A30 Chiverton to Carland Cross
Environmental Statement

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Ground Investigation Report Addendum

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Table of Contents

Table of Figures

Table of Tables

Executive summary

This Ground Investigation Report (GIR) Addendum has been produced following a second phase of ground investigation (GI) completed in October 2017 to further inform the proposed duelling of the A30 between Chiverton and Carland Cross. The first phase of GI, completed between January 2017 and March 2017, is reported in the GIR [1]. The second phase of GI was required to investigate three areas that the first phase was unable to investigate. The second phase of GI was also unable to investigate one of these areas and a third phase is therefore required, after which this GIR Addendum will be updated.

This report builds on the content presented in the GIR [1] and the Preliminary Sources Study Report (PSSR) [2] to refine the ground model for the project and fill in various gaps in the ground model along the proposed route. This report also assesses the results of in situ and laboratory testing to determine characteristic geotechnical parameters for subsequent use in design. The following provides a brief summary of the contents and main findings of this GIR Addendum:

Section 2 clarifies the scope of the second phase of GI and why it was required. It sets out the current proposals for the route, including numbers of structures.

Section 3 provides updates on the general geological setting of the scheme following a recent review of aerial photography and an updated mining search and risk assessment.

Section 4 details the extent of both phases of GI in terms of numbers of exploratory holes, in situ testing, and laboratory testing (geotechnical and geo-environmental).

Section 5 summarises the soil and rock encountered along the route. It concludes that the ground conditions generally confirm published geology, comprising the following materials:

- **Made ground** - encountered in various locations
- **Superficial Deposits** (Alluvium or solifluction debris/Head) - encountered in various locations. Areas in which Superficial Deposits are still anticipated despite not having been identified are highlighted
- **Porthtowan Formation** - from the western end of the route to approximate chainage 7+900m
- **Grampound Formation** - from approximate chainage 7+900m to approximate chainage 13+600m
- **Trendrean Mudstone Formation** - from approximate chainage 13+600m to the eastern end of the route

Each of the formations comprises interbedded sedimentary rocks (with varying degrees of metamorphosis) and has been observed in various states of weathering (from fresh rock to residual soil). Design groundwater levels have been developed at a number of locations where groundwater monitoring has been undertaken and a geotechnical long section showing the identified thicknesses of the strata encountered along the route is presented.

Section 6 details the results of geotechnical testing. A series of figures showing test results and derived values are presented and a set of site-wide characteristic values are derived for the materials' geotechnical parameters (including strength and stiffness).

Section 7 details the results of chemical testing and assesses the possibility of contamination posing a risk to human health. Contamination in relation to groundwater is also assessed.

Section 8 presents a summary of the geotechnical risks currently present on the project and sets out suggested mitigation measures to minimise their likelihood and/or severity.

1 Introduction

1.1 Scope and objective of the report

1.1.1 This report forms an addendum to the Ground Investigation Report (GIR) [1] to account for additional ground investigation (GI) undertaken since its issue (latest revision P02) in September 2017. The GIR presents the findings of the "Phase 1" GI undertaken by Structural Soils Limited (SSL) between 23 January 2017 and 31 March 2017. This addendum is required as a result of the "Phase 2" GI undertaken by SSL between 2 October 2017 and 27 October 2017.

1.1.2 This report should be read in conjunction with the GIR [1] and includes only changes and additions to it. Text, figures, and tables that have not been affected by the Phase 2 GI are not reproduced. The aims of this report are as given in the GIR, namely to:

- Describe the ground investigation undertaken.
- Present a representative ground model for the length of the proposed scheme.
- Provide geotechnical parameters for design.
- Develop a geotechnical risk register for the proposed works.

1.2 Description of project

1.2.1 The proposed route has been developed since the GIR [1] was produced. This addendum includes changes or additions required as a result of the development of the proposed scheme. The current route proposal is shown on the appended drawings.

1.2.2 Current proposals for the scheme include the following structures:

- 3No overbridges (including 1No "green bridge")
- 7No underbridges
- 2No walking, cycling, and horse riding (WCH) underpasses
- 10No small span structures
- 1No existing bridge requiring assessment
- 2No retaining walls

1.2.3 The GI within two sections of the route was in abeyance at the time of production of the GIR [1]: the section from Ch.7+100m to Ch.8+500m (due to local resident considerations) and the section from Ch.12+700m to Ch.12+900m (due to access issues and ecological considerations). Furthermore, no boreholes were undertaken as part of the Phase 1 GI around the proposed Chiverton Cross underbridges due to access issues.

1.2.4 All of the sections not covered by the Phase 1 GI were included in the scope for the Phase 2 GI. However, a lack of progress regarding access has meant that the section from Ch.12+700m to Ch.12+900m has still not been covered by GI.

1.3 Geotechnical Category of project

1.3.1 No changes or additions.

1.4 Other relevant information

1.4.1 No changes or additions.

2 Existing information

2.1 Topographical maps

2.1.1 No changes or additions.

2.2 Geological maps and memoirs

2.2.1 No changes or additions.

2.3 Aerial photographs

2.3.1 A review of aerial photography has been undertaken and is reported in the PEIR [3]. This report makes the following conclusions regarding the geology of the area:

- The faults interpreted in the British Geological Society 1:50,000 maps are not strongly expressed in the aerial imagery. Short lengths of ravines and small valleys appear to be, however, controlled by minor unmapped faults.
- A number of hydrological and hydrogeological features, including springs, seepages, and poorly drained ground, are evident.

2.3.2 For further details, refer to the Air Photo Interpretation Report [4], which is included within Appendix G.

2.4 Records of mines and mineral deposits

2.4.1 A review of mines and mineral extraction within the area was carried out to supplement the information contained within the GIR [1], which refers to a mining search carried out by Cornwall Consultants in 2002 and a DEFRA non-coal mining database search carried out by Peter Brett Associates, both of which are included as appendices to the Hyder Consulting Ltd PSSR [5]. Additional activities included reviewing and interpreting aerial photographs and appointing Cornwall Consultants Ltd to undertake an updated mining search and risk assessment. The recent Cornwall Consultants Ltd report contains further information on lode outcrops, shafts and quarries, in particular at Chiverton Cross, Nanteague Farm and Wheal Ennis (or otherwise referred to as Journeys End). The findings are summarised below, but for further information, refer to the mining search [6], which is included within Appendix H.

2.4.2 The 1:100,000 scale Mineral Resources map of Cornwall indicates the scheme alignment to be underlain by a sandstone resource (of interbedded sandstone and shale/slate). This resource would have been mined locally from small quarry pits.

2.4.3 Historically metalliferous minerals have also been extracted across the South West of England, throughout which shallow prospecting was widespread. Costean (trial) pits were dug to in order to discover the mineral lodes, then often mined by openworks (linear excavations) along the outcrop and later by means of shafts, adits and levels driven away from the shafts. The ore was extracted from between the levels to leave narrow chasms. Steam pumping engines introduced in the 19th century enabled deeper workings. Industrial decline by the end of the century led to the closure of most mines, often left abandoned without being secured due to a lack of funding and regulations. Most old and shallow mine workings are poorly recorded due to an historical lack of legal requirements.

2.4.4 Cornwall Consultants Ltd were appointed by Arup in August 2017 to conduct a mining search and risk assessment, the results of which are reported in [6]. The aim of this study was to assess the likely existence of, location of, and risk posed by recorded and unrecorded mines within 500m of the proposed alignment.

2.4.5 All mining features described within this section, including recorded and suspected shafts, adits, mines, quarries, and lode outcrops are presented on the Geotechnical Features Plan within Appendix D. Also presented on these figures are the mining risk zones as defined by the Cornwall Consultant Ltd mining search.

2.4.6 Six named mine sites and four unnamed trial workings were identified within the search area. A further four trial sites or mines lie on the search area boundary and may have associated unrecorded workings that enter the search area.

2.4.7 Inferred or recorded lode outcrops traverse the roadway at six locations and there is the potential for unrecorded prospective mine workings to exist on these outcrops. Unrecorded workings on lode outcrops are the most widespread adverse features in the region and give rise to the greatest number of problems for land development. Such workings can comprise partially filled and/or voided slope workings that extend from surface to adit level and onto much deeper levels of the mine.

2.4.8 In addition to the outcrop of lodes that traverse the alignment, an elvan (quartz porphyry) dyke traverses the alignment at approximate chainage 14+000m. Elvan has the potential to contain metalliferous ores and therefore unrecorded workings might exist here in addition to the known surface quarries and opencast workings.

2.4.9 There are no recorded or suspected shafts, adits or deep workings beneath the scheme alignment, although it is interpreted that an adit exists beneath the scheme at approximate chainage 0+450m. This assumes the major shafts associated with the Burra Burra Mine were drained by an adit and discharged in the valley to the south-east or connected to the former Prince Coburg Mine to the west.

2.4.10 The Engine Shaft (closest to the road on the eastern side) intercepted the inclined lode at a depth of 18 fathoms (33 metres). This might be the depth of the adit, because it would have been reasonable for the engine shaft to connect to it vertically. The adit would be a near-horizontal tunnel with approximate dimensions 1.0m wide by 1.8m high.

2.4.11 The Cornwall Consultants Ltd report [6] concludes that the interpreted land instability risk to the scheme arising from past extractive metalliferous mining is low. Moderate risk zones have been assigned to those features that are indirectly related to extractive metalliferous mining activity, whereas a high risk zone has been applied to all features directly related to extractive metalliferous mining activity, irrespective of their proximity to the roadway.

2.4.12 Non-intrusive geophysical investigations have been undertaken to investigate high risk areas further. A description of the scope of works is provided within Section 3.5 and the results of these investigations are discussed in Section 4.7.

2.5 Land use and soil survey information

2.5.1 No changes or additions.

2.6 Archaeological and historical investigations

2.6.1 No changes or additions.

2.7 Existing ground investigations

2.7.1 No changes or additions.

2.8 Consultation with Statutory Bodies and Agencies

2.8.1 No changes or additions.

2.9 Flood Records

2.9.1 No changes or additions.

2.10 Contaminated land

2.10.1 No changes or additions.

2.11 Other relevant information

Hydrogeology

2.11.1 Environmental Agency hydrogeological mapping provides information on annual average rainfall, groundwater flows in aquifers, surface water, and groundwater features in England. The entire site is classed as a 'Secondary A' aquifer for bedrock geology. These aquifers consist of permeable layers that store water at a local rather than strategic scale, in some cases forming an important base flow to rivers.

2.11.2 The location of aquifers in superficial geology generally corresponds to the position of Head and Alluvial deposits. Superficial deposit aquifers in this area are all either 'Secondary A' or 'Secondary undifferentiated' aquifers. This indicates that they comprise permeable layers capable of supporting water supplies at local rather than strategic scale, and in some cases form an important source of base flow to rivers.

2.11.3 The Environmental Agency Groundwater Vulnerability Map identifies the vulnerability of groundwater to contamination in England and Wales. It is based on the soil leaching class, drainage properties, Superficial Deposits properties, and groundwater flow regime in the area. It indicates the risk posed to groundwater from surface activities by categorising ground conditions into six vulnerability classes. These maps indicate that the majority of the scheme lies within minor aquifer low and minor aquifer intermediate Groundwater Vulnerability Zones. A high minor aquifer groundwater vulnerability zone overlaps the scheme approximately 500m south-west of Two Barrows Junction.

2.11.4 Two 'Zone 1' groundwater source protection zones directly underlie the scheme alignment. Zones 1 and 2 are defined as the 50 and 400-day travel time from any point below the water table to the source respectively. During a site walkover a spring/borehole chamber was discovered within the vicinity of these source protection zones and it is anticipated that this chamber represents the groundwater abstraction.

2.11.5 It is known that a large number of springs are exploited for both domestic and agricultural uses. Records of Private Water Supplies (registered with the Local Authority) are presented within the Geotechnical Features Plan in Appendix D. This shows that numerous active Private Water Supplies exist within 250m either side of the proposed scheme. Information is available on the volume and type of water supply (i.e. spring fed/borehole fed). At the time of writing it is understood that the scheme will directly affect at least one Private Water Supply, including the potable supply to the south of the alignment at approximate chainage 11+000m.

2.11.6 Records of Environment Agency Abstraction Licences have not been presented due to the sensitivity of such data and restrictions on commercial use of the data. Nonetheless several abstractions are located within 250m either side of the proposed scheme.

2.11.7 The Public Consultation has identified two additional unregistered supplies that will require resupply. This includes an alternative supply of livestock water that is currently taken from a pond to the north of the scheme at Ch. 10+450m and from a quarry at Ch. 12+700m. For replacement water supplies additional consultation with the Local Authority and Environment Agency is required to understand water quality.

2.11.8 The scheme alignment crosses the headwater streams of several watercourses. These streams are typically fed by springs emerging within close proximity to the scheme alignment.

Groundwater Flooding

2.11.9 BGS data contained with the Groundsure report [7] indicate the scheme alignment to traverse areas having a moderate to high susceptibility to groundwater flooding within Superficial Deposits. High potential areas are defined as having the potential for groundwater flooding at the surface and moderate potential areas have the potential for groundwater flooding to affect structures below ground level.

2.11.10 The flood risk assessment, which considers groundwater flooding, is included in the Road Drainage and Water Environment Chapter of the PEIR [3]. Groundwater flooding areas within the 250m scoping area are summarised within Table 2-1.

Table 2-1 Summary of groundwater flooding areas within the scheme study area

Groundwater flooding susceptibility	Chainage extents (m)	Approx. distance from scheme alignment (m)	Association
Moderate to high[1]	5+990 to 6+100	Crosses scheme between Ch.5+990 to Ch.6+080. Extends south-east	Seepage[2]

Very High[1]	6+000 to 6+210	30m to 145m south-east	Tertiary River[1], Seepage[2]
Very High[1]	8+900 to 9+500	80m to >300m south-east	Tertiary River, Seepage
Very High[1]	10+940 to 11+150	Crosses scheme at Ch.10+960 to Ch.11+060. Extends 80m north and >250m south	Tertiary River[1], Seepage[2], 2No. Pond[3]
Source: [1] Groundsure Report [7] [2] Air Photo Interpretation Consultancy [4] [3] PSSR [2]			

Hydrology

2.11.11 The proposed scheme generally traverses a boundary between two watersheds. Several springs emerge along the flanks of this watershed boundary, flowing to the north and south. The River Gannel and its tributaries flow to the north, and Rivers Kenwyn, Tresillian and Allen and tributaries flow to the south. All surface water features, including streams, springs, seepages and poorly drained ground are presented in Appendix D. A summary of the hydrological features within the study area are presented in Table 2-2.

Table 2-2 Summary of watercourse features

Watercourse Feature	Chainage (m)	Approx. distance from scheme alignment (m)	Comments
Headwater stream[1]	0+180	30m east	Both merge 250m east of the scheme before eventually joining the Truro River to the east.
Headwater stream[1]	1+210	150m north-west	Flows north.
Headwater stream[1]	1+500	150m east	Flows east before meeting a pond 700m east of the scheme.
Headwater stream[1]	3+700	130m south	Flows south eventually merging with the River Kenwyn.
Headwater stream[1]	6+060	80m south-east	Flows south-east as tertiary river, then secondary river 180m from scheme. Merges with a pond 220m south-east of the scheme. Eventually merging with the River Allen.
Headwater stream[1]	7+210	150m south-east	Flows east, eventually merging with the River Allen.
Headwater stream[1]	8+850	135m north-west	Both merge together at Ch.8+900 45m north-west of the scheme before crossing beneath at Ch.8+910. River flows east before joining a river network eventually merging with the River Allen.
Headwater stream[1]	8+910	80m north-west	
Headwater stream[1]	9+250	Beneath scheme	Flows south-east crossing under the scheme at Ch.9+250. River flows south-east before joining a river network eventually merging with the River Allen.

Watercourse Feature	Chainage (m)	Approx. distance from scheme alignment (m)	Comments
Headwater stream[1]	11+030	150m north-west	Flows south-east, crossing scheme at Ch.11+040. Continues to flow south-east.
Headwater stream[1]	13+680	55m north-west	Flows north-east
Source: [1] Groundsure Report			

3 Field and laboratory studies

3.1 Walkover survey

3.1.1 No changes or additions.

3.2 Geomorphological/geological mapping

3.2.1 No changes or additions.

3.3 Ground Investigations

3.3.1 The results of the Phase 1 GI were issued by SSL as a factual report, which is appended to the GIR [1], and AGS data on 24 July 2017. The results of the Phase 2 GI were issued by SSL as a factual report, which is included as Appendix I, and AGS data on 7 February 2018.

Description of Fieldwork

3.3.2 The Phase 2 GI is a development of the Phase 1 GI covering areas missed by the Phase 1 GI and targeting revised locations of structures and ponds as defined by the current alignment proposal. The aims of the GI and the standards used by the GI are unchanged from those reported in the GIR [1].

3.3.3 A number of specified investigatory holes (11No boreholes and 4No trial pits) were cancelled due to on-site issues such as access and landowner refusal. Table 3-3 details the number of investigatory holes that were undertaken. The locations of these are shown on the drawings in Appendix D and Appendix E.

Table 3-3 Numbers of exploratory holes

Exploratory holes	Phase 1	Phase 2	Total
Borehole (cable percussive with rotary core follow-on)	73	23	96
Trial pit (machine excavated)	103	49	152

In situ tests

3.3.4 Standard penetration tests (SPTs) were undertaken in all boreholes to assess the mechanical properties of the geological materials encountered. A number of other in situ tests were undertaken as outlined in Table 3-4.

Table 3-4 Summary of in situ tests

In situ test	Number of tests (repeats)		
	Phase 1	Phase 2	Total
SPTs	187	86	273
Plate load tests (PLTs)	23 x2 1 x1	18 x2	41 x2 1 x1
Hand shear vane tests (HSVs)	11 x3 2 x2 1 x1	3 x3 1 x2 2 x1	14 x3 3 x2 3 x1
Rising head tests	2	0	2
Soakaway tests	5 x3 3 x1	7 x3 2 x2 3 x1	12 x3 2 x2 6 x1

3.3.5 Plate loading tests were undertaken to inform the highway pavement design. These tests have been interpreted by SSL to determine derived values of CBR. A pair of tests was generally undertaken at each location to determine an average.

3.3.6 Hand shear vane tests were undertaken in a number of trial pits, generally in sets of three.

3.3.7 Soakaway infiltration tests were undertaken to inform the design of attenuation ponds for drainage. These tests have been interpreted by SSL to determine derived values of infiltration rate. A set of three tests was generally undertaken at each location to determine an average. The results of these tests inform drainage design at specific locations only.

3.3.8 Groundwater monitoring has been undertaken using standpipe piezometers that were installed in the boreholes listed in Table 3-5. Monitoring was typically targeted at cuttings and structures. Response zones were decided by the engineer on site.

3.3.9 Monitoring is ongoing at the time of writing and utilises data logging divers (as well as dip meter readings taken whenever data is downloaded from the divers). The results of this monitoring are presented in 0.

Table 3-5 Summary of groundwater monitoring

	Monitored boreholes		
	Phase 1	Phase 2	Total
Number of installations	13	6	19
Lists of monitored boreholes	BH-R-004 BH-R-010 BH-R-013 BH-R-017 BH-R-027 BH-R-041 BH-S-005 BH-S-012 BH-S-019 BH-S-032	BH-201 BH-207 BH-213 BH-216 BH-303 BH-309	-

	Monitored boreholes		
	Phase 1	Phase 2	Total
	BH-S-036 BH-S-042 BH-S-049		

3.4 Drainage studies

3.4.1 No changes or additions.

3.5 Geophysical surveys

3.5.1 Geophysical surveys were undertaken as part of the Phase 2 additional GI carried out by TerraDat (sub-contractor to SOCOTEC) in May 2018. The factual report [8] is provided within Appendix J. These investigations were carried out at 8No high risk areas across the alignment of the scheme, as defined by the Cornwall Consultants mining search [6], with the aim of providing information on mining related features. The investigation areas and their locations are summarised by Table 3-6.

Table 3-6 Summary of geophysical investigation areas

Geophysics investigation area	Grid reference (centre point of investigation area)		Aims of geophysics survey	Area (hectares)
	Easting (m)	Northing (m)		
Chiverton Cross Junction	174954	047369	Features associated with potential prospective working of the surface outcrop of the ‘unnamed lode’ Location of a shaft associated with the Silver Valley Mine	1.4
Chiverton Cross North	175380	047830	Features associated with ‘unnamed mine’	0.5
Callestick Vean	177506	048771	Features associated with potential prospective working of the surface outcrop of two Perran Virgin lodes	2.4
Nanteague Farm	179463	049726	Features associated with potential prospective working of the surface outcrop of the Great South Chiverton lode Location of two possible shafts associated with the Great South Chiverton mine	0.7
Twobarrows Junction	180697	050942	Features associated with potential prospective working of possible mineralisation along north-south trending fault	0.7
Boxheater Junction South	182628	052718	Location of two possible trial shafts	0.6

Geophysics investigation area	Grid reference (centre point of investigation area)		Aims of geophysics survey	Area (hectares)
	Easting (m)	Northing (m)		
Journey's End	183538	053531	Features associated with potential prospective working of the surface outcrop of an 'unnamed lode' associated with Wheal Ennis Location of three shafts associated with Wheal Ennis	1.3
Carland cross	185220	054386	Features associated with surface workings associated with Wheal Mitchell	2.1

3.5.2 The following geophysical techniques were used:

- Magnetics (Geometrics G858);
- Electromagnetics (Geophex GEM-2);
- Electrical Resistivity Tomography (ERT) (IRIS Syscal); and
- Microgravity (Scintrex CG-5).

3.5.3 The exact details of the methodology employed by the geophysics contractor are described in the contractor's report [8], within Appendix J.

3.5.4 The results of the geophysical surveys are discussed in Section 4.7.

3.6 Pile tests

3.6.1 No changes or additions.

3.7 Other field work

3.7.1 No changes or additions.

3.8 Laboratory investigation

3.8.1 The results of two phases of GI have been provided by SSL in the form of factual reports and in AGS format as referred to in the previous section. Samples were taken from all boreholes and trial pits for the purposes of geotechnical and geo-environmental laboratory tests.

Table 3-7 Summary of laboratory testing (geotechnical)

Laboratory test	Number of tests		
	Phase 1	Phase 2	Total
Particle size distribution (PSD) tests	112	54	166
Atterberg limits tests	110	36	146
Moisture content (MC) tests	111	47	158
Linear density tests	10	0	10
Immersion density tests	9	0	9
Particle density tests	45	0	45
Triaxial tests (drained)	0	1	1

Laboratory test	Number of tests		
	Phase 1	Phase 2	Total
Shear box tests	35	8	43
Oedometer tests	0	1	1
Compaction tests (4.5kg hammer)	0	9	9
Compaction tests (2.5kg hammer)	42	2	44
Unconfined compressive strength (UCS) tests	3	4	7
Point load (PL) tests	871	425	1,296
LA abrasion tests	0	6	6
Slake tests	33	23	56
BRE SD1 tests	104	40	144

Table 3-8 Summary of laboratory testing (geo-environmental)

Chemical test	Number of tests		
	Phase 1	Phase 2	Total
Soil Samples			
Heavy Metals	21	48	69
pH, ammoniacal nitrogen, cyanide, phenols, TOC	21	48	69
Asbestos screen	21	48	69
USEPA 16 PAHs	6	48	54
CWG TPH	6	48	54
BTEX compounds	6	48	54
Herbicides and Pesticides Suite	6	0	6
Leachate Analysis			
Heavy Metals	16	0	16
pH, ammoniacal nitrogen	16	0	16
USEPA 16 PAHs	16	0	16
CWG TPH	16	0	16
BTEX compounds	16	0	16
Groundwater Analysis			
Heavy Metals	0	12	12*
pH, ammoniacal nitrogen, cyanide, phenols, DOC, BOD, COD	0	12	12
USEPA 16 PAHs	0	12	12
CWG TPH	0	12	12
BTEX compounds	0	12	12

*One or more further rounds of groundwater testing and analysis for heavy metals is likely to be required.

4 Ground summary

4.1.1 Refer to Appendix E for the geotechnical long section showing the locations and thicknesses of the materials encountered during the GI, which are outlined in this section.

4.2 Topsoil

4.2.1 Topsoil was present at most exploratory hole locations, most frequently with a thickness of circa 0.3m. The maximum thickness encountered was 0.9m.

4.3 Made Ground

4.3.1 Only the localised occurrences of Made Ground detailed in Table 4-9 were identified.

Table 4-9 Occurrences of Made Ground

Approximate chainage	Exploratory hole	Thickness of Made Ground (m)
0+420m	TP-201	0.20
4+070m	TP-219	>1.00*
6+060m	TP-P-009	0.45
7+320m	BH-212	0.15
8+680m	BH-S-032	0.15
12+080m	BH-R-030	0.55
13+280m	TP-R-088A	0.50
13+380m	TP-363B	>3.00*
13+850m	BH-R-040C	0.45
14+040m	BH-R-041	0.90

*These trial pits did not extend deep enough to prove the base of the Made Ground.

4.3.2 Of these occurrences, only TP-219, BH-R-030, TP-363B, and BH-R-041 encountered a thickness of Made Ground greater than circa 0.5m. As discussed in the GIR [1], the instance of Made Ground encountered by BH-R-030 may have been as a result of workings from the possible nearby mine shaft.

4.3.3 There is a remaining area of uncertainty where Made Ground may be present as a result of workings related to the quarry between 12+700m and 12+900m, as indicated by the BGS map of the area. As noted in Section 2, no investigations have been undertaken in this area of the site to date due to access and ecological restraints.

4.4 Superficial Deposits

4.4.1 According to geological maps of the area, Superficial Deposits are largely absent along the proposed route. However, there are locations where the route crosses the upper parts of valleys where Superficial Deposits such as solifluction debris/Head or Alluvium may be anticipated. There is unlikely to be any clear distinction between these possible material types and the term “Superficial Deposits” has been used to cover both possibilities.

4.4.2 A review of the topography and BGS maps of the area indicates that Superficial Deposits may be expected at the approximate chainages shown in Table 4-10.

Table 4-10 Confirmation of Superficial Deposits at possible locations

Approximate chainage at which Superficial Deposits may be anticipated	Exploratory hole(s) that encountered Superficial Deposits close to anticipated location	Thickness of Superficial Deposits (m)
6+000m	BH-S-020 (6+000m) TP-352 (6+040m) TP-P-009 (6+060m)	0.61 0.25 0.40
7+100m	None, which (given the positions of BH-R-017 and BH-211) suggests that if Superficial Deposits are present then they could only be under the southern extent of the proposed earthworks here.	Not proven
8+900m	TP-P-013 (8+870m)	1.40
9+250m	BH-220 (9+260m) TP-R-060 (9+260m)	0.20 >2.00
11+050m	None, however, Superficial Deposits may still be anticipated at the base of the valley immediately to the east of the proposed Penny-Come-Quick side road and underbridge.	Not proven
13+100m	None, however, Superficial Deposits may still be anticipated under the northern slope of the proposed embankment at this location.	Not proven
13+600m	None, however, Superficial Deposits are still expected under the northern slope of the proposed embankment at this location. Shallow slope movements have been observed on site, possibly due to periglaciation and/or water (there is a spring near the top of the slope), hence there are existing stability concerns at this location.	Not proven

4.4.3 Superficial Deposits were logged in four exploratory holes in locations where they were not expected:

- 0.25m in BH-R-101 (1+160m)
- 0.07m in TP-204 (1+170m)
- 0.05m in BH-S-010 (1+950m)
- 0.10m in BH-S-012 (4+830m)

4.4.4 BH-S-020 (Ch.6+000m) and TP-P-013 (Ch.8+870m) are the only instances in which a thickness of Superficial Deposits greater than 0.5m was encountered.

4.4.5 Descriptions in the logs are generally of soft to firm yellowish/orangish brown slightly sandy slightly gravelly CLAY with some instances of dark organic matter.

4.5 Bedrock

4.5.1 The bedrock encountered during the Phase 2 GI is as described in the GIR [1]. The route is underlain by Porthtowan Formation from the western end of the route to approximate chainage 7+900m, by Grampound Formation from approximate chainage 7+900m to approximate chainage 13+600m, and by Trendrian Mudstone Formation from approximate chainage 13+600m to the eastern end of the route. The base of these formations was not proven in any of the boreholes.

4.5.2 Each of these formations comprise interbedded mudstones, siltstones, and sandstones of Devonian age and exhibiting varying degrees of metamorphosis; this geology is known locally as "Killas". The GIR [1] aimed to provide geotechnical parameters for individual rock types within each formation (except for when weathered to soil), however, this report adopts a different approach in which geotechnical parameters are determined for formations. This is deemed a more practical approach that is better suited to the engineering assessments required for design and the level of detail that will be available in terms of ground models. Note, however, that in certain circumstances (consideration of excavatability and reuse in particular) distinction may be required between material types.

4.5.3 Each formation has been encountered in all six of the weathering grades defined by BS EN ISO 14689-1:2003; from fresh rock (Grade 0) to residual soil (Grade 5). The degree of weathering affects how the material behaves and the following groups are proposed as the basis for the ground model presented in this report:

- Grade 0-2. Rock materials for which parameters such as UCS apply.
- Grade 3. Transitional material for which rock or soil parameters may apply.
- Grade 4-5. Soil materials for which parameters such as angle of internal shear resistance and undrained shear strength apply (depending on particle size, material permeability, and loading rate).

4.5.4 The weathering profile was not always encountered "in order" and more weathered materials were encountered below less weathered materials relatively frequently. This is not unexpected and may be due to a number of factors such as preferential groundwater pathways.

4.5.5 Typical thicknesses of weathering grades are presented in Table 4-11. No thicknesses are presented for Grade 0-2 materials as the base of these formations were not proven.

Table 4-11 Summary of strata thicknesses

Strata	Average thickness (m)
Grade 4-5 Porthtowan Formation	1.6
Grade 3 Porthtowan Formation	0.7
Grade 4-5 Grampound Formation	1.3
Grade 3 Grampound Formation	0.5
Grade 4-5 Trendrean Mudstone Formation	1.0
Grade 3 Trendrean Mudstone Formation	0.5

4.5.6 Deeper weathering to Grade 4-5 than encountered elsewhere on the route was apparent in BH-220 and BH-306, which were located in the valley at approximate chainage 9+250m. As stated in Section 5.3, Superficial Deposits were anticipated here, however, only 0.2m was logged. 10.1m of completely weathered rock/residual soil was found in BH-220, which is seemingly an example of “rotten Killas” [9]. There are multiple possible reasons for this deep weathering, the underlying cause likely being that the valley itself is the result of faulting which disturbed the rock.

4.6 Groundwater

4.6.1 The water monitoring data presented in 0 shows locations with rapid responses (short, sharp peaks following rainfall events), seasonal responses (gradual variation over months with a maximum in the winter), and combinations of the two. Design groundwater levels, based on the higher groundwater levels experienced in the winter with consideration given to the extent and persistency of short-term response to rainfall, are illustrated in Appendix E and listed in Table 4-12 below.

Table 4-12 Summary of groundwater levels derived from monitoring

Borehole	Design groundwater level	
	Elevation (mAOD)	Depth (mBGL)
BH-R-004	140.75	2.87
BH-R-010	105.00	2.14
BH-R-013	92.50*	6.17
BH-R-017	78.25	1.10
BH-R-027	117.50	2.90
BH-R-041	131.25	4.67
BH-S-005	137.50	6.15
BH-S-012	111.75	4.99
BH-S-019	79.00	2.78
BH-S-032	74.75*	5.61
BH-S-036	106.75	4.55
BH-S-042	141.00	5.80
BH-S-049	140.50	3.27
BH-201	137.50	2.17
BH-207	130.75	4.47
BH-213	99.00	2.91
BH-216	103.25	3.59
BH-303	81.25	3.04
BH-309	120.50	1.99
*Standpipe piezometers were dry throughout the monitoring period, a conservatively high groundwater level approximately 0.25m below the base of the response zone has been assumed.		

4.6.2 The depths to groundwater vary from 1.10m to over 6m below existing ground level. These design groundwater levels have been used to interpret a design groundwater profile for the entire route, which is shown in Appendix E. This interpretation involves engineering judgement based on topography and the location of nearby watercourses and springs, particularly where there are significant distances between monitored boreholes.

4.7 Mining

4.7.1 As discussed in Section 3.5, the high risk zones defined by the Cornwall Consultants report [6] were targeted for surface geophysical investigations and the findings have been used to inform a reassessment of the level of risk associated with land stability in these areas (see Table 4-13).

4.7.2 The results are presented in the SOCOTEC Factual Report [8] and are also summarised in Table 2-1 (with reference to features presented in Appendix F). Further studies are required to investigate a number of the anomalies, details of which will be discussed within the Geotechnical Design Report.

Table 4-13 Summary of geophysical investigations for mining

Geophysical investigation area	Aim of investigation	Relative level of risk[1] according to Cornwall Consultants [6]	Summary of findings of geophysical surveys[2]	Concluding remarks and residual relative level of risk following geophysics[1]
Chiverton Cross Junction	Investigate features associated with potential prospective working of the surface outcrop of an unnamed load.	Mineral lode directly related to extractive metalliferous mining activity, therefore **high risk** 20m buffer surrounding unnamed lode.	Small scattered magnetic anomalies (F1.2, F1.4 and F1.8) encountered. Sub vertical zone of low resistivity (F3.1 and F3.4), possibly representing the location of the mineral lode.	Evidence of mineral lode traversing the scheme, however no clear evidence of mine entrances or shallow mine workings. Road will be approximately a grade and the mineral lode identified to be 35m south of mapped zone, therefore a **medium risk** has been applied.
Chiverton Cross North	Investigate features associated with possible quarry	Quarry directly related to extractive metalliferous mining, therefore **high risk** 20m buffer surrounding quarry.	Small scattered magnetic anomalies (F1.1). Several low resistivity zones (F3.2, F3.4, F3.5 and F3.6) that are probably unrelated to mining, but possibly related to weathering.	No clear evidence of quarry, therefore the level of risk has been reduced to **low risk**.
Callestick Vean	Investigate features associated with potential prospective working of the surface outcrop of two Perran Virgin lodes	Mineral lodes directly related to extractive metalliferous mining activity, therefore **high risk** 20m buffer surrounding two Perran Virgin lodes.	Increased magnetic response (F1.8 and F1.9) along the alignment of the Perran Virgin lode (west) may indicate shallow worked/disturbed ground. Scattered magnetic dipole features (F1.1 to F1.5) will require ground truthing to confirm feature. Sub-vertical zone of decreased resistivity (F3.1 to F3.6) indicates the location of mineral lodes. Bowl shaped depressions of resistivity within bedrock (F3.7) may indicate worked ground. Features coincide with magnetic dipole features F1.4 and F.5.	Evidence of mineral lodes traversing the scheme and some evidence of worked/disturbed ground. Single feature associated with the Perran Virgin lode (east) close to development therefore a **high risk** has been applied. Perran Virgin lode (west) identified to be a broader zone than previously thought, possibly up to 50m wide. The road is approx. at grade and some features close to the development need further confirmation/ground truthing, therefore a **high risk** has been applied.

Geophysical investigation area	Aim of investigation	Relative level of risk[1] according to Cornwall Consultants [6]	Summary of findings of geophysical surveys[2]	Concluding remarks and residual relative level of risk following geophysics[1]
Nanteague Farm	Investigate features associated with potential prospective working of the surface outcrop of the Great South Chiverton lode and two possible shafts associated with the Great South Chiverton mine	Mineral lodes associated with fault directly related to extractive metalliferous mining activity, therefore **high risk** 20m buffer surrounding the Great South Chiverton lode.	Presence of services and metal gates affects the signal across much of the site. Parallel linear magnetic feature (F1.2) extends across the inferred outcrop of the Great South Chiverton lode. Sub vertical low resistivity zone (F3.2) corresponding to broad, low density feature (F3.1) to the west of the suggested fault zone. Likely caused by fracturing of the rock in the fault zone.	Evidence of mineral lode and fault potentially traversing the scheme along the approximate mapped location. The road is approx. at grade and there is no clear evidence of mine entrances or shallow mine workings, however the quality of the survey was impacted by surface features, therefore a **medium risk** has been applied.
Twoburrows Junction	Investigate features associated with potential prospective working of possible mineralisation along north-south trending fault	Fault indirectly related to extractive metalliferous mining activity, therefore **medium risk** 20m buffer surrounding fault.	Single larger area of increased magnetic response (F1.2) and several scattered magnetic anomalies (F1.3 and F1.4) indicating buried ferrous object. Significant, sub-vertical decreases in resistivity indicating the presence of a fault zone or mineral lode, within the bedrock.	Evidence of fault zone traversing the scheme, possibly orientated north-south as opposed to the mapped north-west to south-east orientation. The road will be on a small embankment, with side roads in cutting. Evidence of a magnetic feature that needs confirmation/ground truthing, therefore **medium risk** has been applied.
Boxheater Junction South	Investigate the location of a possible trial shaft	Shaft directly related to extractive metalliferous mining, therefore **high risk** 20m buffer surrounding shaft.	Numerous magnetic anomalies (F1.1 to F1.3). Microgravity survey indicates an isolated low density feature (F1.2) thought to represent the location of the shaft.	Evidence of shaft location with below ground void. The shaft is in proximity to the side road tie in point and temporary compound, therefore **high risk** remains.
Journey's End	Investigate features associated with	Mineral lodes associated with fault	Large isolated magnetic anomaly (F1.2) and smaller, but still significant isolated magnetic anomaly (F1.3),	Confirmation of northern shaft location and confirmation of the absence of a

Geophysical investigation area	Aim of investigation	Relative level of risk[1] according to Cornwall Consultants [6]	Summary of findings of geophysical surveys[2]	Concluding remarks and residual relative level of risk following geophysics[1]
	potential prospective working of the surface outcrop of an unnamed lode and two shafts associated with Wheal Ennis	directly related to extractive metalliferous mining activity, therefore **high risk** 20m buffer surrounding the unnamed lode and two shafts associated with Wheal Ennis.	both of which could be associated with mine shafts. Various linear magnetic anomalies (F1.4a to F1.4d and F1.5) possibly associated with costean pitting and/or an old tramway serving the shaft. Ground truthing will be required to confirm all features. Sub vertical low resistivity zone (40m wide) (F3.1 to F3.3), representing fractured rock in the fault zone and/or mineral lodes. This zone is located approximately 20m to the west of the mapped location.	shaft to the south. The toe of the proposed embankment is within a few metres of the confirmed shaft location. Possible worked/disturbed ground associated with linear anomalies and fault zone/mineral lode approximately 20m to west, therefore **high risk** remains.
Carland cross	Investigate features associated with surface workings associated with Wheal Mitchell	Elvan indirectly related to extractive metalliferous mining activity, therefore **medium risk** 20m buffer surrounding elvan outcrop.	Large zone of increased magnetic response (F1,1) correlates with expected location of quarry and the presence of a depression observed within the field. Strong magnetic anomalies (F2.2) and lineations (F2.1) are coincident with conjectured backfilled quarry. Broad and subtle decrease in ground conductivity and magnetic anomaly may indicate the presence of disturbed ground or spoil.	Confirmation of the presence of backfilled quarries. The road is on embankment. Some features need further confirmation/ground truthing, however a **medium risk** has been applied.

Notes

1) Low = no evidence of features related to mining. Medium = confirmation of position of mineral lode/fault, which does not extend beyond the limits of the survey area; and/or confirmation of the location of possible mining features greater than 20m away from scheme (that need more detailed studies) - High = confirmation of the location of possible mining features that directly impact the scheme (that need more detailed studies).
2) Refer to figures in Appendix F for feature locations indicated by FX.X.

5 Ground conditions and material properties

5.1 General

5.1.1 This section presents tests results and derived values for a number of key geotechnical properties. Site-wide characteristic values are then determined from these using cautious selection or statistical methods, as defined in BS EN 1997-2:2007. Location-specific values for design of individual structures and earthworks may be determined at later stages of design upon review of the test results and derived values relevant to the specific location.

5.1.2 Where test results and derived values are directly comparable to the results presented in the GIR [1] then conclusions are made in relation to those results. However, if reinterpretation using a different approach has been undertaken then the Phase 1 and Phase 2 results and derived values have been collated and interpreted as one data set.

UCS and PL tests

5.1.3 Table 5-14 presents the results of the 7No UCS tests that were undertaken as part of the Phase 2 GI. The limited number of tests was a result of the lack of suitable intact test specimens recovered due to the frequent close spaced discontinuities in the cores recovered. As a consequence, these results may be more representative of the stronger examples of rock encountered along the route rather than typical.

Table 5-14 Summary of UCS tests

Borehole	Material	Depth (mBGL)	UCS (MPa)
BH-206	Grade 0 Porthtowan Formation	7.25	4.2
BH-211	Grade 3 Porthtowan Formation	9.28	0.063*
BH-217	Grade 0 Grampound Formation	5.72	4.5
BH-307	Grade 0 Grampound Formation	7.52	6.8
BH-S-046	Grade 1 Grampound Formation	6.5	8.6
BH-S-051	Grade 1 Grampound Formation	5.5	12.9
BH-S-051	Grade 1 Grampound Formation	7.6	17.3

*This exceptionally low result suggests that the sample contained a fracture that meant it was essentially a non-intact sample. This result has therefore not been considered.

5.1.4 1,231No PL tests, which can be used to estimate an equivalent UCS, were undertaken. This sample size gives a more representative view of the rock strength, however, the lack of suitable UCS test results mean that derivation of a site-specific correlation is unlikely to be reliable. The UCS results are, however, plotted along with PL test results wherever possible.

5.1.5 The UCS test results presented in Table 5-14 indicate that the rocks encountered along the route can generally be classified as weak according to BS EN USI 14689-1:2003. While a correlation factor between PL and UCS of circa 24 (as adopted by WSP) may be suitable for harder rocks, it is generally proposed that a value of 16 is more suitable for weaker rocks [10]. A factor of 16 has therefore been selected for this site.

5.1.6 All three types of PL test (axial, diametral, and irregular) were undertaken. Review of the PL test results shows that the axial tests imply higher strengths than the irregular tests, which in turn imply higher strengths than the diametral tests. This is as expected since the materials present are generally foliated in a sub-horizontal orientation, hence the diametral tests are able to easily "split" the specimens.

5.1.7 All three types of PL test are valid and have been collated and processed as one data set. This approach avoids the loss of data resulting from choosing one test type and, since in reality a rock is likely to be stressed in multiple directions, may well provide a more realistic measure of available strength.

5.1.8 Note that although UCS should be representative of in situ intact strength it is only one factor affecting rock mass behaviour and the nature of discontinuities within the material is a vital consideration during design.

Soil Particle Sizes

5.1.9 Grade 4-5 materials have been logged as either predominantly clay, silt, sand or gravel. It is generally accepted that if more than 35% of a soil's particles are smaller than the silt/sand boundary then it will behave as a fine-grained (cohesive) material whereas if less than 35% of a soil's particles are smaller than the silt/sand boundary then it will behave as a coarse-grained (granular) material. This distinction is based on whether excess pore water pressures can build up in the short term after stress changes in the soil and has great significance in terms of mechanical behaviour. Note that this distinction between cohesive and granular material is based on the engineering behaviour of the material and is not the same as the classification given in the Specification for Highway Works (SHW).

5.1.10 Soils that have been logged as predominantly clay or silt have been categorised as fine-grained whereas soils that have been logged as predominantly sand or gravel have been categorised as coarse-grained. This categorisation is required for the selection of suitable geotechnical parameters, particularly where correlations are used. The validity of this approach can be assessed by comparing particle size distribution curves against the "35% rule", however, this is not definitive and there is a "grey area"/overlap between the two types of soil.

SPTs

5.1.11 Derived N_{60} values have been calculated from the N values presented by SSL for all SPT results using a simple energy ratio multiplier. All SPT results discussed and presented within this report are therefore derived N_{60} values. No further corrections or normalisations have been applied. The following correlations have been used throughout this section:

- [Coarse-grained strength] Angles of internal shear resistance have been determined using the correlation from N60 value given by Peck, Hanson, and Thornburn [11].
- [Coarse-grained stiffness] Drained Young's moduli have been determined using E' = 1 x N60 (MPa) in line with the GIR [1] based on Stroud [12].
- [Fine-grained strength] Undrained shear strengths have been determined using cu = f1 x N60 (kPa) where f1 = 5 kPa in line with the GIR [1] based on a correlation given by Stroud and Butler [13].

- [Fine-grained stiffness] Undrained Young's moduli can be calculated assuming elastic behaviour from Eu ≈ 1.2 x E' where drained Young's moduli have been determined using E' = 0.9 x N60 (MPa) based on Stroud [12].

Density

5.1.12 26No particle density tests were undertaken on Porthtowan Formation, giving an average result of 2.74Mg/m^3. 14No particle density tests were undertaken on Grampound Formation, giving an average result of 2.74Mg/m^3. 3No particle density tests were undertaken on Trendrean Mudstone Formation, giving an average result of 2.76Mg/m^3. A particle density of 2.74Mg/m^3 is therefore proposed for all bedrock materials.

5.1.13 19No bulk density tests (9No by immersion and 10No by linear measurement) were undertaken on Porthtowan Formation and Grampound Formation materials. These tests resulted in values ranging from 1.83Mg/m^3 to 2.18Mg/m^3 with an average of 2.02Mg/m^3. It is unclear how representative these results are of the in situ material and the higher and lower values did not correspond with particular weathering grades.

Plate loading tests

5.1.14 The results of the PLTs, reported as CBR values to inform highway pavement design, are presented in Appendix A. The CBR percentages are derived values determined by SSL in their factual reports [14] and [15]. The CBRs are slightly higher for coarse-grained soils (averages for formations between 6.0% and 6.5%) than for fine-grained soils (averages for formations between approximately 4.0% and 4.5%).

Soakaway tests

5.1.15 As discussed in the previous section, soakaway tests were undertaken to inform infiltration rates for attenuation pond design. These tests cannot be attributed to single materials (the sides of the trial pits used for the tests often contain multiple strata) hence the results cannot be used to determine geotechnical parameters for materials. The results of these tests, which are derived values determined by SSL in [14] and [15], therefore only inform attenuation pond design at specific locations and are presented in Appendix B.

BRE SD1 testing

5.1.16 A total of 104No soil samples from the Phase 1 investigations and 40No soil and 12No groundwater samples from the Phase 2 investigations were tested to determine the Aggressive Chemical Environment for Concrete (ACEC) and Design Sulphate class (DS) for the site soils in accordance with the guidance given in BRE SD1 [16].

5.1.17 On the basis of the recorded sulphate concentrations of both soil and groundwater samples the following DS and ACEC class is proposed for the entire route:

- DS1 – AC1

5.2 Topsoil

5.2.1 No quantitative geotechnical assessment required.

5.3 Made Ground

5.3.1 Made Ground is inherently variable hence the results of testing in it do not relate to a single material. As such, it is not deemed appropriate to determine site-wide characteristic geotechnical properties. Made Ground is present only at discrete locations, listed in Table 4-9, each with unique origins, and must be assessed on a case by case basis during detailed design.

5.4 Superficial Deposits

5.4.1 4No MC tests were undertaken on Superficial Deposits during Phase 1. The results of these tests range from 13% to 45% with an average of 27% and show no trend with depth.

5.4.2 4No Atterberg limit tests were undertaken on Superficial Deposits during Phase 1. The results of these tests show this material to comprise silt of intermediate to high plasticity.

5.4.3 No SPTs were undertaken in Superficial Deposits.

5.4.4 2No HSV tests (sets of three readings) were undertaken in Superficial Deposits (both in TP-P-009) during Phase 1, which gave average peak shear strengths of 86kPa and 37kPa and average residual shear strengths of 34kPa and 17kPa. The depths of these tests were 0.55m and 0.75m respectively.

5.4.5 1No shear box test (set of three readings) was undertaken on Superficial Deposits. This test was part of Phase 1, however, it has been reinterpreted based on peak stresses at failure (rather than with depth). A conservative line drawn through these results gives a strength of φ' = 33°.

5.5 Porthtowan Formation

Grade 0-2 Porthtowan formation

5.5.1 41No SPTs were undertaken in Grade 0-2 Porthtowan Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation. The resulting N_{60} values are presented in Figure 5-1 and show no trend with depth but are generally greater than 40.

5.5.2 1No UCS test and 614No PL tests were undertaken on Grade 0-2 Porthtowan Formation, the results of which are presented as Figure 5-1 and summarised as Table 5-15. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting UCS values for the whole formation. There is no visible trend with depth and there is a high degree of scatter, which the histogram presented as

5.5.3 Figure **5-3** shows in terms of a distribution.

Table 5-15 Summary of UCS values for Grade 0-2 Porthtowan Formation

	No tests	Range (MPa)	Mean (MPa)	5% (MPa)	95% (MPa)
Grade 0	145	0.16 – 61.76	6.11	0.53	17.60
Grade 1	337	0.00 – 44.00	4.96	0.48	16.51
Grade 2	132	0.00 – 50.08	3.99	0.16	11.34
Grade 0-2	614	0.00 – 61.76	5.02	0.40	17.55

5.5.4 As expected, the derived UCS results for Grade 0 material imply a higher characteristic value while the derived UCS results for Grade 2 material imply a lower characteristic value. It would be overly conservative, however, to take a characteristic UCS for the combined Grade 0-2 material based on only the Grade 2 results, particularly since Grade 2 material accounts for only 23% of the bedrock encountered during the GI. The moderately conservative line drawn through the derived values on Figure 5-1 gives a characteristic UCS of 1.5MPa.

5.5.5 21No pairs of slake durability tests were undertaken on Grade 0-2 Porthtowan Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 71.9% to 100% (excluding one result of 46.7% obtained from BH-303) and giving an average of 93.9%.

5.5.6 1No LA test result was obtained for Grade 0-2 Porthtowan Formation during Phase 2, giving a Los Angeles coefficient of 68.

Grade 3 Porthtowan Formation

5.5.7 24No SPTs were undertaken in Grade 3 Porthtowan Formation, the results of which are plotted as Figure 5-4. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation. The resulting N_{60} values show a trend with depth such that a characteristic line has been proposed with $N_{60} = 5 + 9z$ (z is depth below ground level), however, a generally conservative N_{60} value of 25 may be assumed. Based on the correlations listed in Section 6.1 this relates to a strength of $\varphi' \approx 35°$, as might be expected for gravel.

5.5.8 1No UCS test (omitted as explained in the previous section) and 86No PL tests were undertaken on Grade 3 Porthtowan Formation, the results of which are presented as

5.5.9 Figure 5-5. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting UCS values for the whole formation. As with the Grade 0-2 material these results show no trend with depth and exhibit a high degree of scatter, which the histogram presented as

5.5.10 Figure 5-6 shows in terms of a distribution. The moderately conservative line drawn through these derived values on

5.5.11 Figure 5-5 gives a characteristic UCS of 0.5MPa.

5.5.12 2No compaction tests were undertaken on Grade 3 Porthtowan Formation during Phase 1. These resulted in optimum MCs of 12% and 13%, which are within 3% of the in situ MC of the samples.

5.5.13 8No pairs of Slake durability tests were undertaken on Grade 3 Porthtowan Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 72.8% to 98.5% (excluding one result of 53.5% obtained from TP-R-024) and giving an average of 91.5%.

5.5.14 2No LA test results were obtained for Grade 3 Porthtowan Formation during Phase 2, giving Los Angeles coefficients of 36 and 78. Both samples were described as very weak phyllite recovered as clayey gravel.

Grade 4-5 Porthtowan Formation

5.5.15 Completely weathered rock (Grade 4) and residual soil (Grade 5) are likely to contain small lumps of the intact bedrock (sand, gravel, and cobbles) and a matrix of the constituent materials (clay, silt, and sand). The classification of Grade 4-5 materials as coarse-grained or fine-grained depends on the proportions of the constituent parts, as discussed in the previous section.

Coarse-grained

5.5.16 56No PSD tests were undertaken in coarse-grained Grade 4-5 Porthtowan Formation, the results of which are presented as Figure 5-7. The Phase 2 results are all within the envelope of the Phase 1 results. The classification of these samples as coarse-grained is shown to be correct according to the “35% rule” in the majority of cases.

5.5.17 25No MC tests were undertaken on coarse-grained Grade 4-5 Porthtowan Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-10). The results of these tests show no trend with depth and are generally between 10% and 25% with an average of 12%.

5.5.18 20No Atterberg limit tests were undertaken on the less than 425micron matrix of the coarse-grained Grade 4-5 Porthtowan Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-9). The results of these tests class the matrix material as silt of intermediate plasticity.

5.5.19 79No SPTs were undertaken in coarse-grained Grade 4-5 Porthtowan Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation (Figure 5-11). The resulting N_{60} values show no trend with depth but are generally greater than 15. Based on the correlations listed in Section 6.1 this relates to a strength of $\varphi' \approx 32°$ and a stiffness of $E' \approx 15$MPa.

5.5.20 6No shear box tests were undertaken in coarse-grained Grade 4-5 Porthtowan Formation. These tests were all part of Phase 1, however, they have been reinterpreted based on peak stresses at failure (rather than with depth). The conservative line drawn through these results on Figure 5-12 gives a strength of $\varphi' = 33°$, which agrees well with the SPT results.

5.5.21 18No compaction tests were undertaken on coarse-grained Grade 4-5 Porthtowan Formation. The Phase 1 results and Phase 2 result are comparable hence all results have been combined for consideration. The optimum MCs resulting from these tests range from 11% to 14% with an average of 12.75%.

The initial MCs of the samples used for these tests were typically within 3% of optimum.

5.5.22 4No pairs of slake durability tests were undertaken on coarse-grained Grade 4-5 Porthtowan Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 87.7% to 97.7% and giving an average of 93.6%.

5.5.23 No LA tests were undertaken on coarse-grained Grade 4-5 Porthtowan Formation.

Fine-grained

5.5.24 11No PSD tests were undertaken in fine-grained Grade 4-5 Porthtowan Formation, the results of which are presented as Figure 5-8. The Phase 2 results are all within the envelope of the Phase 1 results. The classification of these samples as fine-grained is shown to be correct according to the “35% rule” in all but one case.

5.5.25 51No MC tests were undertaken in fine-grained Grade 4-5 Porthtowan Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-10). The results of these tests show no trend with depth and are generally between 12% and 25% with an average of 17%.

5.5.26 50No Atterberg limit tests were undertaken in in fine-grained Grade 4-5 Porthtowan Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-9). The results of these tests class this material as clay of low plasticity to silt of intermediate plasticity. The in situ MC of fine-grained Grade 4-5 Porthtowan Formation is on average 8.6% lower than the plastic limit.

5.5.27 18No SPTs were undertaken in fine-grained Grade 4-5 Porthtowan Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation (Figure 5-13). The resulting N_{60} values imply a trend with depth (particularly when considered in conjunction with the results from Grampound Formation) such that N_{60} = 15 for z ≤ 1.5m and N_{60} = 4.5 + 7z *for z > 1.5m*. Based on the correlations listed in Section 6.1 this relates to strength of c_u ≈ 75kPa *for z ≤ 1.5m* and c_u = 22.5 + 35z kPa *for z > 1.5m* (and stiffness of E_u ≈ 16MPa *for z ≤ 1.5m* and E_u = 5 + 7.5z MPa *for z > 1.5m*).

5.5.28 5No HSVs (sets of one, two, and three readings) were undertaken in fine-grained Grade 4-5 Porthtowan Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration. These tests gave average peak shear strengths of 84kPa to 109kPa (overall average of 95kPa) and 2No average residual shear strengths of 52kPa and 53kPa.

5.5.29 17No drained shear box tests were undertaken in fine-grained Grade 4-5 Porthtowan Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting based on peak stresses at failure (rather than with depth). The conservative line drawn through these results on Figure 5-14 gives a strength of φ’ = 30° with c’ = 2kPa.

5.5.30 8No compaction tests were undertaken on fine-grained Grade 4-5 Porthtowan Formation. The single result obtained during Phase 2 exhibited a higher initial MC than any other test but a similar optimum MC. The optimum MCs resulting from both phases of tests range from approximately 9% to 18% with an average of 15%. The initial MCs of the Phase 1 samples used for these tests were typically within 3% of optimum, with the initial MC from Phase 2 being 16% higher.

5.5.31 1No pair of slake durability tests were undertaken in fine-grained Grade 4-5 Porthtowan Formation, giving results of 50.4% and 47.2%.

5.5.32 No LA tests were undertaken on fine-grained Grade 4-5 Porthtowan Formation.

5.6 Grampound Formation

Grade 0-2 Grampound Formation

5.6.1 29No SPTs were undertaken in Grade 0-2 Grampound Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation. The resulting N_{60} values are presented in Figure 5-15 and show no trend with depth but are generally greater than 40.

5.6.2 5No UCS tests and 499No PL tests were undertaken on Grade 0-2 Grampound Formation, the results of which are presented as

5.6.3 Figure **5-16** and summarised as Table 5-16. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting UCS values for the whole formation. There is no visible trend with depth and there is a high degree of scatter, which the histogram presented as

5.6.4 Figure **5-17** shows in terms of a distribution.

Table 5-16 Summary of UCS values for Grade 0-2 Grampound Formation

	No tests	Range (MPa)	Mean (MPa)	5% (MPa)	95% (MPa)
Grade 0	40	0.16 – 74.40	9.92	0.64	46.42
Grade 1	407	0.00 – 54.56	6.43	0.48	20.80
Grade 2	52	0.00 – 40.16	6.72	0.16	29.91
Grade 0-2	499	0.00 – 74.40	6.74	0.40	21.28

5.6.5 As expected, the derived UCS results for Grade 0 material imply a higher characteristic value while the derived UCS results for Grade 2 material imply a lower characteristic value. It would be overly conservative, however, to take a characteristic UCS for the combined Grade 0-2 material based on only the Grade 2 results, particularly since Grade 2 material accounts for only 23% of the bedrock encountered during the GI. The moderately conservative line drawn through the derived values on

5.6.6 Figure **5-16** gives a characteristic UCS of 1.5MPa.

5.6.7 9No pairs of slake durability tests were undertaken on Grade 0-2 Grampound Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 86.1% to 97.7% and giving an average of 94.7%.

5.6.8 No LA tests were undertaken on Grade 0-2 Grampound Formation.

Grade 3 Grampound Formation

5.6.9 12No SPTs were undertaken in Grade 3 Grampound Formation, the results of which are plotted as Figure 5-18. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation. The resulting N_{60} values show a trend with depth such that a design line has been proposed with $N_{60} = 5 + 9z$, however, a generally conservative N_{60} value of 25 may be assumed. Based on the correlations listed in Section 6.1 this relates to a strength of $\varphi' \approx 35°$, as might be expected for gravel.

5.6.10 No UCS tests but 8No PL tests were undertaken on Grade 3 Grampound Formation, the results of which are presented as

5.6.11 Figure **5-19**. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting UCS values for the whole formation. As with the Grade 0-2 material these results show no trend with depth.

5.6.12 There are not enough PL results to reliably interpret a characteristic UCS value for Grade 3 Grampound Formation on its own. It is suggested that since the same characteristic UCS value was found for the Grade 0-2 Grampound Formation as Grade 0-2 Porthtowan Formation that this material may be attributed the same characteristic UCS value as the Grade 3 Porthtowan Formation. A characteristic UCS value of 0.5MPa has therefore been adopted.

5.6.13 8No compaction tests were undertaken on Grade 3 Grampound Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration. The optimum MCs resulting from these tests range from approximately 10% to 14% with an average of 12%. The initial MCs of the samples used for these tests were typically within 3% of optimum.

5.6.14 6No pairs of slake durability tests were undertaken on Grade 3 Grampound Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 84.7% to 96.7% and giving an average of 92.9%.

5.6.15 1No LA test was undertaken on Grade 3 Grampound Formation during Phase 2, giving a LA coefficient of 38. This sample was described as weak psammite recovered as sandy gravel.

Grade 4-5 Grampound Formation

5.6.16 Completely weathered rock (Grade 4) and residual soil (Grade 5) are likely to contain small lumps of the intact bedrock (sand, gravel, and cobbles) and a matrix of the constituent materials (clay, silt, and sand). The classification of Grade 4-5 materials as coarse-grained or fine-grained depends on the proportions of the constituent parts, as discussed in the previous section.

Coarse-grained

5.6.17 40No PSD tests were undertaken in coarse-grained Grade 4-5 Grampound Formation, the results of which are presented as Figure 5-20. The results from Phase 2 generally lie within the envelope of the results from Phase 1. The classification of these samples as coarse-grained is shown to be correct according to the "35% rule" in all but two cases.

5.6.18 7No MC tests were undertaken in coarse-grained Grade 4-5 Grampound Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-23). The results of these tests show no trend with depth and are generally between 12% and 22% with an average of 16%.

5.6.19 5No Atterberg limit tests were undertaken on the less than 425micron matrix of the Grade 4-5 Grampound Formation. The Phase 2 result is comparable with the Phase 1 results hence all results have been combined for consideration (Figure 5-22). The results of these tests class the matrix material as silt of intermediate plasticity.

5.6.20 52No SPTs were undertaken in coarse-grained Grade 4-5 Grampound Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation (Figure 5-24). The resulting N_{60} values show no trend with depth but are generally greater than 15. Based on the correlations listed in Section 6.1 this relates to a strength of $\varphi' \approx 32°$ and a stiffness of $E' \approx 15MPa$.

5.6.21 2No shear box tests were undertaken on coarse-grained Grade 4-5 Grampound Formation. All results have been combined while reinterpreting based on peak stresses at failure (rather than with depth). The conservative line drawn through these results on Figure 5-25 gives a strength of $\varphi' = 33°$, which agrees well with the SPT results.

5.6.22 5No compaction tests were undertaken on coarse-grained Grade 4-5 Grampound Formation. The Phase 1 result and Phase 2 results are comparable hence all results have been combined for consideration. The optimum MCs resulting from these tests range from 10% to 16% with an average of 13.6%. The initial MCs of the samples used for these tests were typically within 4% of optimum.

5.6.23 4No pairs of slake durability tests were undertaken on coarse-grained Grade 4-5 Grampound Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 82.1% to 97.8% (excluding one result of 67.8% obtained from TP-S-008) and giving an average of 94.0%.

5.6.24 1No LA test was undertaken on coarse-grained Grade 4-5 Grampound Formation during Phase 2, giving a Los Angeles coefficient of 35. This sample was described as slightly silty clayey gravel (gravel consisting of phyllite).

Fine-grained

5.6.25 12No PSD tests were undertaken in fine-grained Grade 4-5 Grampound Formation, the results of which are presented as Figure 5-21. The classification of these samples as fine-grained is correct according to the “35% rule” in all but one case.

5.6.26 39No MC tests were undertaken in fine-grained Grade 4-5 Grampound Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-23). The results of these tests show no trend with depth and are generally between 10% and 30% with an average of 20%.

5.6.27 39No Atterberg limit tests were undertaken in fine-grained Grade 4-5 Grampound Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration (Figure 5-22). The results of these tests class this material as clay of low plasticity to silt of high plasticity. The in situ MC of fine-grained Grade 4-5 Porthtowan Formation is on average 8.5% lower than the plastic limit.

5.6.28 11No SPTs were undertaken in fine-grained Grade 4-5 Grampound Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting N_{60} values for the whole formation (Figure 5-26). The resulting N_{60} values imply a trend with depth (particularly when considered in conjunction with the results from Porthtowan Formation) such that N_{60} = 15 for z ≤ 1.5m and N_{60} = 4.5 + 7z *for z > 1.5m*. Based on the correlations listed in Section 6.1 this relates to strength of c_u ≈ 75kPa *for z ≤ 1.5m* and c_u = 22.5 + 35z kPa *for z > 1.5m* (and stiffness of E_u ≈ 16MPa *for z ≤ 1.5m* and E_u = 5 + 7.5z MPa *for z > 1.5m*).

5.6.29 8No HSVs (sets of one, two, and three readings) were undertaken in fine-grained Grade 4-5 Grampound Formation during Phase 1. These tests gave average peak shear strengths of 76kPa to 177kPa (overall average of 119kPa) and 4No average residual shear strengths of 43kPa to 77Pa (overall average of 55kPa).

5.6.30 14No drained shear box tests were undertaken in fine-grained Grade 4-5 Grampound Formation. There is no difference evident between Phase 1 and Phase 2 results hence all results have been combined while reinterpreting based on peak stresses at failure (rather than with depth). The conservative line drawn through these results on Figure 5-27 gives a strength of φ' = 30° with c' = 2kPa.

5.6.31 3No compaction tests were undertaken on fine-grained Grade 4-5 Grampound Formation. The Phase 1 and Phase 2 results are comparable hence all results have been combined for consideration. The optimum MCs resulting from these tests range from approximately 10% to 18% with an average of 13%. Two of these samples had initial MCs equal to optimum while one had an initial MC 9% higher than optimum.

5.6.32 2No pairs of slake durability tests were undertaken on fine-grained Grade 4-5 Grampound Formation. The results from Phase 1 and Phase 2 are comparable, ranging from 96.7% to 97.9% and giving an average of 97.3%. Both samples were described as soft to firm slightly sandy/silty gravelly clay (gravel consists of phyllite).

5.6.33 1No LA test was undertaken on fine-grained Grade 4-5 Grampound Formation during Phase 2, giving a Los Angeles coefficient of 64.

5.7 Trendrean Mudstone Formation

Grade 0-2 Trendrean Mudstone Formation

5.7.1 1No SPT was undertaken in Grade 1 Trendrean Mudstone Formation during Phase 1, resulting in an extrapolated N_{60} value of 630. As with the other formations, a degree of variability is expected in N_{60} values for this material and it is deemed that the value of 630 is an exceptionally high value that is not representative of this material.

5.7.2 The test results from the Porthtowan Formation and Grampound Formation led to identical characteristic values being taken for many parameters. It is reasonable to assume that all three formations, being of similar age and comprising similar materials, exhibit largely comparable properties. Minimal testing was undertaken in the Trendrean Mudstone Formation, since it only underlies the very end of the proposed route, and it is therefore suggested that where insufficient test results are available to characterise this formation the parameters determined for the other formations will be adopted. Grade 0-2 Trendrean Mudstone Formation is therefore assumed to have N_{60} values greater than 40.

5.7.3 No UCS tests but 24No PL tests were undertaken on this material during Phase 1, the results of which are presented as Figure 5-28. These results have been reinterpreted to determine UCS values for Grade 0-2 of the whole formation. There is no visible trend with depth and there is a high degree of scatter. There are not enough results to reliably determine a characteristic UCS value hence the characteristic UCS value of 1.5MPa determined for both the Grade 0-2 Porthtowan Formation and the Grade 0-2 Grampound Formation is to be adopted.

5.7.4 No slake durability tests or LA tests were undertaken on Grade 0-2 Trendrean Mudstone Formation.

Grade 3 Trendrean Mudstone Formation

5.7.5 1No SPT was undertaken on Grade 3 Trendrean Mudstone Formation during Phase 1, resulting in an N_{60} value of 525. As with the Grade 0-2 Trendrean Mudstone Formation this is deemed unrepresentative and the design line of N_{60} = 5 + 9z (and strength of φ' ≈ 35°) determined for Grade 3 Porthtowan and Grampound Formations shall be adopted.

5.7.6 No UCS or PL tests were undertaken on this material. The same characteristic UCS of 0.5MPa determined for Grade 3 Porthtowan and Grampound Formations shall therefore be adopted.

5.7.7 2No compaction tests were undertaken on Grade 3 Trendrean Mudstone Formation during Phase 1 resulting in optimum MCs of 9.6% and 11% compared to in situ MCs of 11% and 9.9% respectively.

5.7.8 1No pair of slake durability tests were undertaken on Grade 3 Trendrean Mudstone Formation during Phase 1. The results of these tests were 93.9% and 94.8%.

5.7.9 No LA tests were undertaken on Grade 3 Trendrean Mudstone Formation.

Grade 4-5 Trendrean Mudstone Formation

5.7.10 Completely weathered rock (Grade 4) and residual soil (Grade 5) are likely to contain small lumps of the intact bedrock (sand, gravel, and cobbles) and a matrix of the constituent materials (clay, silt, and sand). The classification of Grade 4-5 materials as coarse-grained or fine-grained depends on the proportions of the constituent parts, as discussed in the previous section.

Coarse-grained

5.7.11 6No PSD tests were undertaken in coarse-grained Grade 4-5 Trendrean Mudstone Formation, the results of which are presented as Figure 5-29. The Phase 2 results are comparable to the Phase 1 results and the classification of all of these samples as coarse-grained is correct according to the “35% rule”.

5.7.12 2No MC tests, one in Phase 1 and one in Phase 2, were undertaken in coarse-grained Grade 4-5 Trendrean Mudstone Formation (Figure 5-31). These resulted in MCs of 11% and 19%

5.7.13 2No Atterberg limit tests, one in Phase 1 and one in Phase 2, were undertaken on the less than 425micron matrix of the Grade 4-5 Trendrean Mudstone Formation (Figure 5-30). The results of these tests class the fine-grained proportion of this material as silt of intermediate to high plasticity.

5.7.14 2No SPTs were undertaken in coarse-grained Grade 4-5 Trendrean Mudstone Formation during Phase 1, resulting in N_{60} values of 22 and 315. As with the less weathered Trendrean Mudstone Formation it is deemed unrepresentative to use these values and the N_{60} value of 15 attributed to coarse-grained Grade 4-5 Porthtowan and Grampound Formations will be adopted. Based on the correlations listed in Section 6.1 this relates to a strength of $\varphi' \approx 32°$ and a stiffness of $E' \approx 15$MPa.

5.7.15 No shear box tests were undertaken in coarse-grained Grade 4-5 Trendrean Mudstone Formation.

5.7.16 2No compaction tests were undertaken on coarse-grained Grade 4-5 Trendrean Mudstone Formation, one during Phase 1 and one during Phase 2. The optimum MCs resulting from these tests were 10% and 15%. The higher result came from a sample taken immediately below topsoil that was more clayey. The initial MCs of the samples used for these tests were up to 6% higher than optimum.

5.7.17 No slake durability tests or LA tests were undertaken on coarse-grained Grade 4-5 Trendrean Mudstone Formation.

Fine-grained

5.7.18 No PSD tests were undertaken in fine-grained Grade 4-5 Trendrean Mudstone Formation.

5.7.19 1No MC test was undertaken in fine-grained Grade 4-5 Trendrean Mudstone Formation during Phase 1 (Figure 5-31), resulting in a MC of 21%.

5.7.20 1No Atterberg limit test was undertaken in in fine-grained Grade 4-5 Trendrean Mudstone Formation during Phase 1 (Figure 5-30). The results of this test classes this material as clay of intermediate plasticity to silt of high plasticity.

5.7.21 1No SPT was undertaken in coarse-grained Grade 4-5 Trendrean Mudstone Formation during Phase 1, resulting in an N_{60} value of 14. This agrees with the characteristic line adopted for fine-grained Grade 4-5 Porthtowan and Grampound Formations of $N_{60} = 15$ for $z \leq 1.5$m and $N_{60} = 4.5 + 7z$ *for* $z > 1.5m$. Based on the correlations listed in Section 6.1 this relates to strength of $c_u \approx 75$kPa *for* $z \leq 1.5m$ and $c_u = 22.5 + 35z$ kPa *for* $z > 1.5m$ (and stiffness of $E_u \approx 16$MPa *for* $z \leq 1.5m$ and $E_u = 5 + 7.5z$ MPa *for* $z > 1.5m$).

5.7.22 No HSVs were undertaken in fine-grained Grade 4-5 Trendrean Mudstone Formation.

5.7.23 1No drained shear box test was undertaken in fine-grained Grade 4-5 Trendrean Mudstone Formation during Phase 1. This has been reinterpreted based on peak stresses at failure (rather than with depth). The results of this test plot above the characteristic line for fine-grained Grade 4-5 Porthtowan and Grampound formations (Figure 5-32), however, similar individual results were produced for the other formations so the same characteristic line shall be adopted.

5.7.24 No compaction tests were undertaken on fine-grained Grade 4-5 Trendrean Mudstone Formation.

5.7.25 No slake durability tests or LA tests were undertaken on fine-grained Grade 4-5 Trendrean Mudstone Formation.

5.8 Summary of characteristic parameters from testing

Table 5-17 Classification and reuse parameters

Material		γ* (kN/m³)	In situ MC (%)	Plasticity index (%)	Optimum MC (%) *Nearest 0.5%*	Slake durability (%)	Los Angeles coefficient
Made Ground		-	-	-	-	-	-
Superficial Deposits		-	13 - 45	13 - 27	-	-	-
Porthtowan Formation	Grade 0-2	21	-	-	-	94	68
	Grade 3	20	-	-	12.5	92	36 / 78
	Coarse-grained Grade 4-5	19	10 - 20	6 - 32**	13.0	94	-
	Fine-grained Grade 4-5	19	10 - 25	7 - 29	15.0	49	-
Grampound Formation	Grade 0-2	21	-	-	-	95	-
	Grade 3	20	-	-	12.0	93	38
	Coarse-grained Grade 4-5	19	10 - 20	15 - 25**	13.5	94	35
	Fine-grained Grade 4-5	19	10 - 30	9 - 31	13.0	97	64
Trendrean Mudstone Formation	Grade 0-2	21	-	-	-	-	-
	Grade 3	20	-	-	10.5	94	-
	Coarse-grained Grade 4-5	19	10 - 20	18 - 28**	12.5	-	-
	Fine-grained Grade 4-5	19	21	16	-	-	-

*As discussed in 6.1.4 testing implied bulk densities of circa 20kN/m³ for all weathering grades, however, since these test results are not deemed to be fully representative of in situ conditions these values incorporate guidance from BS 8002:2015.

**Relates to the less than 425micron matrix present around the larger grains.

Table 5-18 Strength and stiffness parameters

Material		SPT (N_{60} value)	UCS (MPa)	S_u (kPa)	φ' (°)	c' (kPa)	E_u (MPa)	E' (MPa)
Made Ground		-	-	-	-	-	-	-
Superficial Deposits		-	-	35	33*	0*	-	-
Porthtowan Formation	Grade 0-2	40	1.5	-	-	-	-	-
	Grade 3	5 + 9z	0.5	-	35	0	-	25
	Coarse-grained Grade 4-5	15	-	-	33	0	-	15
	Fine-grained Grade 4-5	15 for z≤1.5m 4.5+7z for z>1.5m	-	75 for z≤1.5m 22.5+35z for z>1.5m	30	2	16 for z≤1.5m 5+7.5z for z>1.5m	13.5 for z≤1.5m 4+6.5z for z>1.5m
Grampound Formation	Grade 0-2	40	1.5	-	-	-	-	-
	Grade 3	5 + 9z	0.5	-	35	0	-	25
	Coarse-grained Grade 4-5	15	-	-	33	0	-	15
	Fine-grained Grade 4-5	15 for z≤1.5m 4.5+7z for z>1.5m	-	75 for z≤1.5m 22.5+35z for z>1.5m	30	2	16 for z≤1.5m 5+7.5z for z>1.5m	13.5 for z≤1.5m 4+6.5z for z>1.5m
Trendrean Mudstone Formation	Grade 0-2	40	1.5	-	-	-	-	-
	Grade 3	5 + 9z	0.5	-	35	0	-	25
	Coarse-grained Grade 4-5	15	-	-	33	0	-	15
	Fine-grained Grade 4-5	15 for z≤1.5m 4.5+7z for z>1.5m	-	75 for z≤1.5m 22.5+35z for z>1.5m	30	2	16 for z≤1.5m 5+7.5z for z>1.5m	13.5 for z≤1.5m 4+6.5z for z>1.5m

*The one set of shear box tests available indicates a drained strength of Superficial Deposits of approximately φ'=33° and c'=0kPa. However, based on descriptions of the Superficial Deposits it is deemed unlikely that it this material is as strong as the coarse-grained Grade 4-5 materials (particularly if solifluction debris/Head, which is likely to be well-sheared). These parameters should therefore be lowered in line with typical strengths given in the published guidance and/or location-specific back analysis where appropriate during design.

Note that values are identical for the three formations (only weathering grade affects choice of strength and stiffness parameters).

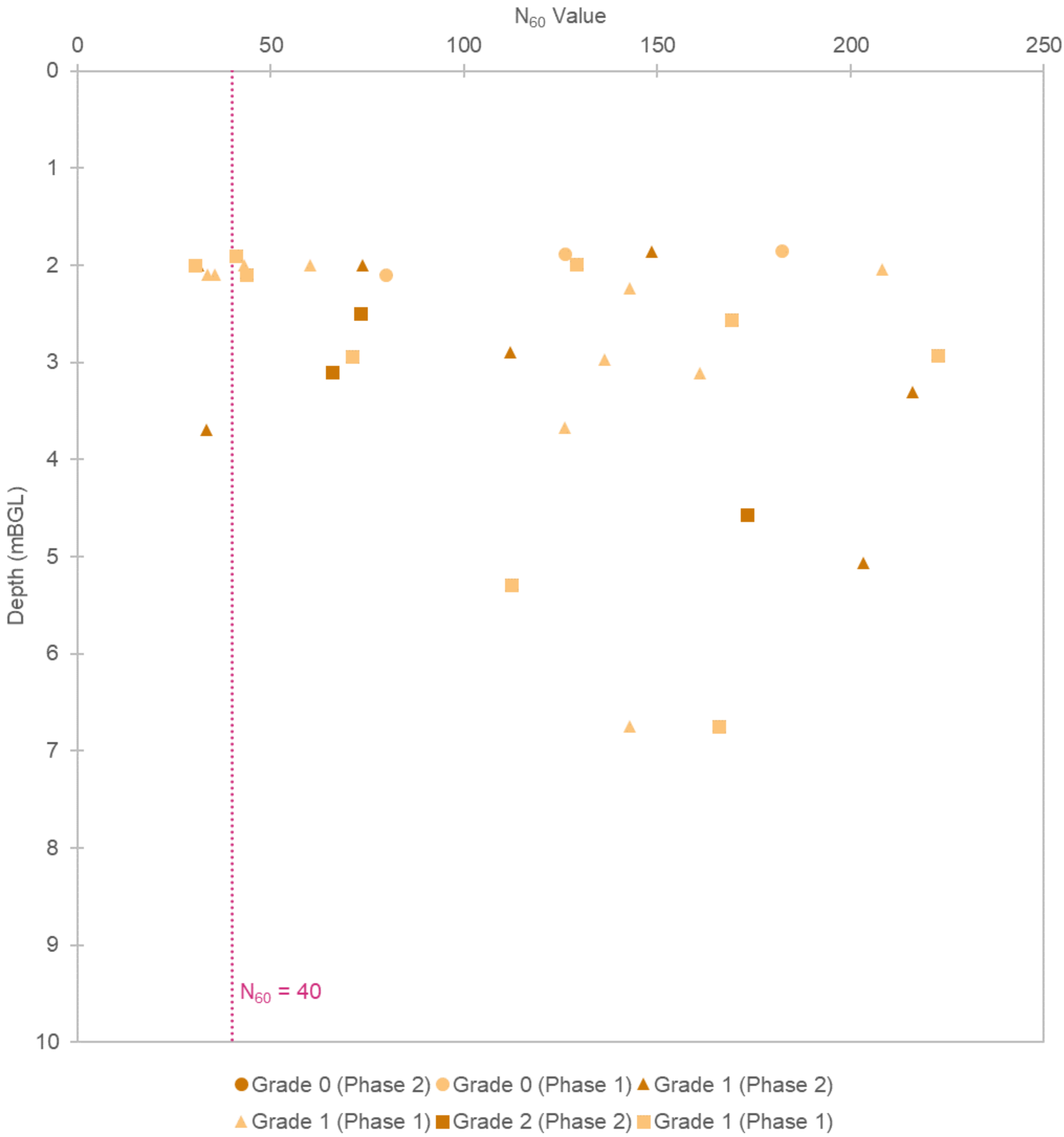

Figure 5-1 SPT results of Porthtowan Formation Grade 0-2 with depth

Figure 5-2 PL and UCS test results of Porthtowan Formation Grade 0-2 with depth

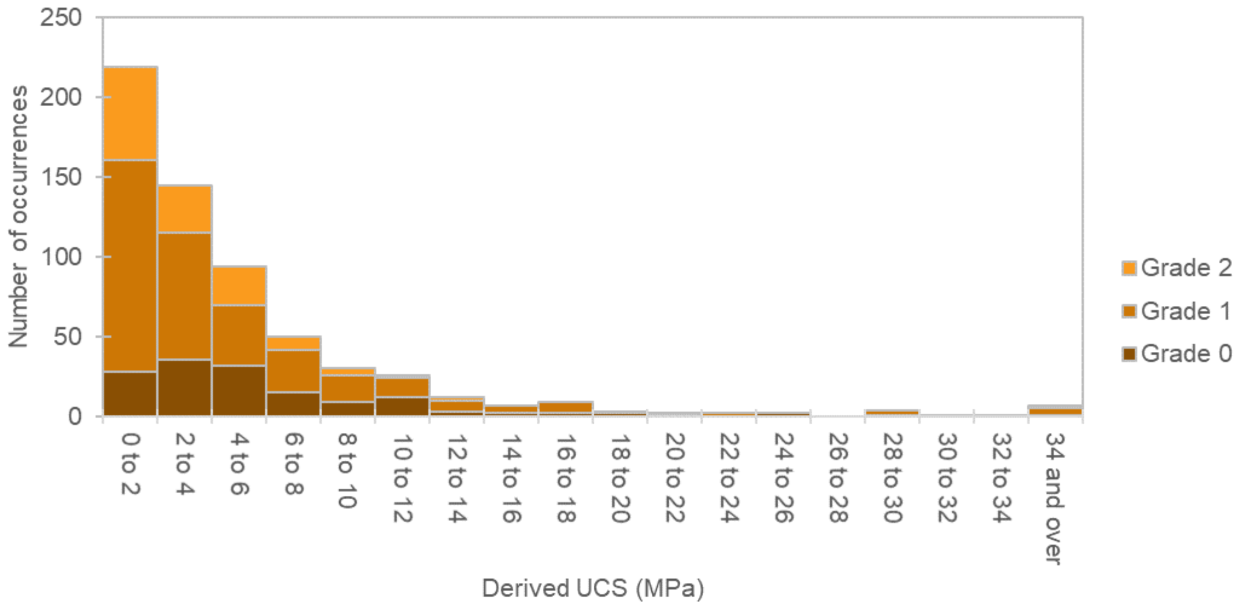

Figure 5-3 Distribution of PL test results of Porthtowan Formation Grade 0-2

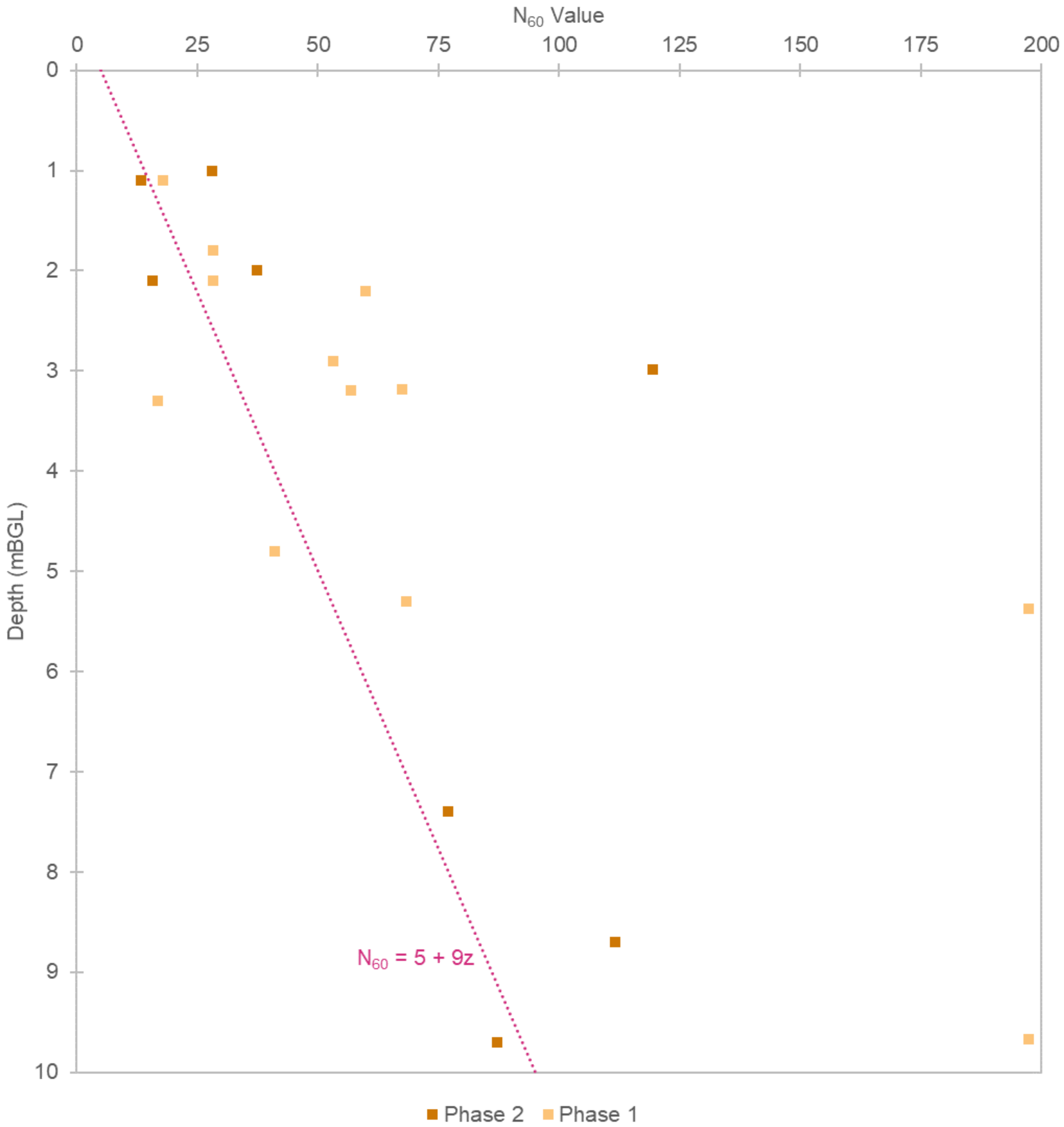

Figure 5-4 SPT results of Porthtowan Formation Grade 3 with depth

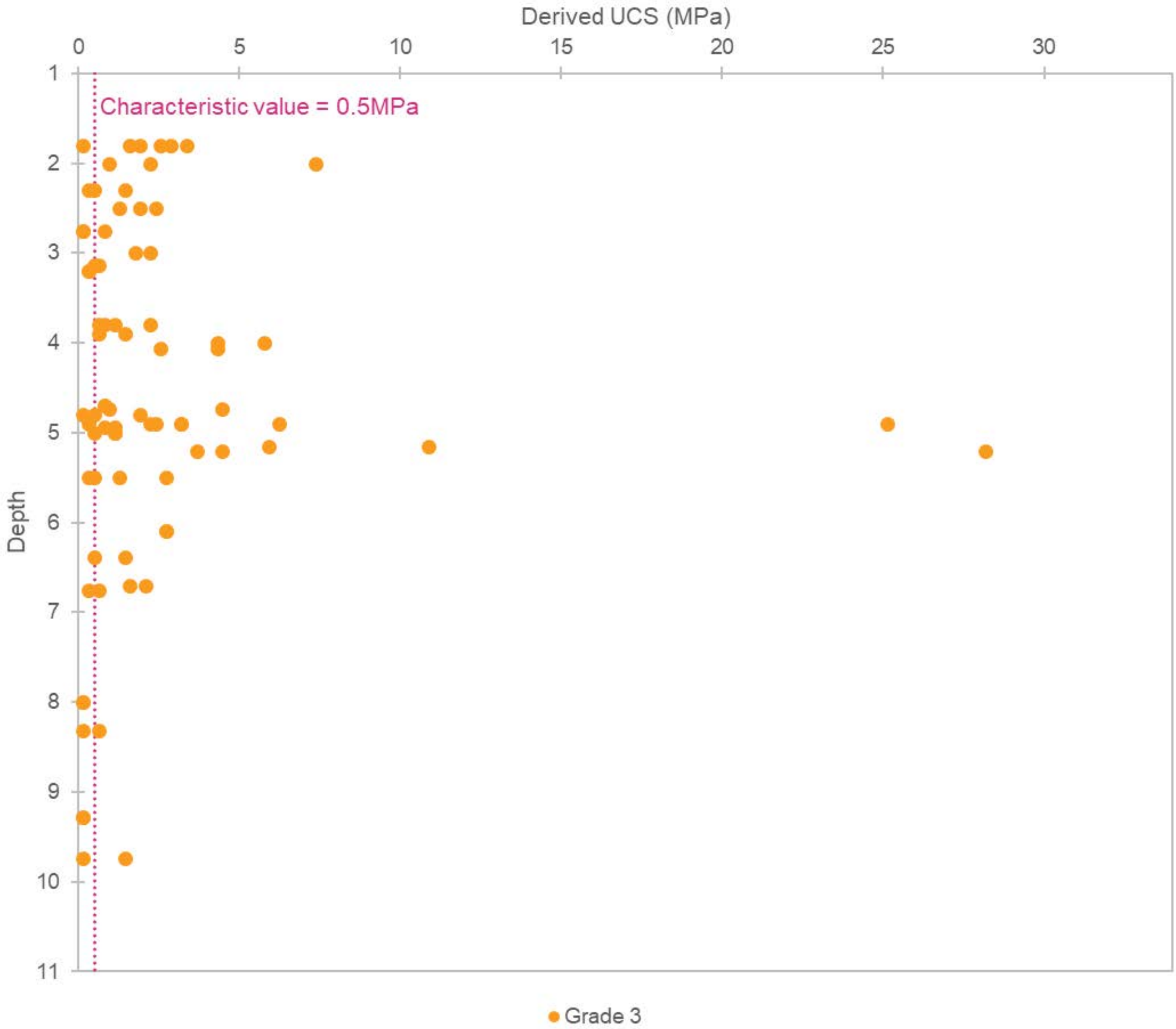

Figure 5-5 PL test results of Porthtowan Formation Grade 3 with depth

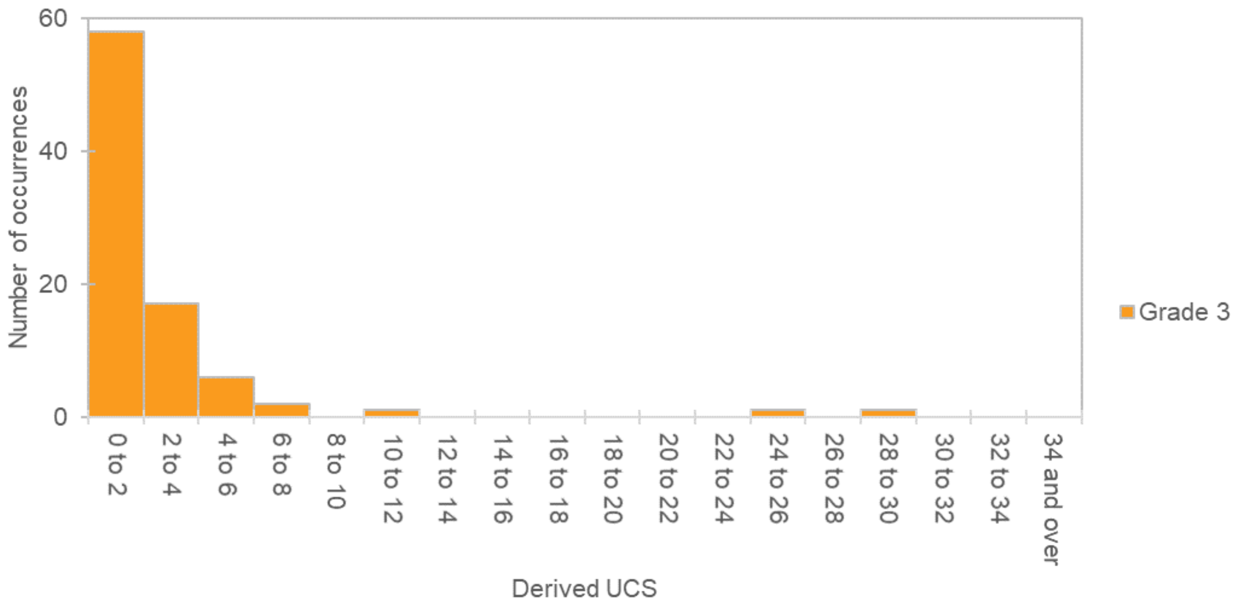

Figure 5-6 Distribution of PL test results of Porthtowan Formation Grade 3

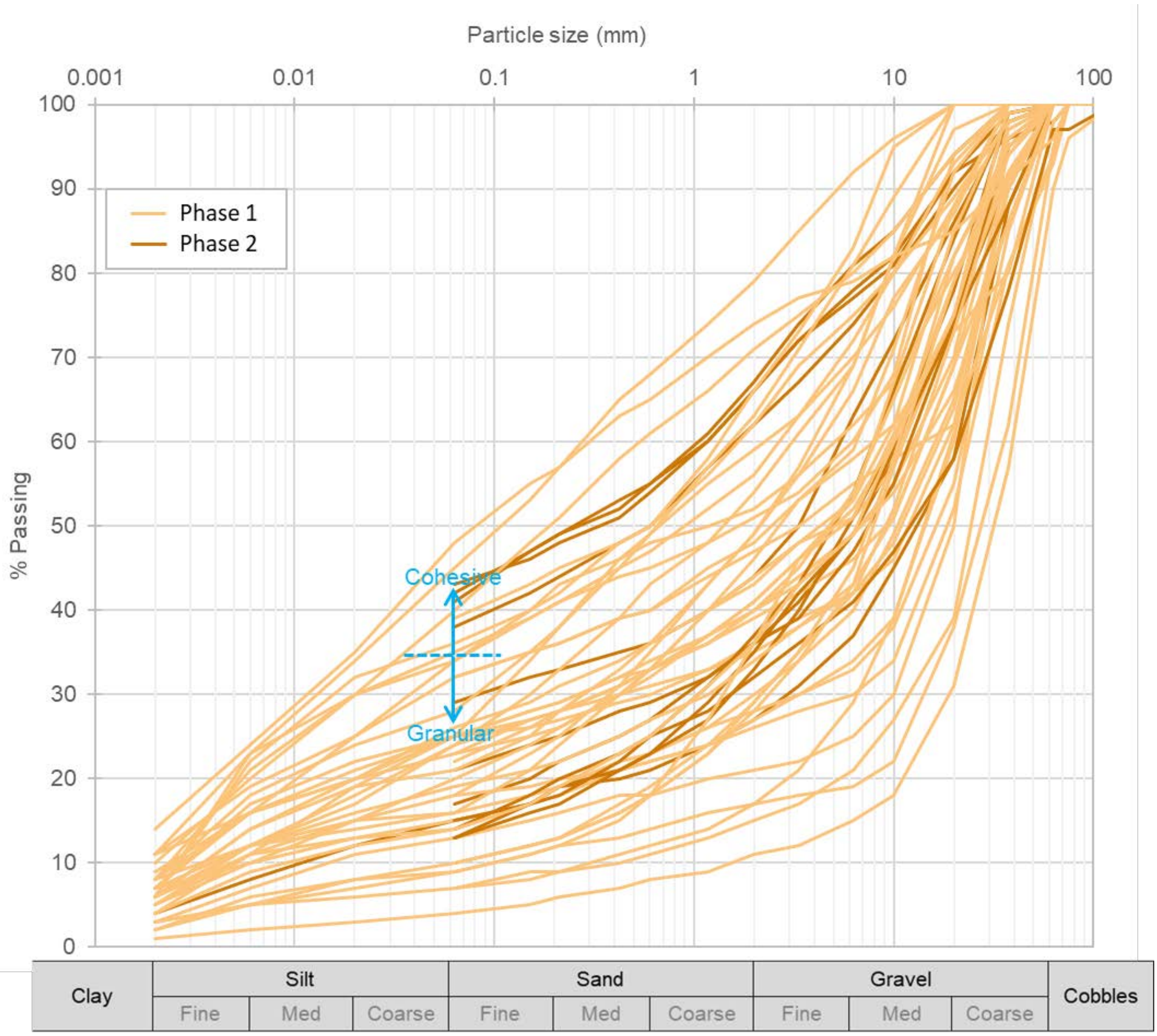

Figure 5-7 PSD results of COARSE Porthtowan Formation Grade 4-5

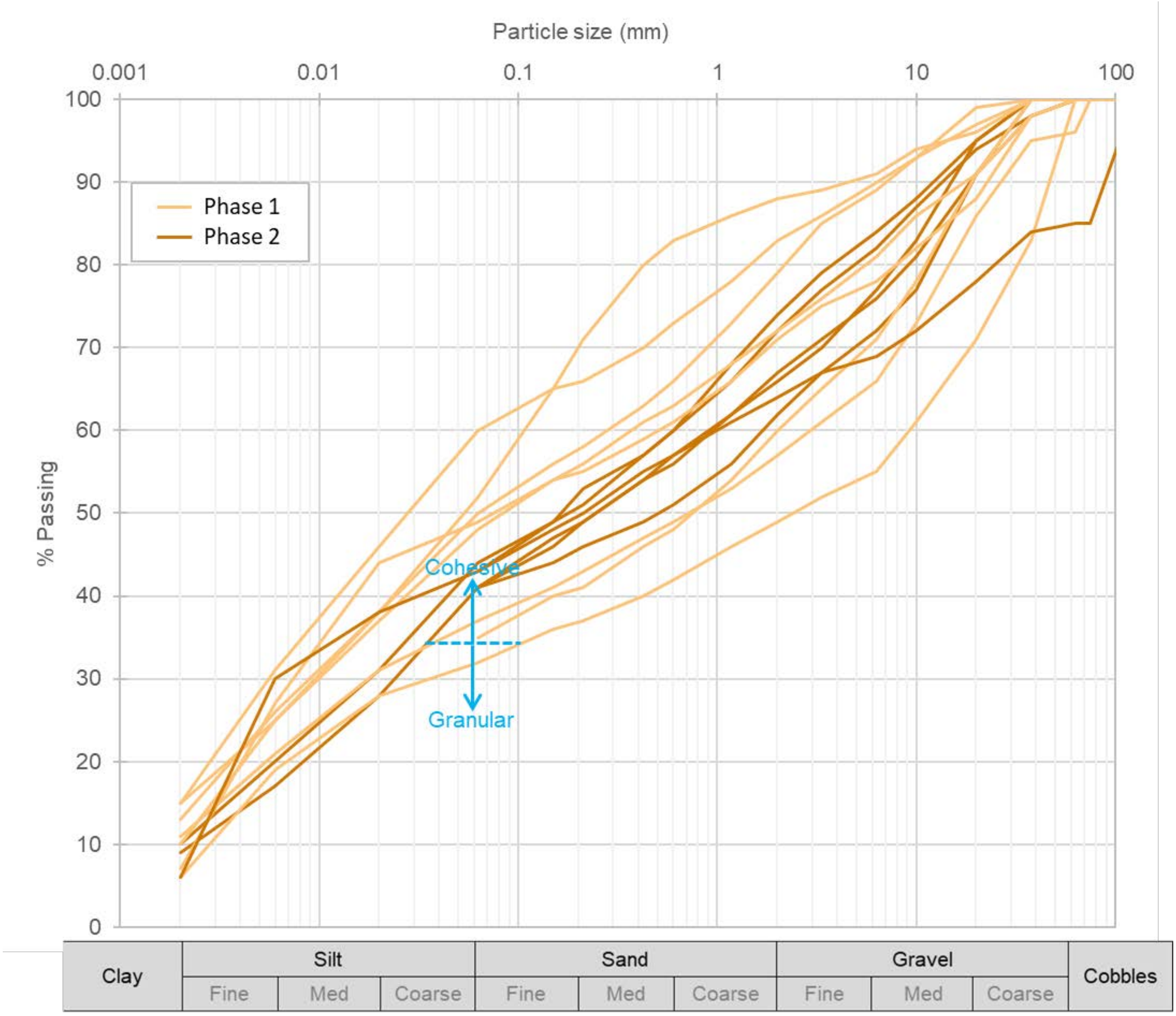

Figure 5-8 PSD results of FINE Porthtowan Formation Grade 4-5

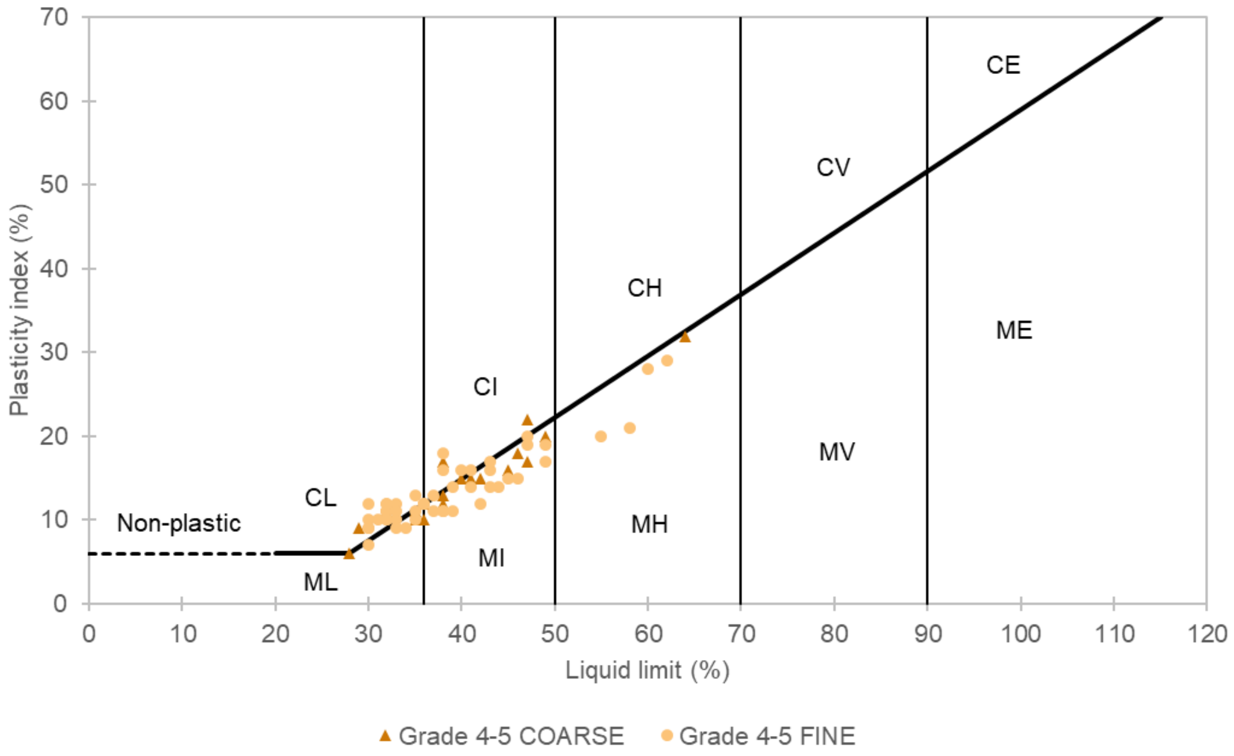

Figure 5-9 Plasticity chart of Porthtowan Formation Grade 4-5

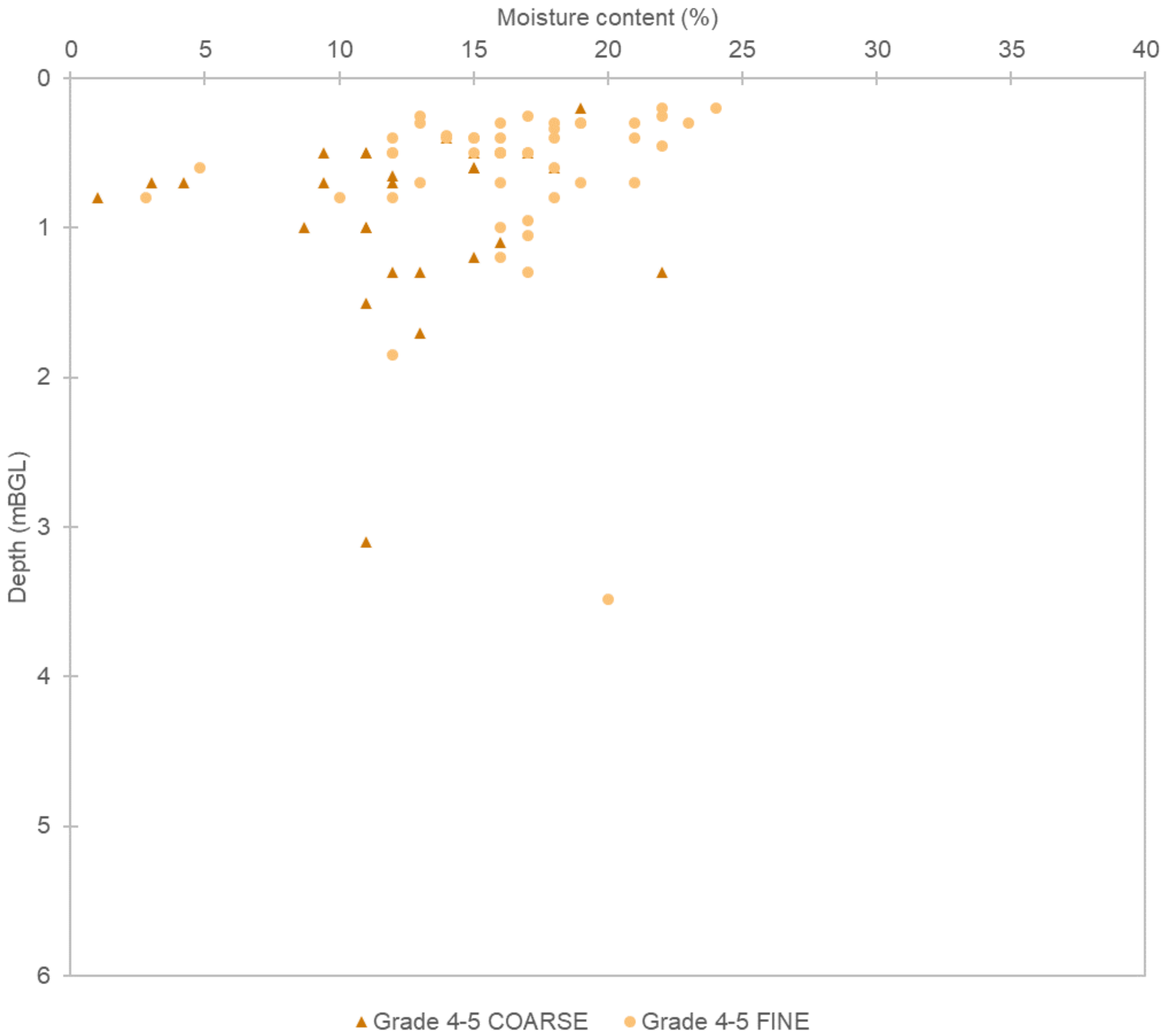

Figure 5-10 Moisture content results of Porthtowan Formation Grade 4-5 with depth

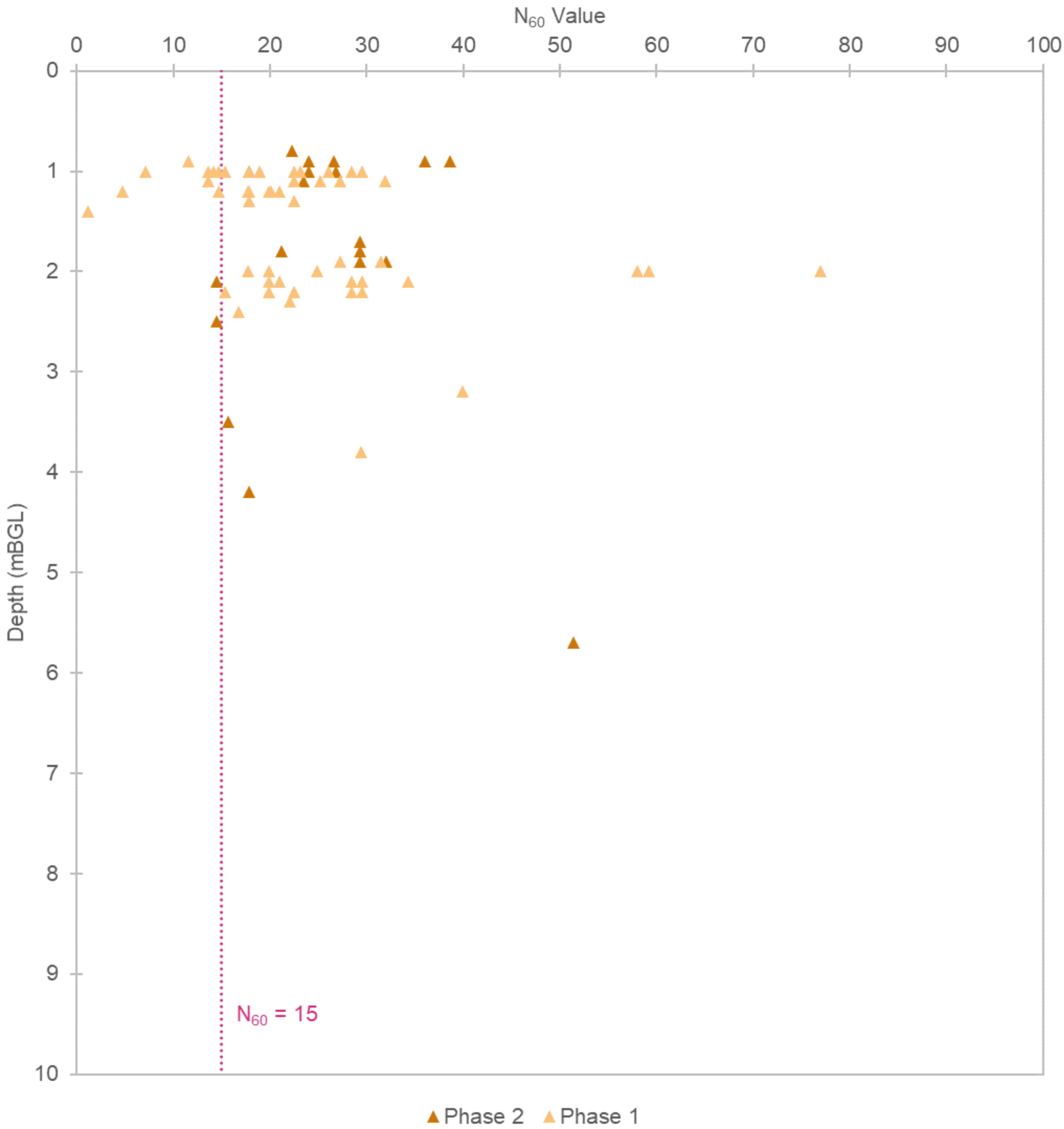

Figure 5-11 SPT results of COARSE Porthtowan Formation Grade 4-5

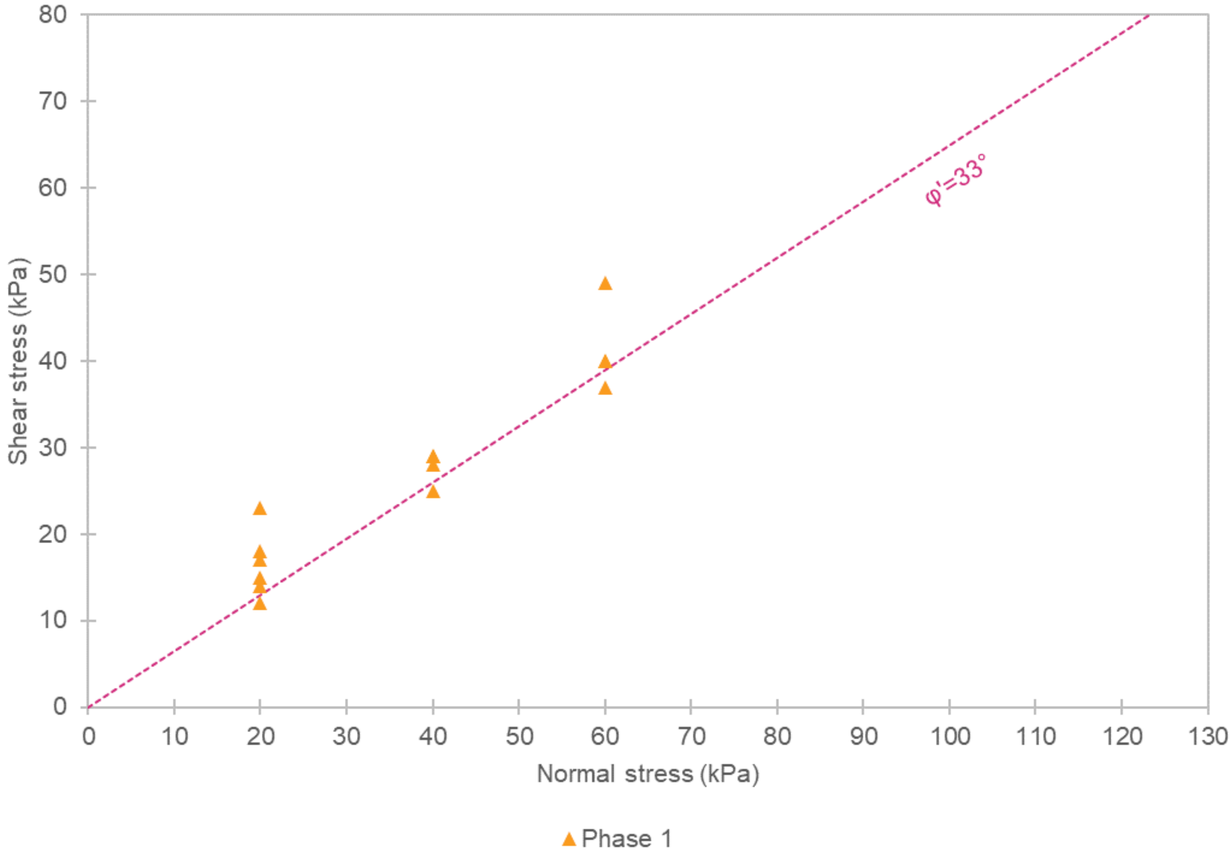

Figure 5-12 Shear box test results COARSE Porthtowan Formation Grade 4-5

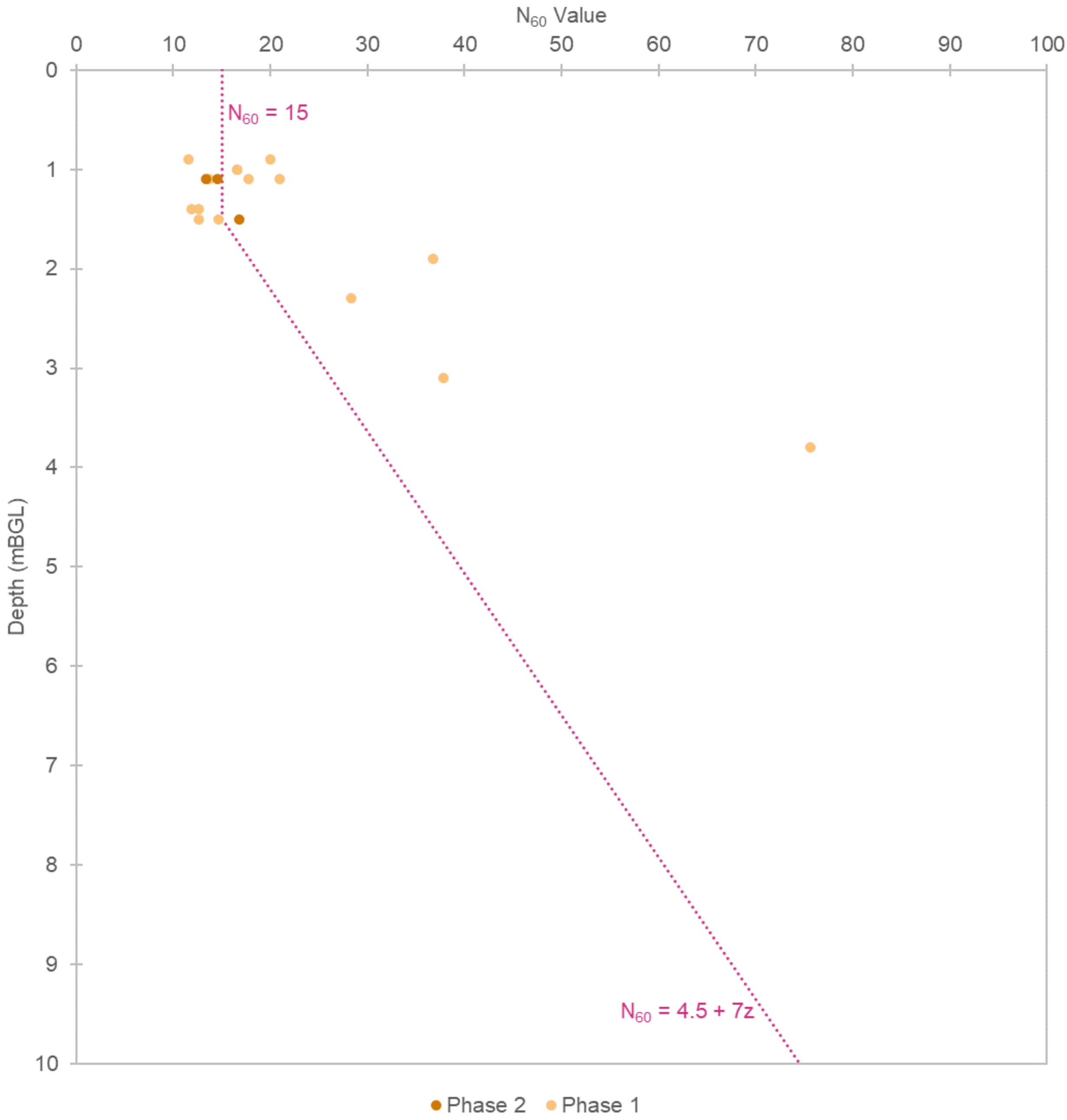

Figure 5-13 SPT results of FINE Porthtowan Formation Grade 4-5

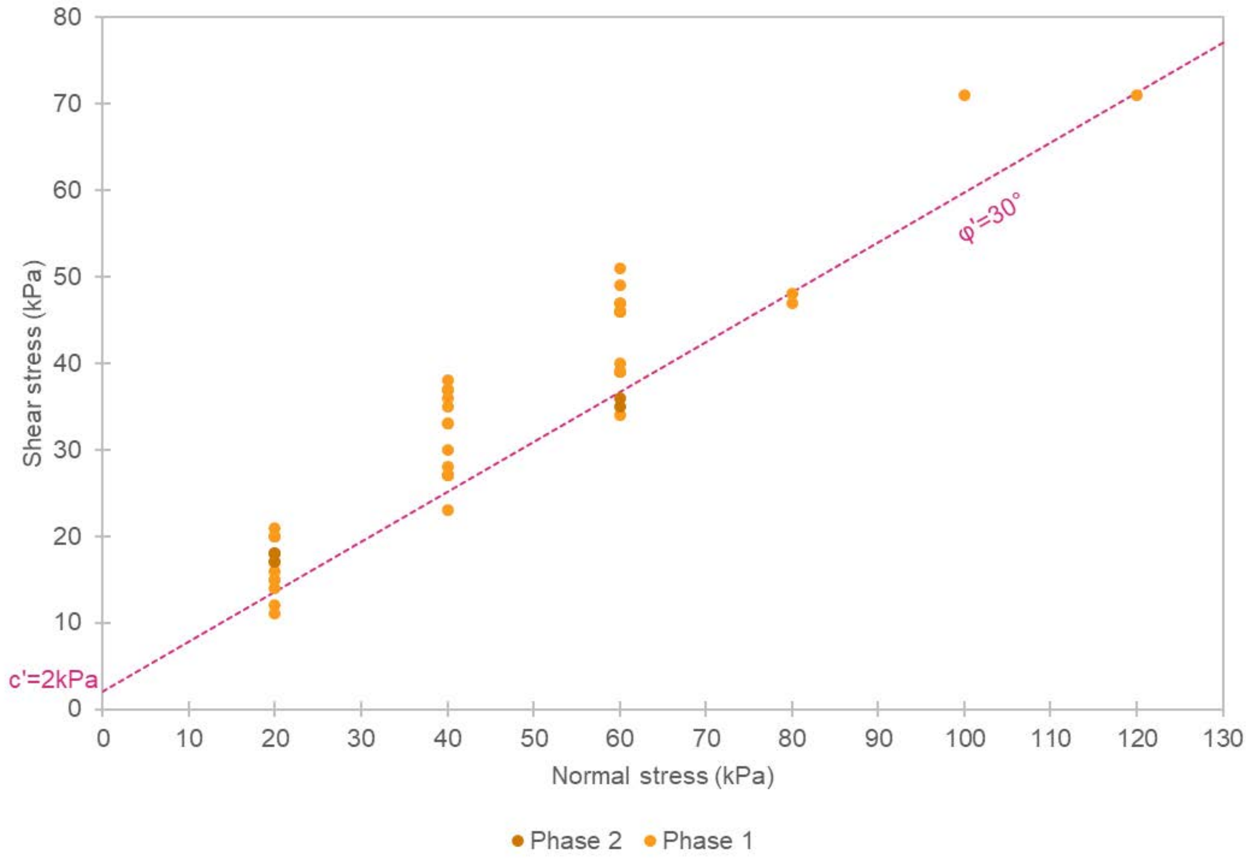

Figure 5-14 Shear box test results FINE Porthtowan Formation Grade 4-5

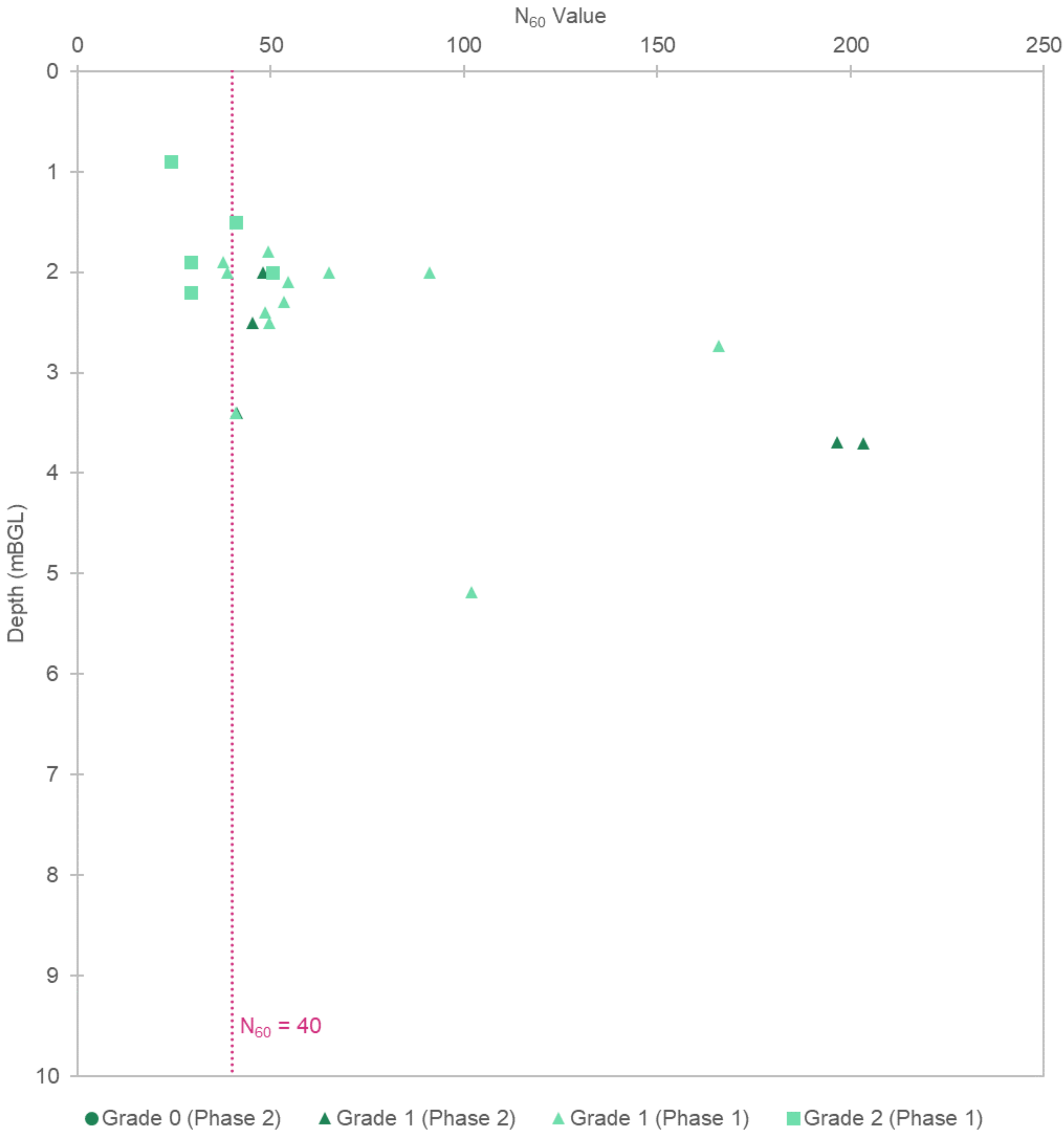

Figure 5-15 SPT results of Grampound Formation Grade 0-2 with depth

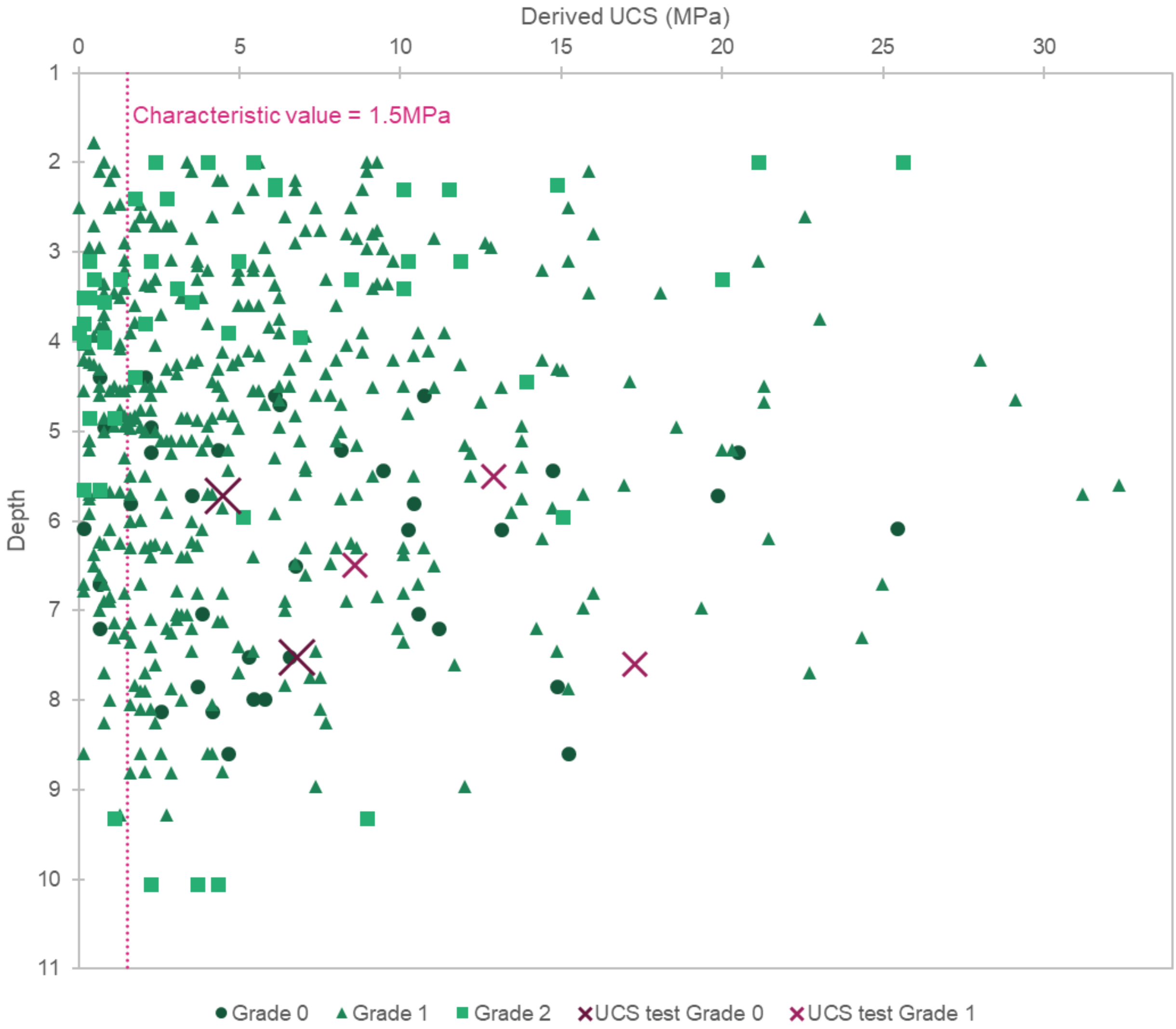

Figure 5-16 PL and UCS test results of Grampound Formation Grade 0-2 with depth

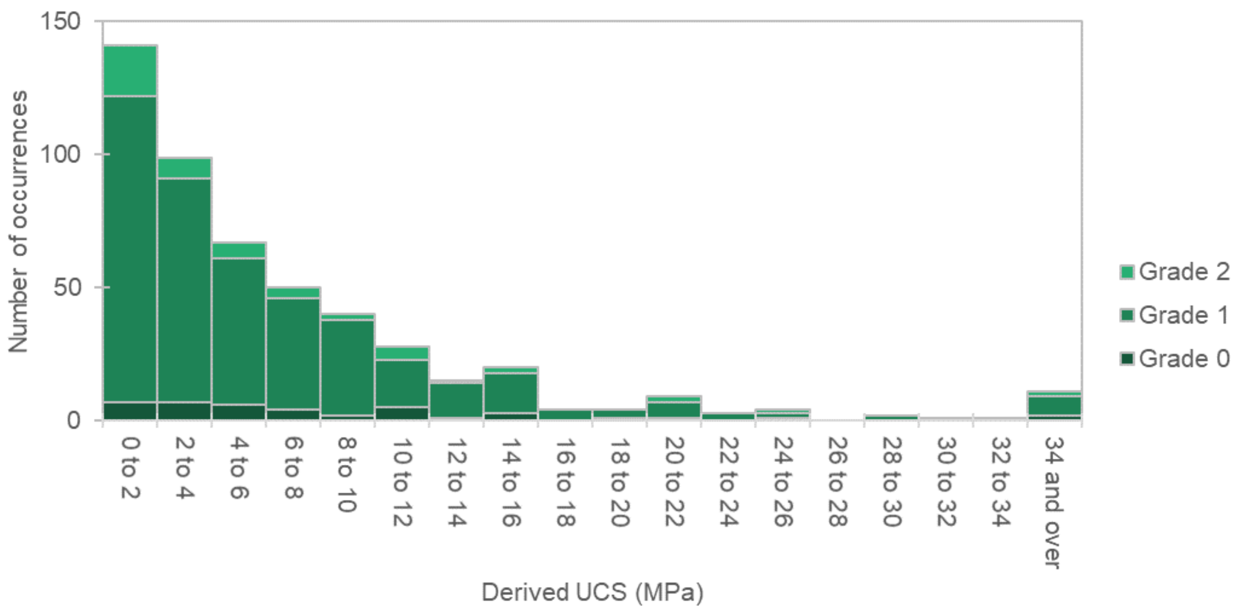

Figure 5-17 Distribution of PL test results of Grampound Formation Grade 0-2

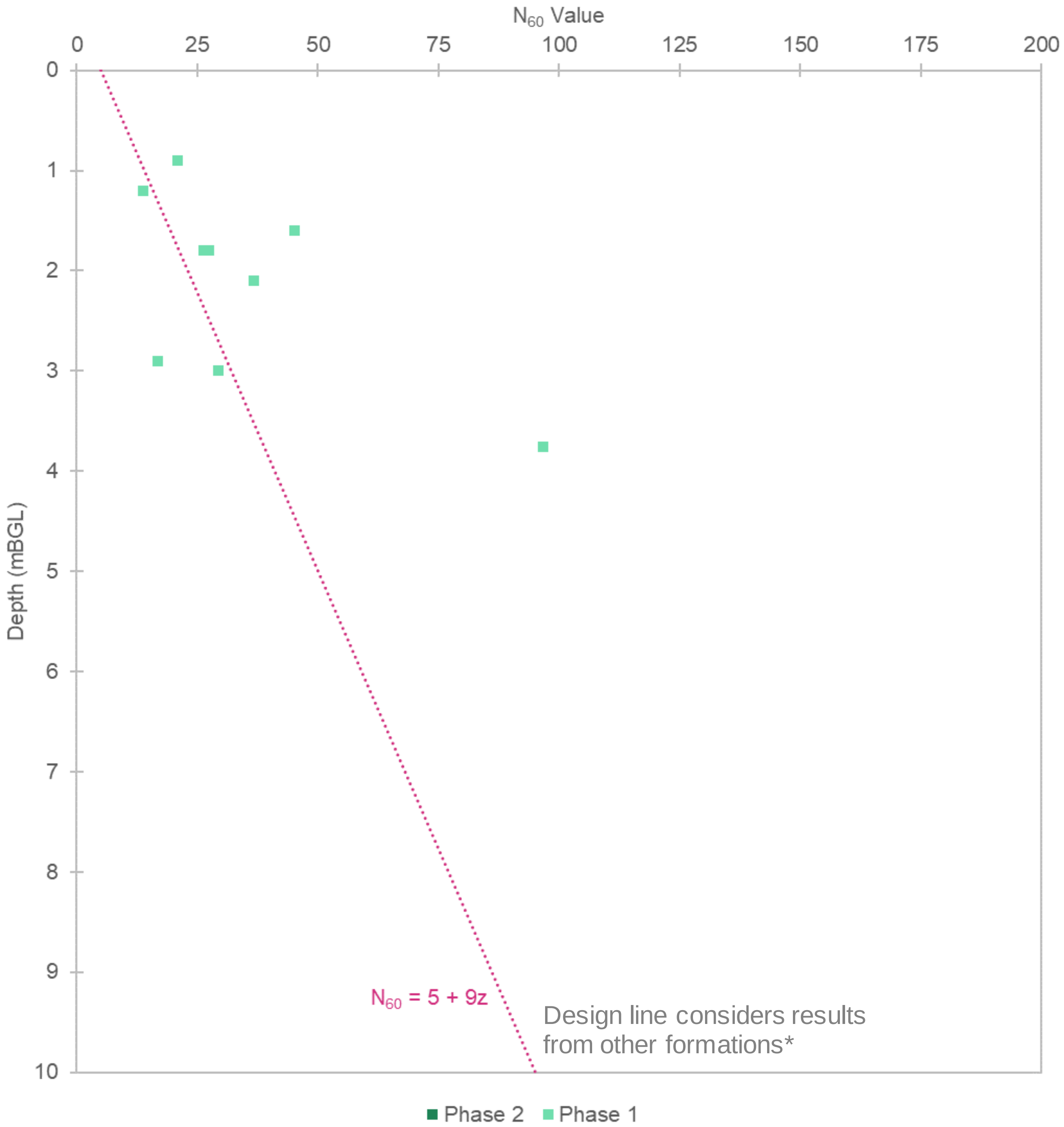

Figure 5-18 SPT results of Grampound Formation Grade 3 with depth

*In order to draw conclusions from a larger data pool and avoid unnecessarily attributing different values of parameters to different formations this design line is based on the combined SPT data for Grade 3 Porthtowan Formation as well as Grade 3 Grampound Formation.

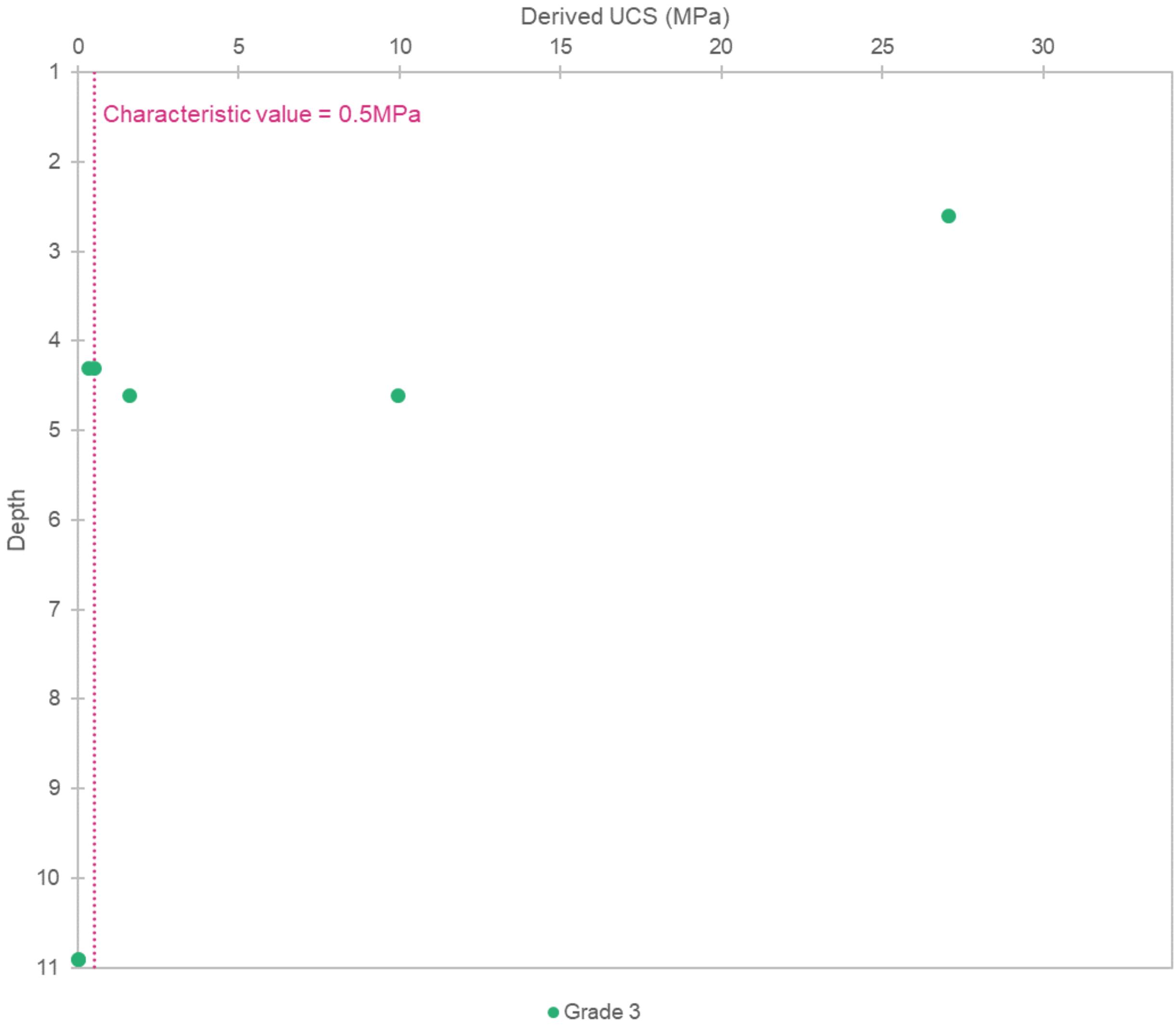

Figure 5-19 PL test results of Grampound Formation Grade 3 with depth

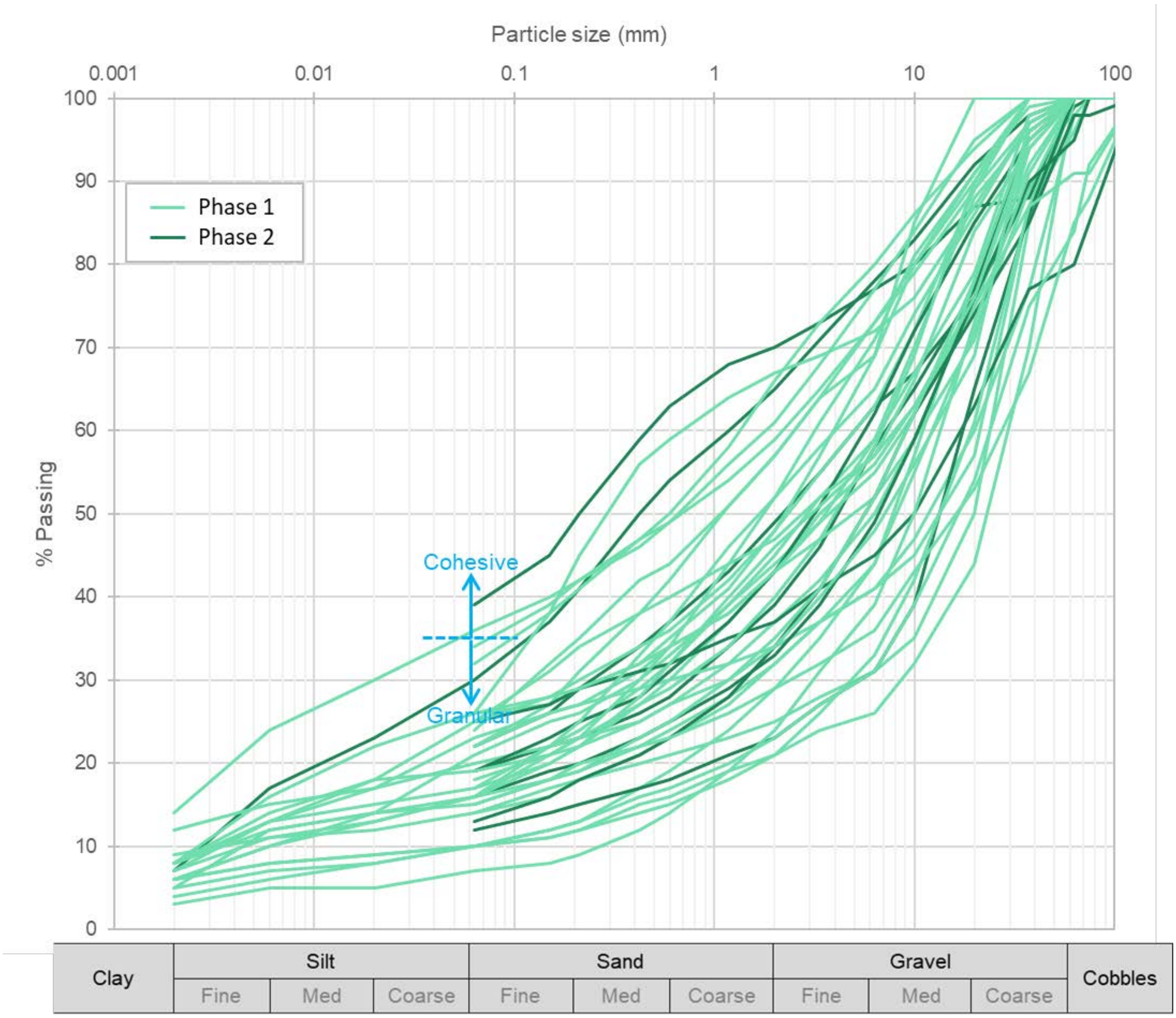

Figure 5-20 PSD results of COARSE Grampound Formation Grade 4-5

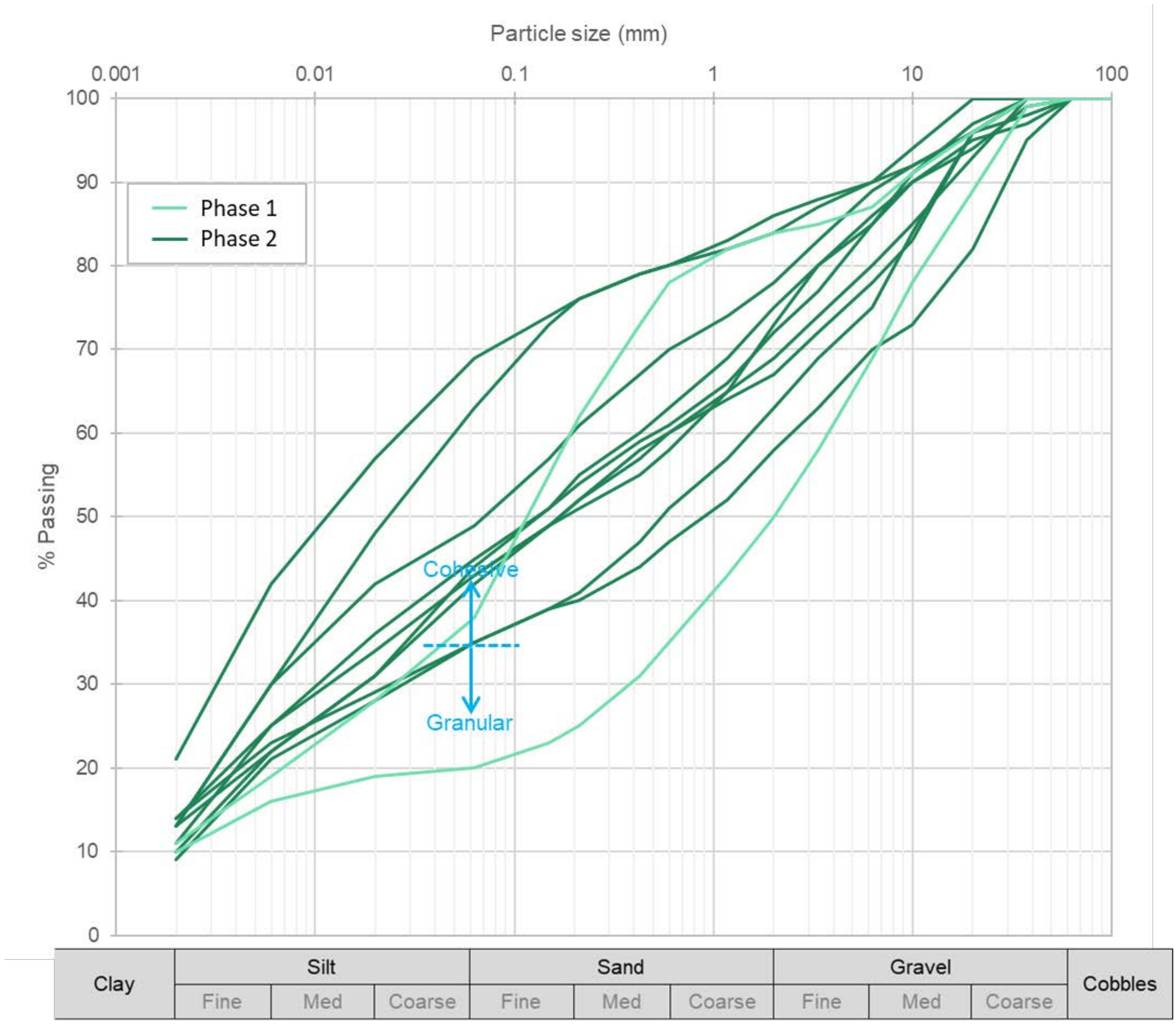

Figure 5-21 PSD results of FINE Grampound Formation Grade 4-5

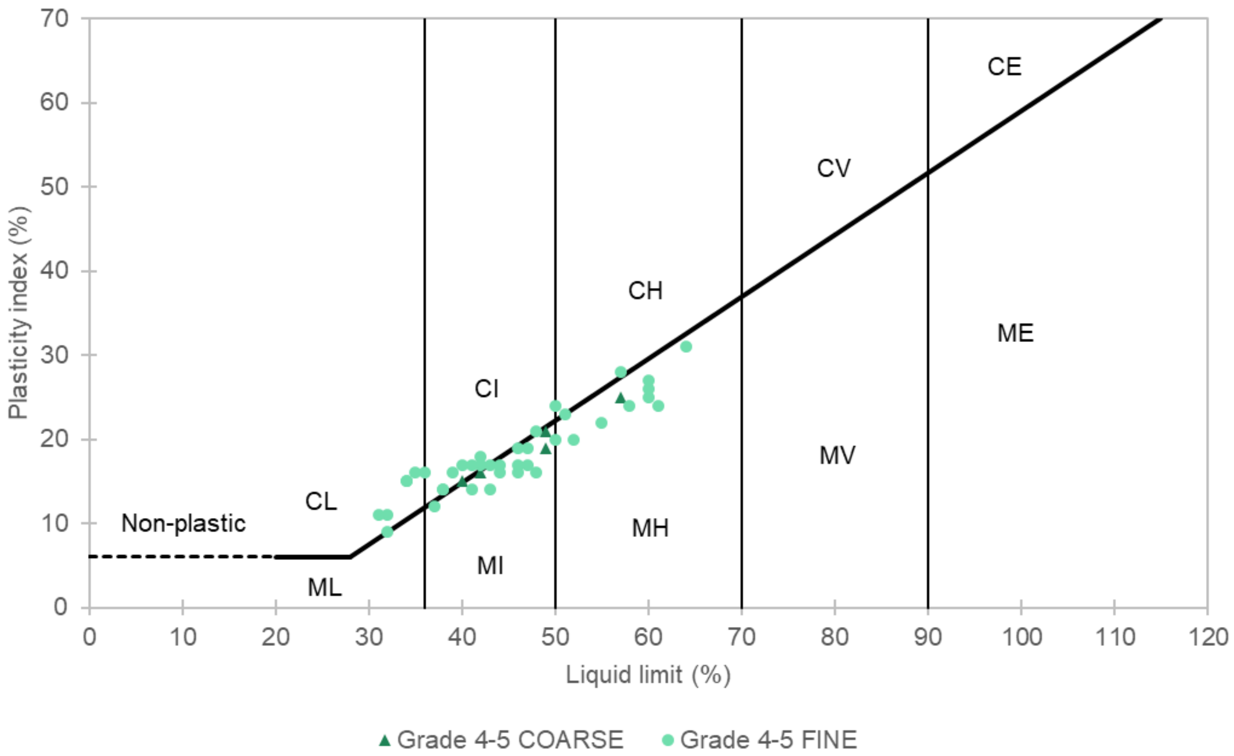

Figure 5-22 Plasticity chart of Grampound Formation Grade 4-5

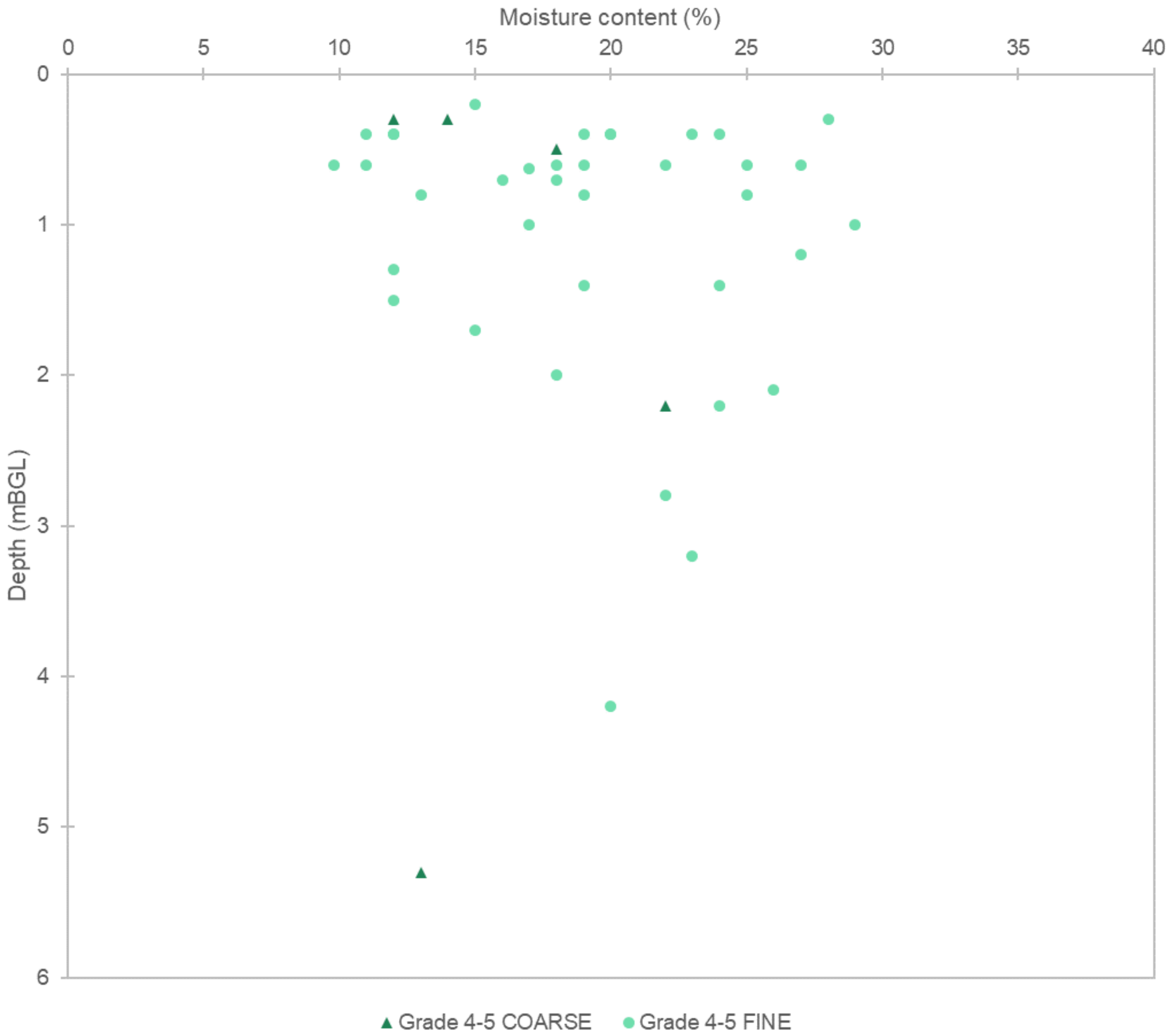

Figure 5-23 Moisture content results of Grampound Formation Grade 4-5

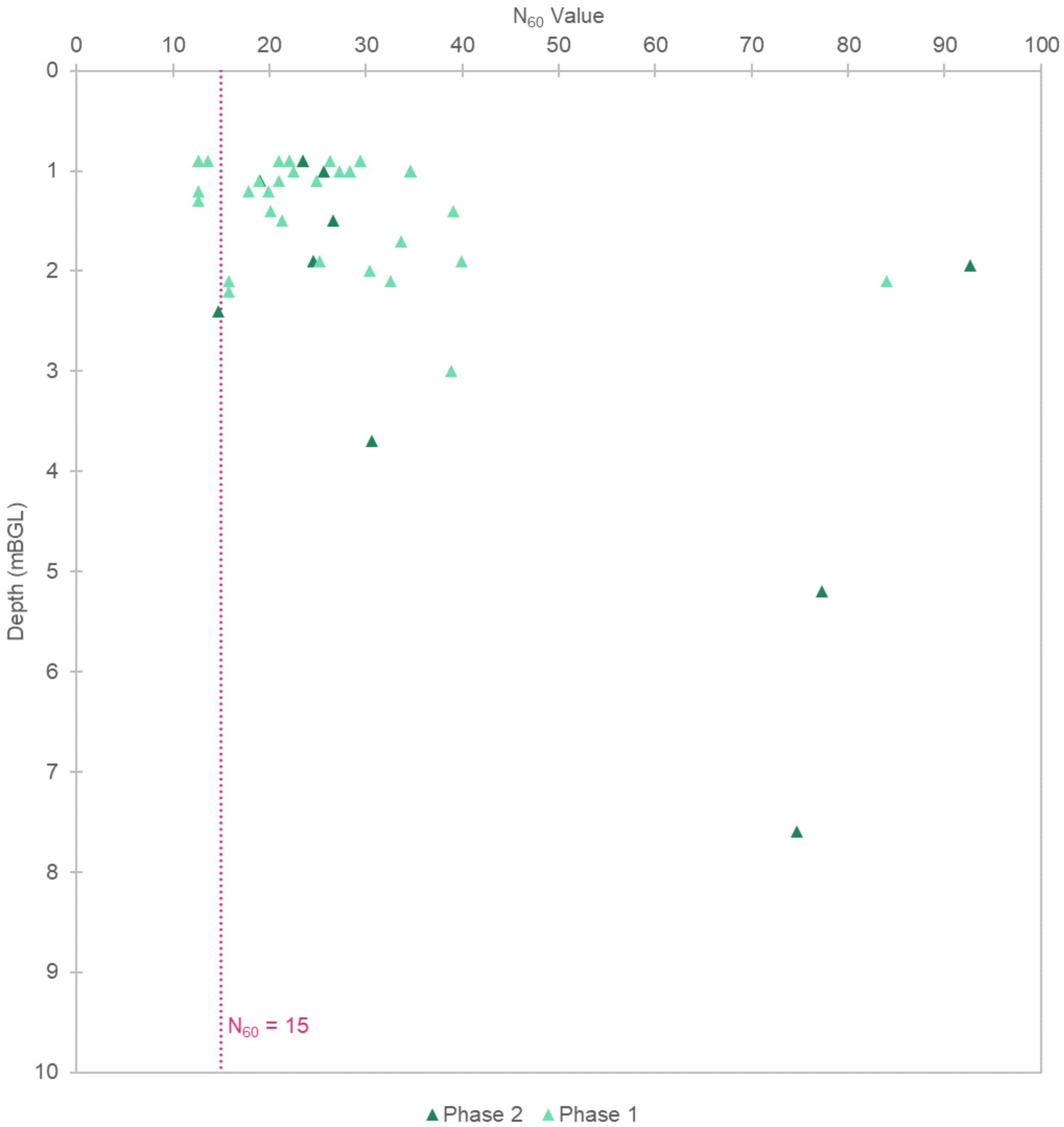

Figure 5-24 SPT results of COARSE Grampound Formation Grade 4-5

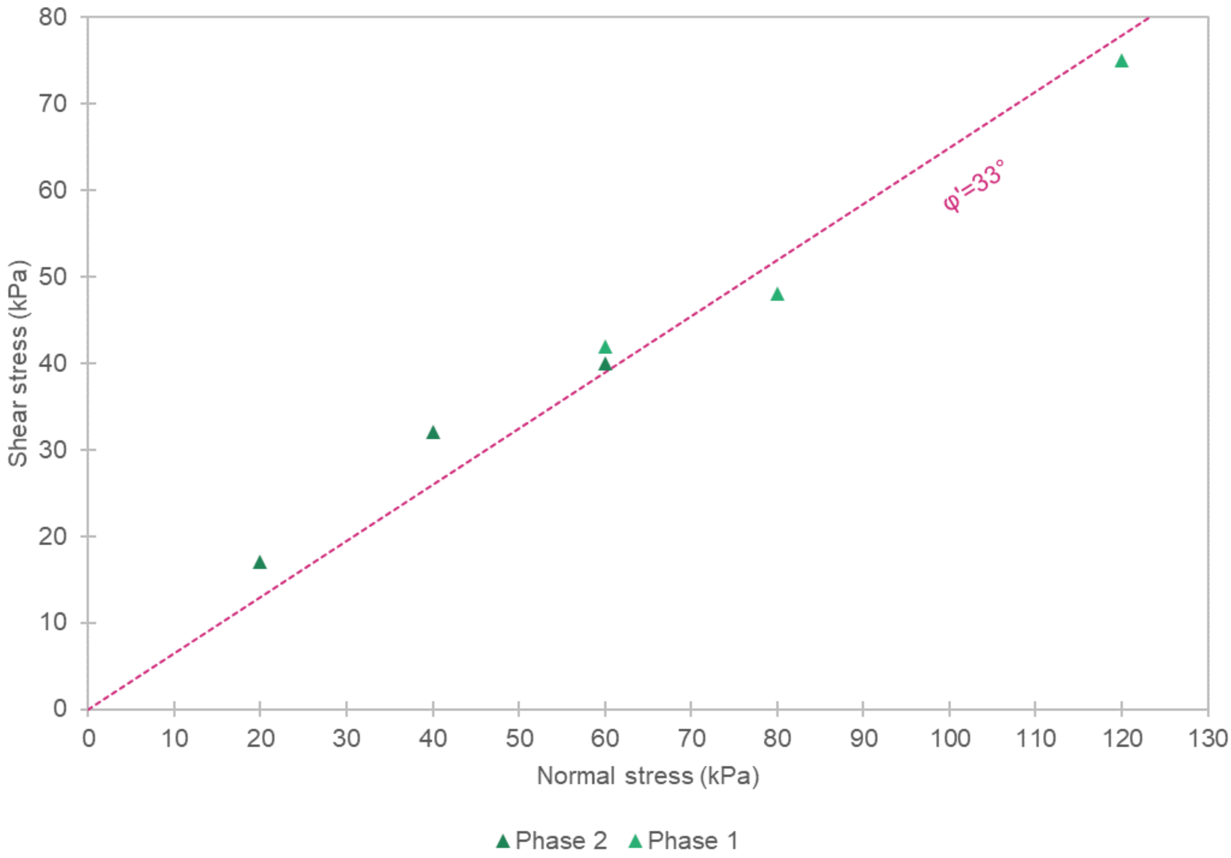

Figure 5-25 Shear box test results COARSE Grampound Formation Grade 4-5

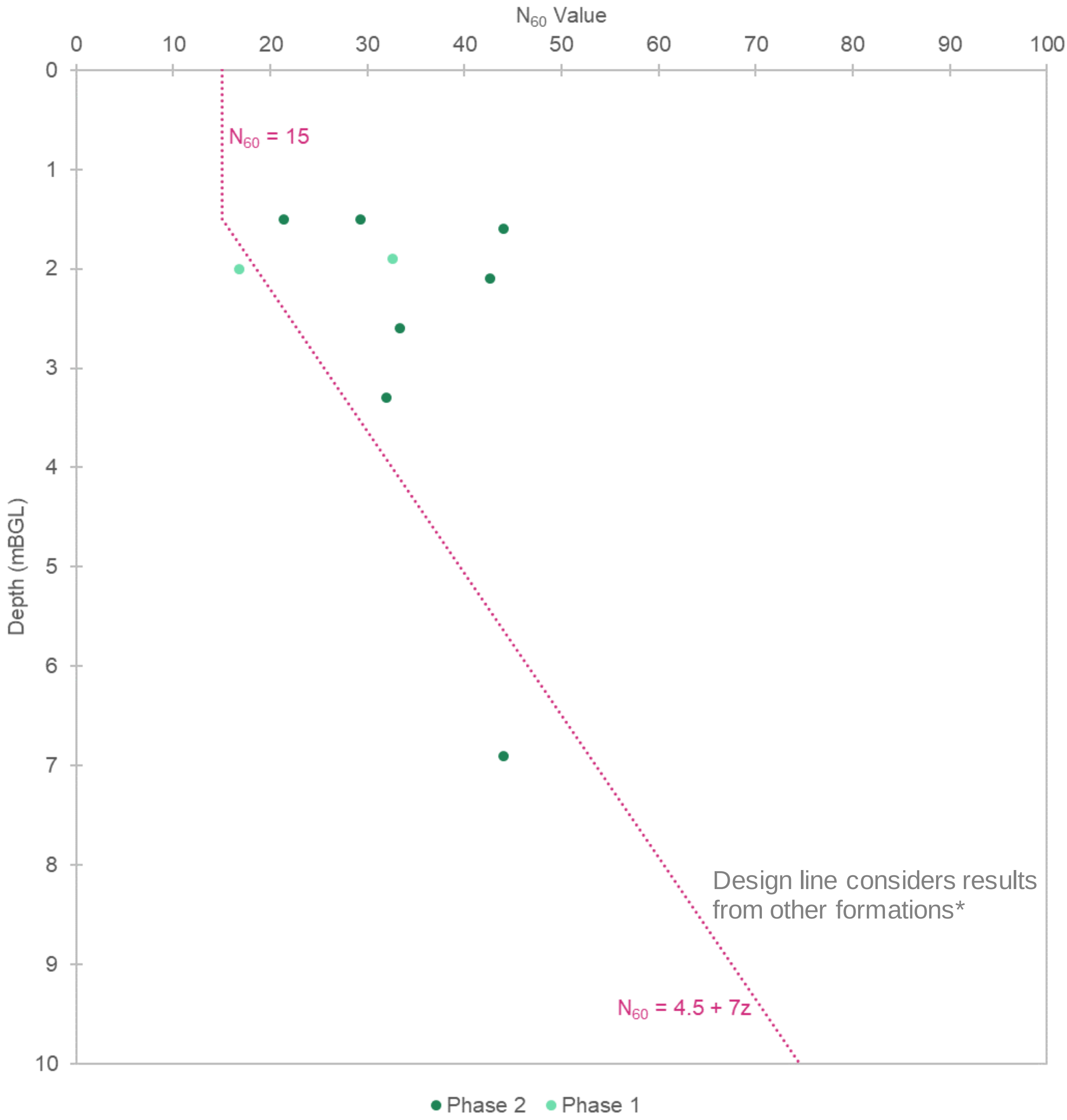

Figure 5-26 SPT results of FINE Grampound Formation Grade 4-5

*In order to draw conclusions from a larger data pool this design line is based on the combined SPT data for Grade 3 Porthtowan Formation as well as Grade 3 Grampound Formation.

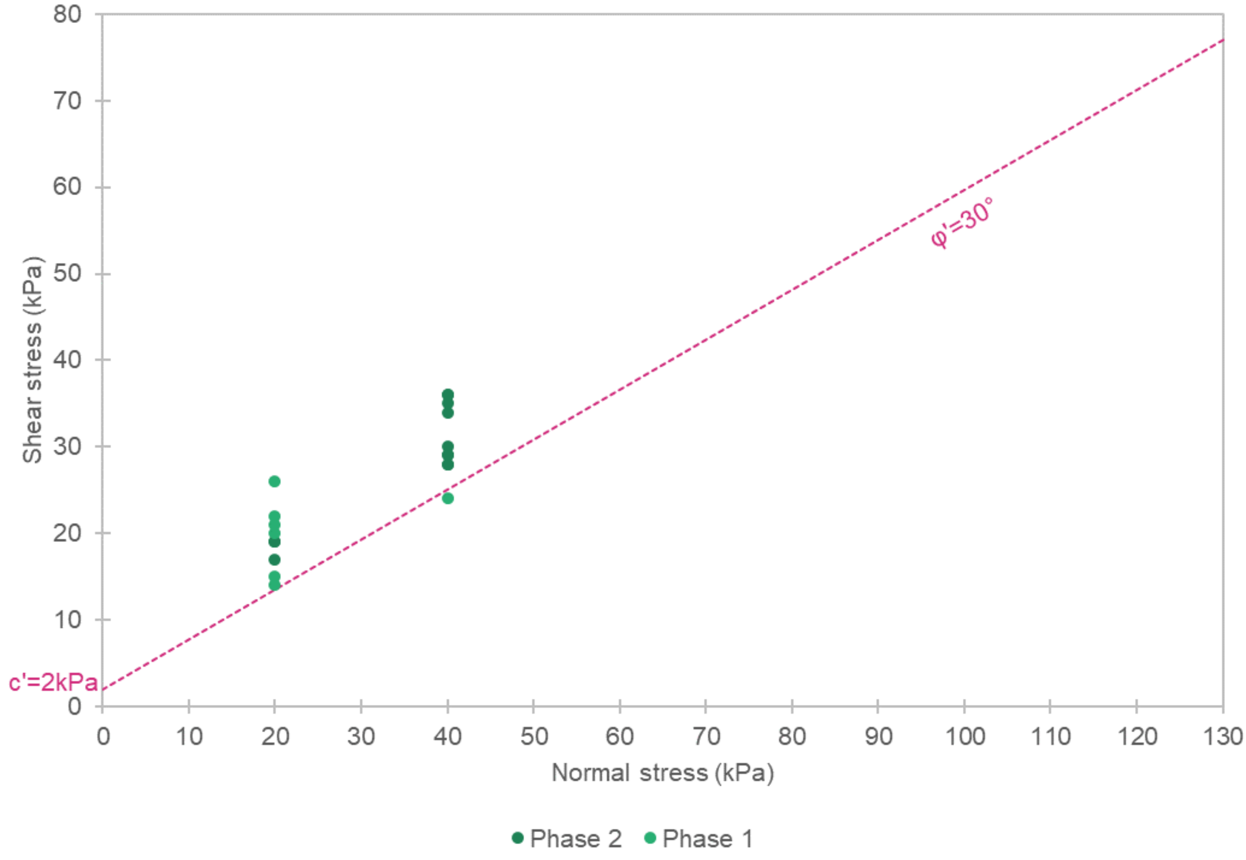

Figure 5-27 Shear box test results FINE Grampound Formation Grade 4-5

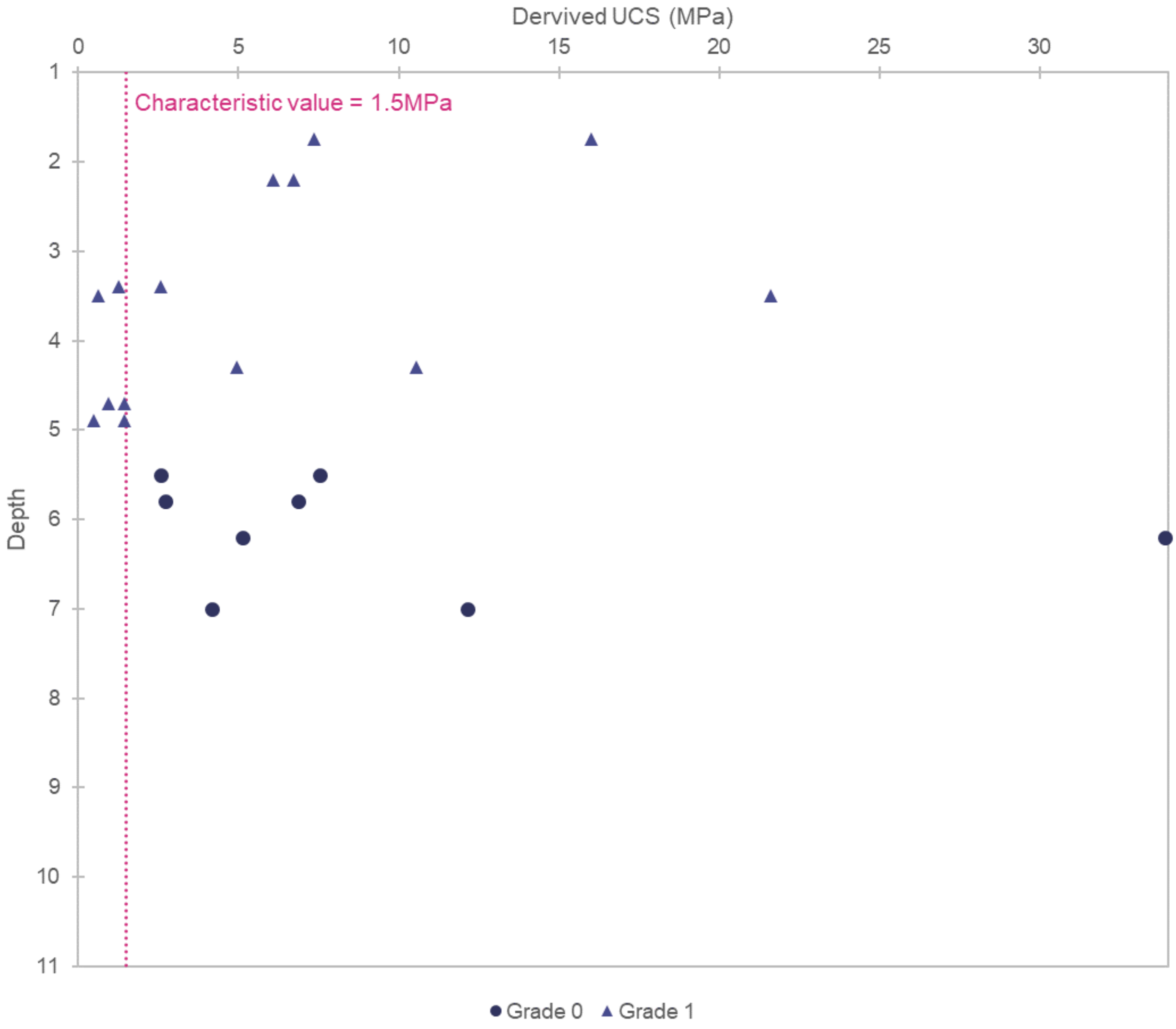

Figure 5-28 PL test results of Trendrean Mudstone Formation Grade 0-2 with depth

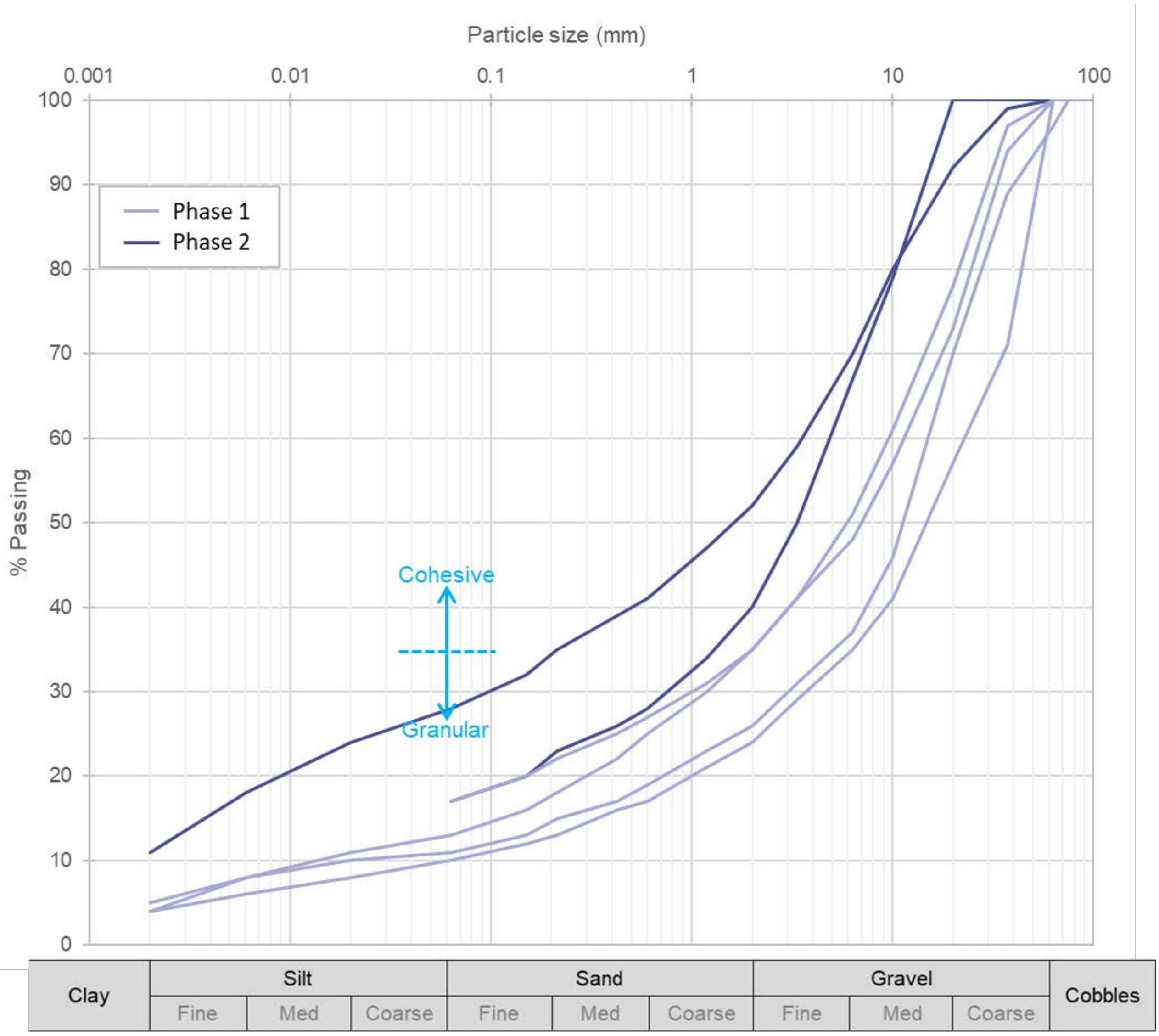

Figure 5-29 PSD results of COARSE Trendrean Mudstone Formation Grade 4-5

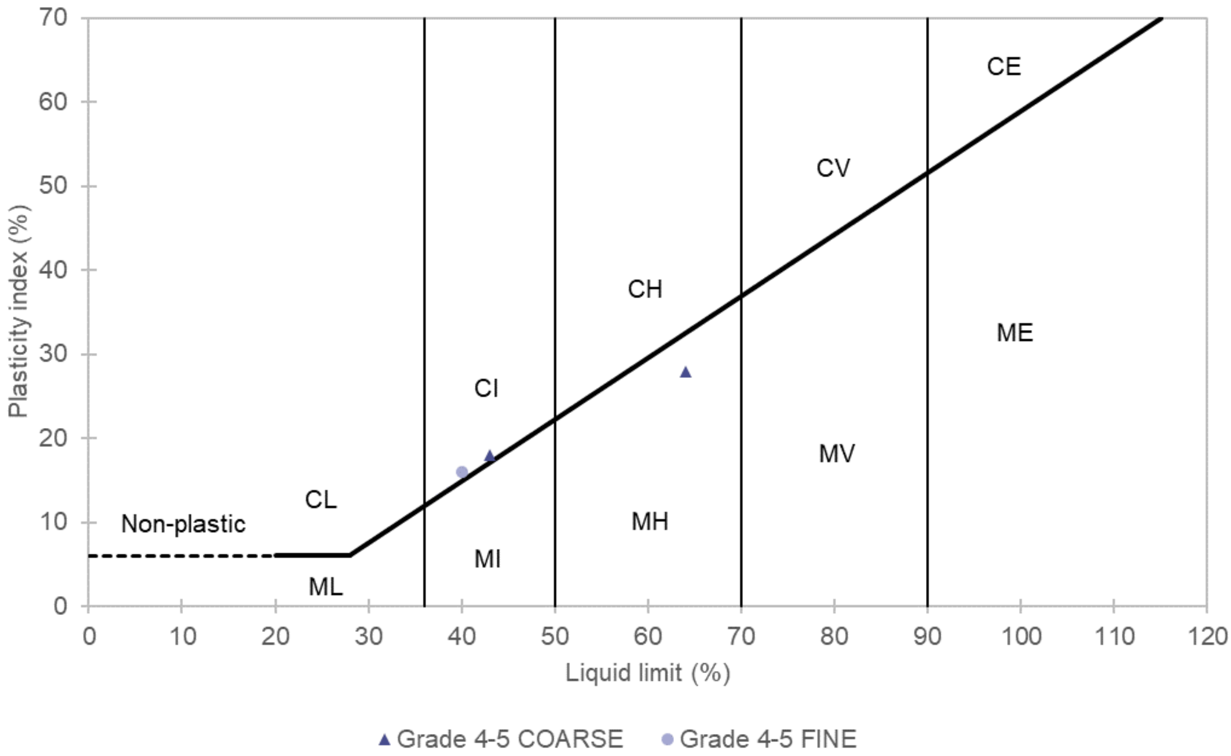

Figure 5-30 Plasticity chart of Trendrean Mudstone Formation Grade 4-5

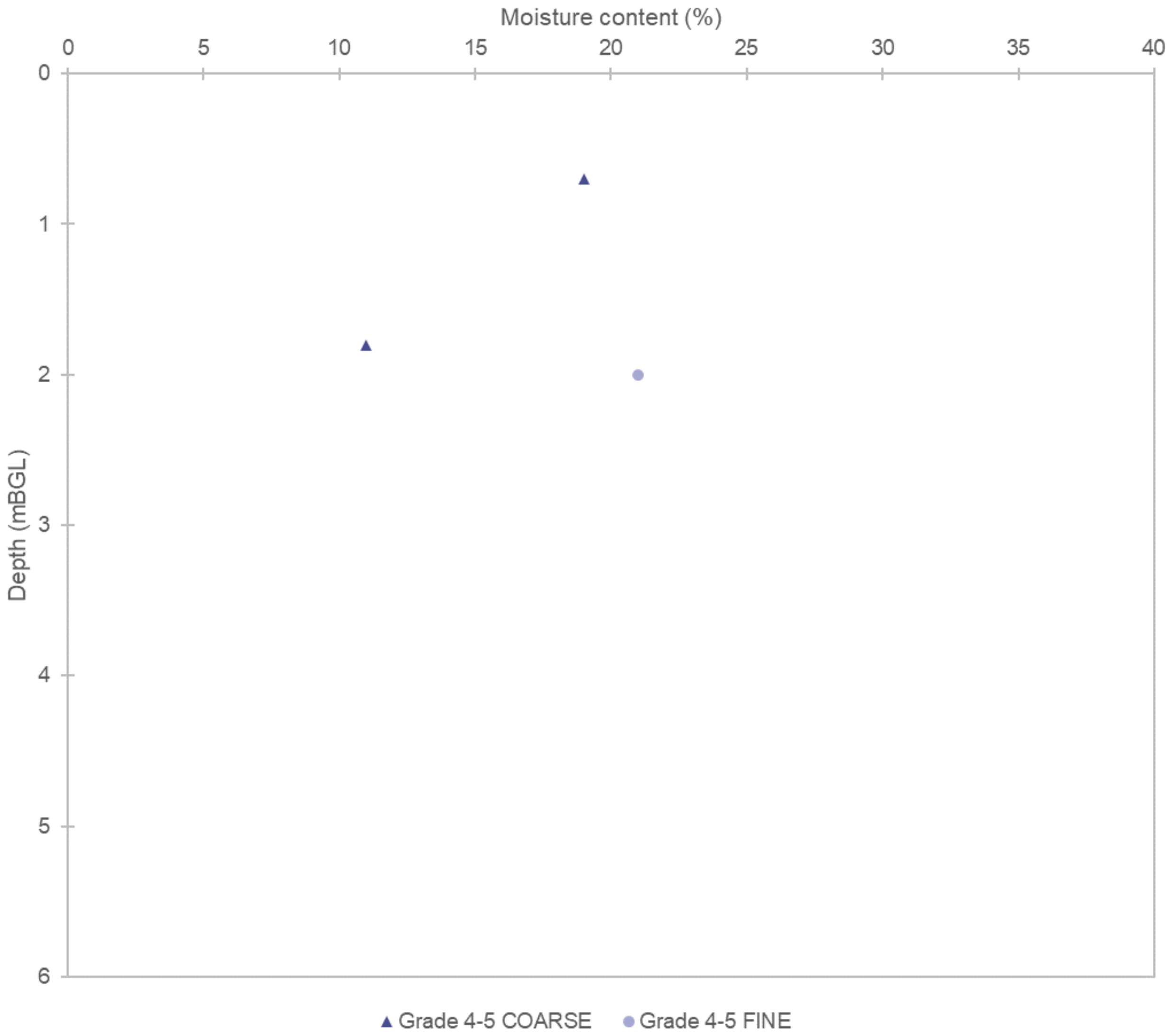

Figure 5-31 Moisture content results of Trendrean Mudstone Formation Grade 4-5

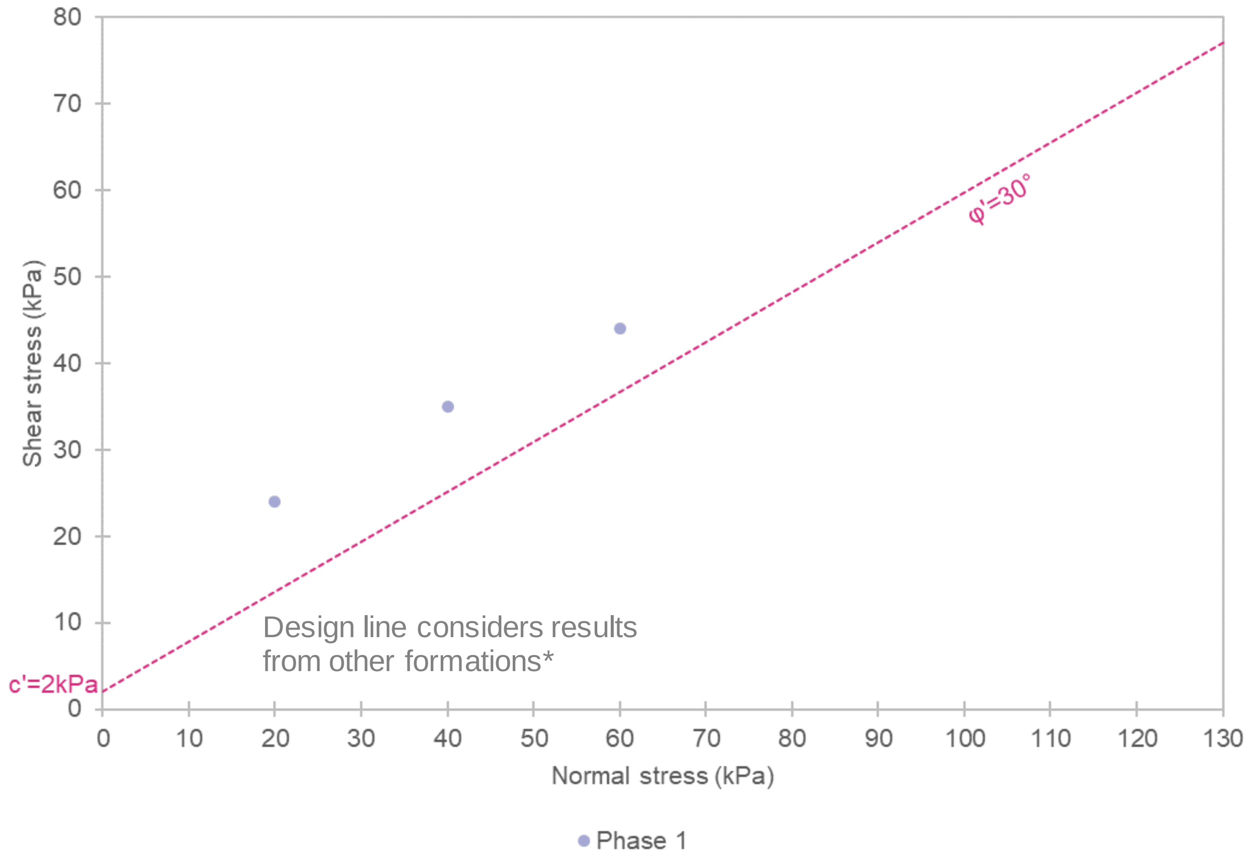

Figure 5-32 Shear box tests FINE Trendrean Mudstone Formation Grade 4-5

*In order to draw conclusions from a larger data pool this design line is based on the design lines for Porthtowan Formation and Grampound Formation, which are conservative here.

6 Contamination

6.1 Conceptual Model

6.1.1 The following assessment in relation to contamination has been prepared following the production of the PEIR [3] and completion of the Phase 2 GI. The previously presented Conceptual Model for the site which was included in the GIR has been updated and included in the PEIR [3]. A summary of the updated conceptual model presented in the PEIR is presented below.

Sources:

6.1.2 The following sources have been identified during review of the desk study information on the site and the results of the GI:

- Made Ground – associated with existing infrastructure, identified during the GI, associated with private development and farm land.
- Historic mining – mine waste, backfilled mining features, mine waters.
- Current or historic land use (non-mining) – petrol filling stations, vehicle servicing, disused/backfilled quarries, former nurseries, agriculture/farming, activities associated with the operation of existing infrastructure (spillage of oils/fuels etc, defective drainage/interceptors).

Pathways:

6.1.3 The following pathways were identified during review of the desk study information and assessment of the development and end use of the scheme:

- Ingestion of soils and dust – soils exposed during works on existing infrastructure, soils exposed in farming practice, during the proposed works, and following completion during subsequent maintenance works.
- Inhalation of soil dust – soils exposed during works on existing infrastructure, soils exposed in farming practice, during the proposed works, and following completion during subsequent maintenance works.
- Inhalation of gases and vapours – gases and vapours from spills and leaks on existing infrastructure, gases from natural and made ground sources.
- Dermal contact with soils and dust - soils exposed during works on existing infrastructure, soils exposed in farming practice, during the proposed works, and following completion during subsequent maintenance works.
- Leaching of contamination – soil borne contamination leaching into underlying groundwater and lateral migration/impact on surface waters.
- Lateral migration of contaminated groundwater.

Receptors:

6.1.4 The following potential receptors were identified during review of the desk study information and assessment of the development and end use of the scheme:

- Existing site users of the current scheme area, including existing infrastructure, workers (businesses and farms), recreational users (ramblers etc.) and residents
- End site users.
- Construction and maintenance workers of the existing infrastructure

- Construction and maintenance workers involved in the proposed development
- Groundwater and surface waters.

6.1.5 Review of the above potential source pathway receptor (SPR) linkages indicate that nearby residents, workers and recreational land users are unlikely to be exposed to potential sources of contamination through ingestion, inhalation and to groundwater and soils through dermal contact on a frequent basis, if at all, for the following reasons:

- The GI generally encountered natural soils across the study site.
- Where Made Ground soils have been encountered these have been generally isolated to small areas and did not display visual or olfactory signs of contamination.
- The most likely source for contamination is either Made Ground associated with the existing road infrastructure or possible mining waste. In the current scenario the former is likely to be largely isolated from these receptors by road surfacing, while the latter is considered likely to be isolated to some degree by vegetation and or topsoil layers.
- Exposure frequency is likely to be relatively sporadic, and in addition the duration is likely to be short term. For example, it is overly pessimistic to assume that an entire walking route would be over exposed, contaminated soils.
- Mitigation measures will be required during construction works to reduce the risk of dust generation.

6.1.6 Review of the possible impact to maintenance workers and construction works indicates that they are considered the most likely to be impacted by the potential sources of contamination for the following reasons:

- Maintenance workers or construction workers may be directly exposed to contaminated soils or Made Ground during works on the existing infrastructure or proposed works. Exposure pathways would include dermal, ingestion and inhalation. Exposure duration is likely to be relatively short term for construction workers, however it is feasible that this could be on a more regular basis for maintenance workers over the lifetime of the worker (e.g. grass cutting on verges).
- Due to likely location of the works (in association with highways) it is considered that there is a higher potential for Made Ground, or contaminated soils to be present.
- Construction workers may come into contact with shallow groundwater, albeit only during the construction phase. Regular maintenance works are not considered likely to involve deep excavations, no direct exposure to groundwater is considered likely to occur. However, the proposed drainage systems may collect groundwater and maintenance workers might come into contact with groundwater through this pathway. Nevertheless, it is considered that contact will be limited through the use of PPE and given the likely low frequency of exposure via this pathway the risk is relatively low.
- Given the likely nature of the site soils, ground gas risk is considered to be low. Furthermore, it is considered that man entry into excavations/confined spaces would be limited and likely to be controlled with mitigation measures and risk assessment to reduce the risk to maintenance and construction workers from ground gasses.

6.1.7 Existing users of the A30, or other highways in the Study site area are not considered likely to be impacted by contamination on the basis of the following:

- Relative isolation within vehicles.
- Their transient nature and likely short term duration.

6.1.8 The possible pathways in relation to controlled waters are considered to be plausible for the following reasons:

- Potential contaminants within the identified sources are considered to be freely leachable from the site soils via infiltration of rain or surface water.
- The investigations to date have indicated the site soils to comprise a mixture of granular and cohesive materials overlying weathered bedrock. While not considered to be highly permeable strata, vertical and lateral migration is still plausible, especially in bands of higher permeability strata or in granular made ground, service runs, or old mining features.

6.1.9 In summary, a review of the conceptual model for the current and proposed end use of the site indicates that the most plausible receptors to site contamination are likely to be construction workers involved in the proposed works and maintenance workers on the existing and future scheme. In addition, it is considered that controlled waters are also potentially at risk from contamination. Site end users, nearby residents and workers are not considered to be at risk on the basis that the potential pathways are not considered to be plausible.

6.2 Human Health Generic Quantitative Risk Assessment

6.2.1 On the basis of the site conceptual model a Generic Quantitative Risk Assessment (GQRA) has been carried out to assess the risk to human health. The previous GQRA undertaken by WSP and included in the GIR [1] included assessment using Commercial End Use screening criteria. It is not considered that these criteria would be suitable to assess the likely risks to the receptors identified in the site conceptual model and as such the following section presents an updated GQRA using both the Phase 1 and Phase 2 GI data.

6.2.2 As part of the Phase 1 works, a total of 21No. soil samples were submitted for chemical testing. Of these 5No. were from Made Ground soils, 4No. were from weathered bedrock, 1No. was obtained from alluvium, and 11No. were obtained from topsoil.

6.2.3 During the supplementary Phase 2 works, a further 48No. soil samples were submitted for chemical testing. Of these 7No. were from Made Ground soils, 26No. were from weathered bedrock, 1No. was obtained from alluvium, and 14No. were obtained from topsoil.

6.2.4 The soil samples obtained were analysed for the following contaminants:

- Heavy metals, cyanide, sulphate, phenols, and pH;
- Asbestos screen and identification;
- Speciated total petroleum hydrocarbons (TPH CWG);
- Speciated polycyclic aromatic hydrocarbons (USEPA 16 PAH) and benzene, toluene, ethylbenzene and xylenes (BTEX)
- Pesticides and herbicides suites.

6.2.5 In relation to the risk from contaminated soils the identified receptors are maintenance and construction workers. Published generic screening criteria for the exposure scenario associated with this type of work are not available since current published guidance is tailored to the assessment of the end use of the site and chronic exposure periods. On this basis the assessment criteria chosen for the following GQRA are for residential with plant uptake end use. These criteria are considered to be risk conservative given the likely exposure scenario encountered by a maintenance and construction workers, however they are likely to be suitable to establish if further discussion or detailed assessment is required.

6.2.6 The results of the screening assessment indicate that the majority of chemical concentrations fall below the applied screening criteria with the following exceptions:

- 8No. concentrations of arsenic;
- 2No. concentrations of lead;
- 1No. concentration of benzo(a)pyrene;
- 1No. concentration of dibenzo(ah)anthracene.

6.2.7 Exceedances for arsenic were encountered in samples of made ground from 0.6m bgl in TP-P-09, samples from 0.5m above ground level (bund) and 0.2m bgl in TP-201, samples from 0.4 to 0.5m and 0.9 to 1.0m bgl in TP-219. Review of the soil descriptions for TP-P-09, TP-201, and TP-219 does not indicate a potential source of the arsenic aside from the general description of Made Ground. The other exceedances for arsenic were noted in residual soils in samples from 0.4 to 0.5m bgl in TP-363, 0.7 to 0.8m bgl in TP-365, and a sample from 0.3m bgl in BH-213. It is concluded that these concentrations may be reflective of elevated background concentrations associated with the natural soils in this geography.

6.2.8 Exceedances for lead were encountered in samples of Made Ground from 0.65m bgl in BH-R-030 and 0.00 to 0.15m bgl in BH-212. Review of the soil descriptions for both exploratory holes does not indicate any potential sources for the lead aside from the general description of Ground including coal. Review of the location of BH-R-030 indicates that the borehole lies approximately 100m north of the location of an old shaft and heap (possible mine waste) shown on the historic mapping from 1879 to 1958. The mapping following 1958 no longer shows the shaft or heap, the heap may well have been re-graded across the area and it may be that this is the cause of the elevated lead in this location. BH-212 is in close proximity to an old shale quarry and as such it is conceivable that the Made Ground in this area may be backfilled waste material used to fill this feature.

6.2.9 Exceedances for both benzo(a)pyrene and dibenzon(ah)anthracene were only recorded in a single sample from 0.9 to 1.0m bgl in TP-219. Review of the soil descriptions for TP-219 indicates asphalt present within the Made Ground, potential drainage infrastructure and an oily sheen on the groundwater encountered in the trial pit. These may well be the source of these elevated levels of PAH.

6.2.10 Despite the above exceedances, in general the soils encountered during the investigation works have shown little evidence of contamination with concentrations of contaminants falling below the applied residential with plant uptake screening criteria. It is considered that the screening criteria are likely to be overly conservative in relation to assessing the risk to maintenance and construction workers, and that it is likely that much of the risk identified by the exceedances would be mitigated by the likely use of personal protective equipment (PPE). On this basis it is not considered that a risk to human health is present to the identified receptors.

6.3 Controlled Waters Generic Quantitative Risk Assessment

6.3.1 In order to assess the likely impact on controlled waters a GQRA based on the results of leachate analysis and groundwater analysis obtained during the Phase 1 and Phase 2 GI has been undertaken.

6.3.2 As part of the Phase 1 GI leachate analysis was undertaken on a total of 16No soil samples. 4No samples were from Made Ground deposits, 4No. from weathered bedrock and a further 8No. from topsoil samples. However, no groundwater sampling and testing was undertaken.

6.3.3 As part of the Phase 2 GI a total of 12No. groundwater samples were obtained from installations in boreholes from across the scheme.

6.3.4 Leachate preparations were analysed for the following contaminants:

- Heavy metals, ammoniacal nitrogen, and pH;
- TPH CWG;
- USEPA 16 PAH and BTEX.

6.3.5 Groundwater samples were analysed for the following contaminants:

- Heavy metals, ammoniacal nitrogen, free and total cyanide, phenols, sulphide, chemical oxygen demand, biological oxygen demand, dissolved organic carbon, calcium, and pH;
- TPH CWG;
- USEPA 16 PAH and BTEX.

6.3.6 As discussed in Section 2.11, the Study area is situated above Secondary A Aquifers, there are numerous water courses and springs within the study area, and in addition, there are abstraction licenses within the study area. On this basis leachate results were screened against Freshwater Environmental Quality Standards (FEQS) or UK Drinking Water Standards (UKDWS), whichever was most conservative. In addition, priority hazardous substances will be screened against their laboratory Limit of Detection (LOD). For ambient level concentrations of particular contaminants the catchment area has been defined as the River Fal catchment area. Where hardness dependant FEQS values have been used, in the absence of site specific data the most conservative FEQS values have been adopted. Similarly, where particular FEQS are derived from assessment of site specific calcium and dissolved organic carbon data the relevant bioavailable FEQS values have been adopted and the bioavailable concentrations of contaminants have been calculated.

6.3.7 The results of the leachate screening assessment indicate the following:

- The majority of heavy metals are below the applied screening criteria with the exception of copper, lead, and zinc which are discussed further below.
- Numerous concentrations of PAH compounds are recorded above the laboratory limit of detection which are discussed further below.

- Two samples indicated leachable levels of TPH fractions, this is discussed further below.

6.3.8 Elevated levels of copper were observed in excess of the applied FEQS of 1µg/l, ranging between 2.0µg/l to 22.0µg/l in 13 of 16No. samples of Made Ground, topsoil and weathered bedrock. The highest concentrations were observed in samples recovered from the Made Ground.

6.3.9 Elevated levels of lead were found in excess of the applied FEQS of 1.2µg/l (bioavailable) ranging between 2.0µg/l to 26µg/l in 14 of 16No. samples, 4No.of these samples were from the Made Ground soils while 10No. were from natural soils, including topsoil.

6.3.10 Elevated levels of zinc were found to be in excess of the applied FEQS of 16.7g/l (10.9µg/l+ ambient 5.8µg/l) ranging between 21 and 158µg/l in 4No. samples. 1No. sample was from the Made Ground soils and 3No. were obtained from natural topsoil or residual soils.

6.3.11 Numerous concentrations of PAH compounds have been detected above the LOD of 0.02µg/l. One sample of Made Ground from BH-R-041 at 0.45m bgl showed exceedances in all compounds except acenaphthylene and dibenzo(ah)anthracene. Other exceedances were noted for acenaphthene, acenaphthylene, naphthalene, pyrene, and fluoranthene, and a few exceedances were noted for benzo(a)anthracene, benzo(k)fluoranthene, and fluorene. Review of the soil descriptions for the samples with PAH exceedances does not indicate any obvious source of the PAHs. It is notable that those PAHs with higher molecular weights and/or higher organic carbon partition co-efficients are generally absent from the leachate testing, suggesting a lack of mobility in the soil environment. It is considered that the lack of indefinable source for the PAHs, their presence in the topsoil samples, natural ground samples as well as Made Ground soils suggests that they may be derived from diffuse pollution, possibly from vehicle emissions in association with the nearby highway or from past historic activity in relation to the area's past mining history. It is not considered that the PAH concentrations in the leachate are a significant risk to controlled waters.

6.3.12 The results of the groundwater screening assessment indicate the following:

- Exceedances of the screening criteria for cadmium, copper, chromium, mercury and zinc which are discussed further below.
- Numerous concentrations of PAH compounds are recorded above the laboratory limit of detection in a single sample.
- Three samples indicated levels of TPH fractions above the laboratory limit of detection, this is discussed further below.

6.3.13 A single elevated level of cadmium was observed in a sample from BH-S-005 at a concentration of 0.4µg/l against the FEQS of 0.1µg/l.

6.3.14 Bioavailable assessment of the copper concentrations has indicated that 4No. bioavailable concentrations of copper ranging from 1.25µg/l to 2.54µg/l exceeded the FEQS of 1.0µg/l. These copper exceedances were recorded in samples from BH-S-005, BH-S-019, BH-S-049, and BH-R-027.

6.3.15 Two elevated concentrations of mercury were recorded, one at 15.0µg/l against the FEQS of 0.1µg/l in BH-201, and one at 7.0µg/l in BH-216.

6.3.16 Bioavailable assessment of nickel concentration has indicated a single elevated bioavailable concentration of 4.23µg/l in exceedance of the FEQS of 4.0µg/l from BH-S-042.

6.3.17 Bioavailable assessment of zinc concentrations has indicated 6No. bioavailable concentrations ranging from 13.0µg/l to 45.0µg/l exceed the FEQS of 16.7µg/l. These exceedances were recorded in samples from BH-S-005, BH-R-010a, BH-S-019, BH-R-027, BH-S-042, and BH-216.

6.3.18 Numerous PAH compounds were recorded above the laboratory limit of detection in BH-201.

6.3.19 TPH Aliphatic fractions C_5 to C_6 were recorded at concentrations of between 1.0µg/l and 5.0µg/l in samples from BH-S-005, BH-201, and BH-309. Aliphatic C_6 to C_8 were recorded at concentrations of 5.0µg/l and 6.0µg/l in BH-201 and BH-309 respectively. Aliphatic C_{16} to C_{21} and C_{21} to C_{35} were recorded at concentrations of 41.0µg/l and 78.0µg/l respectively in a sample from BH-S-005.

6.3.20 TPH Aromatic fractions C_7 to C_8 and C_8 to C_9 were recorded at concentrations of 5.0µg/l and 1.0µg/l respectively in a sample from BH-201. Aromatic $C_{16}$to C_{21} was recorded at a concentration of 12.0µg/l in a sample from BH-S-019.

6.3.21 In summary, generally the groundwater chemical analysis has indicated some exceedances of heavy metal concentrations, and generally organic contaminant concentrations below the laboratory limit of detection except for a single sample showing PAHs and TPH and three others showing TPH detections. With regards to the potential sources of heavy metals, review of the locations where exceedances have occurred does not indicate an obvious spatial relationship, some occurrences are noted in boreholes near known former mining areas but at the same time other exceedances are present in boreholes not near known mining features. It is suspected that the heavy metal concentrations may be reflective of typical background concentrations in this area given the past mining history and likely metalliferous mineralisation present in the local geology. In relation to the organic contaminants encountered a review of the location of the boreholes from which exceedances noted do not indicate potential sources for the hydrocarbon contaminants. BH201 showed elevated PAHs and TPH fractions, however the borehole is in an area of open agricultural land, away from likely potential sources of PAH contamination. BH-S-005, BH-S-019, and BH-309 all showed variable levels of TPH fractions, review of their locations also indicated that the exploratory holes were in agricultural areas, away from any development and likely source of hydrocarbon contamination.

6.3.22 It is recommended that a second round of groundwater sampling and analysis is undertaken to determine if the recorded concentrations are repeatable and possibly reflective of a more significant groundwater contamination issue.

7 Geotechnical risk register

7.1.1 Table 7-1 summarises the risks identified for this project in relation to the ground along with a list of proposed mitigation measures. It includes both health & safety and project risks. Risk has therefore not been numerically rated as this would involve two separate classification frameworks.

7.1.2 Refer to the Designer's Risk Register for a full assessment of the health & safety risks on the project. Refer to the Project Risk Register for a full assessment of the programme and cost risks to the project.

7.1.3 Table 7-19 is considered "live" and should continue to be developed as the project progresses. Refer to the latest version at each stage of the project.

Table 7-19 Geotechnical risk register

Risk	Location	Mitigation measures
Instability of cut slopes Adverse geological structure or groundwater conditions could cause localised slope failure.	Cuttings site-wide	Design of cuttings with appropriate conservatism and drainage measures (cut-off ditches at the crest). Construction phase inspections will be required and any suspect areas are to be addressed in line with the proposals of the GDR.
Instability of excavation sides Failure of vertical excavation sides during construction.	Excavations site-wide	Carry out full temporary works design and use of supports and groundwater control measures as required.
Instability of embankments on sidelong ground Soft sloping ground (in particular slopes covered by solifluction deposits) or groundwater conditions provide an increased risk of embankment failure.	Some embankments site-wide, in particular at 13+075 and 13+700	Appropriate embankment design, particularly in terms of drainage. Consider a granular starter layer to control groundwater pressures beneath fill. Excavation and replacement and/or counterfort drainage trenches may be required.
Reuse of site-won material Laboratory testing of a relatively small number of samples cannot reliably determine the suitability or performance of all site-won material and the cut/fill balance may not be realistic. Additional export and import would then be required.	Site-wide	Identification of suitable fill to import as a contingency measure. Identification of opportunities to reuse material that is not of the desired class for construction of embankments etc.
Deterioration of fill material Excavated material is susceptible to changes in moisture content and breakdown through excessive handling.	Site-wide	The construction methodology must include for protection of stockpiles and minimising any adverse impacts of material intended for use as fill.

Risk	Location	Mitigation measures
Groundwater flow into cuttings Excavations below the water table will generate seepage into the cutting, which could cause flooding or internal erosion and eventual failure of the cutting slopes.	Site-wide	Site-wide GI including groundwater monitoring. Appropriate design accounting for the results of the GI. Drainage design, including cut-off ditches at the crests of cuttings to prevent surface water flowing in from surrounding areas. Monitoring during construction phase to confirm flows and contingency measures put in place to deal with higher flows than expected.
Differential settlement between structures and adjacent earthworks Noticeable bumps can develop where embankments meet new or existing bridges for example.	Numerous locations	Earthworks to be constructed according to series 600 of the SHW and best practice. Particular consideration to be given at interfaces and how to account for or remediate differential settlement between embankments and structures.
Unknown ground conditions and hydrogeological regime at flooded quarry Poorer ground than expected causing slope failure or preferential flow paths from the quarry to the adjacent cutting causing flooding or internal erosion and eventual failure of the cutting slopes.	Ch.12+700m to Ch.12+900m	GI is required at this location to confirm the ground conditions. Water monitoring is currently being undertaken to determine the variation and driving factors of the water level in the quarry.
Locally soft soils Superficial Deposits or more highly weathered zones of bedrock pose a risk in terms of stability and settlement (particularly differential). Creep settlements may extend this risk into the operational phase.	Site-wide	Inspection by a competent geotechnical engineer during construction to identify unpredicted soft spots in founding material.
Variable weathering profile and depth to rockhead Variable ground conditions within the same rock/soil type could produce excessive differential settlements of structures	Structures	Inspection during construction to confirm assumptions made during detailed design. Contingency measures such as removal and replacement to be applied if e.g. the two foundations of a bridge appear to be on different materials.
Mining features A number of high and medium risk areas have been identified under or near the proposed alignment and other unexpected workings may exist. These could cause sudden large settlements through subsidence or even plant or structures falling into voids.	Numerous (refer to geotechnical features plan in Appendix D and the results of geophysical investigations in Appendix E)	Multiple phases of mining investigation have been undertaken to identify areas where this is a risk. Geophysical surveys have been undertaken to provide further information on the level of risk at each area. Further studies/ground truthing required at certain geophysical anomalies. Probing required between certain features and the proposed road to prove absence of voids. Possible probing at set foundation locations at construction phase to identify any voids.

Risk	Location	Mitigation measures
Locally hard bedrock Unexpectedly competent material might cause excavation to require larger plant and longer programmes	Cuttings and excavations site-wide	Assessment of excavatability based on rock mass properties in various locations. Possible excavatability tests in locations of deep cuts. Programme and plant allowances for digging trenches for services and/or drainage through hard material.
Contaminated ground Construction workers' health may be affected by contamination, particularly where it is unexpected (previously not identified) and suitable protection measures have therefore not been enforced.	Site-wide	Contractor to undertake appropriate H&S risk assessment and to enforce controls for identifying and reacting to unexpected contamination by the application of appropriate mitigation measures.
Aggressive ground conditions Damage to buried concrete due to more aggressive ground than expected	Site-wide	Interpretation of BRE suite test results both generally for each material and location-specific at concrete structures where possible. Specification of suitable concrete according to BRE Special Digest 1:2005.
Buried obstructions Unexpected features in the ground that must either be protected or cannot be excavated	Site-wide	Major services are to be identified during the process and designed for. The contractor must allow for suitable contingency (and must have safe systems of work in place) for dealing with unexpected obstructions.
Buried abandoned oil pipeline Presence of unused pipeline in the ground crossing the route	Ch.11+700m and Ch.12+200	Obstruction to be removed or protected during construction with measures deemed appropriate by the contractor.
Proximity of high pressure gas line Risk of damage if not located accurately or conversely an unnecessary impact on the route	Ch.5+100m to Ch.6+300m and Ch.12+920m to Ch.13+450m	Pipeline location to be confirmed and protected during works.

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Appendix A Plate load test results

Table 8-20 Summary of plate load test results

Trial pit	Depth (m)	Geology	CBR (%)
TP-363	0.5	Gmp-Grade 4 (FINE)	3.8
TP-R-013	0.5	Ptn-Grade 4 (COARSE)	13
TP-202	0.5	Ptn-Grade 4 (COARSE)	8
TP-202	0.5	Ptn-Grade 4 (COARSE)	6.2
TP-205	0.5	Ptn-Grade 4 (COARSE)	3.1
TP-205	0.5	Ptn-Grade 4 (COARSE)	8
TP-208	0.5	Ptn-Grade 4 (FINE)	7.8
TP-208	0.5	Ptn-Grade 4 (FINE)	3.9
TP-211	0.5	Ptn-Grade 4 (COARSE)	1.9
TP-211	0.5	Ptn-Grade 4 (COARSE)	3.8
TP-214	0.5	Ptn-Grade 5 (FINE)	2.2
TP-214	0.5	Ptn-Grade 5 (FINE)	3.2
TP-216	0.5	Ptn-Grade 4 (COARSE)	7.6
TP-216	0.5	Ptn-Grade 4 (COARSE)	2.5
TP-218	0.5	Ptn-Grade 4 (COARSE)	7.6
TP-218	0.5	Ptn-Grade 4 (COARSE)	5.4
TP-219	0.5	Made Ground	3.4
TP-219	0.5	Made Ground	3.1
TP-224	0.9	Ptn-Grade 5 (FINE)	3.9
TP-224	0.9	Ptn-Grade 5 (FINE)	2.5
TP-231	0.5	Gmp-Grade 4 (COARSE)	8.4
TP-231	0.5	Gmp-Grade 4 (COARSE)	4.5
TP-353	0.5	Ptn-Grade 4 (COARSE)	3.2
TP-353	0.5	Ptn-Grade 4 (COARSE)	3.4
TP-354	0.5	Ptn-Grade 4 (COARSE)	5.6
TP-354	0.5	Ptn-Grade 4 (COARSE)	3.4
TP-355	0.5	Gmp-Grade 4 (FINE)	3.9
TP-355	0.5	Gmp-Grade 4 (FINE)	3.3
TP-359	0.5	Gmp-Grade 4 (COARSE)	4.9
TP-359	0.5	Gmp-Grade 4 (COARSE)	11
TP-360	0.5	Gmp-Grade 4 (COARSE)	5.2
TP-360	0.5	Gmp-Grade 4 (COARSE)	3.1
TP-361	0.5	Gmp-Grade 4 (COARSE)	4
TP-361	0.5	Gmp-Grade 4 (COARSE)	4.5
TP-362	0.5	Gmp-Grade 4 (COARSE)	4.2
TP-362	0.5	Gmp-Grade 4 (COARSE)	3.2
TP-363	0.5	Gmp-Grade 4 (FINE)	6.9
TP-R-003	0.3	Topsoil	2.2
TP-R-003	0.3	Topsoil	1.6
TP-R-012	0.5	Ptn-Grade 5 (FINE)	3.3
TP-R-012	0.5	Ptn-Grade 5 (FINE)	4.1

Trial pit	Depth (m)	Geology	CBR (%)
TP-R-014	0.5	Ptn-Grade 4 (COARSE)	6.7
TP-R-014	0.5	Ptn-Grade 4 (COARSE)	7.4
TP-R-016	0.3	Ptn-Grade 5 (FINE)	3.5
TP-R-016	0.3	Ptn-Grade 5 (FINE)	2.1
TP-R-018	0.5	Ptn-Grade 5 (FINE)	14
TP-R-018	0.5	Ptn-Grade 5 (FINE)	4.9
TP-R-021	0.5	Ptn-Grade 5 (COARSE)	8.6
TP-R-021	0.5	Ptn-Grade 5 (COARSE)	18
TP-R-028	0.5	Ptn-Grade 5 (FINE)	3
TP-R-028	0.5	Ptn-Grade 5 (FINE)	8
TP-R-039	0.5	Ptn-Grade 5 (COARSE)	4.3
TP-R-039	0.5	Ptn-Grade 5 (COARSE)	7.4
TP-R-041	0.5	Topsoil	7.4
TP-R-041	0.5	Topsoil	3.8
TP-R-043	0.5	Ptn-Grade 5 (COARSE)	6.4
TP-R-043	0.5	Ptn-Grade 5 (COARSE)	3.6
TP-R-045	0.5	Topsoil	6.6
TP-R-045	0.5	Topsoil	7.4
TP-R-065	0.5	Topsoil	2.1
TP-R-065	0.5	Topsoil	1.5
TP-R-069	0.5	Gmp-Grade 5 (COARSE)	2.4
TP-R-069	0.5	Gmp-Grade 5 (COARSE)	17
TP-R-070	0.5	Gmp-Grade 5 (FINE)	2.1
TP-R-070	0.5	Gmp-Grade 5 (FINE)	3.6
TP-R-075	0.5	Gmp-Grade 4 (COARSE)	3.1
TP-R-075	0.5	Gmp-Grade 4 (COARSE)	6.4
TP-R-077	0.5	Topsoil	5.1
TP-R-077	0.5	Topsoil	2.3
TP-R-079	0.5	Gmp-Grade 4 (COARSE)	4.8
TP-R-079	0.38	Topsoil	8.4
TP-R-082	0.5	Gmp-Grade 5 (COARSE)	8.8
TP-R-082	0.5	Gmp-Grade 5 (COARSE)	6.8
TP-R-084	0.5	Gmp-Grade 5 (COARSE)	4.2
TP-R-084	0.5	Gmp-Grade 5 (COARSE)	13
TP-R-094	0.5	Trd-Grade 5 (COARSE)	5.9
TP-R-095	0.5	Trd-Grade 5 (COARSE)	6.4
TP-R-025	0.5	Ptn-Grade 5 (COARSE)	5.7
TP-R-025	0.5	Ptn-Grade 5 (COARSE)	11
TP-S-002	0.5	Ptn-Grade 5 (FINE)	2.2
TP-S-002	0.5	Ptn-Grade 5 (FINE)	1.5
TP-S-004	0.5	Ptn-Grade 4 (FINE)	11
TP-S-004	0.5	Ptn-Grade 4 (FINE)	10

Appendix B Soakaway test results

Table 8-21 Summary of soakaway test results

Trial pit	Depth (m)	Geology	Soil infiltration rate (m/s)	Comments
TP-201	2.80	Ptn-Grade 3	6.99×10^{-5}	Non standard
TP-201	2.64	Ptn-Grade 3	2.61×10^{-5}	Non standard
TP-210	2.88	Ptn-Grade 2	2.10×10^{-4}	
TP-210	2.69	Ptn-Grade 2	1.79×10^{-4}	
TP-210	2.56	Ptn-Grade 2	1.62×10^{-4}	
TP-212	2.39	Ptn-Grade 2	1.74×10^{-4}	
TP-212	2.24	Ptn-Grade 3	1.37×10^{-4}	
TP-212	2.19	Ptn-Grade 3	7.93×10^{-5}	
TP-220	2.51	Ptn-Grade 3	3.95×10^{-4}	
TP-220	2.38	Ptn-Grade 3	3.09×10^{-4}	
TP-220	2.29	Ptn-Grade 3	3.62×10^{-4}	
TP-222	2.59	Ptn-Grade 2	2.12×10^{-3}	
TP-222	2.39	Ptn-Grade 2	2.11×10^{-3}	
TP-222	2.31	Ptn-Grade 2	2.33×10^{-3}	
TP-233	3.01	Gmp-Grade 5 (FINE)	2.63×10^{-5}	Non standard
TP-235	2.80	Gmp-Grade 4 (FINE)	1.95×10^{-6}	Non standard
TP-239	0.84	Gmp-Grade 2	2.14×10^{-5}	Non standard
TP-239	0.83	Gmp-Grade 2	1.86×10^{-5}	Non standard
TP-240	2.52	Gmp-Grade 2	2.39×10^{-3}	
TP-240	2.33	Gmp-Grade 2	1.59×10^{-3}	
TP-240	2.26	Gmp-Grade 2	1.78×10^{-3}	
TP-356	2.45	Gmp-Grade 2	6.48×10^{-4}	
TP-356	2.26	Gmp-Grade 3	4.00×10^{-4}	
TP-356	2.16	Gmp-Grade 3	4.53×10^{-4}	
TP-357B	2.55	Gmp-Grade 4 (COARSE)	1.94×10^{-4}	
TP-357B	2.38	Gmp-Grade 4 (COARSE)	1.25×10^{-4}	
TP-357B	2.29	Gmp-Grade 4 (COARSE)	1.13×10^{-4}	
TP-363B	3.00	Made Ground	5.82×10^{-6}	Non standard
TP-365	2.69	Trd-Grade 4 (COARSE)	3.52×10^{-4}	Non standard
TP-365	2.60	Trd-Grade 4 (COARSE)	2.02×10^{-5}	Non standard
TP-P-001	1.81	Ptn-Grade 5 (COARSE)	1.66×10^{-4}	
TP-P-001	1.71	Ptn-Grade 5 (COARSE)	1.52×10^{-4}	
TP-P-001	1.63	Ptn-Grade 5 (COARSE)	1.40×10^{-4}	
TP-P-004	2.16	Ptn-Grade 5 (COARSE)	3.71×10^{-4}	
TP-P-004	2.16	Ptn-Grade 5 (COARSE)	2.76×10^{-4}	
TP-P-004	2.09	Ptn-Grade 5 (COARSE)	3.84×10^{-4}	
TP-P-005	2.17	Not recorded	3.02×10^{-4}	
TP-P-005	2.11	Not recorded	2.78×10^{-4}	
TP-P-005	2.05	Not recorded	2.96×10^{-4}	
TP-P-009	2.83	Ptn-Grade 4 (COARSE)	N/A	Non standard

Trial pit	Depth (m)	Geology	Soil infiltration rate (m/s)	Comments
TP-P-013	2.32	Ptn-Grade 4 (COARSE)	1.63×10^{-5}	Non standard
TP-P-014	1.56	Gmp-Grade 4 (FINE)	N/A	Non standard
TP-P-015	1.18	Gmp-Grade 2	6.24×10^{-5}	
TP-P-015	1.18	Gmp-Grade 2	3.65×10^{-5}	
TP-P-015	1.12	Gmp-Grade 2	3.59×10^{-5}	
TP-P-017	1.26	Gmp-Grade 3	3.70×10^{-3}	
TP-P-017	1.17	Gmp-Grade 3	2.92×10^{-3}	
TP-P-017	1.16	Gmp-Grade 3	2.29×10^{-3}	

Appendix C Water monitoring data and interpretation

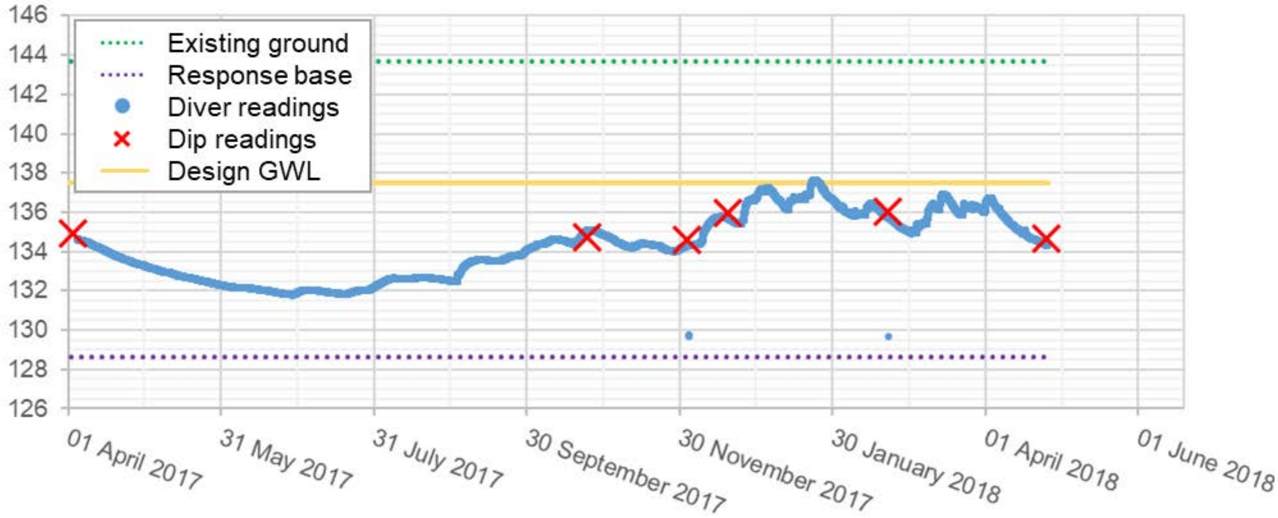

Figure 8-1 BH-S-005 (Ch.0+990m) design groundwater level 137.50mAOD

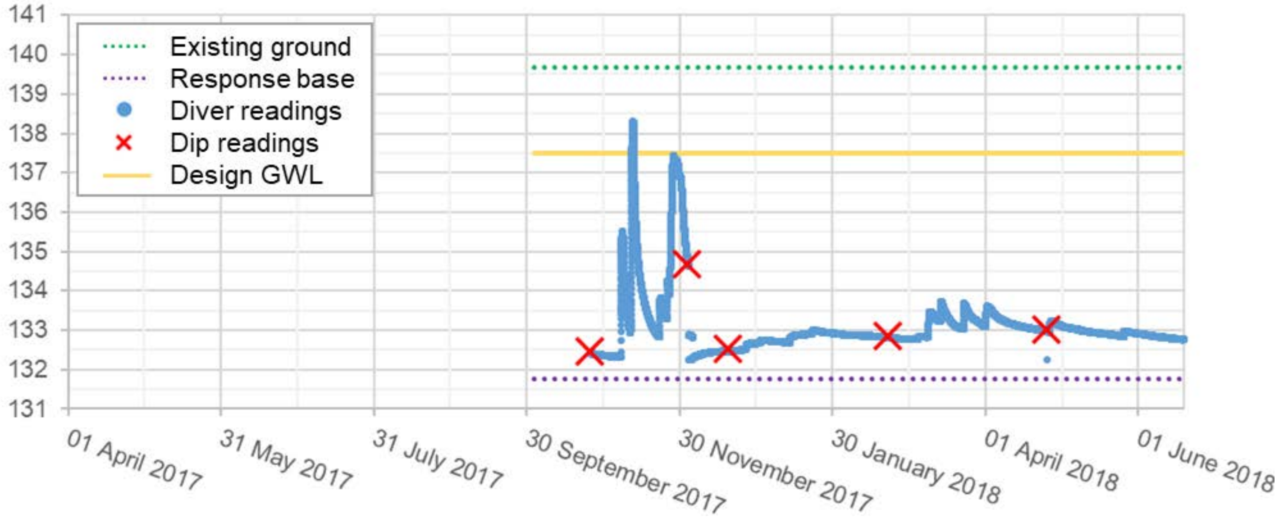

Figure 8-2 BH-201 (Ch.1+405m) design groundwater level 137.50mAOD

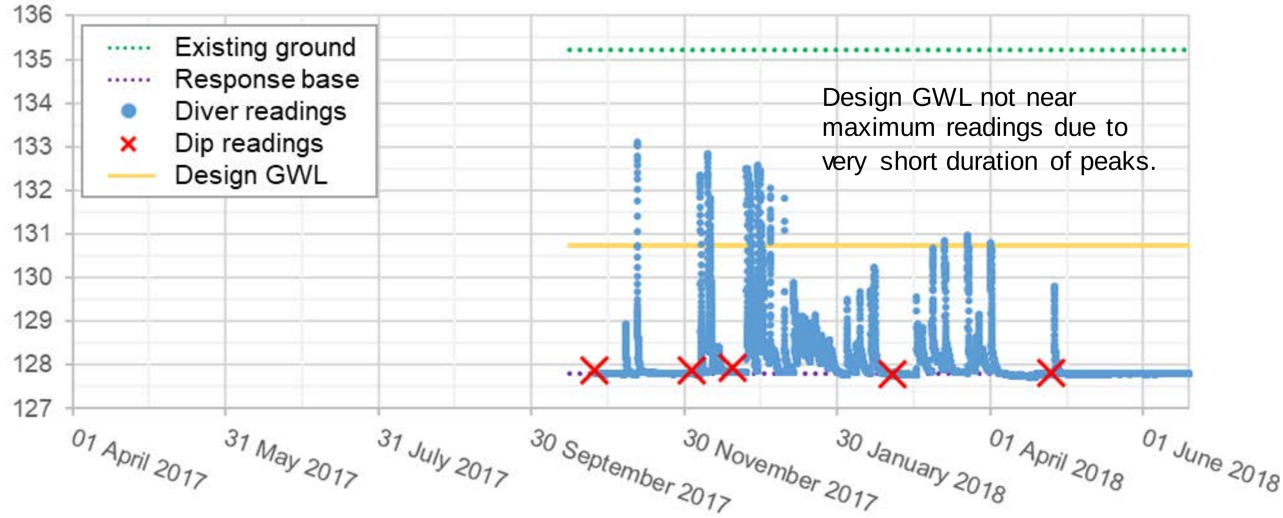

Figure 8-3 BH-207 (Ch.1+540m) design groundwater level 130.75mAOD

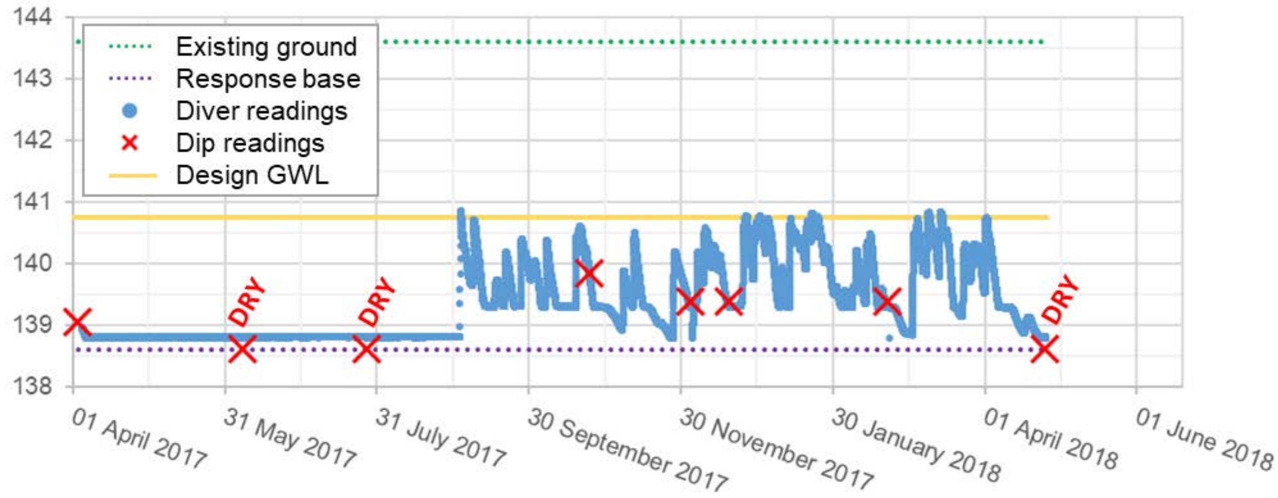

Figure 8-4 BH-R-004 (Ch.2+920m) design groundwater level 140.75mAOD

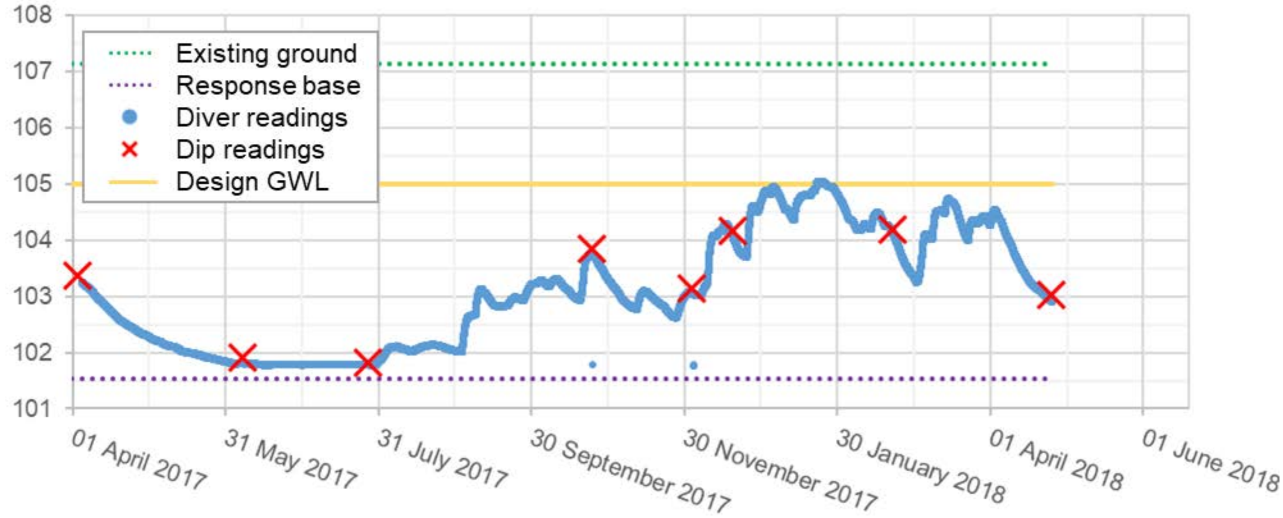

Figure 8-5 BH-R-010 (Ch.3+970m) design groundwater level 105.00mAOD

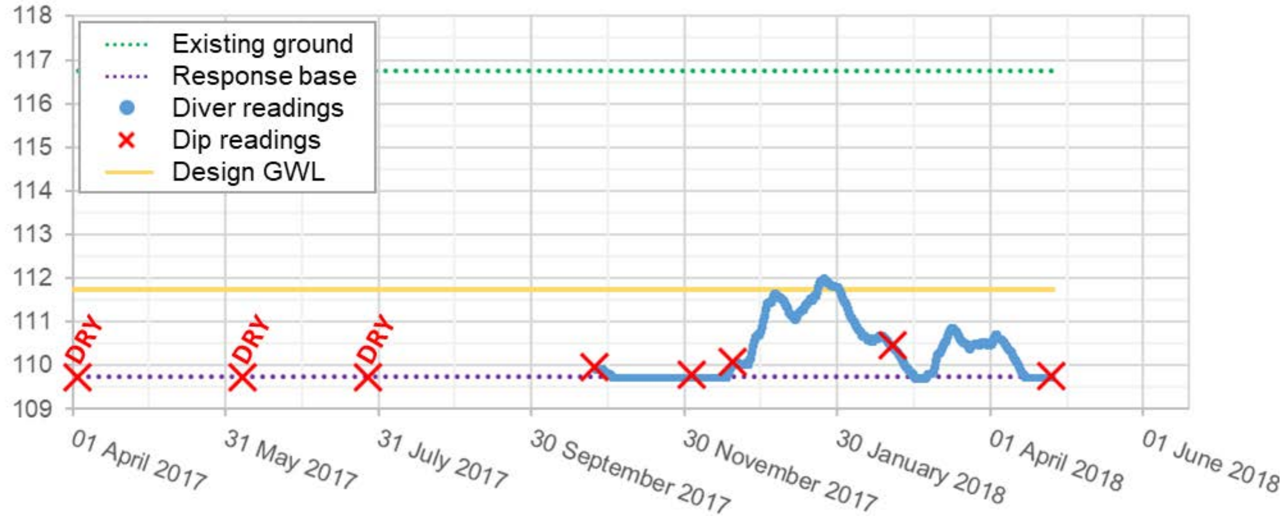

Figure 8-6 BH-S-012 (Ch.4+830m) design groundwater level 111.75mAOD

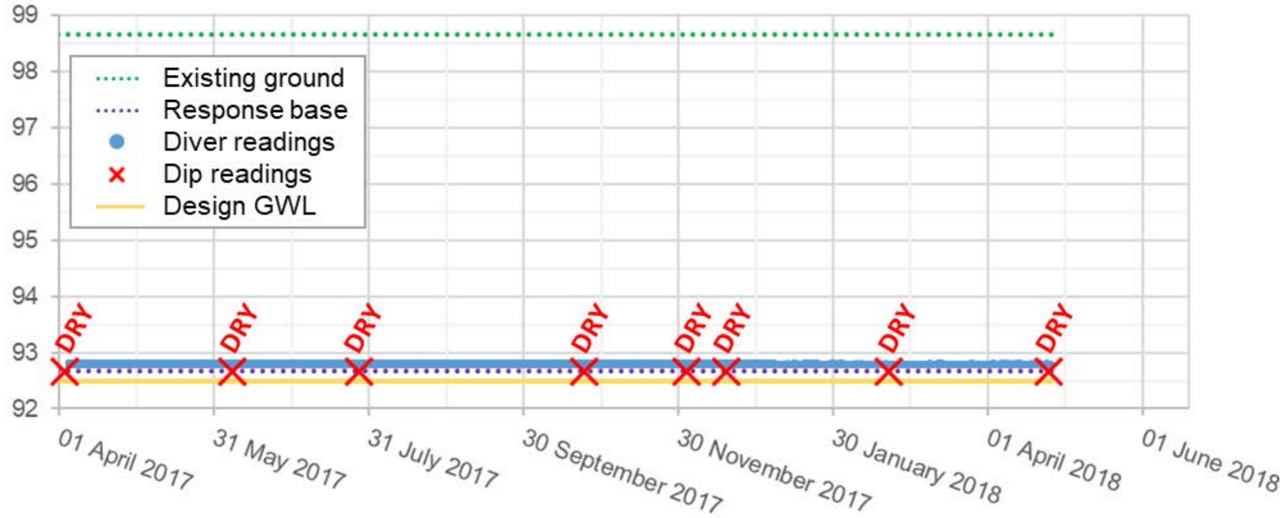

Figure 8-7 BH-R-013 (Ch.5+815m) design groundwater level 92.50mAOD

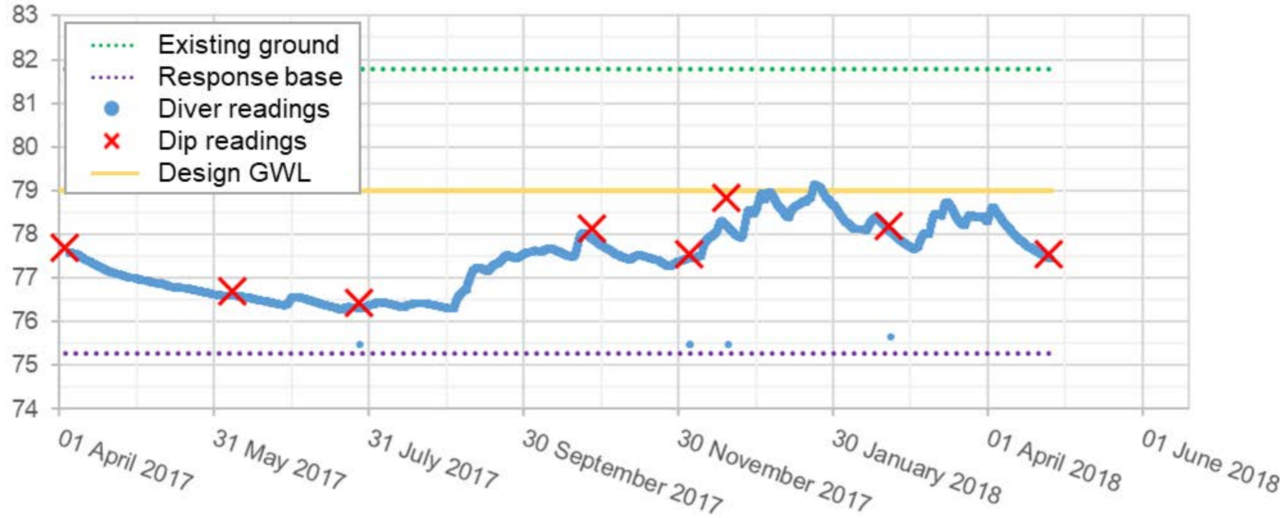

Figure 8-8 BH-S-019 (Ch.5+990m) design groundwater level 79.00mAOD

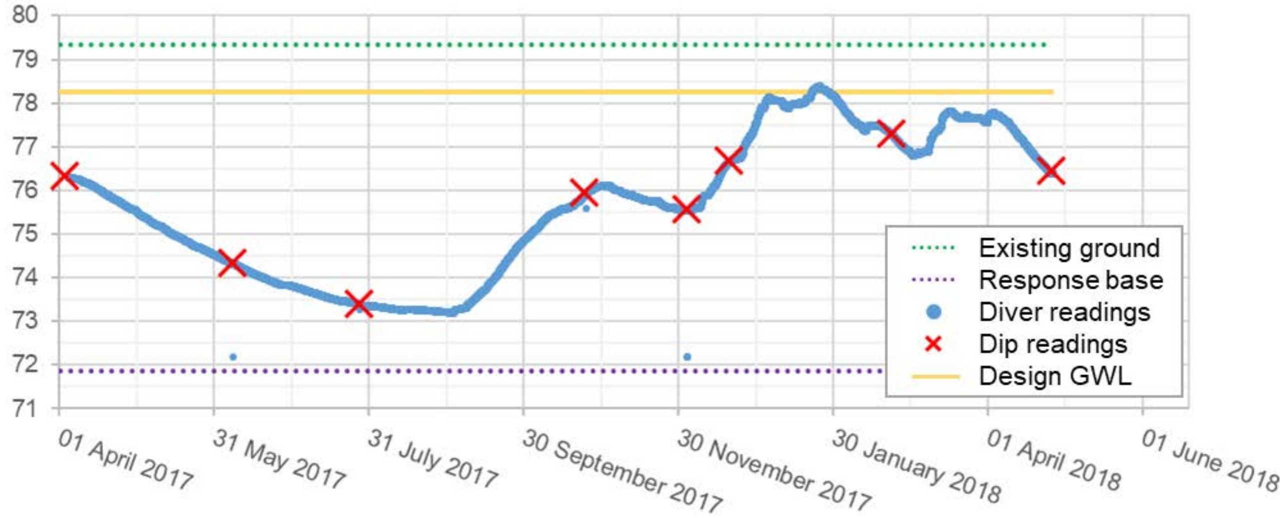

Figure 8-9 BH-R-017 (Ch.7+095) design groundwater level 78.25mAOD

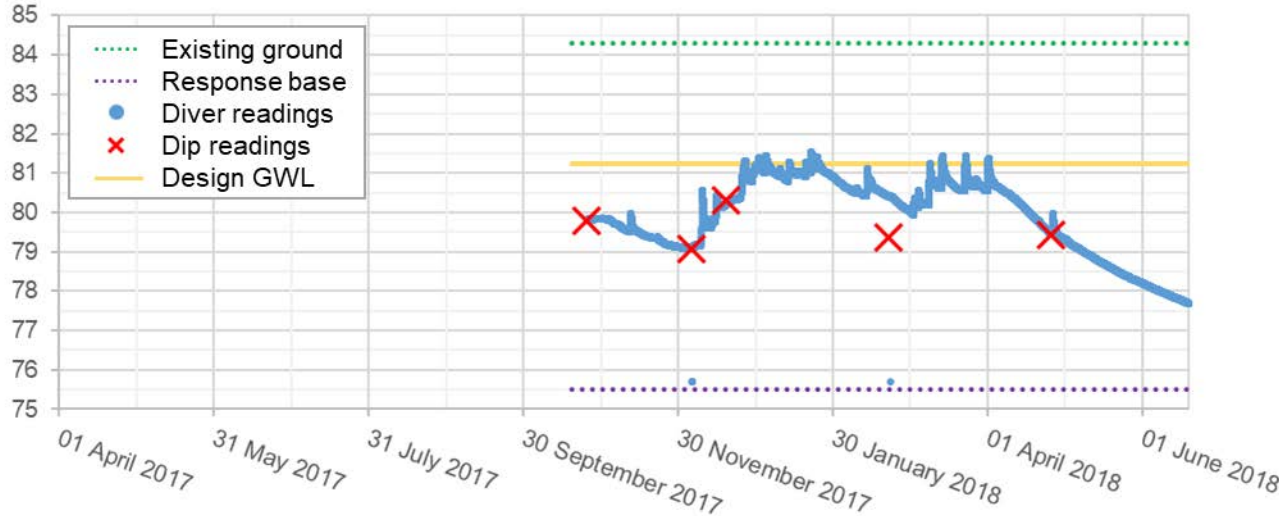

Figure 8-10 BH-303 (Ch.7+315m) design groundwater level 81.25

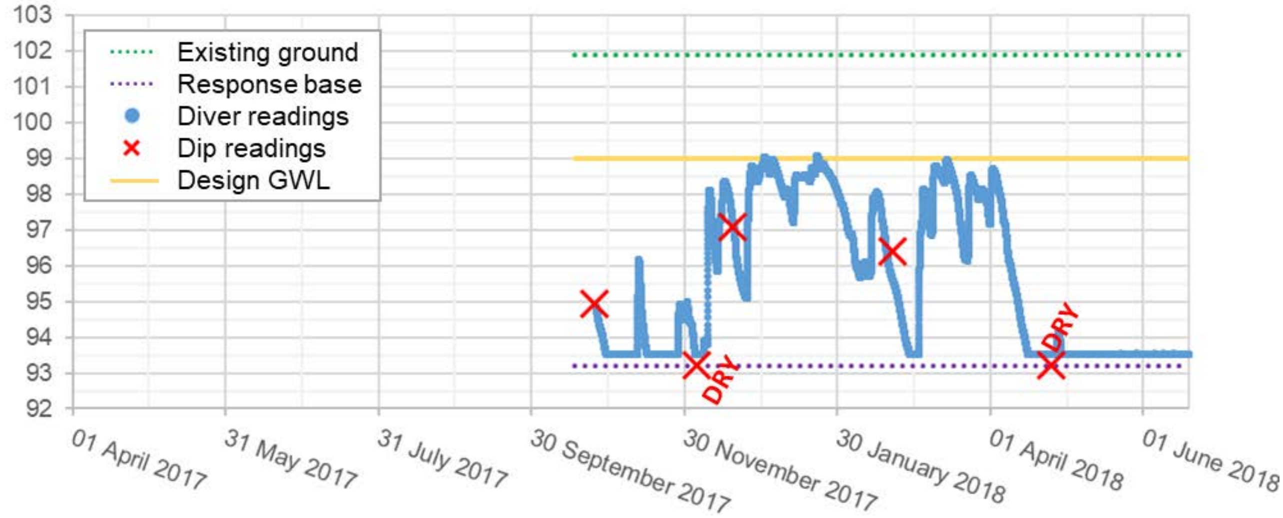

Figure 8-11 BH-213 (Ch.7+670m) design groundwater level 99.00mAOD

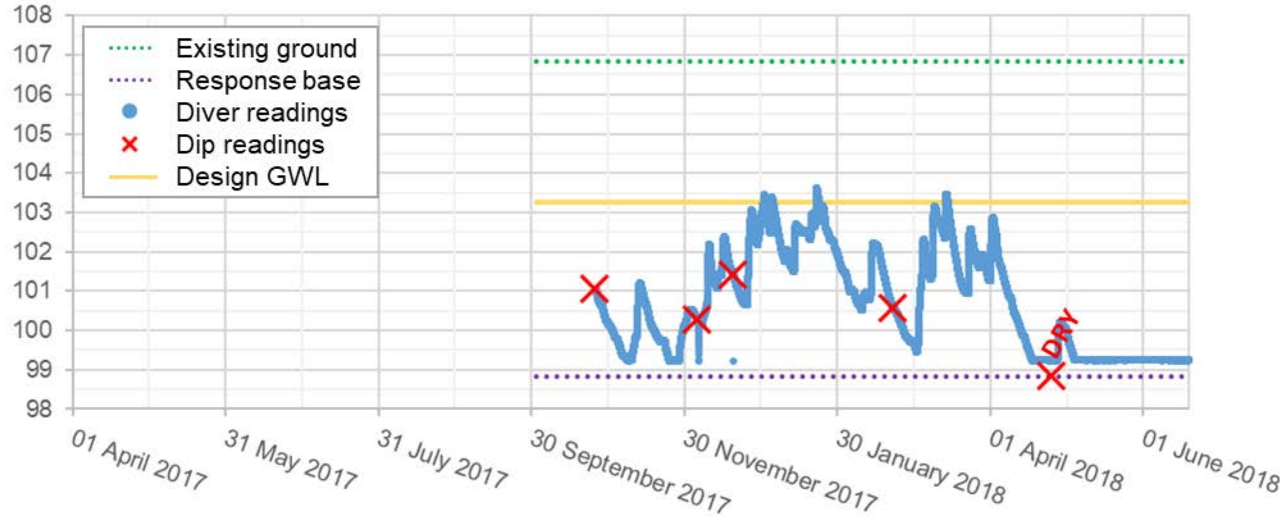

Figure 8-12 BH-216 (Ch.8+315m) design groundwater level 103.25mAOD

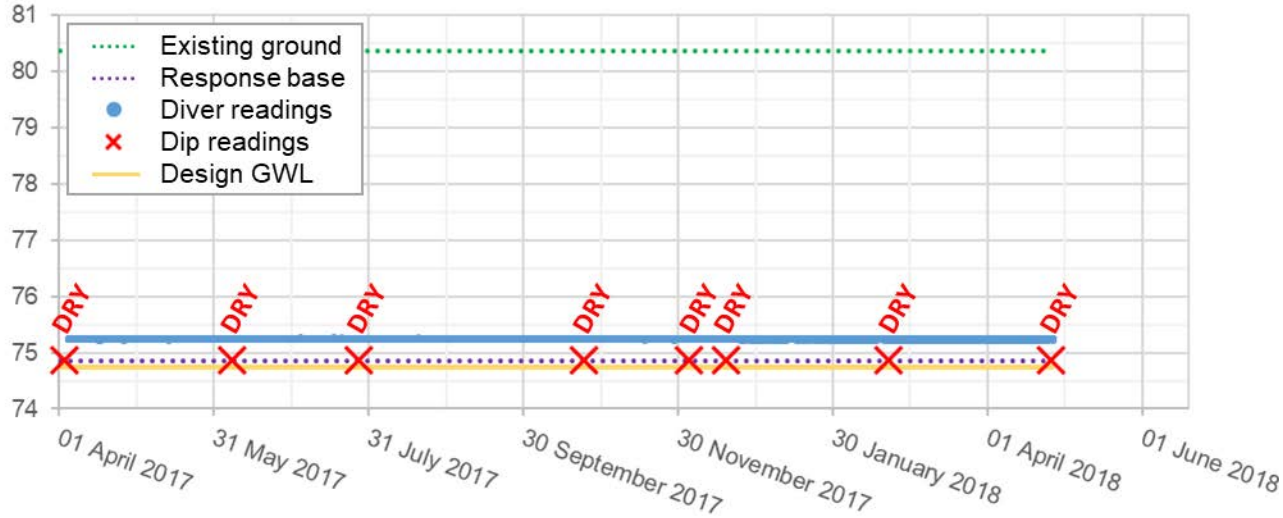

Figure 8-13 BH-S-032 (Ch.8+680m) design groundwater level 74.75mAOD

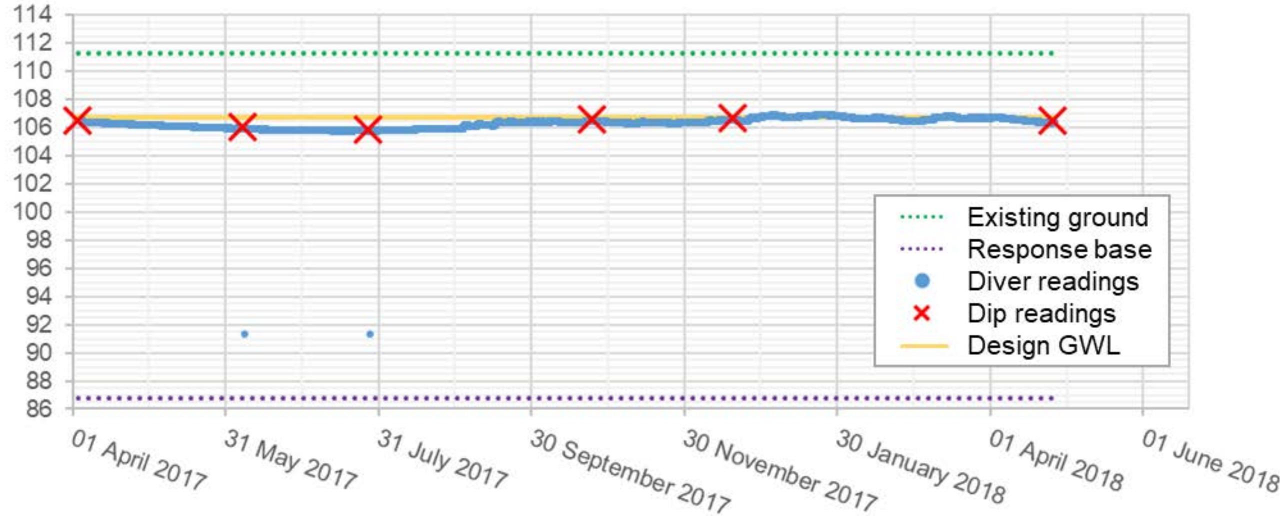

Figure 8-14 BH-S-036 (Ch.10+995m) design groundwater level 106.75mAOD

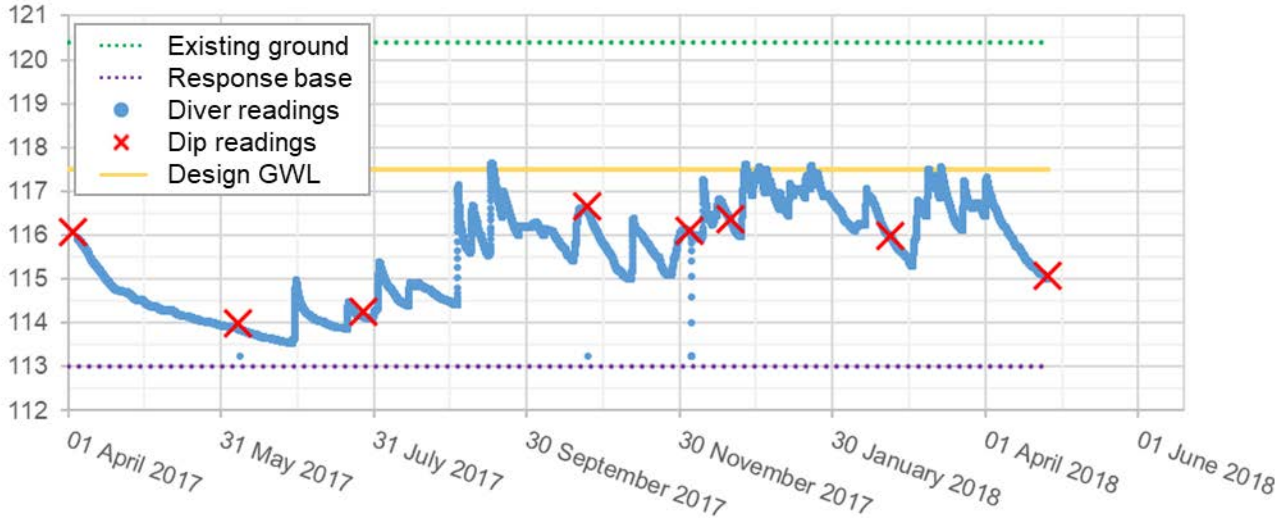

Figure 8-15 BH-R-027 (Ch.11+385m) design groundwater level 117.50mAOD

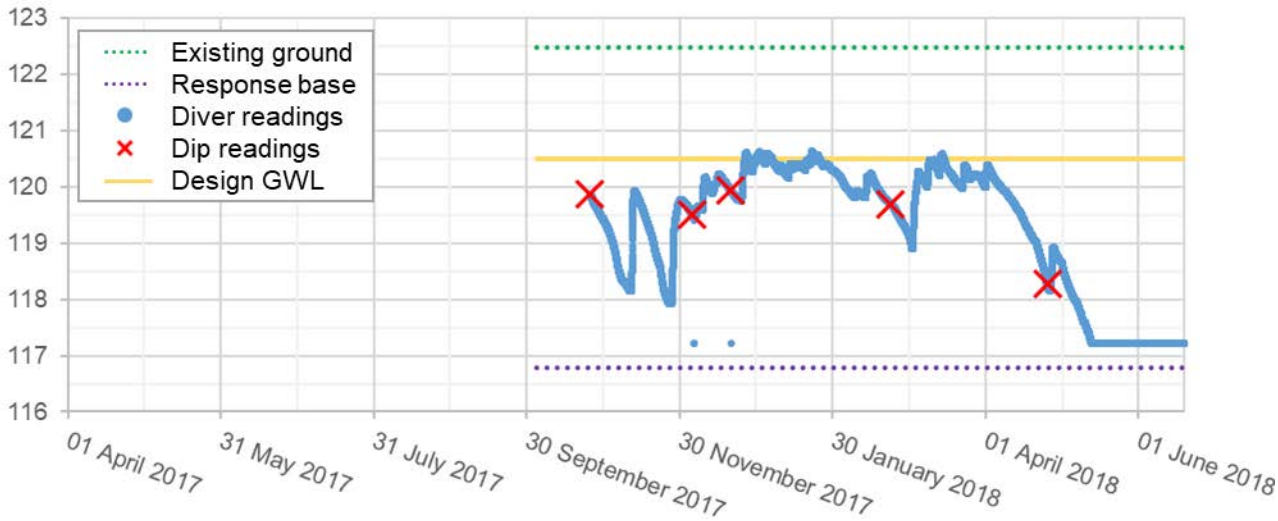

Figure 8-16 BH-309 (Ch.11+520m) design groundwater level 120.50mAOD

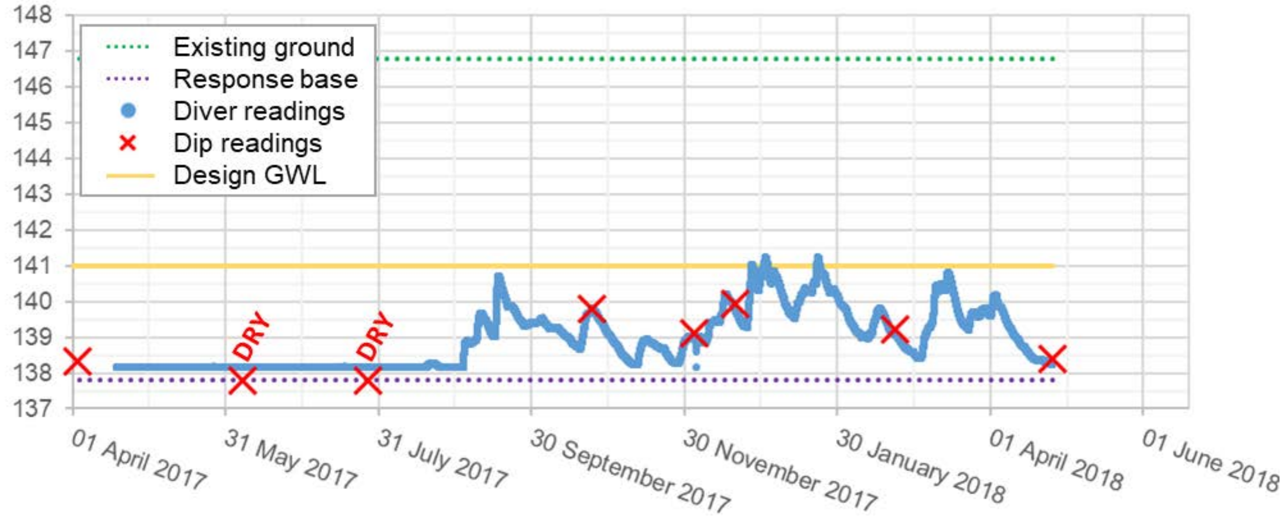

Figure 8-17 BH-S-042 (Ch.12+865m) design groundwater level 141.00mAOD

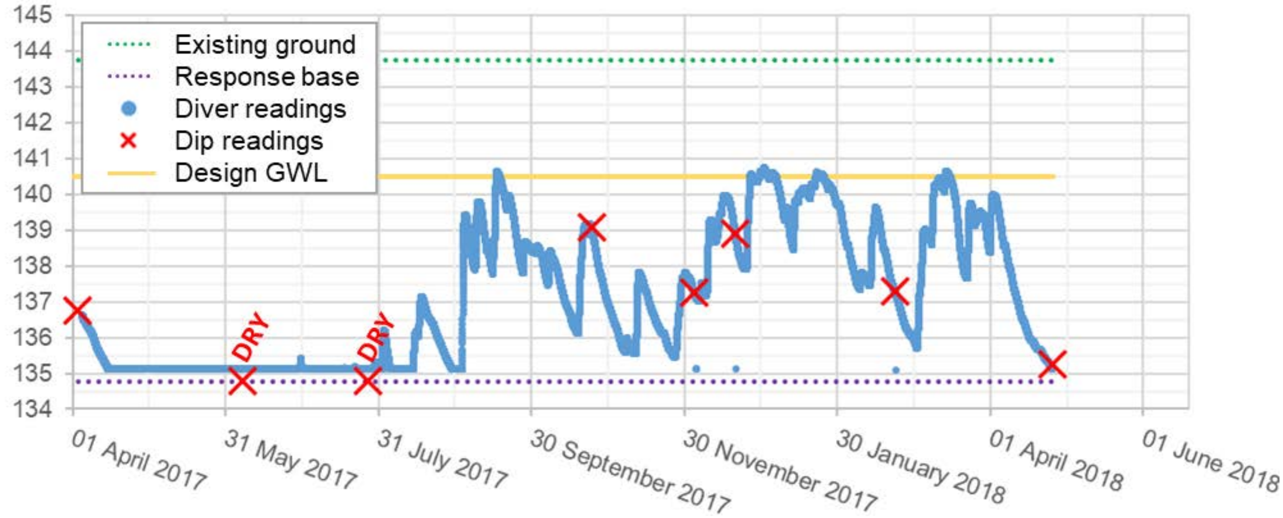

Figure 8-18 BH-S-049 (Ch.13+240m) design groundwater level 140.50mAOD

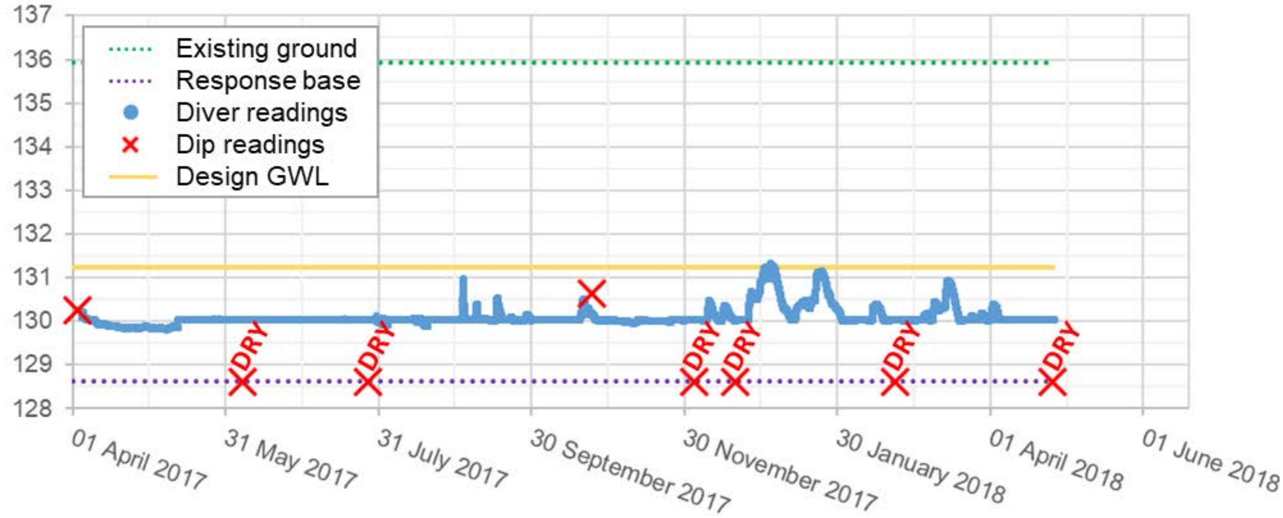

Figure 8-19 BH-R-041 (Ch.14+020m) design groundwater level 131.25mAOD

Appendix D Geotechnical features plan

D.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

Temple Quay House

2 The Square

Temple Quay

Bristol, BS1 6HA

By Telephone: 0300 123 5000

Appendix E Long section

E.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

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Appendix F Geophysical survey features plan

F.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

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Appendix G Aerial photography interpretation report

G.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

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Appendix H Cornwall Consultants metalliferous minerals mining search

H.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

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Appendix I Structural Soils ground investigation factual report

I.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

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Appendix J SOCOTEC ground investigation factual report

J.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

Temple Quay House

2 The Square

Temple Quay

Bristol, BS1 6HA

By Telephone: 0300 123 5000

Appendix K WPD Pylon A101 Retaining Structure Options Report

K.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England
Temple Quay House
2 The Square
Temple Quay
Bristol, BS1 6HA

By Telephone: 0300 123 5000

Appendix L Round Barrow Retaining Structure Options Report

L.1 Available on request from Highways England

Highways England can be contacted:

By Email: A30ChivertontoCarlandCross@Highwaysengland.co.uk

In Writing: Highways England

Temple Quay House

2 The Square

Temple Quay

Bristol, BS1 6HA

By Telephone: 0300 123 5000

If you need help accessing this or any other Highways England information, please call **0300 123 5000** and we will help you.

Creative M170036