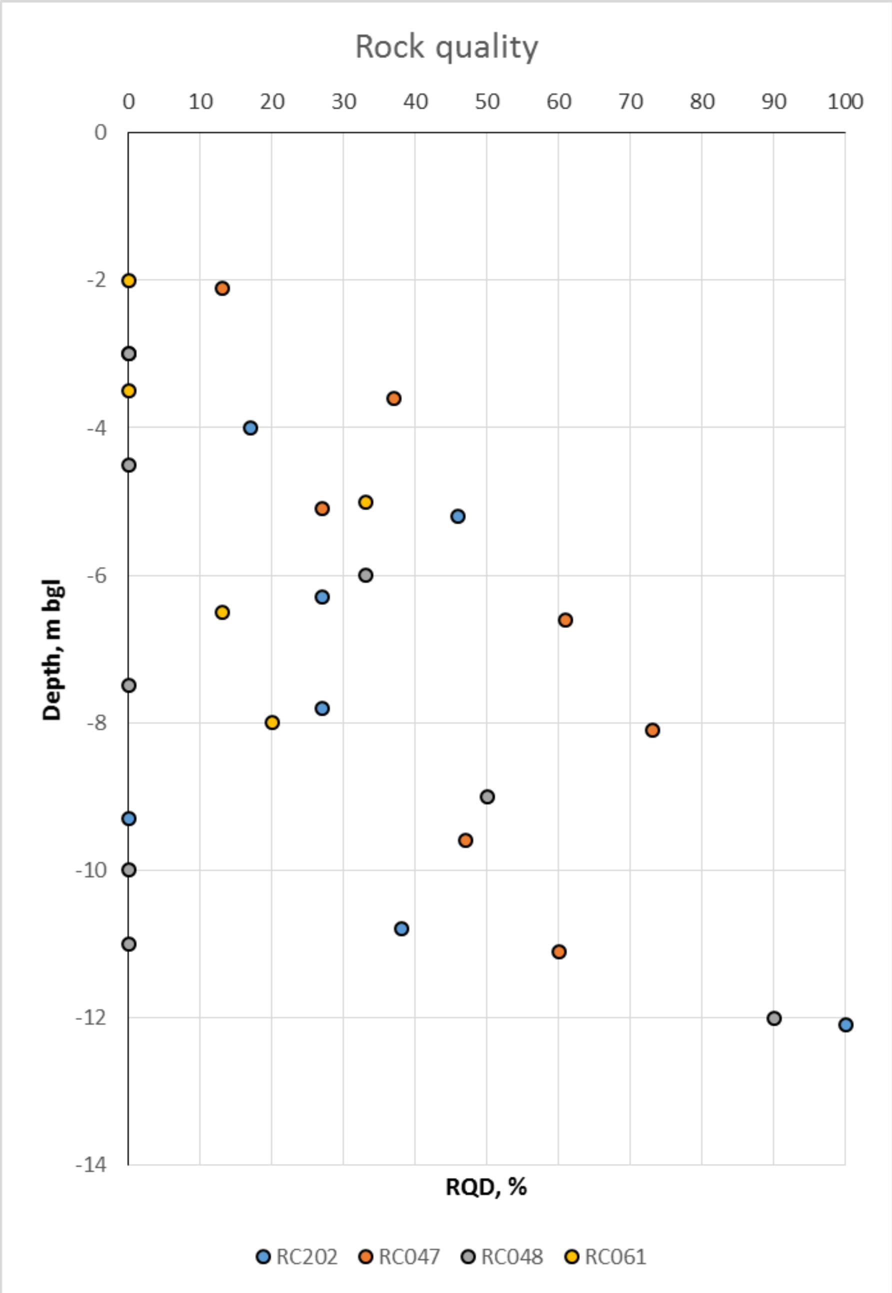
RC108 Dolerite

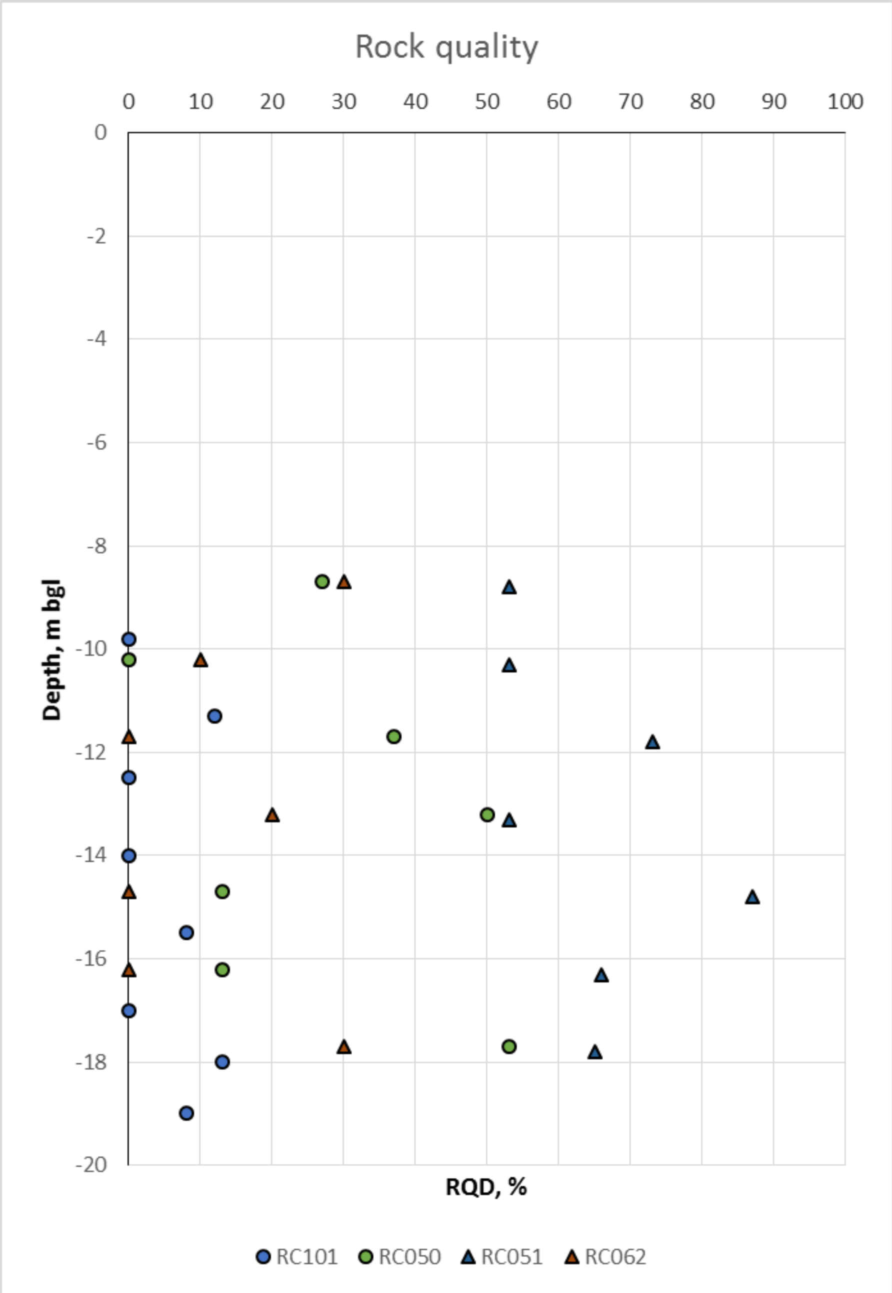
RC108 Dolerite

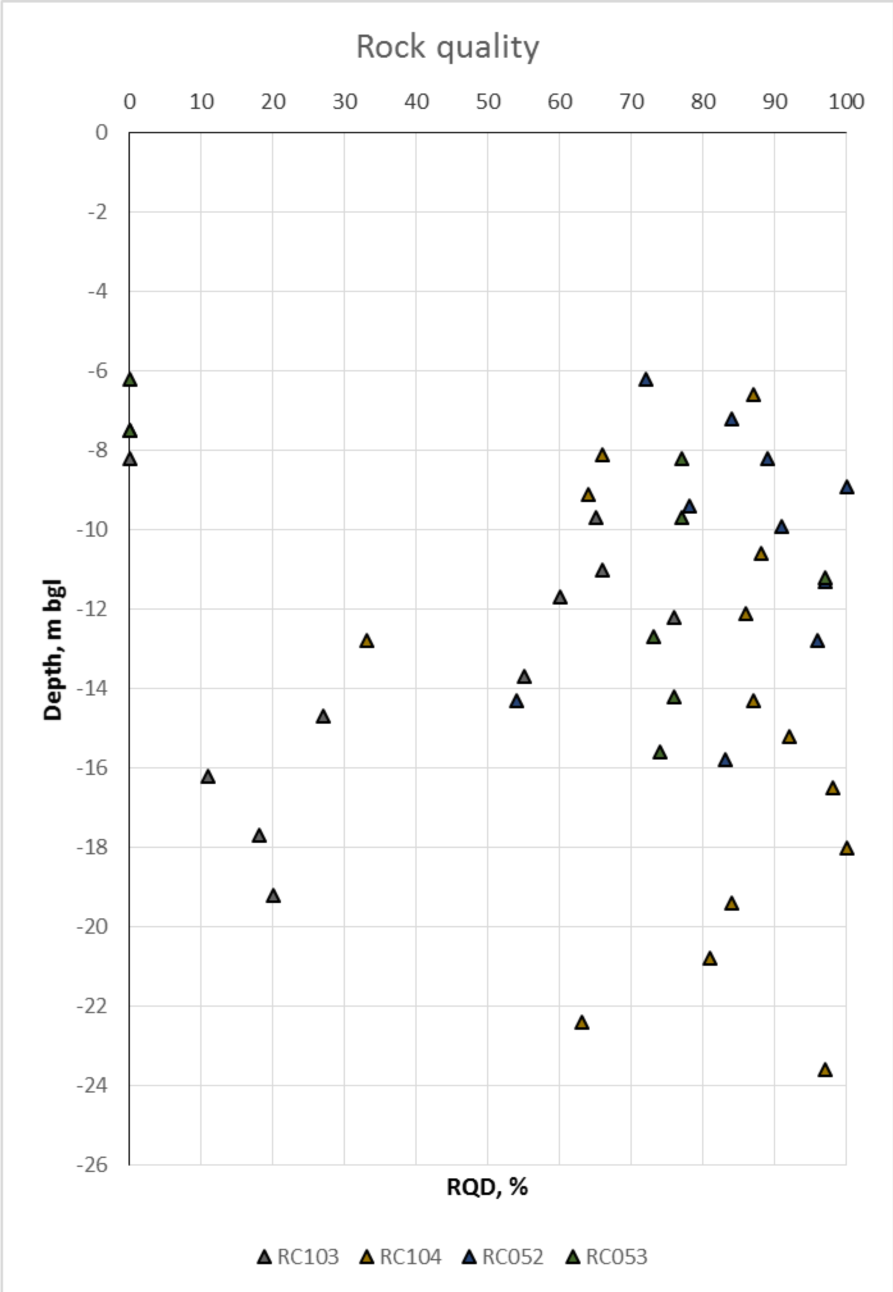
	RC059	**RC109**	**RC110**
	40	37	37
RMR	45	37	40
Class	III	IV-III	IV-III
GSI	20	15	15
Friction ϕ, º	30	28	28
Cohesion, kPa	200	200	200

RC059 Dolerite

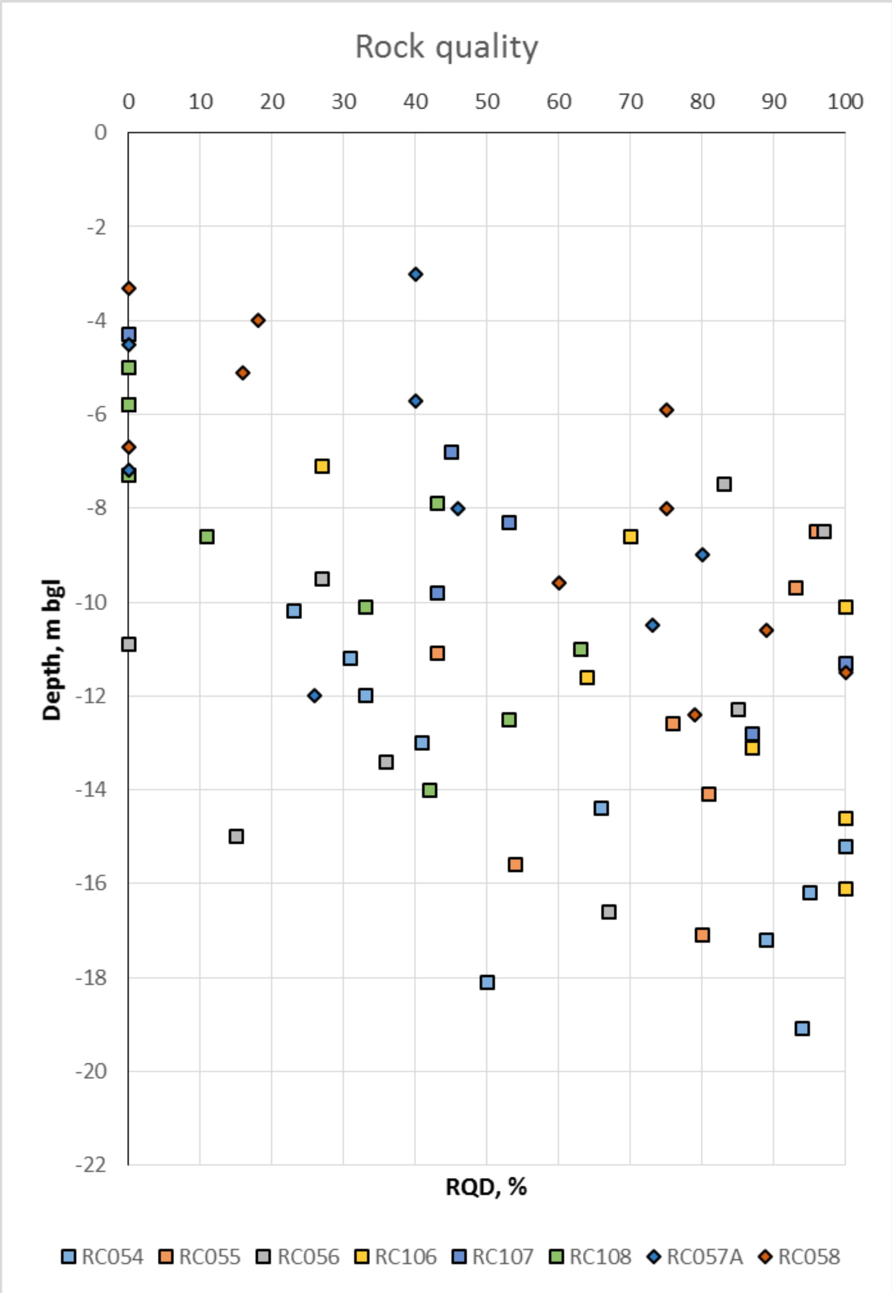
RC109 Dolerite

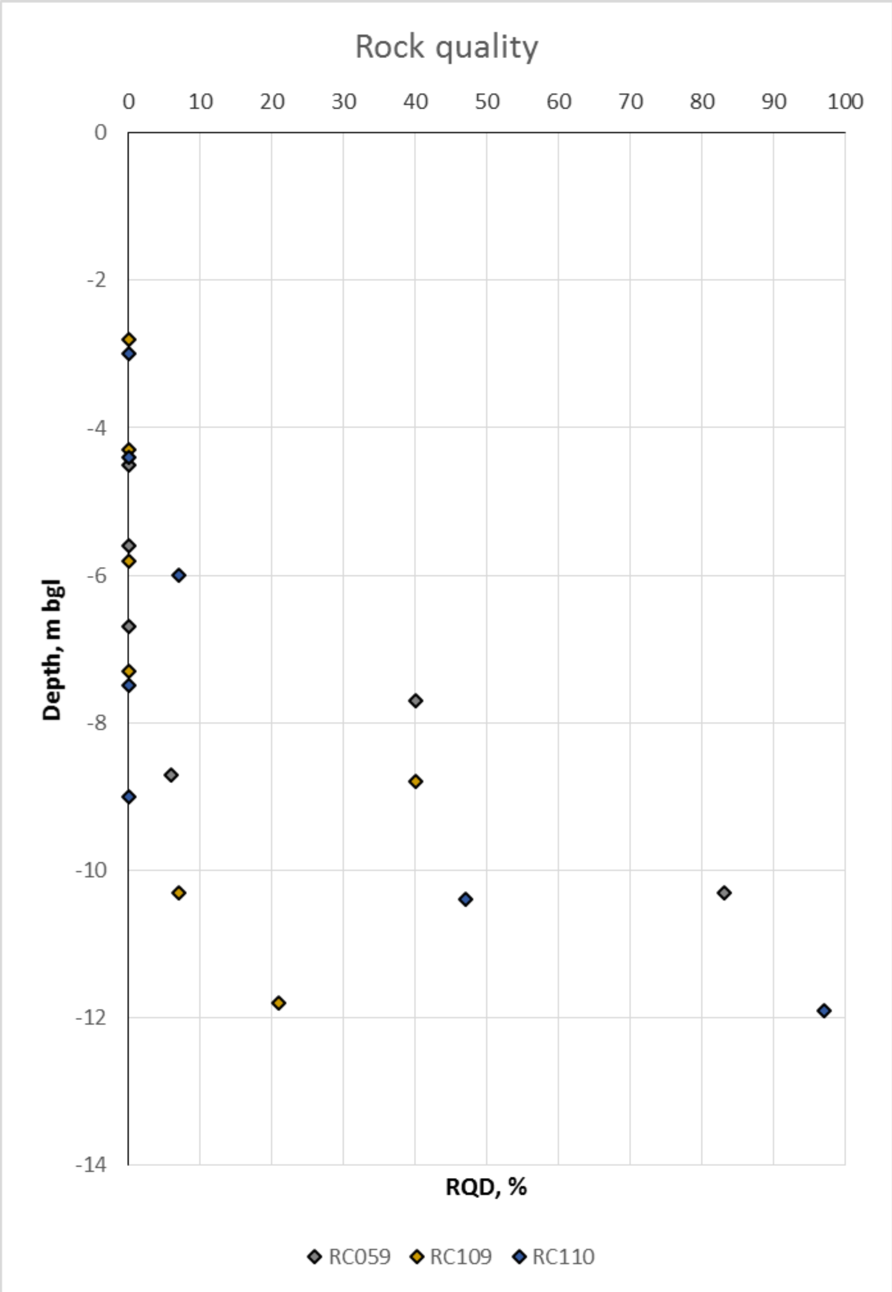

Rock quality
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
Depth, m bgl
RQD, %
RC202
RC047
RC048
RC061


Rock quality
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
Depth, m bgl
RQD, %
RC101
RC050
RC051
RC062



Rock quality
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
Depth, m bgl
RQD, %
RC103
RC104
RC052
RC053


Rock quality
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
Depth, m bgl
RQD, %
RC054
RC055
RC056
RC106
RC107
RC108
RC057A
RC058

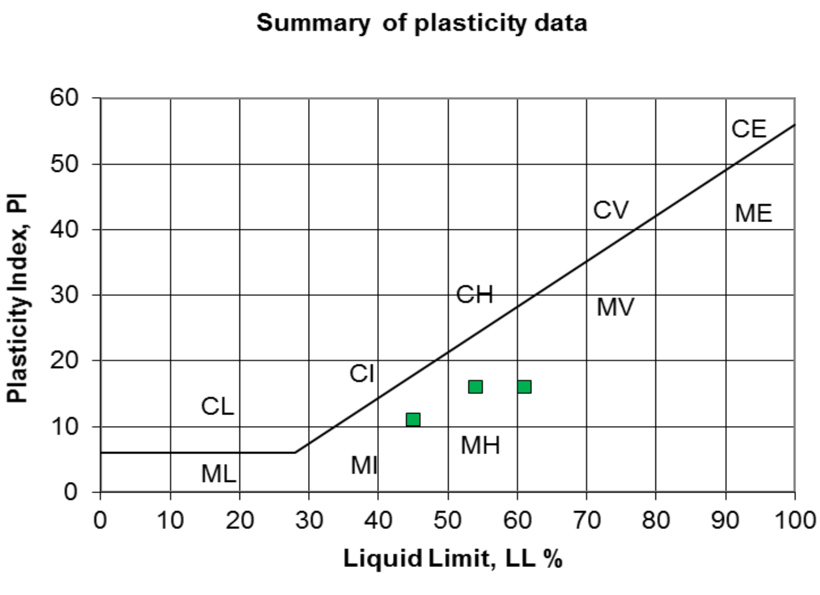

Rock quality
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
Depth, m bgl
RQD, %
RC059
RC109
RC110

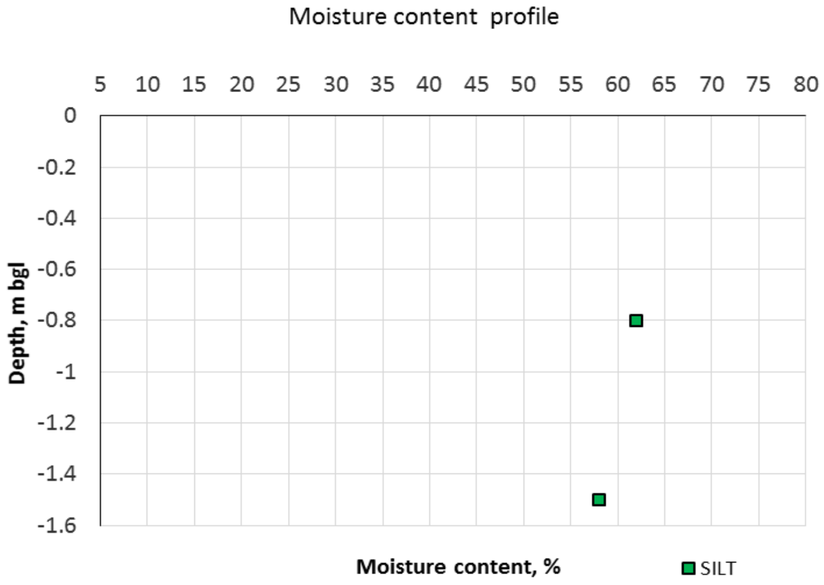
6.1.13 Flow control structures C01; BH001/ BH002 ch0+700m and C02; BH004/ BH005 ch0+150m

The proposed flow control structure comprises; 900mm inlet culvert, a manhole chamber with a 450mm – 600mm diameter pipe to limited flow to /from the existing watercourse.

The upstream flow control, C01 at BH001/ BH002 was characterised by soft to firm sandy SILT of high plasticity, 2.1m (PZ001) to 2.4m thick overlying medium dense sandy very silty GRAVEL to a depth 4.3m bgl to 4.5m bgl (-2.09mOD to -1.4mOD). Below this depth SLATEY-MUDSTONE was encountered. Groundwater was encountered 2.1m bgl to 2.5m bgl (0.06mOD to 0.11mOD). Static groundwater levels at the nearest standpipe wells, BHA02 and BHA03 indicated groundwater 2.3m bgl, 0.7mOD.

The soft to firm sandy SILT was of high plasticity (MH, TPA02 &*TPA04 nearest locations*). Natural moisture content, w 58% and 62% were measured in the general vicintiy (TPA02 & TPA04). The SILT was of medium to high organic content (loss on ignition 29.6% and 59% TPA03). The silt fraction was 31% to 52% with 2% gravels and 56% to 66% sand fractions.





The ratio of natural moisture content to plastic limit (*w*/PL) was 1.3 to 1.6 indicative of very soft deposits (*w*/PL > 1.2, C504).

Standard penetration test N_{SPT} value of 11 was measured, with CPT inferring N_{SPT} = 1 to 6 in the alluvial SILT deposits. With a plasticity index, PI 11 – 16; a factor f_1 = 5.5 (Stroud, 1975) was such to yield undrained shear strengths 5kPa to 60kPa, describing very soft and firm alluvial deposits (BS5930 1999). Derived shear strenght data from the CPT indicated shear strength of 5kPa to 40kPa. Plasticiy data, PI, suggested an angle of friction of 28^o for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

Taking a characteristic N_{SPT}value of 11 in the upper SILT, an unfactored stiffness modulus (Young's modulus, E) of 20MPa to 35MPa is expected (PI 11 and 16, Stroud, 1975).

With high plasticity (MH) the alluvial deposits is expected to be compressible.

Permeability in the SILT was determined based on particle size d_{10} being 0.003mm to, giving a value of the order 10^{-8} ms^{-1}, indicative of very low permeability (C113, Control of groundwater for temporary works).

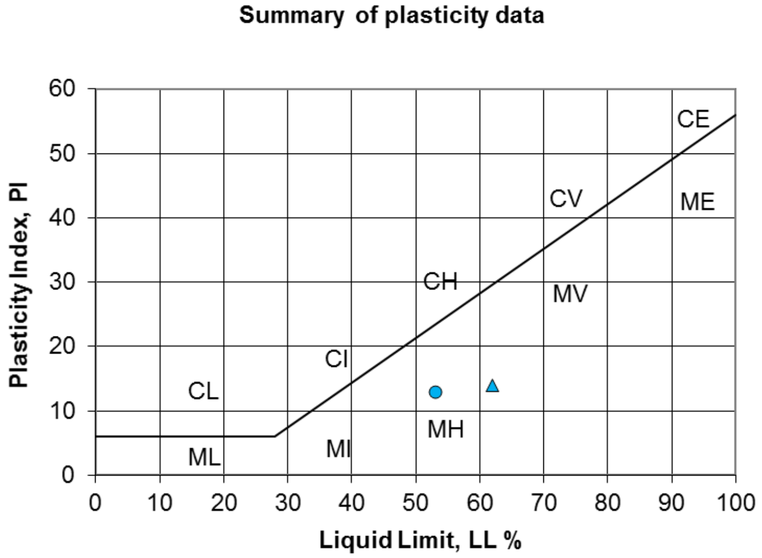
For the GRAVEL grading analysis indicated the gravel fraction was 57% with % silt and 30% sand fractions; 11% Cobble described medium cobble content.

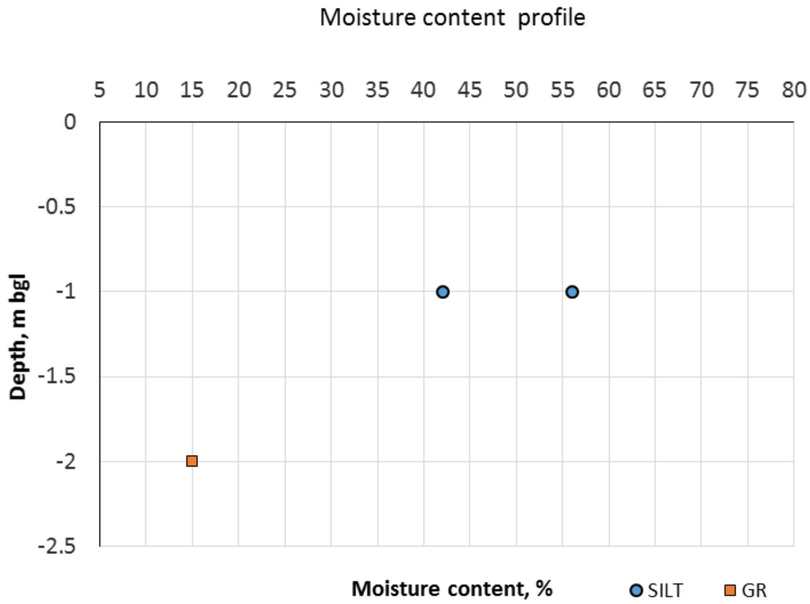
With a single N_{SPT} value of 31 and with CPT inferring N_{SPT} = >30 allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 36^{o} is expected of the medium dense to dense granular deposits. Taking a characteristic N_{SPT} value of 31 in the medium dense to dense GRAVEL deposits a stiffness modulus of 30MPa is expected of the GRAVEL (Menzenbach, 1967, Boeles 1988).

Permeability in the GRAVEL was determined based on particle size d_{10} being 0.6mm to, giving a value of the order 10^{-3} ms^{-1}, indicative of medium permeability (C113, Control of groundwater for temporary works).

The downstream channel flow control, C02 at BH004 and BH005, TP035 was characterised by soft slightly gravelly sandy SILT, 1.0m to 2.1m thick overlying medium dense (slightly) silty very sandy GRAVEL to a depths between 5.2m bgl to 5.8m bgl (-4.01mOD). SLATEY-MUDSTONE was encountered below 5.2m bgl (-3.38mOD). Groundwater was encountered 1.7m bgl to 3.0m bgl (0.85mOD to -1.18mOD).

The soft to firm sandy SILT was of high plasticity (MH). Natural moisture content, w 42% and 56% were measured. The silt fraction was 60% to 75% with 1% to 3% gravels and 24% to 37% sand fractions. Based on a single value the SILT was of low organic content (loss on ignition 5.1%).





The ratio of natural moisture content to plastic limit (*w*/PL) was 0.8 to 1.4 indicative of firm and very soft deposits (*w*/PL <1.0; *w*/PL > 1.2, C504).

Standard penetration tests N_{SPT} value of 10, with CPT inferring N_{SPT} = 5 to 12 in the alluvial SILT deposits. With a plasticity index, PI 13 – 14; a factor f_1 = 5.5 (Stroud, 1975) was such to yield undrained shear strengths 28kPa to 66kPa, describing soft to firm alluvial deposits (BS5930 1999). Derived shear strenght data from the CPT indicated shear strength of 10kPa to 30kPa.Plasticiy data, PI, suggested an angle of friction of 28^o for the sandy SILT (C504; Terzaghi, Peck & Mesri, 1996).

Taking a characteristic N_{SPT} value of 10 in the upper SILT, an unfactored stiffness modulus (Young's modulus, E) of 14MPa to 33MPa is expected (PI 13 and 14, Stroud, 1975).

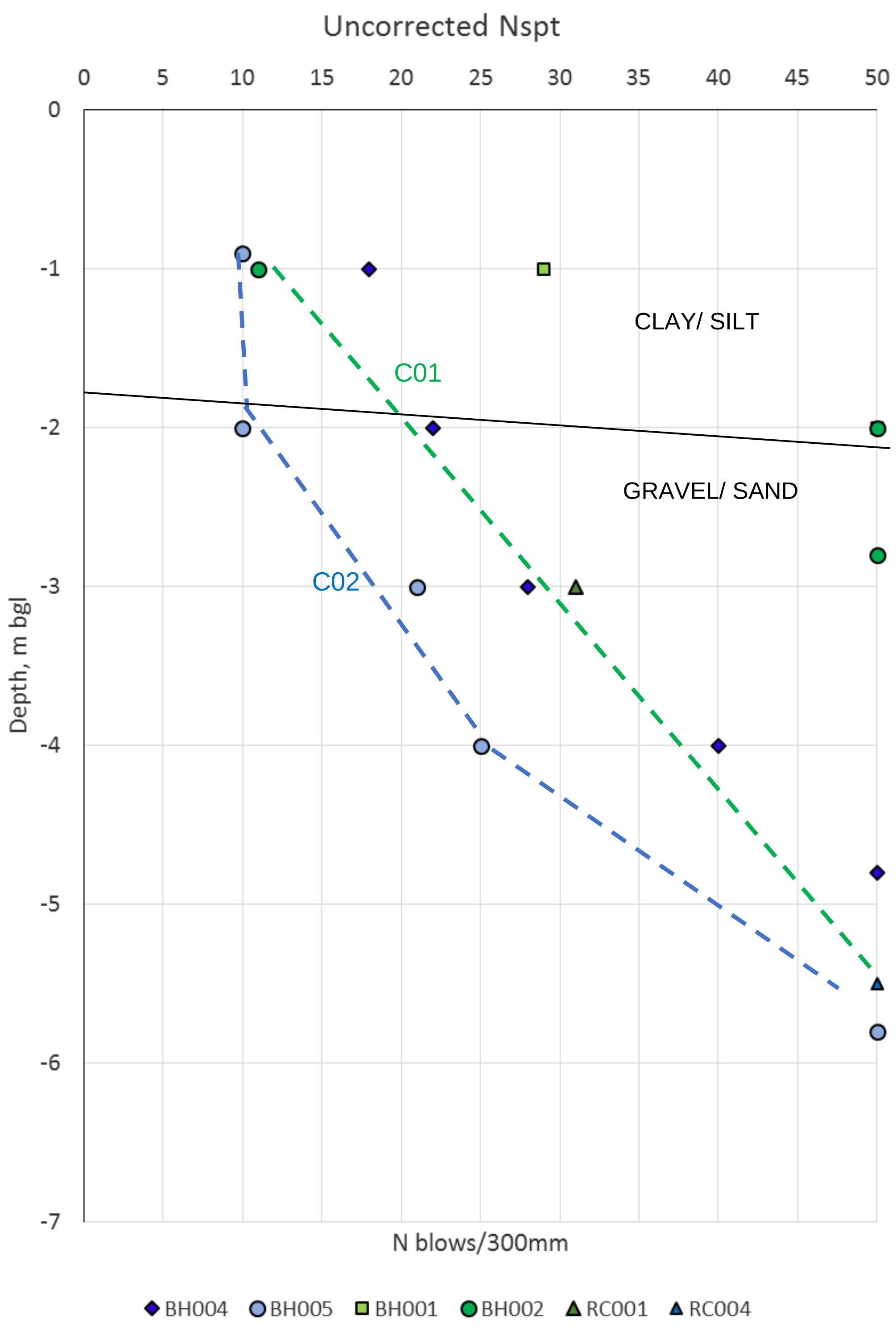
With high plasticity (MH) the alluvial deposits is expected to be compressible.

Permeability in the SILT was determined based on particle size d_{10} being >0.06>3mm to, giving a value of the order >10^{-5} ms^{-1}.

The GRAVEL was characterised by a single moisture content 15%. The gravel fraction was 40% to 58% with 1% to 7% silt and 10% to 33% sand fractions; 8% to 31% Cobble described medium to high cobble content.

With N_{SPT} values 10 to 28, with CPT inferring N_{SPT} = 10 to >25 allowing for the silt and gravel fractions and the particle shape, an angle of friction, ϕ of 30^o to 35^o is expected of the medium dense granular deposits. Taking a characteristic N_{SPT} value of 20 in the medium dense GRAVEL deposits a stiffness modulus of 23MPa to 40MPa is expected of the medium dense GRAVEL (Menzenbach, 1967, Boeles 1988).

Permeability in the GRAVEL was determined based on particle size d_{10} being 2.0 to 0.425mm to, giving a value of the order 10^{-2} ms^{-1} to 10^{-3} ms^{-1}, indicative of medium permeability (C113, Control of groundwater for temporary works).



Uncorrected Nspt
0
5
10
15
20
25
30
35
40
45
50
0
-1
-2
-3
-4
-5
-6
-7
Depth, m bgl
N blows/300mm
CLAY/ SILT
GRAVEL/ SAND
C01
C02
BH004
BH005
BH001
BH002
RC001
RC004

At the upstream location, foundations shall be constructed within the medium dense GRAVEL below 0.3mOD. A presumed bearing capacity of 100kPa is expected for a characteristic Nspt = 11 (BS8004 Code of practice for foundations; 1986).

A standard chamber base of 1.85m is assumed for the proposed 600mm pipe. For the purpose of assessing bearing pressure a foundation width of 2.0m has been used; b_{2000}/b_{600} = 3.3. The inlet culvert is assumed up to 3.0m wide; b_{3000}/b_{600} = 5.0.

Test PLTA01 using the 600mm diameter plate did not induce failure ($\Delta h = 0.1$ R, where R is the plate radius) at loading up to 244kPa within the SILT deposits at a depth 1.0m bgl. The deposits at test location PLTA01 were deemed to have adequate resistance (FoS >3) to localized shear failure with an predicted ultimate bearing pressure of the order 300kPa.

The expected zone of influence of the foundations is not expected to exceed 2.0m to 3.0m (Burland, 1985). For the purpose of the settlement analysis, the SILT may be considered homogenous within this depth. In accordance with Figure K.3 (EN 1997-2:2001), this yielded s_{2000} /s_{600} (s/s_1) settlement ratio 3.5. Predicted settlements of the order 18mm are expected under the imposed design loading, 100kPa.

Summary of PLT data

Location	Δh_{600}, mm at design loading, 100kPa	Subgrade reaction, K_s, $MN/m^2/m$	Estimated CBR, %
PLTA01	5.0	34	3

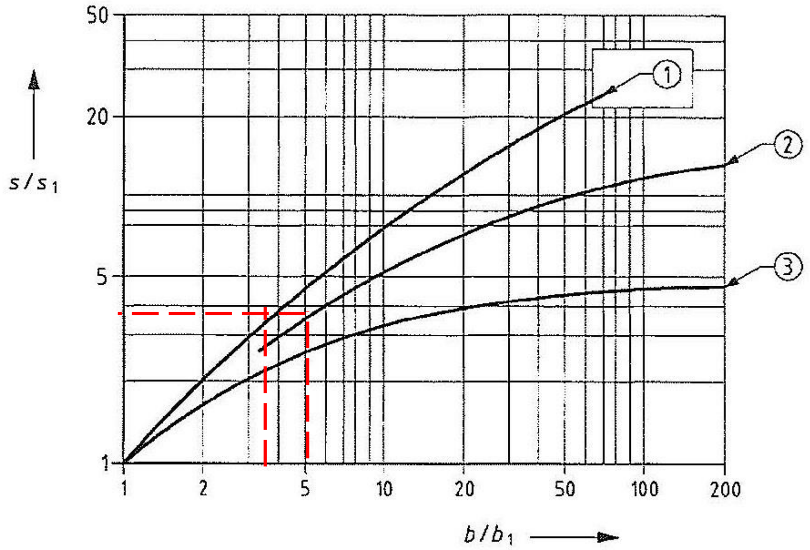
Summary of predicted basic settlements

Location	Δh_{600}, mm at design loading (100kPa)	Proposed dimensions, B, m	Predicted settlement, Δh, mm
PLTA01	5.0	2.0	*17.5*

The foundation shape has been considered, assessing the L/B ratio, (L up to 6m for the intake and spillway) yielding a shape adjustment factor of 1.3.

Summary of adjusted, predicted total settlements

Location	$\Delta h_{2000/3000}$, mm at design loading (100kPa)	Adjusted settlement, Δh, mm
PLTA01	17.5	23.0



Key

b/b_1 is width ratio
s/s_1 is settlement ratio
1 loose
2 medium dense
3 dense

Figure K.3 — Graph for calculations of settlement based on plate loading tests

Plate loading test PLTA01 indicated an allowable bearing capacity of 100kPa in the firm SILT for settlement up to 25mm. A review of the grading analysis of the sandy SIT deposit indicated high sand fractions (55^ to 68%) and so settlement is expected to be immediate. In the medium dense GRAVEL settlement of 0.1mm/kPa is expected for a 2m to 3m foundation (Burland, Broms & DeMello, 1977) 10mm settlement is expected in the GRAVELS. An allowable bearing pressure of 100kPa is recommended below a depth 1.0m bgl subject to a review of the intake geometry and elevations. Careful consideration shall be given to scour control.

At the downstream location, foundations shall be constructed within the medium dense GRAVEL below -0.21mOD. A presumed bearing capacity of 100kPa is expected for a characteristic Nspt = 10 (BS8004; 1986).

Test PLTA02 using the 600mm diameter plate did not induce failure (Δh = 0.1 R, where R is the plate radius) at loading up to 139kPa within the SILT deposits at a depth 0.5m bgl. The deposits at test location PLTA02 were deemed to have adequate resistance (FoS >3) to localized shear failure with an predicted ultimate bearing pressure of the order 200kPa.

The expected zone of influence of the foundations is not expected to exceed 2.0m to 3.0m (Burland, 1985). For the purpose of the settlement analysis, the SILT may be considered homogenous within this depth. In accordance with Figure K.3 (EN 1997-2:2001), this yielded s_{2000} /s_{600} (s/s_1) settlement ratio 3.5. Predicted settlements of the order 18mm are expected under the imposed design loading, 50kPa.

Summary of PLT data

Location	Δh_{600}, mm at design loading, 50kPa	Subgrade reaction, K_S, $MN/m^2/m$	Estimated CBR, %
PLTA02	4.0	19	2

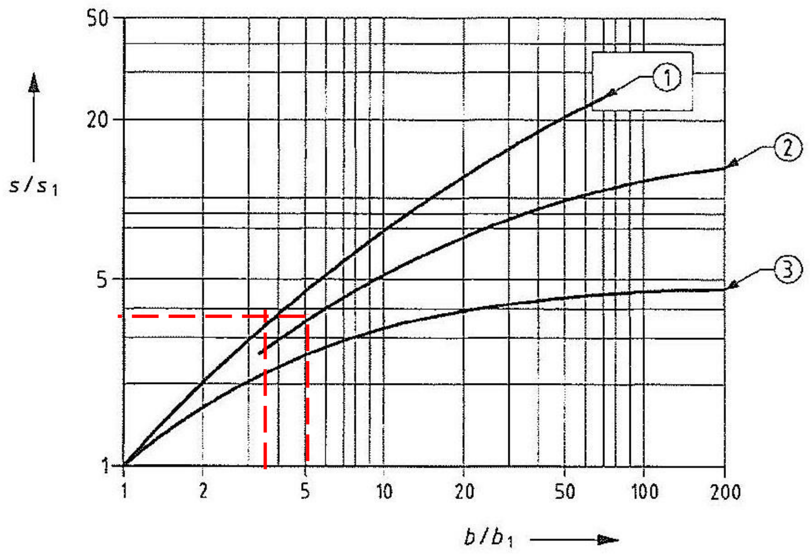
Summary of predicted basic settlements

Location	Δh_{600}, mm at design loading (50kPa)	Proposed dimensions, B, m	Predicted settlement, Δh, mm
PLTA02	5.0	2.0	*17.5*

The foundation shape has been considered, assessing the L/B ratio, (L up to 6m for the intake and spillway) yielding a shape adjustment factor of 1.3.

Summary of adjusted, predicted total settlements

Location	$\Delta h_{2000/\ 3000}$, mm at design loading (50kPa)	Adjusted settlement, Δh, mm
PLTA01	17.5	23.0



Key

b/b_1	is width ratio
s/s_1	is settlement ratio
1	loose
2	medium dense
3	dense

Figure K.3 — Graph for calculations of settlement based on plate loading tests

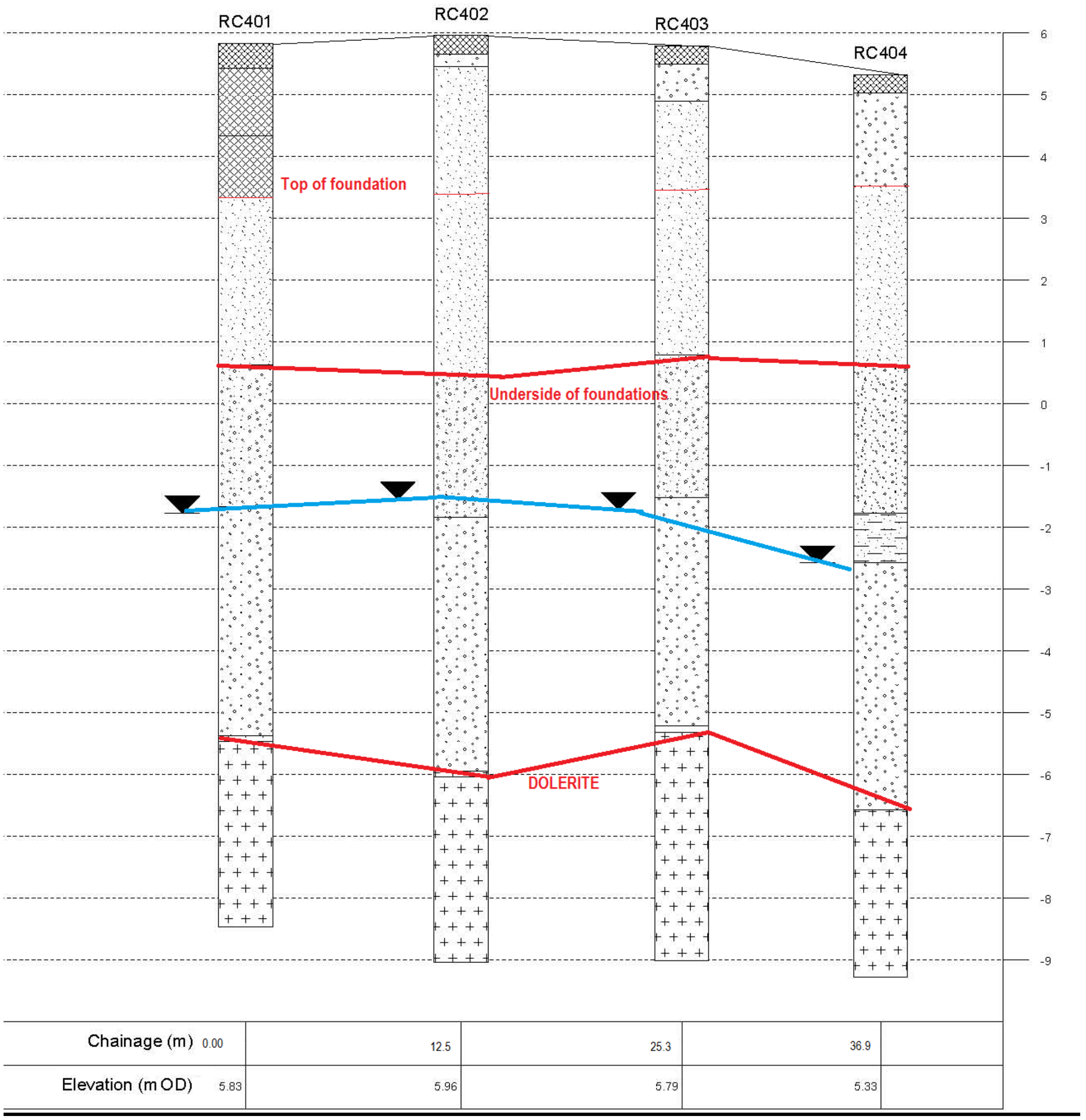
Plate loading test PLTA02 indicated an allowable bearing capacity of 50kPa in the firm SILT for settlement up to 25mm. In the medium dense GRAVEL settlement of 0.1mm/kPa is expected for a 2m foundation (Burland, Broms & DeMello, 1977) 10mm settlement is expected in the GRAVELS. An allowable bearing pressure of 50kPa is recommended below a depth 1.0m bgl subject to a review of the intake geometry and elevations.

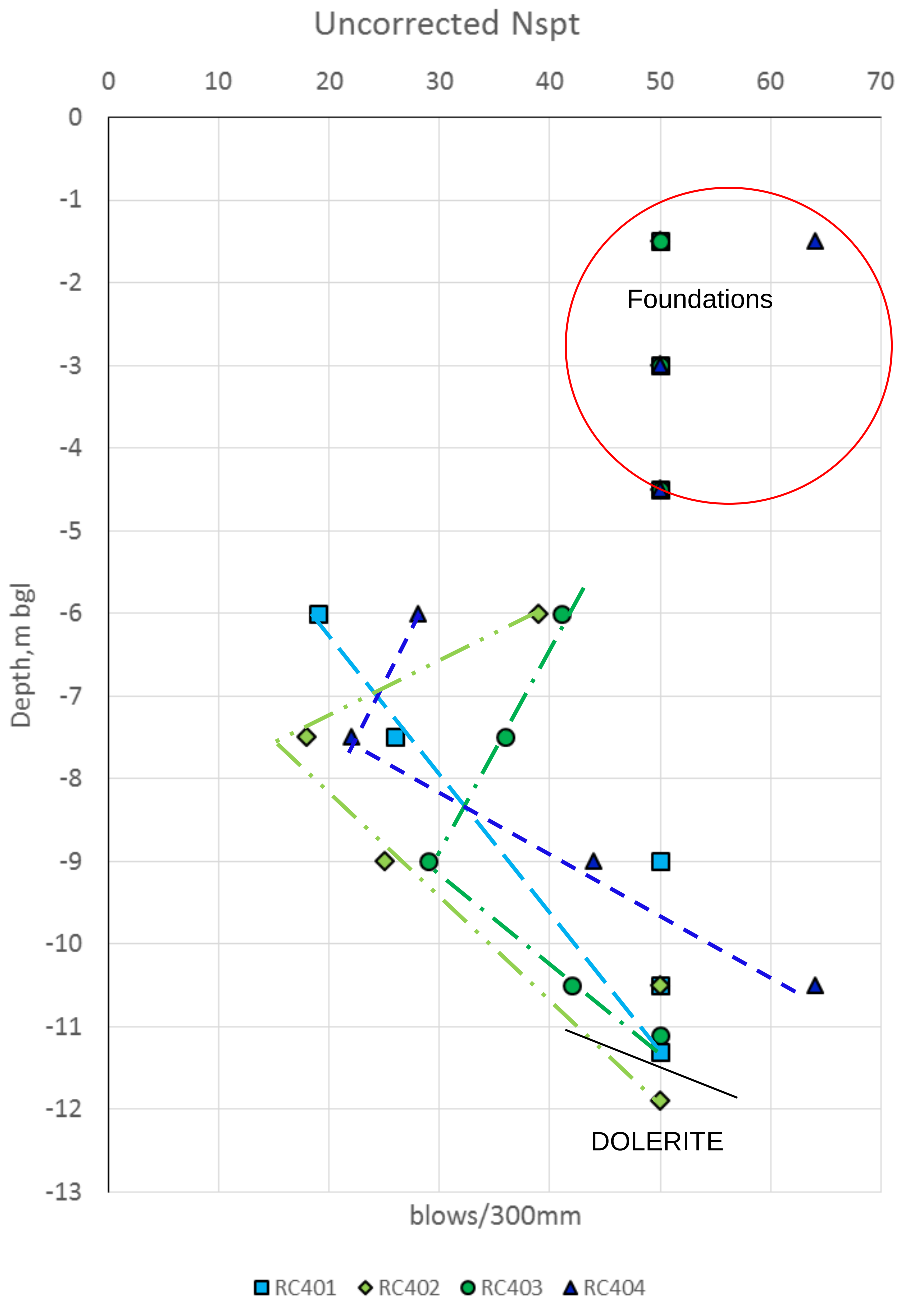
Embankment loading around the manhole chamber and deposition area, DA1, is expected to be up to 2.0m high (40kPa). Settlement of the order 20mm can be expected. It is recommended that the embankment be constructed allowing for settlement to occur before constructing the intake, pipework and spillway elements of the flow control structure where the static cone data indicated a compressible, very soft fine grained deposit 1.0m bgl to 2.0m bgl. With end tip resistance, q_c 0.01MPa measured, an elastic modulus of 0.005MPa is expected. This did not correlate with the plate load test data. A grid of dynamic probing is recommended beneath the plan area of the earthworks structure to fully assess the presence of soft deposits below 1.0m bgl.

6.1.14 Enniscorthy Bridge, B02; RC401 to RC404 ch5+535m

It is proposed to underpin the existing bridge structure.

The ground conditions at the existing bridge structure were such that Made ground was present to a depth 2.5m bgl behind the abutment. The foundations were encountered 2.5m bgl (3.33mOD) being 2.7m thick (0.46mOD to 0.79mOD, underside of foundations). The pad foundation sat on medium dense sandy GRAVEL deposits to a depth 11.0m bgl to 11.9m bgl. Below this DOLERITE was encountered -5.21mOD (RC403) to -6.57mOD (RC404). There was a consistency across the section.





A characteristic Nspt of 20 is recmmended for the medium dnese silty sandy GRAVEL deposits. An angle of friction, ϕ 33^{o} is provided. The relative density varied with elevated values attributed to coarse Cobble particles below depths 9.0m bgl. A characteristic Nspt of 35 may be used providing an angle of friction ϕ 38^{o}.

No assessment of permeability of the granular deposits was undertaken and so grouting as a means of underpinning carried a risk.

No core was recovered in the structural elements of the bride piers and foundations and so the strenght of the concrete has not been assessed. It is recommended that coring of these elements be undertaken to assess the concrete else some form of indirect assessment rebound hardness or similar be employed. Where is is proposed to form a structural connection between the foundations and underpinning system.

Piles designed for both friction/ end bearing within the dense GRAVEL may be considered subject to the imposed structural loading on the bridge abutments and Piers.

It is recommended that rotary bored mini piles be considered socketing into the bedrock. The Dolerite rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (weak to very strong I_{P50}, 0.23MPa – 7.15MPa; UCS 35MPa to 60MPa), fracture spacing (90mm – 530mm) and condition (slightly to moderately weathered), Rock Quality Designation (RQD 0% to 100%) and groundwater (assumed ‘wet’ to ‘damp’ within the zone of influence) was undertaken. The rock mass rating, RMR range was 37 to 59, describing a variable rock mass; Class IV and Class - III poor to fair Dolerite. The variation was primarily associated with non-intact weathered zones in the upper sections of the rock mass. A geological strength index, GSI (Hoek and Brown) of 25 to 55 is assumed for the non-intact to partially disturbed blocky rockmass.

	RC401	**RC402**	**RC403**	**RC404**
RMR	49	42	37	40
	59	51	47	49
Class	III	III	IV-III	III
GSI	55	25/50	25	25/50
Friction ϕ , º	32	30	26	30
Cohesion, kPa	250	200	150	200

RC402 was uncharacteristically weak at 12.7m bgl (I_{P50} 0.23MPa) noting the core was weathered/ altered to a depth 13.5m bgl.

RC402 Dolerite

RC404 was similarly weathered in the upper 1.5m to a depth 13.0m bgl. Piles shall socked below a depth 14.0m bgl or -8.0mOD at RC402, RC403 and RC404.

RC404 Dolerite

A shorter nominal socket shall be achieved at RC401 below -5.5mOD (11.3m bgl).

RC401 Dolerite

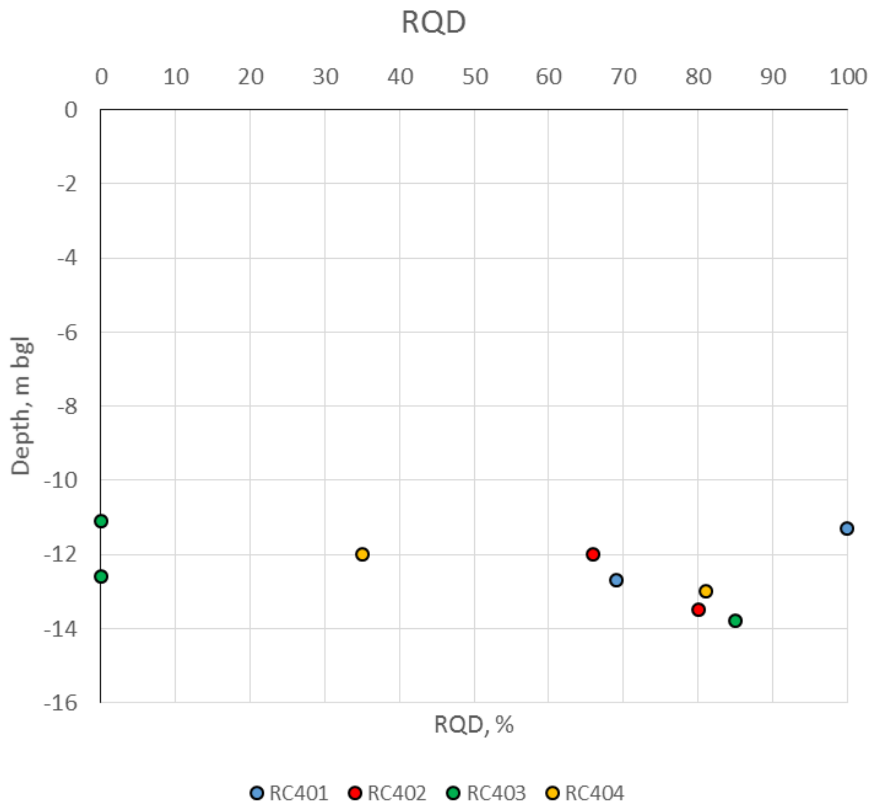
A design UCS strength, σ_c of 35MPa and 60MPa is proposed for the DOLERITE. A partial factor of safety of 1.4 is applied. Based on empirical correlation an adhesion factor (coefficient of side resistance), α of 0.12 to 0.23 is proposed (Ciria report, 'Piled foundations in weak rock', R181, 1999/ Williams *et al* 1980 - for values of χ = 2.0 to 3.0- Table 4.1[5], R181).

Various empirical equations, α		
σ_c =25MPa	**σ_c =43MPa**	**Reference**
0.05	0.03	IRC 78
0.09	0.07	Rowe & Armitage, 1987
0.05	0.04	Horvath *et al.* 1980
0.08	0.06	Rosenberg & Journeaux, 1976
0.08	0.06	Zang & Einstein, 1998 smooth socket
0.16	0.12	Zang & Einstein, 1998 rough socket
0.13	0.10	Horvath & Kenney, 1979
0.13	0.10	Carter & Kulhawy, 1988

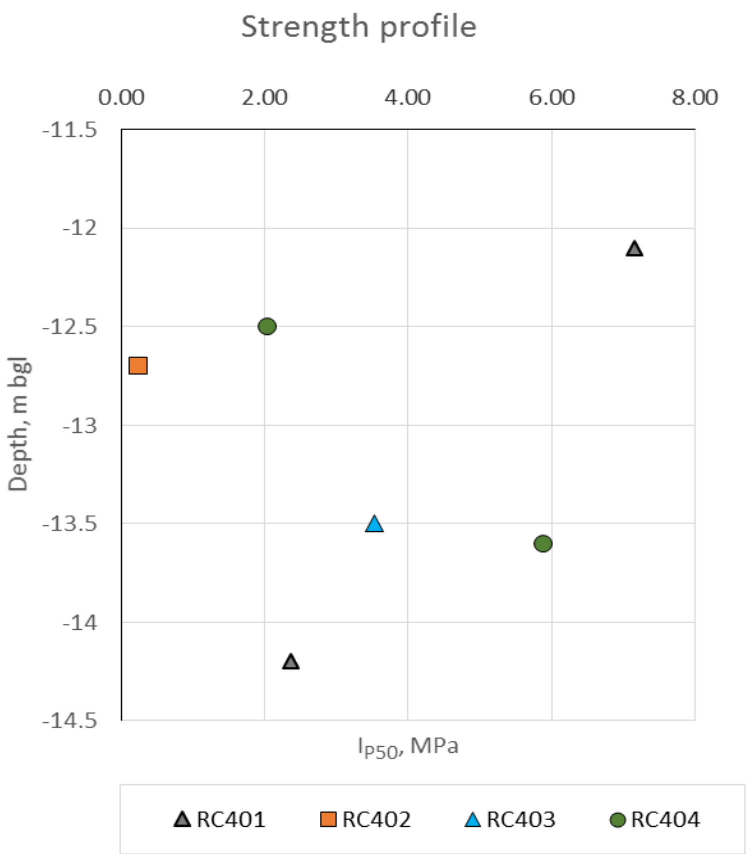
The above table is based on the empirical equation $\alpha = \chi(\sigma_c)^{-0.5}$

An adhesion α of 0.1 is proposed for the DOLERITE.

[5] χ Smooth to rough drilled sockets

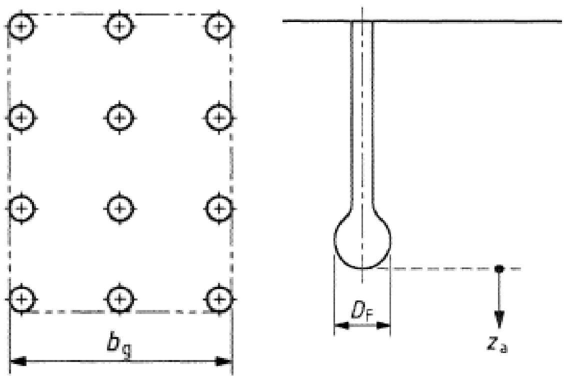


RQD
0
10
20
30
40
50
60
70
80
90
100
0
-2
-4
-6
-8
-10
-12
-14
-16
Depth, m bgl
RQD, %
RC401
RC402
RC403
RC404



Strength profile
0.00
2.00
4.00
6.00
8.00
-11.5
-12
-12.5
-13
-13.5
-14
-14.5
Depth, m bgl
IP50, MPa
RC401
RC402
RC403
RC404

The deatil on the rock mass was limited and it is assumed the rock quality is consistent RQD>60% below a dpeth 12.0m bgl. It should be considered undertaking additional rotary coring to a depth 20m, to assess the rock mass to a depth at least z_a = 5m or 1.0b_g below the underside of the underpinning pile element allowing for a minimum socket 1.5m in the fractured upper zone as per EC7, Part2, Annex B.3.



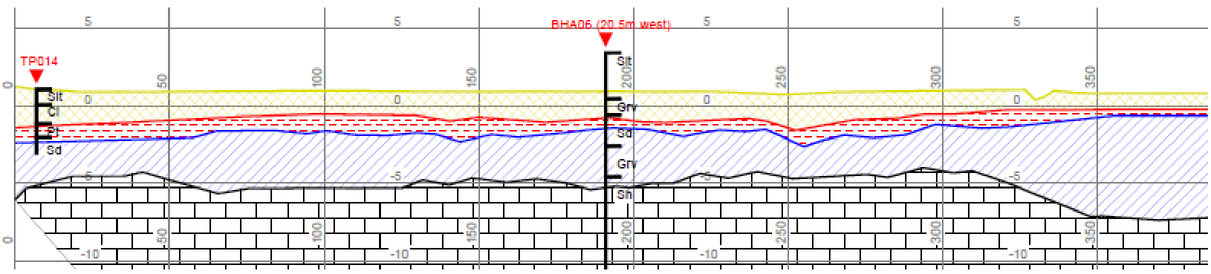
6.1.15 Channel realignment ch4+800 to ch3+600/ Deposition area DA3, ch4+400m to ch4+000m, realignment EX8 and embankment works

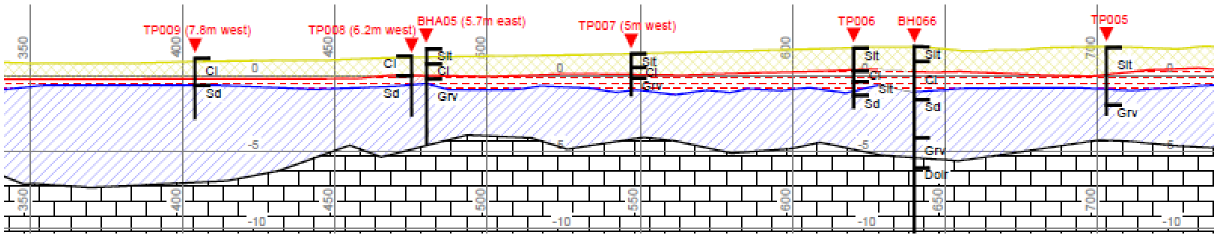
(TP005 – TP014; BHA05, BHA06, BH65 – BH67, BH71, BH72 and BH103, BH106 – BH108)

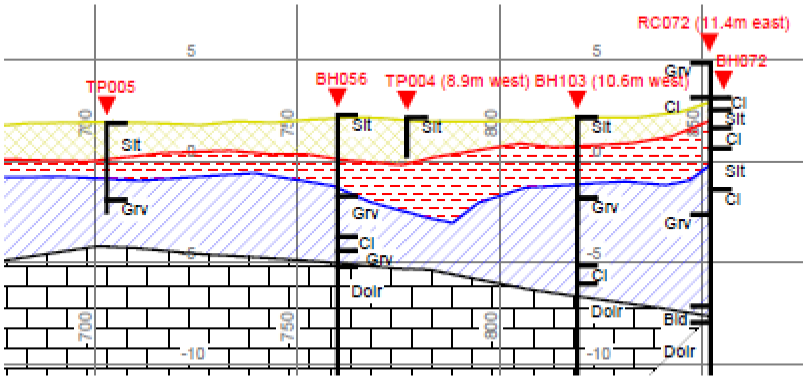
Topsoil, 100mm to 500mm thick was encountered. This was underlain by mixed alluvial deposits; very soft and soft slightly gravelly sandy SILT to depths between 1.3m bgl to 6.1m bgl. TP005 encountered a GRAVEL layer/ lense 200mm thick 1.5m bgl to 1.7m bgl. The SILT was underlain by slightly silty (very) sandy GRAVEL with low Cobble content to depths up to 7.7m bgl. At TP008 and TP010, the SILT was underlain by a silty very gravelly SAND with high Cobble content to depths 3.2m bgl and 3.5m bgl. Below this slightly silty very sandy GRAVEL with low Cobble content was encountered. The SILT deposits were described as peaty and organic at TP012 and TP014. The bedrock profile was varied. Dolerite was encountered 2.8m bgl (RC057A, 3.38mOD) to 12.8m bgl (RC072, -9.69mnOD).

Standard penetration test, Nspt data indicated medium dense GRAVEL (12 – 32). The SILT was described as very soft to firm SILT (3 -13). The SAND were encountered was described as medium dense (16 – 19).

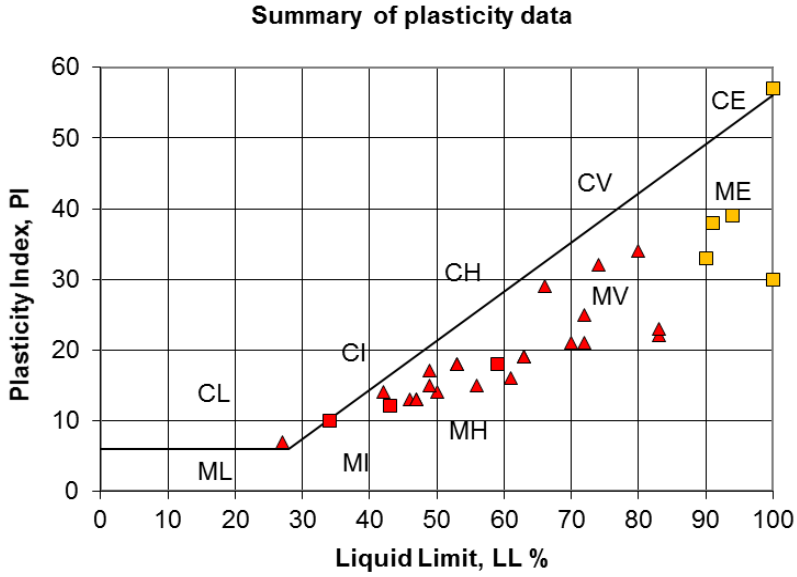
The geophysical survey (R5, S7 and S7') indicated approximately 1m to 2m of superficial deposit; soft (Silt) overlying medium dense (Granular deposits , stiff deposits) 4m to 7m thick. Bedrock (Dolerite) is assumed between -4.5mOD to -7.5mOD.







Groundwater was encountered at depths 1.6m bgl to 7.8m bgl (-3.05mOD to 2.14mOD). The static groundwater level was measured at BHA05 and BHA06 at 0.95m bg and 1.2m bgl (-0.12mOD to 0.25mOD) and at BH066 1.85m bgl (0.09mOD). Groundwater is assumed confined within the granular deposits. Seasonal fluctuations are anticipated but have not been defined. *In situ* groundwater data loggers are recommended to monitor variations in groundwater levels.



The SILT deposits were of mixed and varied plasticity intermediate to 'low to very high' (ML to MV). The SILT was of low to medium organic content (loss on ignition 3.5% to 6.4%). The sand fraction was 5% to 46% with 3% to 33% gravels and 31% to 85% silt fraction. Natural moisture content, w ranged between 10% and 65%. The ratio of natural moisture content to plastic limit (*w*/PL) was 0.6 to 2.1 indicative of stiff to very soft deposits (C504 Engineering in glacial deposits). This correlated with the *in situ* tactile assesment and range of standard penetration test Nspt data. Standard penetration test data, N values ranged between 0 and 16. A characteristic value of Nspt = 4 is recommended for the alluvial high plasticity (MH – ME) deposits and Nspt = 8 for the low plasticity (ML) glacial deposits (Nspt 8 – 14).

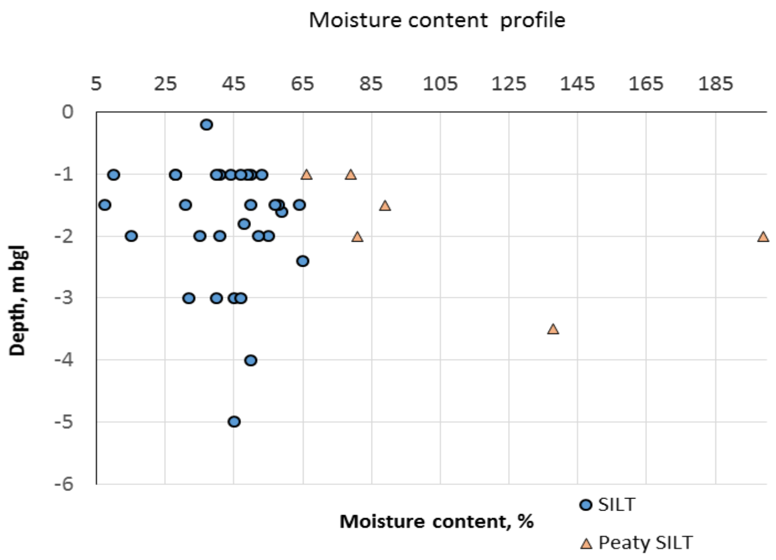
The peaty deposits at TP012 and TP014 were characterised by extremely high plasticity (ME) and loss on ignition values 25% to 41%. Natural moisture content, w ranged between 66% and 199%.

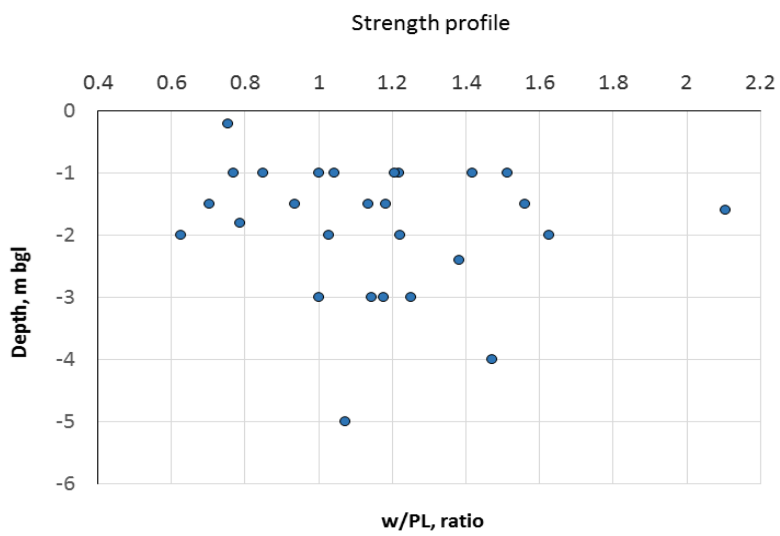
With a plasticity index, PI 7 to 17 and 16 – 39; a factor f_1 = 6 and 4.5 was such that:

$$Cu(kPa) = Nspt \times f_1 \text{ (Stroud, 1975).}$$

This yielded undrained shear strengths of <20kPa to 72kPa for the alluvial deposits, describing very soft to firm deposits (BS5930 1999). With Nspt values 8 to 14 the glacial deposits were described as firm to stiff (Cu 48kPa to 84kPa). Plasticiy data, PI 16 – 39 suggested an angle of friction of 26^{o} to 30^{o} for the sandy SILT and 30^{o} for the glacial deposits, PI 7 – 17 (C504; Terzaghi, Peck & Mesri, 1996).

Taking a characteristic Nspt value of 4 in the sandy SILT, an unfactored stiffness modulus (Young's modulus, E) of 6MPa is expected of the soft sandy SILT (PI 16 – 39, Stroud, 1975). Compressibility of the SILT, mixed alluvial deposits, is expected to be variable being low compressibility (ML) to high compressibility (MH-ME).



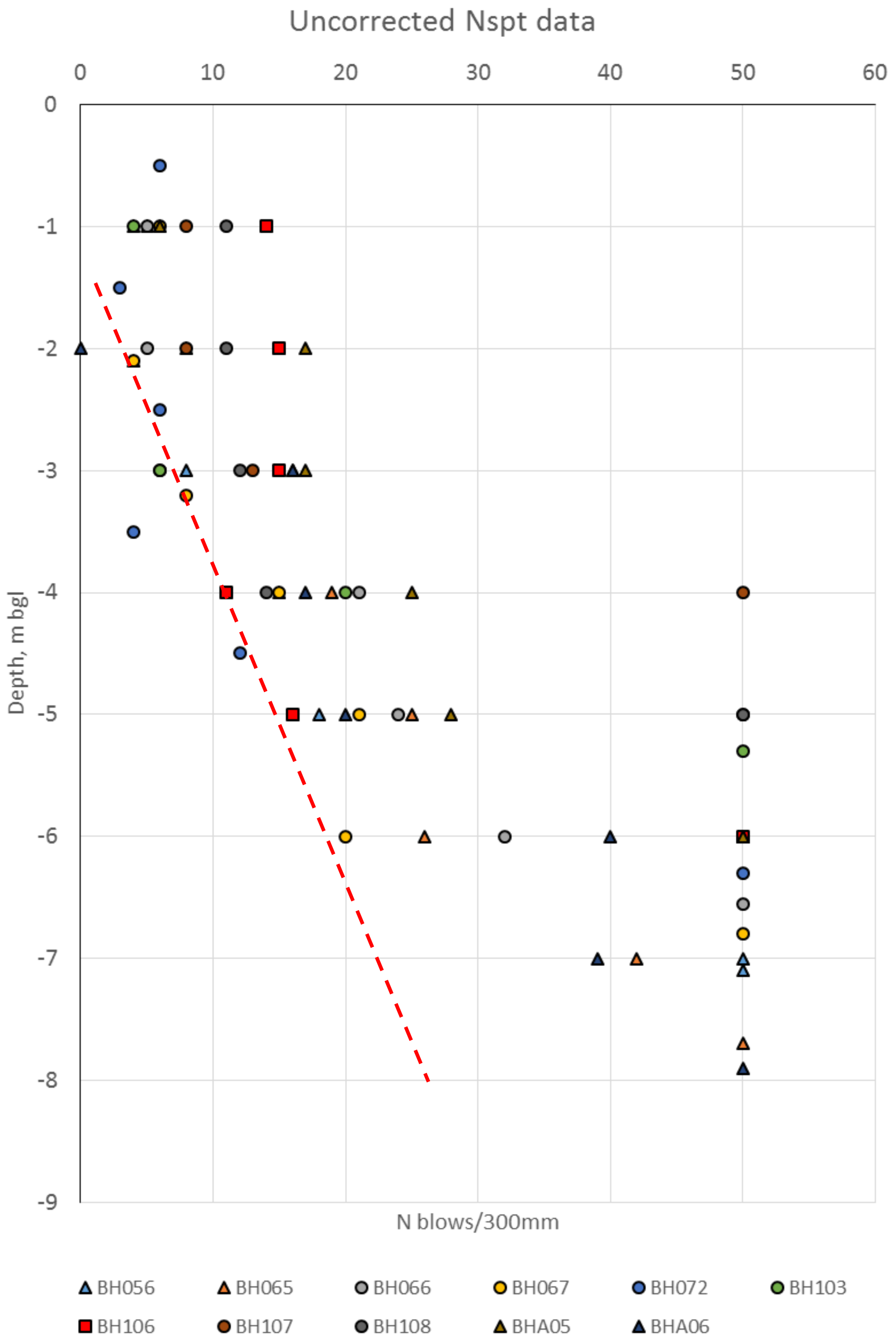


Particle size d_{10} has measured as (0.002mmto 0.003mm) but would be suggestive of a permeability 10^{-8} ms^{-1} being indicative of low permeability (C113, Control of groundwater for temporary works). SILT recompacted at moisture contents 15% to 26% (dry of natural moisture content -11%), provided permeability of 5.0 x 10^{-10} m s^{-1} to 9.2 x 10^{-6} m s^{-1}.

The SAND desposit were characterised by 2% to 5%, silt fraction, 17% to 37% Gravel and 40% to 77% sand fractions. The GRAVEL was characterised by 2% to 15% silt, 10% to 26%, sand and 53% to 82% gravels and a medium to high Cobble content (10% to 29%).

Taking a characteristic Nspt value of 15 in the SAND/ GRAVEL (Nspt 14 – 42) deposits a stiffness modulus of 15MPa is expected of the silty very gravelly SAND/ silty very sandy GRAVEL (Menzenbach, 1967, Bowles 1988).

Particle size(s) d_{10} of 1.18mm to 0.6mm were measured in the GRAVEL indicative of a permeabilities of the order 10^{-2} ms^{-1} to 10^{-3} ms^{-1}. This described medium to high permeability (C113). Particle size(s) d_{10} of 0.3mm to 0.425mm was measured in the gravelly SAND indicative of a permeability of the order 10^{-3} ms^{-1} to 10^{-4} ms^{-1}. This described medium permeability (C113).



Under deposited fill up to 2m settlement, ∆h of 115mm to 150mm is expected (BHA06) for the 2.7m thick SILT deposits.

The geotechnical hazard in this area is considered to be the (long-term) erosion of the fine SILT and Sandy deposits. Appropriate erosion control along the realigned river bank shall be provided. The form of the bank (erosion) protection shall be determined by the re-profiled river bank cross-section. Hard (riprap or other) and soft options (geotextiles and planting) are considered suitable.

The geotechnical hazard in the deposition area is considered to be the shallow slip or shear failure at the toe of fill material. It is recommended to allow for dissipation of pore pressure in the fine grained SILT that the deposition of excavated material be staged, subject to the design top height.

6.1.16 Channel dredging

Grab samples were taken in the River bed and assessed for Waste Acceptance Criteria, WAC; thirty three (33) number. Landfill WAC analysis (specifically leaching test results) must not be used for hazardous waste classification purposes. This analysis is only applicable for hazardous waste landfill acceptance and does not give any indication as to whether a waste may be hazardous or non-hazardous.

Name	Depth, m bgl	Comment
GB01	0.0	inert
GB02	0.0	inert
GB03	0.5	inert
GB04	0.5	inert
GB05	0.0	inert
GB06	0.5	inert
GB06	1.0	inert
GB06	1.5	inert
GB07	0.5	inert
GB07	1.0	inert
GB07	1.5	inert
GB08	0.5	stable non-reactive hazardous, Antimony 0.062mg/kg (10:1), PAH3.8mg/kg
GB08	1.0	inert
GB08	1.5	inert
GB09	0.5	inert
GB10	0.5	inert, PAH 11mg/kg
GB10	1.0	inert
GB10	1.5	inert

Name	Depth, m bgl	Comment
GB11	0.5	inert, PAH 8.3mg/kg
GB11	1.0	inert, PAH 7.3mg/kg
GB11	1.5	inert, PAH 7.9mg/kg, mineral oil 51mg/kg
GB12	0.5	inert
GB12	1.0	inert
GB12	1.5	inert
GB13	0.5	inert
GB13	1.0	inert, PAH 8.7mg/kg
GB13	1.5	inert
GB14	0.5	inert
GB14	1.0	inert
GB14	1.5	inert
GB15	0.5	inert
GB15	1.0	stable non-reactive hazardous, Sulphate 1100mg/kg (10:1)
GB15	1.5	stable non-reactive hazardous, Antimony 0.18mg/kg (10:1)

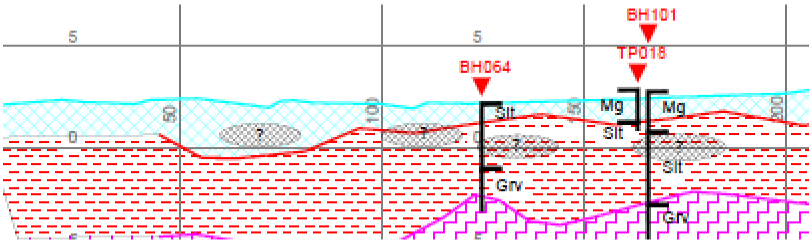
It is noted there is evidence of PAH contamination however it is below the 100mg/kg limit.

Mineral oil was noted at GB11 at 1.5m 51mg/kg.

Inert levels for Antimony were exceeded at GB08 0.5m (0.062mg/kg 10:1) and GB15 at 1.5m (0.18mg/kg 10:1).

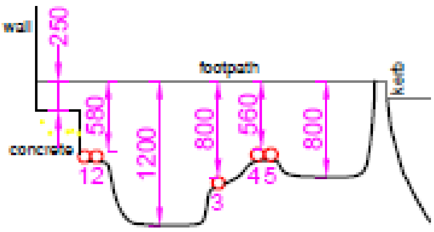
6.1.17 Miscellaneous ground hazards

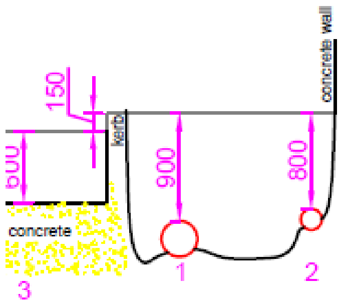
Other ground hazards identified were associated with buried concrete elements in the vicinity of ch4+800m.



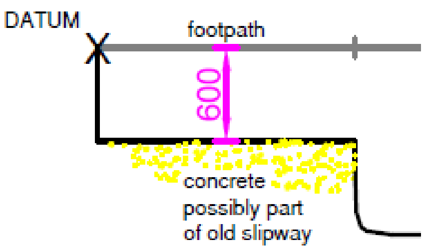
The may be other such obstructions associated with Made ground and existing wall structures along the river channel. The scope of the investigation did not specifically address this form of ground hazard.

Slit trench excavations identified the foundations of the existing quay walls, typically 350mm from the face of the wall, 330mmthick at a depth 0.25m bgl (ST003) to >0.8m bgl (ST006).





ST22 identified a possible old slipway structure 0.6m bgl. Geophysical profile S8 passed in front to the buried structure.



Concrete cover to utilities can also be expected along with possible culvert structures (BH014A).

It is recommended to excavate additional slit trenches to identify quay wall structures at Shannon and Abbey Quays and along the Wexford Road noting previously utilities limited the depth of excavations (ST001 Shannon Quay; ST004, Abbey Quay). Excavations parallel to the quay walls should be considered. It may also be considered using in direct ground penetrating radar, GPR to target shallow buried obstructions at areas of interest along the right bank.

6.2 GROUNDWATER

Groundwater is assessed in the form of a hydrogeological model. Typically Made ground overlay sandy organic SILT in turn overlying GRAVEL and DOLERITE bedrock. Permeability in the SILT is assumed to be uniformly low. Permeability in the GRAVEL and SAND deposits were varied depending on the Silt fraction. Permeability in the DOLERITE was varied depending on the weathering/ fracture condition. *In situ* measurement of permeability was limited. Particle size d_{10} indirectly informed the majority of permeability values presented.

At BH049, d_{10} = 0.15mm yielded permeability k = 10^{-4} ms^{-1} *in situ* value of 4.6 x 1o $^{-4}$ ms^{-1} was measured. Particle size d_{10} is expected to provide for a good estimate of permeability. In the case of the GRAVELS particle size d_{10} indicated much higher permeabilites 10^{-2} ms^{-1} to 10^{-3} ms^{-1} where limited *in situ* tests yielded values as low as 10 $^{-6}$ ms^{-1}. Given the influence of the base of the borehole on measured values where the casing was at the base of the borehole the *in situ* values should be questioned.

In many cased permeability was not determined where it was not possible to develop a test head indicating high permeability in-keeping with d_{10} estimates. Presently the hydrogeological model is considered weak with limited data.

6.2.1 Made ground

Permeability not assessed.

6.2.2 PEAT

Permeability k, 4.0x 10^{-7} ms^{-1}.

6.2.3 Peaty SILT

Permeability k, 2.0x 10^{-4} ms^{-1}.

6.2.4 Sandy SILT

Permeability k, 2.4 x10^{-7} ms^{-1} to 10^{-8} ms^{-1}.

6.2.5 Silty sandy GRAVEL

Permeability k, 10^{-2} ms^{-1} 3.3×10^{-3} ms^{-1} to 5.5×10^{-6} ms^{-1}.

6.2.6 Sand

Permeability k, 10^{-3} ms^{-1} to 10^{-7} ms^{-1}.

6.2.7 DOLERITE

Permeability k, 2.6×10^{-3} ms^{-1} to 5×10^{-7} ms^{-1}.

To further assess permeability and increase the data set under review, the groundwater strike data is assessed and inflow used to determine permeability. Inflow data indicated a permeability of 10^{-4} ms^{-1} in the GRAVELS. This indicated a medium permeability (C113). It must be noted that this was not a permeability test and so the geometry and durations of the observations provide for indicative details rather than direct determination of permeability. It is also note that such an assessment is better suited to fine grained deposits and not coarse Gravels.

> *It is recommended that a Hydrogeologist review the permeability data along with the ground and groundwater conditions to design a suitable programme in situ permeability tests. It is considered using the existing standpipe wells along with newly constructed boreholes to undertaken such an assessment. Soakaway tests may be used to assess the made ground at shallow depths.*

6.3 FOUNDATIONS

Foundations for the proposed flood walls will be constructed in the Made ground deposits. Allowable bearing pressures were varied between 30kPa to 130kPa for an upper limit of 25mm settlement. Differential settlement is expected to be of the order 10mm.

> *Plate loading tests have been recommended to further assess allowable bearing pressures.*

6.4 CHEMICAL

pH >6.5 and sulphate data (<0.010g/l to 0.65g/l) indicated a design class DS-1 in accordance with BRE digest for concrete in aggressive ground / XO i.a.w IS EN 206 with concrete class AC-1.

Exceptions to this were noted at; TPBH107, BH101, BH103 and BH105 with pH 5.5 to pH 6.5 at a depth 1.6m bgl to 3.0m bgl where peaty material and woody inclusions were encountered. Acid resistant concrete AC-2z is recommended where peaty deposits are encountered for a design class DS-1 in accordance with BRE digest for concrete in aggressive ground at the new bridge structure, B5.

Loss on ignition values of 11% to 59% were measured at TP012 (EX8), TP019 (B5), TPA03 (C01), BH028 and BH038 indicative of high organic content and woody inclusions, acid resistant concrete should also be used in these areas.

pH of 9.4 and 11.2 were measured at BH010 and BH038 respectively possibly indicative of hydrocarbon contamination. Olfactory evidence of bituminous contamination was noted at 1.8m bgl at BH003.

> *It is recommended that further environmental sampling is undertaken at BH003, BH010 and BH038 to further assess these locations.*

6.4.1 Pyrite

Assessment of Pyrite at RC101 and RC108 yielded acid soluble sulphate, SO_4 values <0.2%, water soluble sulphate values < 500mg/kg and total Sulphur, S values 0.3% < S <1.0%. The total Sulphur lies outside the ‘pass’ limit <0.3% (I.S.398 Part 1) and so further assessment is needed.

> *It is recommended that petrographic analysis is undertaken to further assess the risk associated with Pyrite.*

6.4.2 Contamination

It is assumed that the material excavated from the river bed shall be disposed of at a suitably licenced landfill where waste acceptance criteria, determined from grad samples, typically identified inert deposits. While the leachable concentrations typically identified inert deposits, no assessment of the absolute levels of heavy metals of other contaminants has been determined. The assessment of these material is restricted to the means of disposal. No environmental risk has been carried out in the absence of absolute concentrations. It is not presently recommended to re-use these deposits.

The following grab samples measured leachable levels >inert limit:

GB13; 1.0m Dissolved organic carbon 610mg/kg exceeding the 500mg/kg limit.

GB15; sulphate 1300mg/kg exceeding the 1000mg/kg inert limit

TP26; sulphate 1100mg/kg exceeding the 1000mg/kg inert limit

These exceedances of the inert limits shall be reviewed by the licenced landfill operator to assess if a relaxation is applicable under their waste management licence.

Polyaromatic hydrocarbons, PAH levels of 3.8mg/kg to 11mg /kg were measured at GB8, GB10 and GB11 between depths 0.5m to 1.5m. While these were below the inert limit of 100mg/kg they identify Polyaromatic hydrocarbon contamination (PAH 2mg/kg – 100mg/kg).

GB08, identified mineral oil concentrations 15mg/kg and 22mg/kg. While these were below the inert limit of 500mg/kg they identify contamination.

GB11, identified mineral oil concentrations 51mg/kg. While these were below the inert limit of 500mg/kg they identify contamination.

TP01, identified mineral oil concentrations 24mg/kg. While these were below the inert limit of 500mg/kg they identify contamination.

ST04, identified mineral oil concentrations 30mg/kg. While these were below the inert limit of 500mg/kg they identify contamination.

It is understood that a waste licence shall be required to deposit material arising from the dredging of the river channel, CH1 at deposition areas; DA1, DA2 and DA3.

A detailed environmental assessment shall be carried out by a suitably qualified Environmental consultant to fully address the issues associated with excavation of deposits and the possible deposition of material and its re-use in embankment fill.

> *It is recommended that absolute concentration of heavy metals be determined, particularly at ST04, TP01, GB8, GB10 and GB11, to fully assess the material arising from the dredging process.*

6.5 RE-USE OF MATERIALS

It is proposed to re-use material excavated from the river banks as general embankment fill associated with the deposition areas, DA1, DA2 and DA3.

Moisture condition values, MCV were typically <MCV5 and as such at natural moisture content, *w* were unsuitable for both general embankment fill (MCV8 – MCV12) and general landscaping fill (>MCV5).

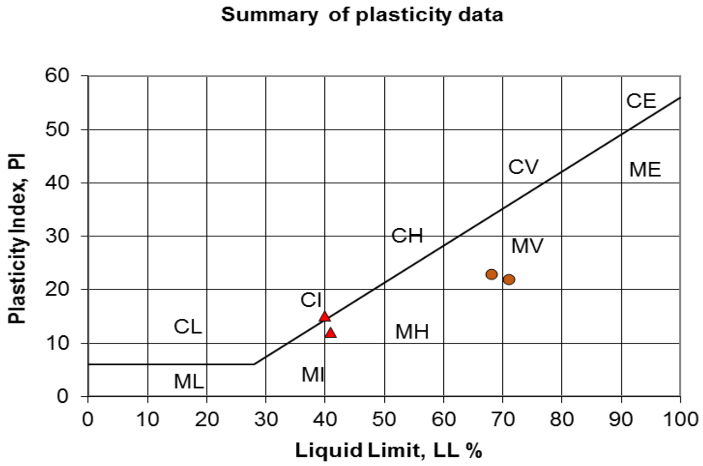
> *It is noted that there was no engineering assessment/ characterisation of the river sediment other that the chemical analysis discussed previous. It is recommended to determine the grading of the river bed deposits to fully assess disposal of dredged deposits to the deposition areas.*

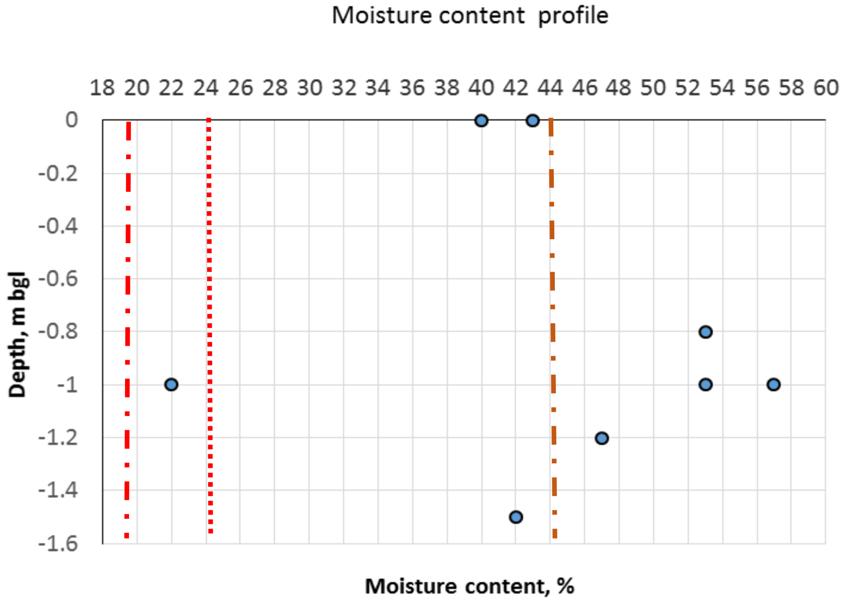
6.5.1 Cutting EX1/ EX2, right bank ch6+500m to ch6+780m

At cutting EX1/ EX2, BHA01, TPA01, TP027 and TP028 an optimum moisture content of 12% was determined for a maximum dry density of 1.31 Mg/m^3 for the very high plasticity deposits (MV) and 19% to 20% for maximum dry density 1.6 Mg/m^3 to 1.66Mg/m^3 for the intermediate plasticity deposits (MI).

Natural moisture content, *w* lie significantly wet of the optimum, +23% moisture for the very high plasticity deposits. However compaction of 95% maximum dry density can be achieved at moisture content up to 44% indicating drying up to 13% will be required. Moisture condition values MCV5 are expected at moisture content 54% with the range MCV8 to MCV8 achievable at 20% to 44% moisture for the very high plasticity deposits.

For the intermediate plasticity deposits natural moisture content is +3% wet of optimum. A moisture content up to 24% a density corresponded to 95% compaction can be achieved. Moisture condition values MCV5 are expected at moisture content up to 24% with the range MCV8 to MCV8 achievable at 6% to 16% moisture for the very high plasticity deposits.





It is expected that the excavate deposits will be suitable for deposition as general landscaping fill with drying up to -8% (MI) and -13% (MV) required to be suitable for embankment fill in the deposition areas.

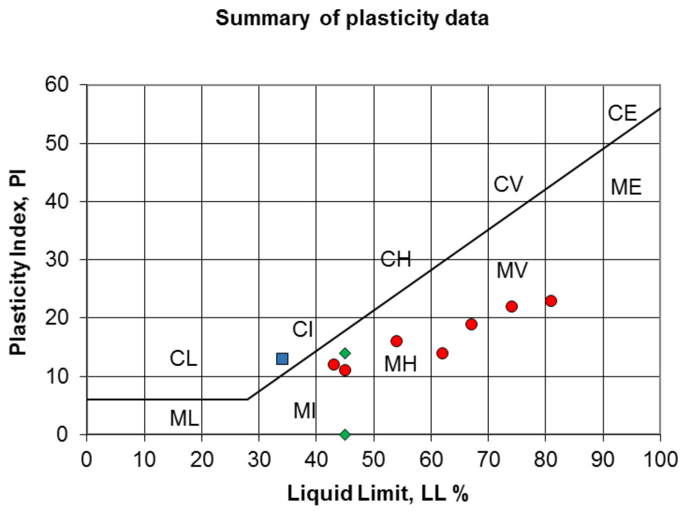
6.5.2 Cutting EX3, left bank ch6+600m to ch5+725m

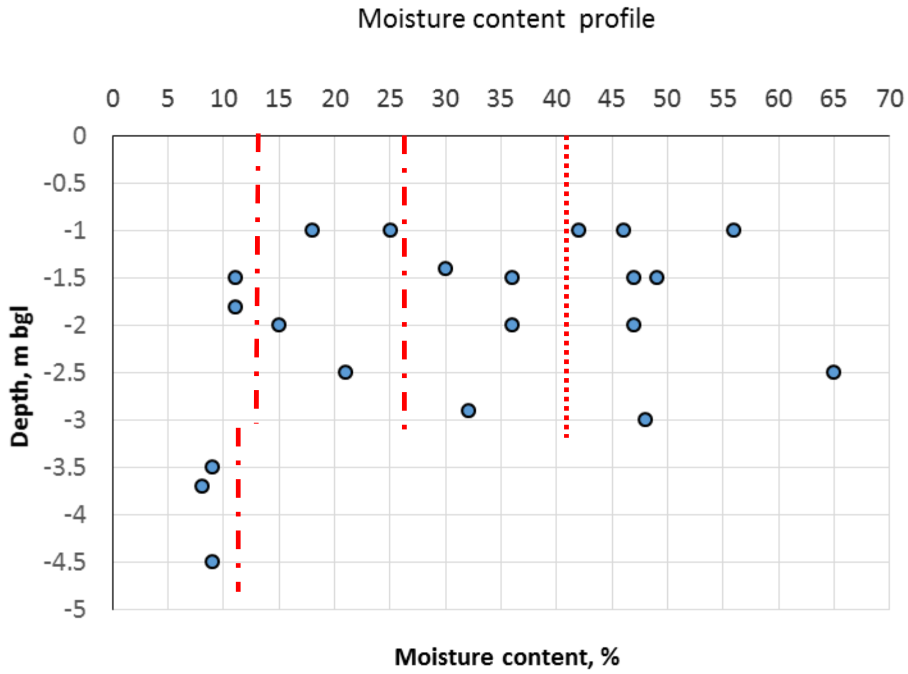
At cutting EX3, TPA06, TPA02, TP033, TP035 and TP029 an optimum moisture content of 13%, 17%, 19% and 26% was determined for a maximum dry density of 1.35 Mg/m^3 to 1.58 Mg/m^3 for the high to very high plasticity deposits (MH- MV). Natural moisture content, *w* lie significantly wet of the optimum, +21% to +39% moisture for the high to very high plasticity deposits. However compaction of 95% maximum dry density can be achieved at moisture contents between 10% to 40%, indicating drying up to -21% will be required. MCV moisture content relationship indicated MCV5 at moisture content 40% to 50% with MCV8 to MCV12 achievable at moisture content 23% to 38%.

Single, moisture condition values MCV0 to MC11 were measured at moisture content 33% to 64%.

Optimum moisture contents of 12% and 16% corresponded to maximum dry density to 1.73 Mg/m^3 and 1.76 Mg/m^3 for the intermediate plasticity deposits (MI). Natural moisture content, *w* lie significantly wet of the optimum, +21% to +39% moisture for the high to very high plasticity deposits. However compaction of 95% maximum dry density can be achieved at moisture contents between 14% to 22%, indicating drying up to -3% will be required. MCV moisture content relationship indicated MCV5 at moisture content 16% to 34% with MCV8 to MCV12 achievable at moisture content 13% to 28%.

For the GRAVEL deposits (TP029 and TP035) an optimum moisture content of 11% for a maximum dry density 1.96Mg/m^3 to 2.03Mg/m^3 was considered. Natural moisture content in these deposits was -3% dry to +3% wet of optimum. Compaction of 95% maximum dry density can be achieved at natural moisture contents.





It is expected that the excavate deposits of intermediate plasticity will be suitable for deposition as general landscaping fill at the range of natural moisture contents and/or with wetting up to +9% (MI) required to be suitable for embankment fill.

It is expected that the excavate deposits of intermediate plasticity will be suitable for deposition as general landscaping fill at the range of natural moisture contents and/or with drying up to -20% (MH-MV) required to be suitable for embankment fill.

6.5.3 Cutting EX4, right bank ch5+725m to ch5+535m

At cutting EX4, BH027, Moisture condition value of MCV5 was measured at moisture content 19%. Natural moisture content was measured between 20% and 33%. It is expected that excavated deposits will not be suitable for re-use at natural moisture contents as landscaping fill in deposition areas. Drying up to -14% may be required.

It is recommended that the deposits be samples and the waste acceptance criteria determined to identify a suitable means of disposal for these deposits, particularly the upper 2.5m of Made ground.

6.5.4 Cutting EX5, left bank ch5+700m to ch5+535m

At cutting EX5, BH008, BH009, BH010, Moisture condition values of MCV0 to MCV3 were measured at moisture content 20% to 29%. Natural moisture content was 20% to 33%. It is not expected that excavated deposits will be suitable for re-use at natural moisture content. A range of suitable moisture content has not been established.

It is recommended that the deposits be samples and the waste acceptance criteria determined to identify a suitable means of disposal for these deposits, particularly the upper 2.2m of Made ground.

Additional laboratory testing may also be carried out to determine a suitable range of moisture contents for re-use of the deposits.

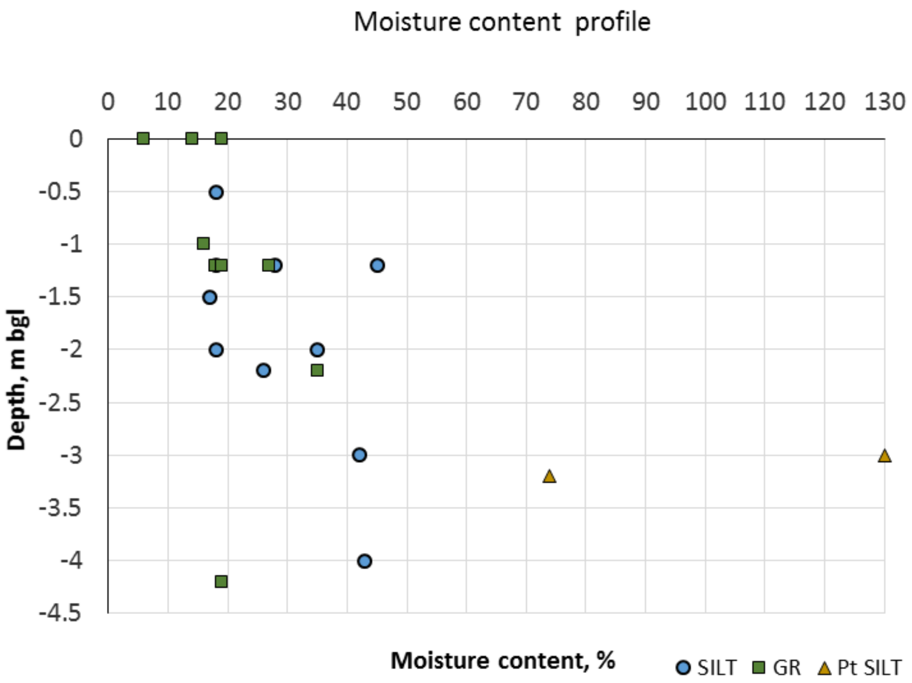
6.5.5 Cutting EX6, right bank ch5+540m to ch5+480m

There were no exploratory works at this location. The nearest location was BH031 indicating GRAVEL deposits to a depth 4.1m bgl.

It is recommended that the deposits be samples and the waste acceptance criteria determined to identify a suitable means of disposal for these deposits, particularly the upper 2.2m of Made ground. Moisture condition value MCV shall also be determined.

6.5.6 Cutting EX7, right bank ch5+380m to ch4+900m

At cutting EX7, BH041, a moisture condition value MCV6 was measured at natural moisture content 72%. Based on limited data it is not expected that excavated deposits will be suitable for re-use at natural moisture content.



It is recommended that the deposits be samples and further laboratory tests be carried out to fully assess these deposits. Eight (8) cable percussive boreholes are a proposed to assess the upper 3.0m of Made ground. Waste acceptance criteria shall also be determined to identify a suitable means of disposal for the Made ground deposits, particularly location BH040.

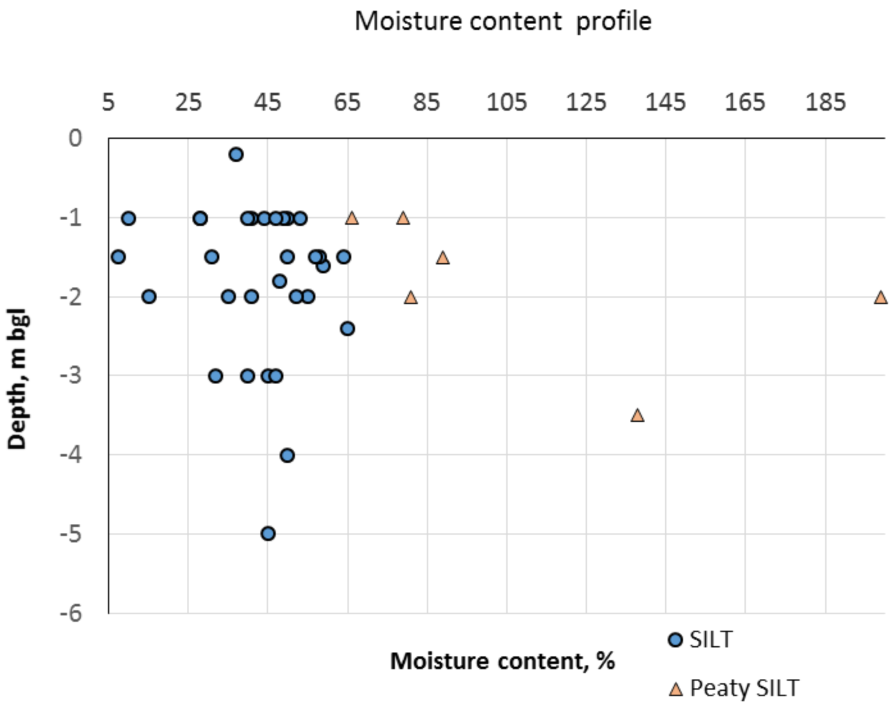
Additional laboratory testing may also be carried out to determine a suitable range of moisture contents for re-use of the deposits.

6.5.7 Cutting EX8, right bank ch5+000m to ch3+600m, new channel,

At cutting EX8, right bank ch5+000m to ch3+600m a new channel is proposed, TP003-TP014, BH101.

An optimum moisture content of 18% to 21% was measured for a maximum dry density of 1.52Mg/m^3 to 1.61Mg/m^3 for intermediate plasticity deposits (MI). However compaction of 95% maximum dry density can be achieved at moisture contents between 22% to 36%, indicating drying up to -3% to -34% will be required. Moisture condition value MCV5 can be achieved at moisture content <40% with MCV8 to MCV12 between 26% to 34%.

An optimum moisture content of 18% to 29% was measured for a maximum dry density of 1.39Mg/m^3 to 1.61Mg/m^3 for high plasticity deposits (MH). However compaction of 95% maximum dry density can be achieved at moisture contents between 14% to 36%, indicating drying up to -15% to -20% will be required. Moisture condition value MCV5 can be achieved at moisture content <42% with MCV8 to MCV12 between 10% to 34%.



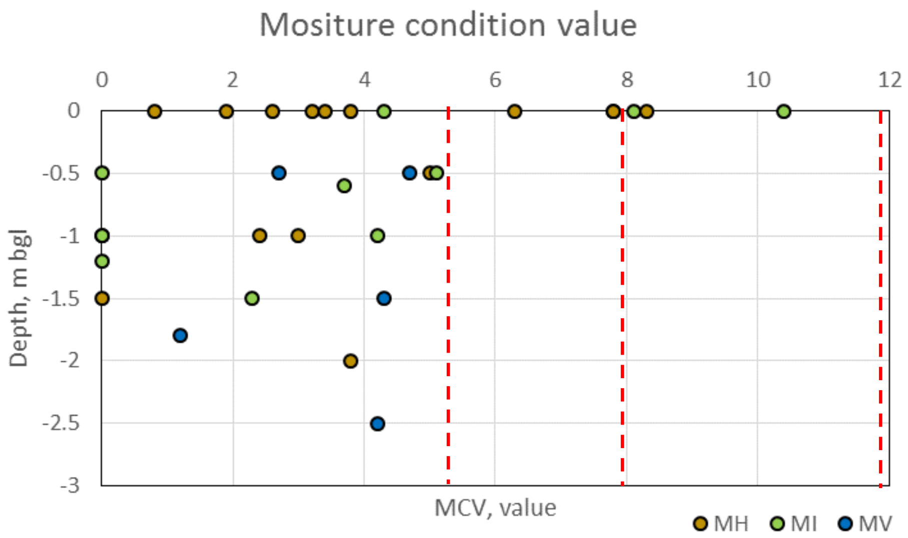
An optimum moisture content of 14% to 31% was measured for a maximum dry density of 1.18Mg/m^3 to 1.48Mg/m^3 for very high plasticity deposits (MV). Compaction of 95% maximum dry density can be achieved at moisture contents between 7% to 48%, indicating drying up to

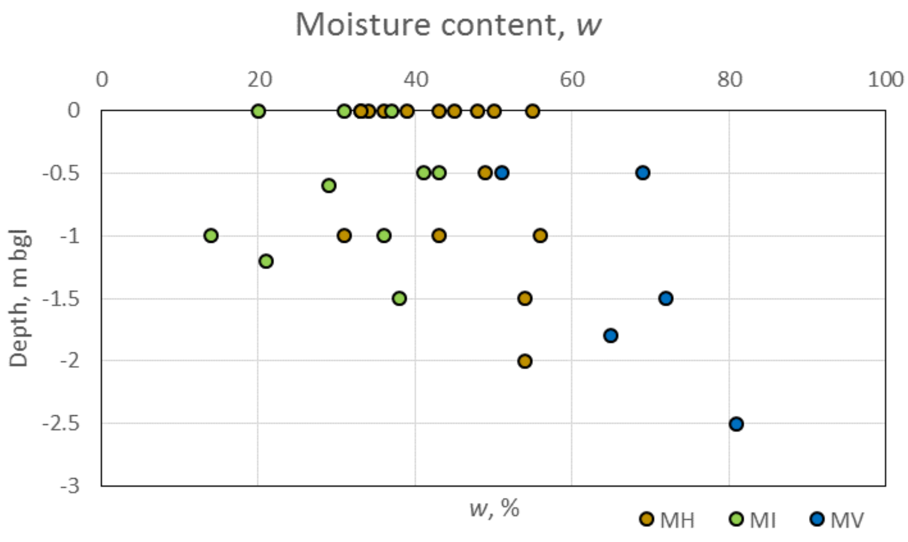
-12% to -31% will be required. Moisture condition value MCV5 can be achieved at moisture content <34|% with MCV8 to MCV12 between 8% to 28%.

Moisture condition value MCV5 can be achieved at moisture content <60% with MCV8 to MCV12 between 24% to 46% for the very to extremely high plasticity deposits (MV-ME).

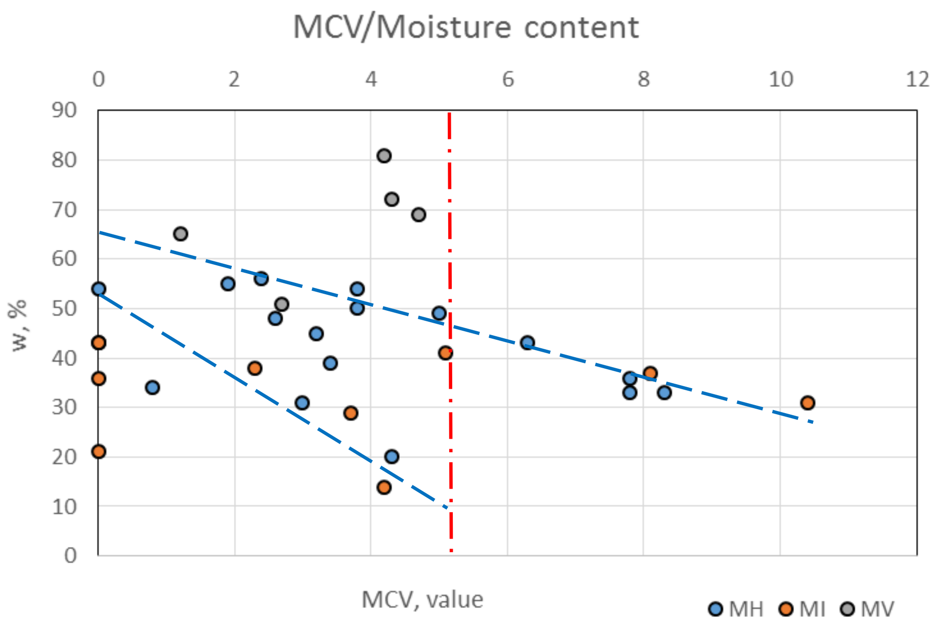
For the GRAVEL deposits (BH101) an optimum moisture content of 9% for a maximum dry density 1.88Mg/m^3 was considered. Natural moisture content in these deposits was -12% dry of optimum. Compaction of 95% maximum dry density can be achieved with drying of -2%.

Single moisture condition values MCV0 to MCV10 were measured at moisture content 14% to 81%. It can be seen the majority of values below MCV 5 and indicate the deposits to be unsuitable for re-use at natural moisture content.





There was no obvious relationship between moisture content and MCV. Natural moisture content tyically increased with depth with corresponding decreasing MCV. Excavated deposits will not be suitable for deposition at natural mositure content. It is recommended to stockpile the excavated deposits to allow them to dry before compacting them in the deposition area. Drying of -8% to -31% moisture is required to achieve MCV5 (general landscaping fill).



6.5.8 Cutting C01/ DA01, left bank ch0+550m to ch0+750m

Some cutting is expected adjacent to the flow control structure. TPA003 indicated an optimum moisture content of 18% to 21% for a maximum dry density of 1.63Mg/m^3 to 1.66Mg/m^3 for the high plasticity SILT deposits (MH). 95% compaction is achieved at 14% to 26% moisture. Moisture condition values MCV8 to MCV12 was achievable at 12% to 24% moisture content. The deposits are wet +8% of optimum moisture content. Drying of -2% to -6% is required at TPA003.

TPA004 indicated an optimum moisture content of 20% for a maximum dry density of 1.12Mg/m^3 for the high plasticity SILT deposits (MH). 95% compaction is achieved at 6% to 48% moisture. Moisture condition values MCV8 to MCV12 was achievable at 30% to 48% moisture content. The deposits are wet +38% of optimum moisture content. Drying of -10% is required at TPA004.

For the GRAVEL deposits (TPA005) an optimum moisture content of 7% for a maximum dry density 2.01Mg/m^3 was considered. Natural moisture content in these deposits was +1% wet of optimum. Compaction of 95% maximum dry density can be achieved at natural moisture content for the deposits at 2.0m bgl.

7 SUMMARY

1. The ground conditions at the site was characterised by; Topsoil 0.10m to 1.0m thick, Made ground 0.2m to 6.3m thick, bituminous construction 40mm to 400mm thick; concrete 0.1m to 5.0m thick; undifferentiated glacial deposits of slightly sandy gravelly CLAY/ SILT, (slightly to very) silty (slightly to very) gravelly SAND, (slightly) silty (very) sandy GRAVEL with variable cobble and boulder content to depths up to 12.8m bgl.

2. Organic deposits of peaty CLAY were identified 1.7m to 1.9m thick. Alluvial deposits of (slightly) sandy SILT were also present.

3. Bedrock encountered within the site was variable and comprised; weak to very strong LIMESTONE 3.0m bgl to 8.6m bgl, weak SLATEY-MUDSTONE between depths 2.1m bgl and 7.0m bgl, weak to strong SHALE 5.1m bgl to 11.3m bgl and weak to very DOLORITE 1.5m bgl to 11.3m bgl.

4. Six (6) BRE365 Soakaway test were carried out over a single (1) drainage cycle. The data is presented in APPENDIX A of the factual report.

5. Three (3) number plate loading tests were carried out. The data is presented in APPENDIX A of the factual report.

6. Static cone penetration testing was carried out by In Situ Ltd. on behalf of PGL. A report is presented within APPENDIX B of the factual report.

7. TRL dynamic probing was carried out at selected trial pit locations six (6) number providing for an estimate of California bearing ratio, CBR. The data is presented in APPENDIX A of the factual report.

8. A non-intrusive geophysical survey utilising 2D resistivity and seismic refraction techniques was carried out by Minerex on behalf of PGL. A report is presented within APPENDIX C of the factual report.

9. Groundwater was encountered between depths of 1.6m bgl and 9.2m bgl. Details are summarised herein and presented on the relevant logs in APPENDIX A of the factual report.

10. Eighteen (18) number standpipe wells were installed to allow for groundwater monitoring, see Section 5.

11. *In situ* falling head permeability tests were carried out in the cable tool and rotary borehole casing(s) at specified borehole locations; BH007A, BH008, BH009, BH011, BH018A, BH022, BH032, BH037, BH038, BH044, BH045, BH055 and BH065; RC012, RC014A, RC025, RC026, RC035, RC047, RC049, RC050, RC051, RC060, RC061, RC062, RC066, RC067 and RC202.

12. Detailed records of the ground and groundwater conditions can be found on the exploratory logs and photographic records presented within APPENDIX A of the factual report. Further details of the ground conditions can be found in the geophysical survey report presented within APPENDIX C of the factual report.

13. Laboratory testing was undertaken to determine the classification, engineering properties and geo-chemistry of the soil and rock encountered during the ground investigation. The data is presented in APPENDIX D of the factual report.

14. The exploratory locations are presented on the location plans presented within APPENDIX E of the factual report.

8 RECOMMENDATIONS

The following is a non-exhaustive scope for infill/ supplementary works. Further works may be deemed necessary by the Engineer subject to the requirements of the detailed design.

1. It is recommended that **fifteen (15**) **trial pit excavations** or **window samples** be undertaken in the vicinity of existing locations; TP028, SK01, SK02, TPA02, TP030 and TP034; BH003, BH010 and BH038; ST04, TP01, GB8, GB10 and GB11 and at EX6. Moisture condition value, MCV shall be determined at EX6, environmental assessment and absolute concentration of heavy metals of excavated deposits shall be determined.

2. It is recommended to continue groundwater monitoring. It is recommended **ten (10) data loggers** are installed at locations; BH012, BH022, BH037, BH055, BHA01, BHA02, BHA03, BHA04, BHA05 and BHA06.

3. It is recommended to carry out **eighteen (18) number plate loading tests** to verify the design bearing capacity at; BH302, BH304, BH305, BH018, BH020, BH023, BH025. BH007, BH009, BH032, BH036, BH038, BH040, L02, L07 and RC1.

4. It is recommended that in situ permeability tests (falling and constant head tests) are recommended at thirteen (13) locations; BH301, BH305, **BH018, BH023, BH025**, **BH008, BH009**, BH012, BH033, BH035, BH037, BH041 and BH043. Several tests are expected at each location in boreholes. It is recommended that a Hydrogeologist review the permeability data along with the ground and groundwater conditions to design a suitable programme in situ permeability tests. Environmental sampling is required along **EX4; EX5** to establish waste acceptance criteria.

5. It is recommended that **sixty three (63) dynamic probes** (40 to 75number, 5m to 10m spacing) shall be considered to a depth 5.0m along the alignment of wall L01. Dynamic probing (20number, 15m spacing) shall also be considered to a depth 4.0m along the alignment of wall L03 and three (3) dynamic probes between BH013 and BH014.

6. It is recommended to excavate **six(6) slit trenches** to identify quay wall structures at Shannon and Abbey Quays, along the Wexford Road and at L02. Excavations parallel to the quay walls should be considered. It may also be considered using in direct ground penetrating radar, **GPR** to target shallow buried obstructions at areas of interest along the right bank.

7. It is recommended to excavate **one(1)** shallow **trial pit excavation** and **plate loading test** be carried out to verify the construction the car park and assess bearing for overlay construction. Environmental PAH samples are recommended in the carpark to assess the existing surfacing.

8. A **pavement condition survey** is also recommended to identify area of defect to allow for appropriate remediation. **Falling weight deflectometer**, FWD stage and stage 2 analysis shall also be considered with regard to pavement overlay design. It is recommended to further assess the pavement construction using TRL cone penetration tests to establish the California bearing ratio, CBR of the formation deposits.

9. It is recommended that **petrographic analysis** is undertaken to further assess the risk associated with Pyrite.

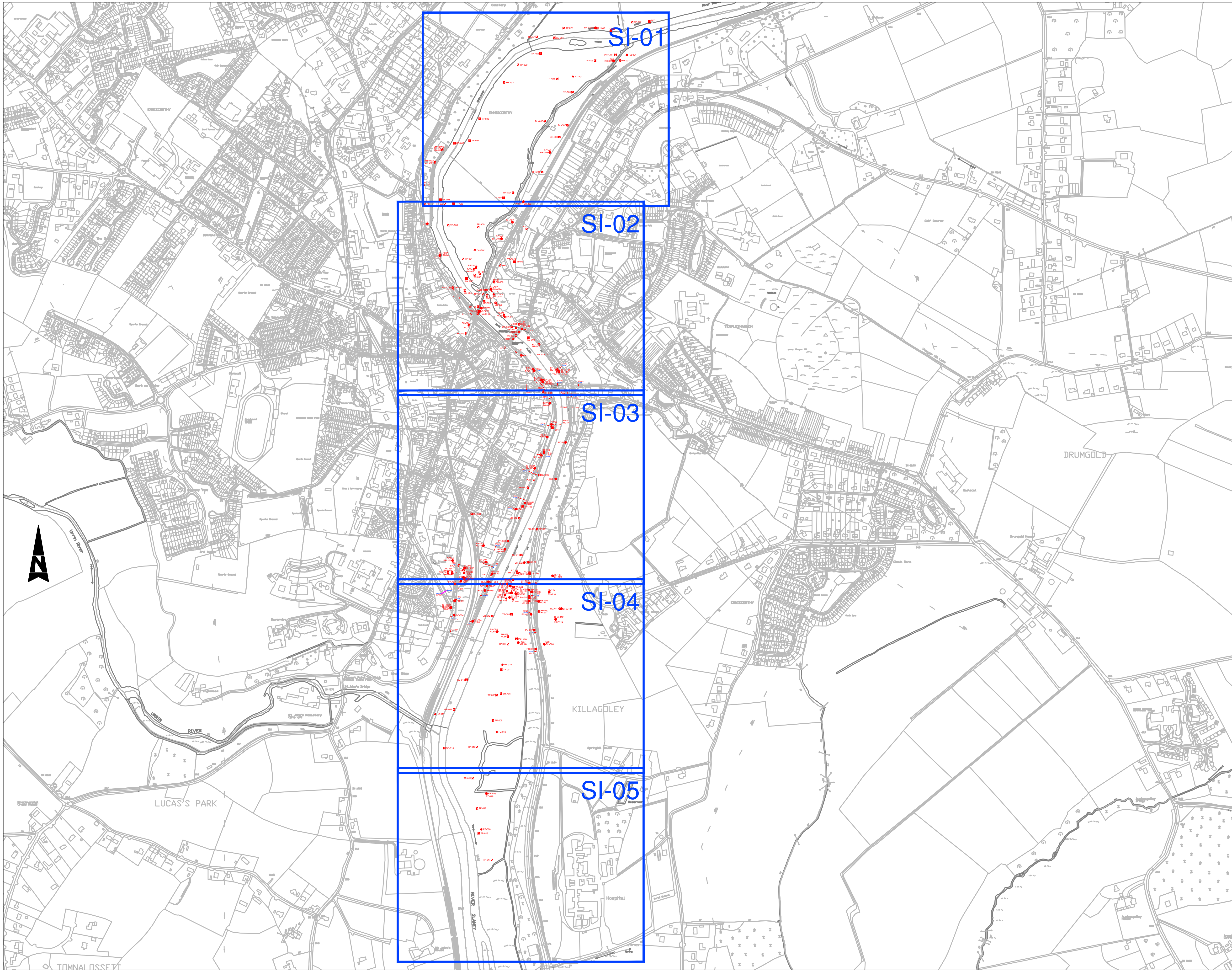
10. It is recommended to carry out further **grab sampling** to determine the grading of the river bed deposits to fully assess disposal of dredged deposits to the deposition areas.

11. **Eight (8) cable percussive boreholes** are a proposed to assess the upper 3.0m of Made ground along EX7. Waste acceptance criteria shall also be determined to identify a suitable means of disposal for the Made ground deposits, particularly location BH040 at EX7. Additional laboratory testing may also be carried out to determine a suitable range of moisture contents for re-use of the deposits.

APPENDIX 1

EXPLORATION LOCATION PLANS

Exploration Location Layout	P16087-SI-A
Exploration Location Plans	P16087-SI-01 to P16087-SI-05



JOB NAME:
ENNISCORTHY
Flood Defence Scheme

Sheet Title:
EXPLORATORY LOCATION LAYOUT

JOB NUMBER:
P16087

DRAWING NUMBER:
P16087-SI-A

DRAWN BY:
Gary Curtin

DATE:
11/09/2017

SCALE:	APPROVED:
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REVISION:
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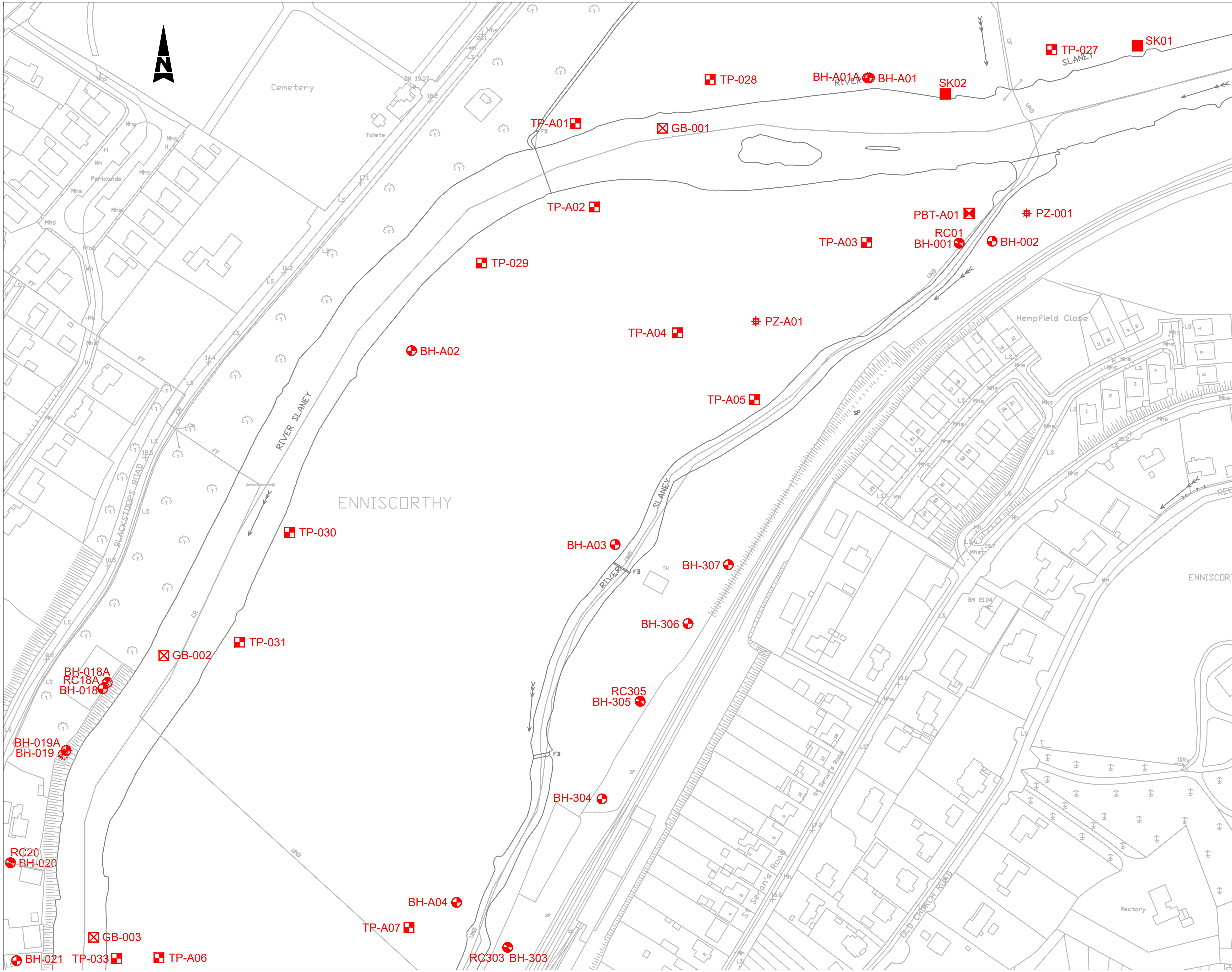

KEY:

ST00 DATUM	Denotes Slit Trench and Datum location
TP-00	Denotes Trial Pit location
BH-0	Denotes Borehole location
PZ-00	Denotes Piezo Cone location
RC00	Denotes Rotary Core location
PC00	Denotes Pavement Core location
GS-00	Denotes Grab Sample location
PBT-00	Denotes Plate Bearing Test location
SK-00	Denotes Soakaway location

JOB NAME:

ENNISCORTHY
Flood Defence Scheme

Sheet Title:

LOCATION PLAN

JOB NUMBER:

P16087

DRAWING NUMBER:

P16087-SI-01

DRAWN BY:

Gary Curtin

DATE:

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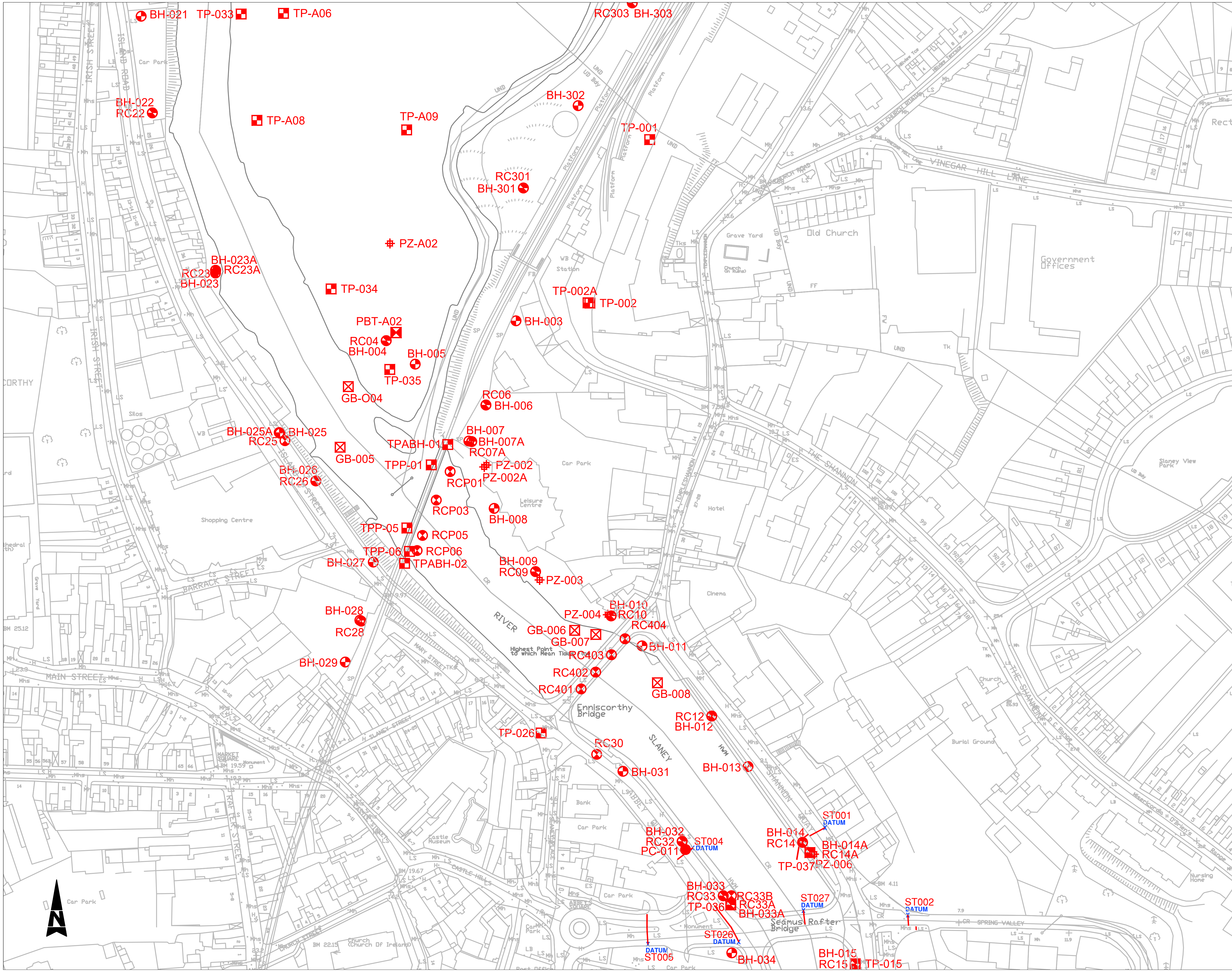

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RC00	Denotes Rotary Core location
PC00	Denotes Pavement Core location
GS-00	Denotes Grab Sample location
PBT-00	Denotes Plate Bearing Test location
SK-00	Denotes Soakaway location

JOB NAME:

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Flood Defence Scheme

Sheet Title:

LOCATION PLAN

JOB NUMBER:

P16087

DRAWING NUMBER:

P16087-SI-02

DRAWN BY:

Gary Curtin

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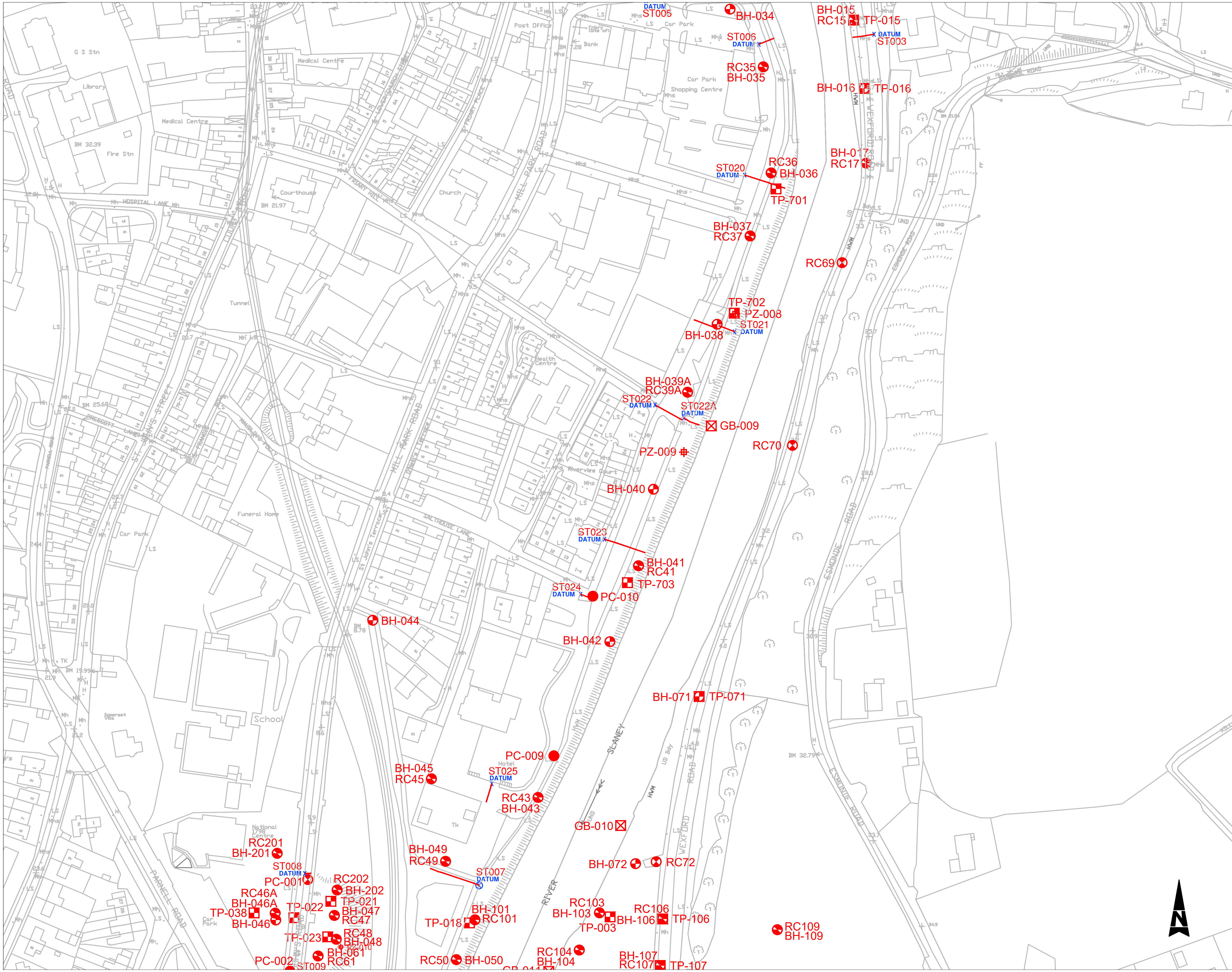

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PC00	Denotes Pavement Core location
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PBT-00	Denotes Plate Bearing Test location
SK-00	Denotes Soakaway location

JOB NAME:

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Flood Defence Scheme

Sheet Title:

LOCATION PLAN

JOB NUMBER:

P16087

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P16087-SI-03

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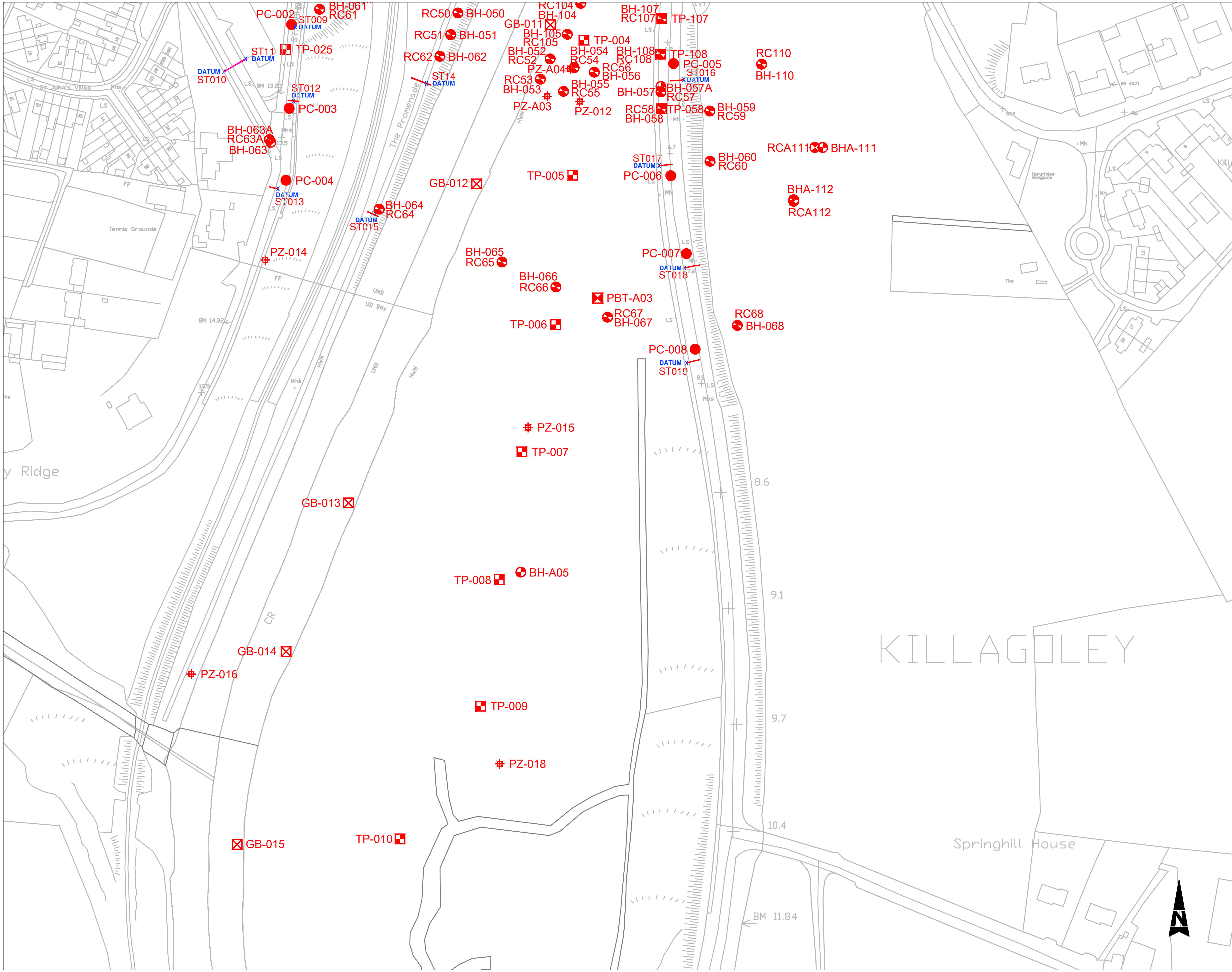

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PC00	Denotes Pavement Core location
GS-00	Denotes Grab Sample location
PBT-00	Denotes Plate Bearing Test location
SK-00	Denotes Soakaway location

JOB NAME:
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Flood Defence Scheme

Sheet Title:
LOCATION PLAN

JOB NUMBER:
P16087

DRAWING NUMBER:
P16087-SI-04

DRAWN BY:
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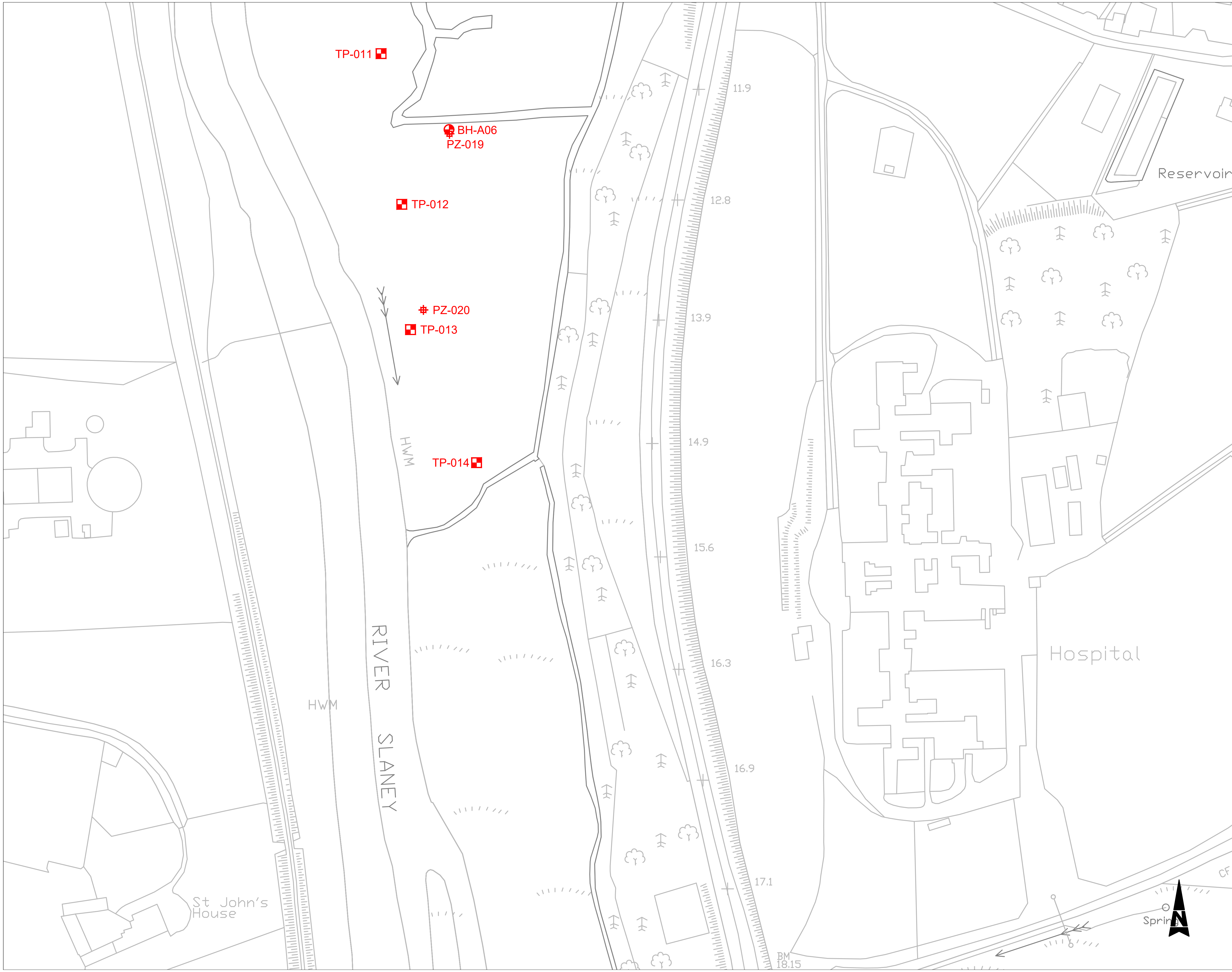

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SK-00	Denotes Soakaway location

JOB NAME:

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Flood Defence Scheme

Sheet Title:

LOCATION PLAN

JOB NUMBER:

P16087

DRAWING NUMBER:

P16087-SI-05

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