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International Advanced Manufacturing Plant

IAMP ONE
Ground Investigation Report (GIR)

Sunderland City Council and South Tyneside Council

Project number: 60283414
M015.003_001

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1. Executive Summary

Sunderland City Council and South Tyneside Council propose to construct the International Advanced Manufacturing Park (IAMP) close to the A19(T) and A1(M). The whole of the site IAMP site is spread across approximately 150 hectares. AECOM has been commissioned to produce a Ground Investigation Report (GIR) and provide Preliminary Engineering Assessment for the first phase of the development only, identified as IAMP ONE, which covers the southern part of the overall site area.

Geological mapping indicates that the assessment area is underlain by superficial deposits identified as the Pelaw Clay Member underlain by soils of the Tyne and Wear Complex. The Tyne and Wear Complex locally rests on bedrock or overlies a lower glacial till. Bedrock comprises the Pennine Upper Coal Measures and the Pennine Middle Coal Measures Formations. Geological maps sheets identify the presence of buried valley channels to the south and north of the overall IAMP site and a dendritic network of main buried valleys, one of which is present immediately north of the River Wear, south of the assessment area. A tributary to this buried valley channel trends north west–south east and crosses through the whole of the Phase ONE site area. Desk study information suggests that variable drift deposits and variable depths to rockhead across the site are anticipated. Significant depth of superficial soils is anticipated to be present in the centre of the infilled glacial valleys.

Preliminary ground investigation for the whole IAMP site has been completed by specialist contractor Dunelm Geotechnical and Environmental Ltd (Dunelm) between 21st July and 29th November 2017. This GIR provides a review of the results from the ground investigation and an interpretation of the ground conditions encountered for ground investigation works within the IAMP ONE site boundary. Geotechnical parameters and material properties of the strata encountered are provided to inform an overall assessment of the ground conditions for the purposes of preliminary engineering assessment. This GIR is intended to provide geotechnical support to a number of technical specialists who are developing the site layout, drainage, pavement and structure foundation designs and aid their assessment of design risks and construction costs for the scheme.

Ground investigation has proved the anticipated geological conditions. Topsoil (e1) or localised generally thin made ground (d1 to d5) was underlain by generally soft and firm Pelaw Clay (b5). The Pelaw Clay is underlain by lacustrine deposits of the Tyne and Wear Complex comprising firm becoming stiff with depth laminated clay (b2) interbedded with bands of sands (b3) or silt (b4). The Tyne and Wear Complex is underlain by a stiff and very stiff over-consolidated Lower Glacial Till (b1) which is inferred to represent the diamicton of the Wear Till Formation. Ground investigation has proved the presence of the infilled glacial valley with superficial soils noted to thicken markedly towards the north east across the centre of the site. Superficial soils are underlain by rocks of the Pennine Upper and Middle Coal Measures Formations which comprised an interbedded sequence of sandstone (a1), mudstone (a2), siltstone (a3) and coal (a4).

Preliminary engineering assessment and a geotechnical risk register is provided and indicates the following:

- Ground aggressivity testing indicates all soils at the site to have an ACEC class of DS-1 AC-1. Data indicates concrete attack due to high soils/ groundwater aggressivity is generally low and may be mitigated by adopting appropriate concrete classification.
- Ground investigation has not identified worked seams below most of the site. However, the ground conditions proved in BH's 28 and 30 shows coal to be interbedded with layers of soft to firm gravelly clay. Therefore, the possibility of unrecorded coal workings, particularly on the eastern portion of the site in the vicinity of Units 4, 5 and 6 cannot be ruled out based on the preliminary information obtained. Further ground investigation to assess this risk to proposed structures is recommended as part of further phases of development and detailed design.
- Earthworks should be undertaken in accordance with the HASHW Series 600 Earthworks Specification. Shallow earthworks are most likely to be within the Pelaw Clay (b5) which was widely encountered below topsoil across most of the site. Given the high plasticity of the Pelaw Clay (b5), careful consideration will be required in assessment of these soils particularly for use in low cutting or embankment construction. A maximum slope angle of 1 vertical to 3.5 horizontal (1V:3.5H) is considered appropriate for the purposes of preliminary design and assessment of earthwork balance quantities.

- Re-use of Pelaw Clay (b5) in bulk earthworks particular in the construction of shallow embankments may be problematic due to the low undrained shear strength of the soils.
- Shallow groundwater is at or close to ground surface, resulting in the potential for surface water flooding generated from groundwater.
- Soakaway testing has shown the Pelaw Clay (b5) to be of low permeability. Groundwater monitoring has shown equilibrium water levels to be at or close to ground level; this combined with the measured permeability indicates that soakaways are not suitable for the discharge of highway and development surface runoff as part of scheme drainage proposals.
- Long term uplift pressures may be generated by pore water pressures within soils constrained beneath the proposed pond base/ liners and provision of a permanent thickened/ deepened cover layer may need to be considered.
- Constraints in forming temporary and permanent excavations (e.g. basements) on site due to the presence of potentially fissured soil (Pelaw Clay (b5)) and shallow groundwater. With the exception of potential constraints in forming excavations, it is not anticipated that there will be any unusual geotechnical constraints affecting service installations.
- Assuming average construction conditions and a high water table, a CBR value of <2.5% should be assumed for construction costing and pavement design. Where a subgrade has a CBR lower than 2.5% it is considered unsuitable support for a pavement foundation and must be permanently improved.
- Lightly loaded structures may be founded on shallow spread foundations or raft foundations bearing within the natural succession below any made ground (d1-d5) and below the depth of influence of any seasonal, climatic or vegetation effects.
- Given the thickness of soft and firm Pelaw Clay (b5), the underlying soft and firm laminated clays (b2), and firm and stiff glacial till (b1) over carboniferous bedrock, for larger heavily loaded structures piled foundations are proposed. Variation in rockhead level below the proposed development units across the infilled buried glacial valley has been proved by ground investigation. Allowance would need to be made for varying pile lengths and pile capacity across the footprint of proposed structure units.
- Floor slabs subject to higher loads or stringent serviceability limits may need to be piled to carry the loads into most competent strata underlying the site.
- Unexploded ordnance (UXO) remains a risk in areas previously identified by UXO survey by others and this risk should be included in the Detailed Design and Construction Risk Registers.

Environmental assessment and risks identified are summarised below:

- The ground investigation provides confirmation that the Phase ONE site is greenfield. The proposed commercial land use is relatively insensitive to contamination therefore it is unlikely that contamination will be a significant constraint
- No gas protection is indicated based on the high water table and cohesive natural Pelaw Clay (b5) found within the Phase ONE site at shallow depth. This mitigates the generation, storage and migration of hazardous ground gases. The unsaturated zone was too thin to monitor with the installed gas instruments; however results of the monitoring support designation of the site as Characteristic Situation CS1. This position should be reviewed subsequent to a fuller assessment of the risk of shallow mining for each new building and possible creation of preferential migration pathways for mine gas.
- A previous study by others suggests that no radon protection is required; however AECOM has not been commissioned to update this assessment.
- Topsoil as would be expected contains Total Organic Carbon concentrations that exceed the Inert Waste limit of 3%w/w. This is not unusual for topsoil and should not provide any unusual difficulty for disposal if it were not possible to accommodate excavated topsoil on the site within the design.
- Preliminary assessment of chemical test data does not highlight constraints to preclude the use of Made Ground within the proposed earthworks. However, it is recommended that made ground is not exposed on cutting slope faces to reduce the risk of run-off of potentially impacted leachate during construction.

2. Introduction

2.1 Scope and objective of the report

Sunderland City Council and South Tyneside Council propose to construct the International Advanced Manufacturing Park (IAMP) close to the A19(T) and A1(M). The whole of the site IAMP site is spread across approximately 150 hectares, as shown on Drawing 60283414_M015_GEO_001.

AECOM has been commissioned to produce a Ground Investigation Report (GIR) and provide Preliminary Engineering Assessment for the site. This report is focussed on the first phase of the development only, identified as IAMP ONE, which covers the southern part of the overall site area as shown on Drawing 60238414-M015-ACM-L1-DR-GE-001.

Preliminary ground investigation for the whole IAMP site has been completed by specialist contractor Dunelm Geotechnical and Environmental Ltd (Dunelm). This GIR provides a review of the results from the ground investigation and an interpretation of the ground conditions encountered for ground investigation works within the IAMP ONE site boundary. It is noted that the ground investigation works were designed to provide a preliminary ground model and geotechnical information for the whole of the IAMP site area for use in further phases of design by others. Additional ground investigation is likely will be required for specific structures, drainage and development subject to the specialist designers' specific requirements at detailed design stage/s.

Geotechnical parameters and material properties of the strata encountered are provided to inform an overall assessment of the ground conditions for the purposes of preliminary engineering assessment. This GIR is intended to provide geotechnical support to a number of technical specialists who are developing the site layout, drainage, pavement and structure foundation designs and aid their assessment of design risks and costs for the scheme. Geotechnical parameters adopted in the preliminary engineering assessment are discussed in Section 6. It is recommended that geotechnical parameters, material properties and groundwater levels are reviewed on an individual earthwork/ structure basis during subsequent detailed design and development phases by others.

A Geotechnical Desk Study Constraints Report has been produced for the wider development area by WSP/ Parsons Brinckerhoff (PB) in March 2016 [Ref. 1]. Reference to this report is advised. This GIR includes a Geotechnical and Geo-environmental Risk Register which updates the geotechnical constraints previously reported by WSP/ PB.

The format of this report is in accordance with the principles set out in the Design Manual for Roads and Bridges (DMRB) HD22/08 [Ref. 2]. This report was prepared in April 2018 and should be treated in light of any subsequent changes in legislation, statutory requirements or industrial practices.

2.2 Description of the project

Sunderland City Council and South Tyneside Council propose to construct the IAMP, which will include the development of 150 hectares of the site, close to the A19(T) and A1(M), for a nationally important and internationally respected location for advanced manufacturing park and European-scale supply chain industries, providing a planned and sustainable employment location that maximises links with Nissan and other high value automotive industries. The overall development is planned to include around 260,000m^2 of commercial space, set alongside new infrastructure and services.

The proposed development will be located on land to the north of the existing Sunderland Nissan car manufacturing plant, south of the A184, east of Usworth Hall and immediately west of Town End Farm. The A19(T) forms the eastern boundary and the A1290 and Washington Road form the southern boundary. The local authority boundary between Sunderland City Council and South Tyneside Council passes through the northern part of the assessment area, on an approximate east to west alignment, partially following the River Don over the west portion of the site.

This report is focussed on the first phase of the development only, identified as IAMP ONE, which covers the southern part of the overall site area as shown on Drawing 60283414-M015-ACM-L1-DR-GE_001. An indicative development masterplan for the IAMP ONE site area is shown on AJA Architects' Drawing 6247-127 included in Appendix A.

The Phase ONE site area is centred on National Grid Reference NZ 335 590. The site is bounded to the south and east by the A1290 with Downhill Lane running along the north boundary of the site. The south east and west parts of the Phase ONE area are bounded by agricultural fields cut by drainage ditches and hedgerows. A length of the west boundary runs directly adjacent to a tributary of the River Don, which converges with the River Don at the corner of the site, approximately 80m south west of Hylton Bridge Farm, which is outside of the site boundary. North Moor Farm is located approximately 200m west of the west boundary and West Moor Farm is located adjacent to the south west corner of the site and the A1290. There are no farmsteads or properties located within the IAMP ONE site boundary. The site is traversed in a north west - south east direction by an unnamed road (formerly Hylton Lane) that provides access from Washington Road over the A1290 to West Pastures (lane), crossing over the River Don at Hylton Bridge. A track trends south west through the site from the unnamed road, providing access to North Moor Farm.

The IAMP ONE development will include up to nine specialist automotive and manufacturing units. The units will be located within the southern area of the overall IAMP site within Sunderland City Council's boundary. The IAMP ONE development will include a new road link from the A1290, associated car parking, service yards, vehicular access, landscaping and drainage ponds.

2.3 Geotechnical Category of project

In accordance with BS EN 1997-1:2004+A1:2013 and Eurocode 7 [Ref. 3] the scheme is considered to fall under Geotechnical Category 2, as it includes conventional types of structure and foundation with no exceptional risk or difficult ground or loading conditions. Examples of Category 2 structures or parts of structures comprise: spread foundations, raft foundations, pile foundations, walls and other structures retaining or supporting soil or water, excavations, bridge piers and abutments, embankments and earthworks, ground anchors and other tie-back systems, and tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirements.

The Geotechnical Category for individual structure units may need to be reviewed by the specialist designer/s as part of further phases of development should more onerous construction, foundation conditions or stringent serviceability limits be required.

2.4 Other relevant information

Previous reports by others specific to Geotechnical and Geo-environmental considerations include:

- International Advanced Manufacturing Park, South Tyneside Council, Desktop Engineering Assessment, July 2014, Mott MacDonald, July 2014 [Ref. 4].
- Sunderland IAMP Geotechnical Desk Study Constraints Report, Sunderland City Council, WSP / Parsons Brinckerhoff, March 2016, REPORT NO 20160314-RH-GEOTECHNICAL DESK STUDY CONSTRAINTS-ISSUE 1.0 [Ref. 1].
- International Advanced Manufacturing Park, Preliminary Environmental Information Report, ARUP, Final, 22 November 2016, Reference 242745, Chapter 8, Geology, Soils and Contaminated Land [Ref. 5]. (http://www.iampnortheast.com/assets/Uploads/Preliminary-environmental-information.pdf).
- Sunderland City Council and South Tyneside Council, International Advanced Manufacturing Park, PSD 16, Geotechnical Technical Background Report (TBR), February 2017, AECOM, in support of overall ARUP Area Action Plan (AAP) [Ref. 6]. (https://www.southtyneside.gov.uk/article/36013/International-Advanced-Manufacturing-Park-Area-Action-Plan).

3. Existing Information

3.1 Topographical maps

The topography of the IAMP ONE development area is generally flat ranging from around 40mOD in the south west corner sloping gently to around 35mOD towards the north east boundary and the River Don, north of Downhill Lane. The A1290 is supported on low embankment, generally between 1 and 1.5m in height, along the southern and eastern boundaries of the Phase ONE site.

3.2 Geological maps and memoirs

The British Geological Survey (BGS) 1:50,000 scale geological map sheet 21 for Sunderland dated 1978 indicates that the assessment area is underlain by superficial deposits identified as Upper or Pelaw Clay, now known as the Pelaw Clay Member, see Drawing 60283414_M015_GEO_DR_005. These clay deposits are described as silty clay containing well dispersed pebbles and cobbles. The deposits are underlain by soils of the Tyne-Wear Complex. The geological Sheet Memoir 21 for Sunderland (British Geological Survey, 1994) describes the Tyne-Wear Complex soils to generally comprise interbedded laminated silty clays and clayey silts, fine grained sands, stoney clays and some gravels. A thin ribbon of Alluvium is recorded along the banks of the River Don and its tributary, which both flow in an approximately west-east direction through the northern part of the assessment area, north of North Moor Farm, between Hylton Grove Farm and Hylton Bridge Farm. The River Don passes below the A19(T) approximately 470m north east of the Phase ONE north site boundary, north of the A19(T)/ A1290 junction. Alluvial deposits are typically described by the BGS on-line lexicon as "normally soft to firm consolidated, compressible silty clay, but can contain layers of silt, sand, peat and basal gravel. A stronger, desiccated surface zone may be present".

Geological mapping shows the superficial deposits are underlain by the Upper and the Middle Coal Measures, now known as the Pennine Upper Coal Measures and the Pennine Middle Coal Measures Formations, respectively, see Drawings 60283414_M015_GEO_DR_006 and 009. These formations were deposited during the Carboniferous Period and comprise an interbedded sequence of mudstone, siltstone, sandstone and coal seams. A geological section shown on Drawing 006 shows that a marine band named the Down Hill Marine Band (DHMB) defines the boundary between the two formations. The projected subcrop of the marine band on the accompanying geological map indicates that all but the north east corner of the IAMP ONE development area is expected to be underlain by rocks of the Pennine Middle Coal Measures Formation. Orientation of the coal seam subcrops suggests that the strata follows the regional dip, falling to the north east across the site. Seam levels in the Hutton coal indicate the rocks dip at approximately 2.8°.

The assessment area is shown to lie within an area of substantial faulting (see Drawing 009), with two named faults identified: the Claxheugh Fault trending north west/ south east through the eastern portion of the assessment area, which has resulted in strata being downthrown to the south west by around 60 to 80m, and the Usworth Fault trending north east/ south west through the northern portion of the assessment area, causing rocks to be downthrown to the south east by 10 to 15m approximately. Several other unnamed faults generally trending north west/ south east have been mapped across the assessment area, some of which have been identified in coal seams at depth.

The BGS GeoIndex Onshore online map viewer http://mapapps2.bgs.ac.uk/geoindex/home.html (Ref. [8]) confirms the site is underlain by Pelaw Clay in turn underlain by the Tyne and Wear Complex. The Pelaw Clay is described as "reddish-brown to dark brown silty clay containing well dispersed pebbles and cobbles (locally abundant), and commonly, small, buff to grey, grotesquely shaped calcareous concretions towards the base of the weathering zone. Clasts are described to be mainly of Carboniferous lithologies (sandstone, mudstone, limestone, coal)". The Pelaw Clay is described to typically have "closely spaced, subvertical, prismatic jointing that formed during dry periglacial conditions".

The BGS reports that there is "generally a planar, subhorizontal to undulating, gradational, glaciotectonic boundary" between the Pelaw Clay the underlying Tyne-Wear Glaciolacustrine Formation, which is described as a "dark grey, laminated silt and clay". The Tyne and Wear Complex locally rests on bedrock or overlies a lower glacial till described on the BGS GeoIndex as a "purplish brown, stony silty sandy diamicton clay of the Wear Till Formation".

The 1:10,560 scale geological map sheets NZ35NE and NZ36SW identify the presence of buried valley channels to the south and north of the overall IAMP site. This interpretation ties in with an insert on the 1:50,000 scale geological map which shows a dendritic network of main buried valleys, one of which is present immediately north of the River Wear, south of the assessment area. A tributary to this buried valley channel trends north west –south east and crosses through the whole of the Phase ONE site area. It is noted that only main buried valley channels have been mapped and that the presence of smaller tributaries branching from the mapped glacial valleys being present below site cannot be discounted. Desk study information suggests that variable drift deposits and variable depths to rockhead across the site are anticipated. Significant depth of superficial soils is anticipated to be present in the centre of the infilled glacial valleys.

Two shallow coal seams (the Top Hebburn Fell and Bottom Hebburn Fell) are mapped to subcrop approximately 1km south west of the Phase ONE site. These coal seams occur within the Pennine Middle Coal Measures Formation. The Hylton Castle seam subcrops approximately 75m north of the site; this seam forms part of the Pennine Upper Coal Measures Formation.

3.2.1 BGS Borehole Logs

There a number of publically available BGS borehole records from within the larger IAMP site but these lie outside of the IAMP Phase ONE area. The strata proved in two representative historical BGS holes are summarised in Tables 1 and 2 below with a copy of the logs and exploratory hole locations included in Appendix B.

Borehole NZ35NW1586 was drilled in 1997 by Dunelm Drilling Company on behalf of (Sunderland) City Consultancy Services for the A1290, Washington. Borehole NZ35NW/77/245 was drilled in 1967 by Le Grand Adsco for 'Sunderland Bypass' on behalf of Durham County Council.

Table 1. Borehole NZ35NW1586 (National Grid Reference: 433672, 559354, adjacent to Downhill Lane)

Depth	Strata Description	Interpreted Geological Unit
0.00 – 1.35	Soil on ashy clay & rubble fill	Made Ground
1.35 – 1.65	Soft dark brown/ black stony clayey soil	Made Ground/ Relict Topsoil
1.65 – 5.60	Stiff reddish brown & grey silty CLAY becoming more sandy with depth	Pelaw Clay Member
5.60 – 10.00	Stiff becoming firm sandy silty laminated CLAY with occasional gravel and moist sand partings	Tyne and Wear Complex

Table 2. Borehole NZ35NW/77/245 (National Grid Reference: 434147, 559863, A19(T) Junction)

Depth	Strata Description	Interpreted Geological Unit
0.00 – 0.15	Topsoil	Topsoil
0.15 – 3.50	Stiff reddish brown silty CLAY with stones	Pelaw Clay Member
3.50 – 6.70	Firm reddish grey laminated silty CLAY	Tyne and Wear Complex
6.70 – 8.80	Hard grey sandy SHALE with occasional thin bands of softer material	Pennine Upper Coal Measures Formation
8.80 – 10.00	Hard grey sandy SHALE	Pennine Upper Coal Measures Formation

The BGS boreholes appear to confirm the anticipated geological sequence described on the published mapping. The made ground proved in Borehole NZ35NW1586 to 1.35m depth is likely to represent former railway embankment fill (which is understood to have been incorporated to partially form the A1290 road embankment). The underlying natural soils encountered in this borehole comprised stiff reddish brown and grey silty clay which became more sandy with depth are interpreted to represent the Pelaw Clay. Stiff laminated clay proved between 5.6 and 10m is considered representative of the Tyne and Wear Complex.

Borehole NZ35NW/77/245 proved a similar sequence of superficial soils although bedrock was encountered; this was described as hard grey sandy shale (assumed mudstone) and proved at depths between 6.7 and 10.0m. However, it is anticipated that the depth of superficial soils will increase significantly over the mapped extents of the infilled glacial valley which extends over the majority of the Phase ONE site area.

3.3 Aerial photographs

Not reviewed as part of this study.

3.4 Records of mines and mineral deposits

Mining risk has been assessed through review of BGS maps, the online Coal Authority Viewer, the BGS Sunderland district memoir and previous reports prepared by others.

The geological maps show two shallow coal seams (Top Hebburn Fell and Bottom Hebburn Fell) are mapped to subcrop approximately 1km south west of the Phase ONE site. These seams are identified as by the Coal Authority but are not identified as a 'high risk coal mining' area. However, it was considered possible at desk study stage (Ref. 1) that unrecorded workings that pre-date Coal Authority records may exist in these seams and rotary coring in rock was recommended. Risks associated with mining activities include collapse of former underground workings or shafts leading to damage of overlying structures and infrastructure. Former workings may present a contamination risk to groundwater and human health risk to site users and infrastructure maintenance operatives from ground gas emissions.

The Hylton Castle Seam subcrops approximately 75m north of the Phase ONE site boundary trending north west to south east direction, see Drawing 60283414_M015_GEO_DR_009. As discussed above in Section 3.2, this seam and other coals are indicated to dip to the north east across the site. It is therefore inferred that this seam does not underlie the site. The BGS memoir indicates the Hylton Castle coal seam contains three shale partings but does not indicate the thickness of this coal. The geological section presented on Drawing 009 indicates that the coal is 21 inches (0.53m) thick. The seam is not identified as an outcrop or a high risk area by the Coal Authority.

The BGS Sheet Memoir 21 (Ref. 33) describes the Hebburn Fell Coals to comprise a group of two or three seams over 10m of strata. Based on the mapped seam subcrops, all of these coals are anticipated to underlie the IAMP ONE site area. Of the two seams, the upper coal is reported to be usually thicker, up to 1.4m. The lower coal is only more than 1m thick in a few places within the Sunderland District and is generally recorded to be less than 0.5m thick. In comparison, the Top Hebburn Fell and Bottom Hebburn Fell Coals were indicated to be 15 to 32 inches thick (0.38 to 0.81m) and 12 to 18 inches (0.30 to 0.46m) thick on the geological section presented as part of Drawing 009.

The memoir indicates that the Hebburn Fell Coals have not been mined underground, although opencast workings near Usworth (approximately 1.5km west of the assessment area) extracted the Top and Bottom Hebburn Fell seams where the upper and lower seams were recorded to be 9m apart. These working are approximately 2km northwest of the IAMP Phase ONE site boundary, and are identified as 'Mine' on the 1:50,000 drift geological map reproduced as Drawing 60283414_M015_GEO_DR_005 in this report. Wardley Colliery was located approximately 1.5km north west of the Phase ONE site. This is later identified as Wardley Colliery Disposal Point (colliery waste spoil heap) and is identified as a geotechnical constraint for the whole of the IAMP development area in the PB Desk Study Report (Ref. 1). The area of the Wardley Colliery Disposal Point is shown on Drawing 60283414-M015-ACM-L1-DR-GE_004 in this GIR.

Further discussion regarding the depth of coal seams and possible unrecorded mine workings proved as part of ground investigation works undertaken is included in Sections 5 and 6 of this GIR.

3.4.1 Coal Authority Mining Report (by others)

A Coal Authority Mining Report specific to the Phase ONE site area is not available at the time of writing this GIR.

A Coal Authority Mining Report was obtained by Mott MacDonald (Ref. 4) with the findings summarised below. A copy of the Coal Authority Mining Report is included in Appendix C. It is noted that the search area extends outside the IAMP Development Consent Order (DCO) boundary and significantly outside the Phase ONE development area.

- The property is in the likely zone of influence from underground coal workings in 8 seams of coal at 190m to 570m depth, and last worked in 1981. Any ground movement from these coal workings should have stopped by now.

- The site is in an area where the Coal Authority believe that there is coal at or close to the surface and the coal may have been worked at some time in the past. The potential presence of coal workings at or close to the surface should be considered prior to any site works or future development activity.
- The site is not in a location where it could be affected by current or planned future underground coal workings. A single mine entry is indicated adjacent to the north western boundary of the site. The Coal Authority report suggests that this was capped in 1975 and re-capped in 1980.
- The site does not lie within the boundary of a coal opencast site, nor within 200m of a current opencast site.
- The site is not within 800m of an opencast site which is being considered by the Coal Authority or licence granted.
- A claim was submitted to The Coal Authority for subsidence damage within fields to the north of the site adjacent to the A184. However, this claim was rejected by The Coal Authority.
- There are no records of mine gas emissions.
- The site has not been subject to remedial works by The Coal Authority.
- The assessment area is in an area where the Coal Authority believes that there is coal at or close to the surface and the coal may have been worked at some time in the past. The Coal Authority advises that the potential presence of coal workings at or close to the surface should be considered prior to any site workings or future development activity.

3.4.2 Minerals

Historical plans, obtained as part of an Envirocheck Report included within the Mott MacDonald report [Ref. 4] for the wider scheme indicate the presence of East House sand and gravel quarry approximately 500m west of the site. Current BGS records reviewed through the BGS GeoIndex website [Ref. 8] do not indicate any present mineral occurrences, active mines or quarries in the Phase ONE development area

3.5 Land use and soil survey information

3.5.1 Land use

It is noted that an Envirocheck Report and associated historical ordnance survey mapping specific to the Phase ONE development area was not available at the time of writing this GIR.

Data relevant to historical development and potentially contaminative uses within/ around the DCO boundary have been reviewed as part of PB/ WSP and Mott MacDonald desk study reports for the IAMP site [Ref. 1 and 4]. Pertinent information is summarised and presented in SCC and STC IAMP Area Action Plan, PSD 16, Geotechnical Technical Background Report, prepared by AECOM in February 2017 [Ref 6].

Key findings relevant to this assessment, amended and assessed as required to reflect changes in the proposed site boundary, are summarised below. Historical land use that could be a constraint to the development is shown on Drawing 60283414-M015-ACM-L1-DR-GE_004.

3.5.1.1 Phase ONE Site Boundary

- Most of the site is shown to be open fields cut by drainage fields and hedgerows from the first available OS Map Edition date 1898 to the latest map reviewed dated 2015. The majority of the land within the Phase ONE boundary remains predominantly farmland in a greenfield setting.
- The first available OS map edition dated 1898 shows the current alignment of the A1290 which forms the east boundary of the site to be the Pontop & South Shields Branch railway line. The railway was replaced by the A1290 in the 1970's.
- Hylton depot is identified on the 1898 OS Map edition (although no specific area is mapped) immediately south of Hylton Lane Level Crossing (this road is no longer named on current mapping – see unnamed road described in Section 2.2 of this GIR). These features are both renamed as Three Horse Shoes Depot and Three Horseshoes Level Crossing by 1921, and no longer identified by 1980.

- West Moor, North Moor and Hylton Bridge farmsteads are identified on the 1898 OS map west of the western site boundary, with the position of these unchanged on the 2015 OS Map edition. Historical OS maps show the presence of water wells associated with the agricultural farmsteads.
- The centre of the Phase ONE site area is shown as 'marshy ground from 1898 up until 1990, indicative of historical high groundwater levels in this area.
- Overhead electricity power lines and pylons are shown crossing the wester corner of the Phase ONE site traversing from south west to north east between 1977 and 2015.

3.5.1.2 Phase ONE Surrounding Area

- An aerodrome identified as RAF Usworth was present south of the site since 1916. The area has been redeveloped since 1984 and is now predominantly occupied by developments associated with the Nissan car manufacturing plant. The proximity of the aerodrome to the site raises the risk that Unexploded Ordnance (UXO) may be present. As a result, further specific desk study was recommended and undertaken by Zetica in 2016. An extract of the report identifying specific UXO risk is included in Appendix D. The risk of UXO at specific exploratory hole locations was examined during the ground investigation, further discussed in Section 6.1.2.
- The North East Land, Sea and Air Museums, formerly the North East Aircraft Museum, is located east of the A1290 close to the south east corner of the Phase ONE site.
- It is noted that an area of rail sidings east of the Phase ONE site and the A1290 was previously identified as a constraint by others. Independent review of historical maps has not identified this feature to be present. Historical plans presented within the North East Land, Sea and Air Museums show this area historically comprised an officer's accommodation block with associated stores and infrastructure. This area is not mapped on the historical OS Maps due to the sensitivity of the land use and the dates over which this these features were present are unknown, but it is inferred that they were most likely removed at the same time as the redevelopment of the former RAF Usworth site occurred in the 1980's. This area is however outside the Phase ONE boundary and does not directly impact the site.

3.5.2 Soil Survey

The site is currently largely agricultural land use. The site is currently largely under agricultural use. Information provided on MAGIC.gov.uk for Agricultural Land Classification (ALC) as supplied by DEFRA indicates that the majority of soils for the site are classified as Grade 3 'Good and Moderate Quality' land, see Drawing 60283414_M015_GEO_DR_007. There is a small area of Grade 2 'Very Good Quality' land located along the banks of the River Don outside of the Phase ONE area.

However, the majority of the land within the IAMP Phase ONE boundary is recorded as unmapped. This is because a subdivision into Grades 3a (Good Quality Agricultural Land) and 3b (Moderate Quality Agricultural Land) was introduced (post 1988) based on a study commissioned by MAFF [Ref. 34], with this study not extending to cover the IAMP ONE site.

The 1:250,000 Soils Map Sheet 1 for Northern England (Soil Survey of England and Wales, 1983) shows the assessment area and surrounding area to be underlain by Foggathorpe 1 (712h) soil with the exception of a small area in the north eastern corner (below the A19(T)) which is underlain by Aberford (511a) soil, see Drawing 60283414_M015_GEO_DR_008. Soils along the eastern boundary, east of the A19(T) have not been surveyed as they are classified as mainly urban and industrial areas.

The Foggathorpe 1 soil association is classified as Glaciolacustrine drift and Till and comprises "clayey and fine loamy over clayey soils, often stoneless". The soil is described as "slowly permeable seasonally waterlogged".

The Aberford soil association is classified as being of Permian, Jurassic and Eocene Limestone geology and comprises "shallow, locally brashy, well drained calcareous fine loamy soils over limestone. Some deeper calcareous soils in colluvium".

3.6 Archaeological and historical investigations

Historical land use of the site is described in Section 3.5. MAGIC Maps [Ref. 7], do not show any historical statutory or non-statutory land designations within the site boundary. A review of any further potential archaeological constraints to the proposed works is outside the scope of this report.

An archaeological watching brief was commissioned by Sunderland City Council (SCC) during the ground investigation works, the results of which are outside the scope of this GIR but may be available for review on request of SCC.

3.7 Existing ground investigations

There have been no previous site specific ground investigations undertaken on the IAMP Phase ONE site but as discussed earlier in Section 3.2.1 above, publically available boreholes relating to past improvements to adjacent road infrastructure are relevant to the wider IAMP development area. Details of these are therefore summarised below.

3.7.1 A1290 (Washington Road) - 1997

This includes four boreholes (10m deep) obtained from the BGS borehole viewer website [Ref. 8], which are located towards the south east corner of the IAMP assessment area. The boreholes provided proved 0.85 to 1.65m of made ground. The A1290 is located on top of a low embankment and based on the historical plans the made ground is likely to be associated with the historical Pontop & South Shields railway line and an adjacent small area of rail sidings. Underlying the made ground, the boreholes encountered sandy silty clay which was recorded to a depth of 10m bgl. One of the boreholes encountered silty sand at depths between 4.75 and 6.60m bgl (1.65m thick).

3.7.2 Sunderland Bypass (A19) - 1967

The BGS borehole viewer identifies 17 boreholes that were drilled along the line of the proposed Sunderland Bypass. This road now forms part of the A19(T), and is located along the eastern boundary of the IAMP site. Ground investigation works for the Sunderland Bypass were carried out by Le Grand Adsco under the instruction of Durham City Council (DCC) in 1967. The historical borehole record sheets show the area to be underlain by topsoil, typically 0.15 to 0.30m thick. This in turn was underlain by sandy silty clays, which included thin layers of sand, generally 0.15 to 0.30m thick. Bedrock recorded below this part of the site was identified as shale and occasional sandstone which were encountered at depths ranging from 1.8 to 14.2m bgl.

3.7.3 Groundwater Conditions

Groundwater was encountered as water strikes during drilling in many of the aforementioned boreholes within the sandy silty clays at depths from 1.8 to 8.35m.

3.8 Consultations with Statutory Bodies and Agencies

Extensive consultation over a period of years has been undertaken by the IAMP Limited Liability Practice (LLP) (a special purpose vehicle incorporated by SCC and Sunderland Tyneside Council (STC)) with statutory bodies, agencies, interested parties and the general public and has been reported separately (by others). SCC, STC and the IAMP LLP hold full details of consultations undertaken and therefore these are not discussed further in this report.

3.9 Flood records

Environment Agency (EA) Flood Maps acquired, by Mott MacDonald, as part of an Envirocheck Report [Ref. 4] indicate areas of the site to fall within Flood zones 2 & 3. More specifically, Flood zone 2 & 3 are located along the River Don and Usworth Burn; these designated areas are described as zones of flooding and extreme flooding, respectively, from rivers or sea without defences.

Flood zone 3 defines part of the IAMP development area which could be flooded from the both River Don and Usworth Burn by a flood that has a 1 per cent (1 in 100) or greater chance of happening each year. Flood zone 2 also covers the area which could be flooded from both the River Don and Usworth Burn by a major flood, with up to a 0.1 per cent (1 in 1000) chance of occurring each year.

3.10 Contaminated land

From desk study information made ground is not anticipated to be widely encountered across the assessment area. The existing A1290 was built along the alignment of a former Pontop & South Shields railway line. The road is generally constructed at grade or on low embankment. The Department of Environment Industry Profile for Railway Land (Ref. 35) indicates imported fill was often utilised during construction of the railways where there was a shortfall of natural excavated material. Imported fill often includes waste material containing clinker and ash. Boiler ash generated by steam locomotives was also often used to form ballast along many railway lines.

There is a 'Depot' identified as Hylton Depot and later as Three Horseshoes Depot, this is not identified to extend over a specific area, however, most likely located east of the railway line (current alignment of A1290) and therefore is outside the IAMP Phase ONE boundary.

3.11 Hydrology

A site specific Envirocheck report is not available for the IAMP ONE site area at the time of issue of the GIR.

The River Don and a tributary (named locally as Usworth Burn) which run from west to east approximately are located north of the IAMP ONE site.

Flood maps obtained from the EA in the Envirocheck report available for a separate study undertaken along the alignment of the A19(T) indicate that land north of the site is at risk from flooding from Rivers or Sea associated with the River Don. The western boundary of the IAMP ONE site is shown to be at risk of flooding associated with the tributary to the River Don. For planning purposes, this area has been classified as a Flood Zone 2 (i.e. has a 1 in 100 or greater annual probability of flooding). A copy of the Envirocheck Flood map is included in Appendix D of this report.

3.12 Hydrogeology

The online EA Aquifer Designation Map (Ref. 9) and Superficial Aquifer Designation map (acquired as part of an Envirocheck report (Ref. 4) indicates that the Pelaw Clay Member beneath the site have been designated as Unproductive Strata, which are described by the EA as 'rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow'.

Thin ribbons along the banks of the River Don are classified as a Secondary A aquifers, which are defined as 'permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers'. These match the outcrops mapped by the BGS as alluvium deposits. Secondary Aquifers include a wide range of rock layers or drift deposits with an equally wide range of water permeability and storage. Secondary aquifers are subdivided into two types, A & B, with A described as: permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as minor aquifers.

The EA and Envirocheck Bedrock Aquifer Map show that the superficial deposits are underlain by a Secondary (A) Aquifer corresponding to the Pennine Upper Coal Measures and Pennine Middle Coal Measures Formations, respectively. These are generally aquifers formerly classified as minor aquifers. According to the groundwater vulnerability map for the area, the aquifer beneath the site is classified as a Minor Aquifer with predominantly a 'low' leaching potential, indicating there is a low likelihood of a pollutant discharged at ground level reaching groundwater stored within superficial and bedrock aquifers.

There are two Groundwater Source Protection Zones within 1km of the site, identified as Zone 3 (total catchment), located 869 and 969m east of the site. There are three Environment Agency licensed groundwater abstractions with 1km of the site, located 931m north east of the assessment area. The site does not fall within a Nitrate Vulnerable Zone (NVZ). These are not considered to significantly impact the IAMP ONE development.

3.13 Other relevant information

Not used.

4. Field and Laboratory Studies

4.1 Walkover Survey

A site walkover survey was undertaken as part of the WSP/ PB Desk Study Report [Ref. 1]. A further documented walkover survey was not undertaken as part of the ground investigation works.

4.2 Geomorphological/geological mapping

No geomorphological or geological mapping was undertaken as part of this report.

4.3 Ground Investigations

BS EN 1997-1:2004+A1:2013 (Eurocode 7) [Ref. 3] recommends a phased approach to ground investigation, with stages of investigation defined as 'Preliminary Investigation', 'Design Investigation' and 'Control and Monitoring Investigation'. Investigations proposed as the first phase of works are considered Preliminary, defined in Eurocode 7 as being able 'to allow the engineer to assess a suitable position for the structure, evaluate its possible effect on adjacent buildings, and consider possible foundations and ground improvements'.

The preliminary ground investigation was designed to provide an overall assessment of the ground conditions for the purposes of preliminary engineering assessment and is intended to provide support to a number of technical teams who are developing the overall site masterplan and assessing design risks and costs for the scheme. Fieldwork was undertaken generally as specified in the contract documents provided by AECOM. The fieldwork was carried out between 21st July and 29th November 2017. Exploratory holes were positioned based on Urbed Illustrative Masterplan layout included as Drawing ZO-DR-U-Masterplan_A_Illustrative_002 Rev E included in Appendix A as shown on Drawing 60283414_M015_GEO_DR_002. Additional exploratory holes were placed to assess ground conditions along the alignment of a primary road including at proposed structure crossings over the River Don and the A19(T).

This report is focussed on the first phase of the development only, identified as IAMP ONE, which covers the southern part of the overall site area as shown on Drawing 60283414-M015-ACM-L1-DR-GE_002.

Preliminary ground investigation for the whole IAMP site has been completed by specialist contractor Dunelm Geotechnical and Environmental Ltd (Dunelm).

The location of the preliminary ground investigation exploratory holes is shown on Drawing 60283414_M015_GEO_DR_002. The preliminary ground investigation works for the overall IAMP site were designed with the following objectives:

- Prove the stratigraphy at the site, nature of made ground (if present) and superficial soils and determination of rock level.
- Determine the type, strength and quality of bedrock across the site by extending selected holes using rotary core drilling.
- Investigate the presence/ absence of unrecorded coal mine workings in the Top and Bottom Hebburn Fell seams which subcrop beneath the south west portion of the site. Investigate the possible presence of workings in the Hylton Castle seam which subcrops in the centre of the site. However, as discussed in Section 3.2 above, based on geological mapping, this coal does not underlie the IAMP ONE site.
- Enable sampling and subsequent testing of soil to obtain geotechnical design parameters and engineering properties of soils for assessment of earthworks acceptability, bearing capacity and foundation design options.
- Installation of groundwater monitoring standpipe piezometers within all exploratory holes.
- Determine the aggressiveness of the soils and groundwater to buried concrete and steel.
- Enabling sampling and subsequent environmental testing of soils to inform the site conceptual model and contamination risk assessment.

- Installation of gas monitoring standpipes where made ground is encountered to quantify potential risk of harmful land gasses.

4.3.1 Description of fieldwork

The preliminary ground investigation for the whole of the IAMP site comprised:

- 40 cable percussive boreholes (BH's) with 33 holes extended into bedrock by rotary coring;
- 1 rotary open hole (BH16B) through superficial soils which was extended into bedrock by rotary coring;
- 28 machine excavated trial pits (TP's) to a maximum depth of 5.0m;
- 6 trial pit soakaways tests (TPS's);
- 17 electronic cone penetration tests (CPT's);
- 6 electronic cone penetration tests with magnetometer testing for UXO (CPTM's);
- 6 road cores through the A1290 to assess the existing highway pavement construction.

Standpipe piezometers or gas monitoring standpipes were installed in all exploratory holes and return visits were scheduled on six occasions over a three month monitoring period following completion of site works to obtain groundwater and ground gas concentrations. At the time of writing this GIR, four return visits had been completed and reported; these visits took place on the 20/12/17, 12/01/18, 26/01/18 and 09/02/18, respectively.

This report relates specifically to the exploratory holes included in the IAMP ONE development area, as shown on Drawing 60283414-M015-ACM-L1-DR-GE_002 and includes discussion of ground conditions encountered in the following positions:

- BH's 13 to 17, 24, 25, 31, 35, 38 to 40, 45 to 49 and 51 to 52;
- TP's 10, 13, 16, 16a, 19, 25 and 35 to 38, TP30 and 31 outside Phase ONE for drainage ponds;
- TPS's 01 to 04 for soakaway testing;
- CPT's 14 to 15, 19, 22 and 26 to 33;
- CPT Magnetometer Testing for UXO (CPTM-04 to 09).
- RC's 01 to 06.

It is noted that no ground investigation works were completed over the area adjacent to the tributary to the River Don where three balance ponds are shown, see Section D on Drawing 6028314-M015-ACM-L1-DR-GE_002. At the time of finalisation of preliminary ground investigation proposals, the location of proposed balancing ponds for the overall IAMP scheme was still under consideration. Trial pits TP30 and TP31 were excavated at indicative pond locations proposed at the time the investigation was undertaken.

4.3.2 Copy of the ground investigation report

Dunelm Geotechnical and Environmental Ltd's Factual Report on Preliminary Site Investigation (Ref. 10) for the works completed across the whole of the IAMP site, Reference D8044, dated 20/02/18 has been issued to SCC under separate cover. It is noted that only a draft of this report was available at the time of writing this GIR.

4.3.3 Results of in situ tests

In situ tests comprised the following:

- Hand shear vane tests carried out during excavation of trial pits and borehole inspection pits.
- Standard Penetration Tests (SPT's) carried out during drilling.
- Soakaway tests were carried out in in TPS 01, 02, 03, 04, 05, 08 and 011 in general accordance with BRE365. TPS's 01 to 04 are located within the IAMP ONE site boundary.
- In situ Plunger Type California Bearing Ratio Tests were carried out at in trial pits TP17, TP18 and TP21. The tests were conducted in accordance with BS1377: 1990. All three exploratory hole positions are however outside the IAMP ONE site boundary and are therefore not discussed within this GIR.

- Dynamic Cone Penetration Tests were carried out at each Road Core position (RC01 – 06). These tests were carried out in accordance with DMRB Vol.7 3.2 HD 29/08. The results of these are presented in Appendix D of the Dunelm Factual Preliminary Site Investigation (SI) Report [Ref. 10]. It is noted that a review of the road core testing results is to be undertaken by an appointed specialist pavement designer as part of ongoing scheme development and detailed design and therefore the results are not discussed further within this GIR.

The in situ test results are provided on the exploratory hole logs as well as is the in situ testing enclosures included in Appendix D of the factual report.

A review and analysis of the results is provided under Sections 5 and 6 of this report as appropriate.

4.4 Drainage studies

A review of drainage requirements is being undertaken by others as part of further phases of detailed design for the wider IMAP development. Discussion of the results of soakaway testing is included in Sections 5 and 6.

4.5 Geophysical surveys

No geophysical surveys were carried out as part of these works.

4.6 Pile tests

No pile tests were carried out as part of these works.

4.7 Other field work

Not used.

4.8 Laboratory Investigation

4.8.1 Description of tests

Laboratory tests carried out on selected soil samples comprised the following:

- Water (moisture) content determination;
- Plastic limit, liquid limit and plasticity index determination;
- Particle size distribution analysis;
- Ground aggressiveness analysis (chloride, total sulphur, sulfate and pH);
- Organic matter content;
- Determination of dry density and moisture content relationship (2.5kg rammer compaction tests);
- One dimensional consolidation properties;
- Undrained shear strength determination in a triaxial cell without pore pressure measurement; and
- Consolidated undrained shear strength in a triaxial cell with pore pressure measurement.
- In situ CBR testing was undertaken in selected trial pits across the IAMP site. However, as discussed in Section 4.3.3, these fall outside the IAMP ONE site boundary.

Laboratory tests carried out on selected rock samples comprised the following:

- Determination of point load index; and
- Determination of unconfined compression strength (and measurement of water (moisture) content).

A summary of the number and type of in situ and laboratory tests results examined for each identified material is provided in Section 6 of this report.

4.8.2 Copies of tests results

Copies of the laboratory tests results are provided in the enclosures included in Appendices E and F of the Dunelm Factual Preliminary SI Report [Ref. 10]. A review and analysis of the results is included in Section 6 of this report.

5. Ground Summary

5.1 General

This ground summary describes the soils and the underlying rocks that were proved across the whole of the IAMP site but is focussed on the ground conditions proved across the Phase ONE development area.

The preliminary ground investigation undertaken by Dunelm has generally confirmed the geology and ground conditions anticipated in the WSP/ PB Geotechnical Desk Study Constraints Report [Ref. 1], from independent review of BGS geological maps and publically available information discussed under Section 3 of this report.

The BGS Sunderland district geological memoir identifies a dendritic network of buried glacial valleys in this part of Wearside associated with the glacial course of the River Wear, south of the assessment area. Geological mapping shows a tributary to the glacial Wear Valley trending in a north west –south east direction which crosses through the whole of the Phase ONE site area. It is noted that only main buried valley channels have been mapped and that the presence of smaller tributaries branching from the mapped glacial valleys being present below site cannot be discounted. These glacial valleys are typically infilled by lacustrine soils of the Tyne and Wear complex comprising medium and high plasticity laminated clays with localised bands of silt and sand. The depth of drift soils is anticipated to thicken significantly over the mapped extent of the infilled glacial valley which has been confirmed by the ground investigation undertaken. Ground investigation data over the whole of the IAMP development area has been analysed in order to plot rockhead as an interpolated contoured surface (mOD) below the Phase One development area, see Drawing 60283414-M015-ACM-L1-DR-GE_003. The drawing shows the glacial valley is orientated north west – south east with rockhead around 15mOD in the centre of the valley, rising to above 35mOD along its margins. This is shown on the insert in this drawing which shows both ground level and inferred rockhead level along Section A-A orientated across the infilled glacial valley.

Geological mapping indicates the site is underlain by Pelaw Clay with Tyne and Wear Complex (mainly laminated clay) present at depth. The Pelaw Clay is described by the BGS as ‘reddish-brown to dark brown silty clay containing well dispersed pebbles and cobbles (locally abundant) with clasts mainly of Carboniferous lithologies (sandstone, mudstone, limestone, coal)’. The BGS indicates that there is ‘generally a planar, subhorizontal to undulating, gradational, glaciotectonic boundary between the Pelaw Clay the underlying Tyne-Wear Glaciolacustrine Formation’, which is described as a ‘dark grey, laminated silt and clay’. The Tyne and Wear Complex locally rests on bedrock or overlies a lower glacial till described on the BGS GeoIndex as a ‘purplish brown, stony silty sandy diamicton clay of the Wear Till Formation’.

Ground investigation has proved topsoil (e1) or localised generally thin made ground (d1 to d5), underlain by generally soft and firm Pelaw Clay (b5). The Pelaw Clay is underlain by lacustrine deposits of the Tyne and Wear Complex comprising firm becoming stiff with depth laminated clay (b2) interbedded with bands of sands (b3) or silt (b4). The Tyne and Wear Complex is underlain by a stiff and very stiff over-consolidated Lower Glacial Till (b1) which is inferred to represent the diamicton of the Wear Till Formation.

Superficial soils are underlain by rocks of the Pennine Upper and Middle Coal Measures Formations which comprised an interbedded sequence of sandstone (a1), mudstone (a2), siltstone (a3) and coal (a4). The surface of the bedrock was noted to be weathered although it was not found to be of significant thickness across the site. Possible coal workings a4 (work) and assumed zones of core loss (AZCL) were also identified.

The ground investigation exploratory hole locations for the Phase One development area are shown on Drawing 60283414-M015-ACM-L1-DR-GE-002. Geological cross sections A-A’ to F-F are presented as Drawings 60283414-M015-ACM-L1-DR-GE_005 to 007, with ground conditions proved along each section discussed in further detail in Section 6 below.

The generalised succession and a strata coding system developed to characterise ground conditions present across the site is summarised in Table 3. Ground conditions, stratigraphical units, in-situ and laboratory test results and material properties are presented and discussed in Section 6.

Table 3. Generalised Strata Succession and Strata Codes

Age/ Period	Strata	Strata Code	Sub Formation Unit (typical description)
Recent	Topsoil	e1	Topsoil - brown or dark brown slightly sandy slightly gravelly clayey topsoil with subangular to rounded, fine to coarse gravel of sandstone, mudstone and coal.
Recent	Made ground - Topsoil	d1	MADE GROUND Topsoil - Dark brown slightly clayey, slightly gravelly sandy topsoil. Gravel is subangular to subrounded, fine to coarse of sandstone, mudstone, limestone, concrete and brick.
		d2	MADE GROUND Topsoil - Dark brown slightly sandy slightly gravelly clayey topsoil. Gravel is angular to subrounded, fine to coarse of sandstone, occasional mudstone, rare coal, glass, ceramic, clay tile and pot.
		d3*	MADE GROUND Topsoil - Dark brown sandy slightly gravelly, clayey topsoil. Sand is fine to coarse. Gravel is fine to coarse, subangular to subrounded of sandstone and coal. Occasional brick, glass and ceramic fragments, rare plastic, rootlets and organic matter noted.
		d4*	MADE GROUND - Firm dark brown sandy slightly gravelly clay with low cobble content. Gravel is angular to subrounded, fine to coarse of concrete, coal and brick. Occasional ash. Cobbles are angular to subrounded of concrete.
		d5	MADE GROUND - Orangish brown slightly sandy gravelly clay of intermediate plasticity. Gravel is subangular to subrounded, fine to coarse of coal, sandstone and brick. **Note: these soils may be mis-logged and represent desiccated Pelaw Clay (b5).**
Recent	Alluvium	c1*	Alluvial Sand - Loose orange or dark brown, slightly clayey, silty SAND. Sand is fine to medium.
		c2*	Alluvial Clay - Firm orange brown slightly gravelly sandy CLAY of intermediate plasticity. Gravel is angular to subangular, fine to coarse of sandstone and quartz.
Quaternary-Pleistocene	Pelaw or Upper Clay	b5	Pelaw Clay – Soft, firm or stiff orange brown, reddish brown, greyish brown, brown or dark brown, mottled light grey, slightly sandy slightly gravelly CLAY of intermediate and high plasticity. Gravel is subangular to rounded, fine to coarse of sandstone, mudstone and coal. The soils are occasionally recorded as laminated.
	Tyne & Wear Complex	b4	Silt - Dark grey clayey sandy SILT. Sand is fine to coarse, or: Soft or firm greyish brown slightly sandy slightly clayey SILT.
		b3	Glacial Sand - Medium dense brown or grey clayey fine to medium SAND.
		b2	Laminated Clay – Soft, firm or stiff thinly laminated brown or grey silty CLAY of high plasticity with silty partings. Light brown and grey silt dustings/ partings on laminae surfaces, or: Soft, firm or stiff laminated brown or greyish brown, slightly sandy, slightly gravelly CLAY of intermediate plasticity. Gravel is subangular to subrounded, fine to medium of coal and sandstone.
	Lower Glacial Till	b1	Lower Glacial Till - Stiff or very stiff brownish grey or dark brown slightly sandy slightly gravelly CLAY. Gravel is subangular to subrounded, fine to coarse of sandstone, mudstone, siltstone and coal.
Carboniferous	Pennine Middle Coal Measures Formation	a1(w)	Weathered Sandstone - Very dense brown, yellow or reddish grey sandy GRAVEL. Gravel is angular to subangular, fine to coarse of sandstone lithorelicts.
		a1	Sandstone – Weak, partially weathered, orange brown fine, predominantly medium to coarse, micaceous SANDSTONE. Fractures are sub-horizontal, planar, smooth with dark red staining. or: Medium strong, partially weathered, light grey fine grained SANDSTONE. Fractures are medium spaced sub-horizontal, planar, smooth, undulose, clean.

Age/ Period	Strata	Strata Code	Sub Formation Unit (typical description)
		a2(w)	Weathered Mudstone - Firm to stiff light grey slightly sandy gravelly CLAY. Gravel is angular to subangular, fine to coarse of mudstone lithorelicts. or: Very dense light grey or reddish brown, slightly clayey, slightly sandy, GRAVEL. Gravel is angular to subangular, fine to coarse of mudstone.
		a2	Mudstone – Extremely weak or very weak distinctly weathered light grey MUDSTONE. Fractures are very closely to closely spaced, sub-horizontal, planar, smooth, clean with dark grey discolouration on fracture surfaces.
		a3(w)	Weathered Siltstone - Very dense light grey sandy GRAVEL. Gravel is angular to subangular of siltstone lithorelicts. or: Stiff, grey mottled brown slightly sandy, slightly gravelly CLAY. Gravel is angular to subangular of mudstone and siltstone
		a3	Siltstone – Very weak, weak or medium strong dark grey SILTSTONE with bands of mudstone. Fractures are very closely to closely spaced, sub-horizontal planar, smooth, clean.
		a4	Coal - Very weak black COAL. Frequently randomly orientated interlocking fractures.
		a4(work)	Potential Coal Workings – very weak mudstone and very weak black coal interbedded with firm grey slightly sandy slightly gravelly clay with mudstone lithorelicts.
		AZCL	Assumed zone of core loss.

*** not proved within the Phase ONE development area although were encountered across the whole of the IAMP site as part of the wider ground investigation works. The occurrence of these soils was found to be localised around the margins of the River Don.**

5.1.1 Geological Section A-A'

Geological Section A-A' is orientated north east – south west across the western side of the Phase ONE development area and is shown on Drawing 60283414-M015-ACM-L1-DR-GE_005. This section includes the footprint extent of Units 2, 7, 8 and 9.

BH51 was drilled approximately 50m south west of the Phase ONE site boundary. Topsoil (e1) was proved from ground level to 0.25m (39.6 to 39.35mOD). The topsoil is underlain by Pelaw Clay (b5) initially described as stiff brown mottled grey slightly sandy, slightly gravelly clay with subangular to subrounded, fine to coarse gravel of mudstone and siltstone to 2.35m (37.25mOD) becoming firm to stiff reddish brown slightly sandy gravelly clay to 3.2m (36.4mOD). The presence of stiff mottled clay close to existing ground is typical of a weathered desiccated surface 'crust' formed as a result of post depositional weathering resulting from seasonal wetting and drying, surface drainage changes, soil oxidation and weathering resulting in the leaching out of carbonates.

Based on BH51, at the south west extent of the Phase ONE development area, the Pelaw Clay (b5) is directly underlain by weathered siltstone (a3(w)) which are considered to be part of the Pennine Middle Coal Measures. The top of the siltstone is was completely weathered, described as very dense light grey sandy angular to subangular gravel of siltstone to 4.15m (35.45mOD). The weathered siltstone was underlain by weak partially weathered, grey fine grained sandstone (a1) with closely to medium spaced, sub-horizontal, planar, smooth fractures which are infilled with light grey clay to 8.35m (31.25mOD) becoming medium strong, partially weathered, light grey fine grained sandstone with medium spaced sub-horizontal, planar, smooth, undulose and clean fractures between 9.11m and the base of the hole at 13.15m (30.49 to 26.45mOD). The sandstone was interbedded with a band of weak to medium strong, partially weathered, grey siltstone (a3) with closely spaced, sub-horizontal, planar, smooth and clean fractures between 8.35 and 9.11m (31.25 to 30.49mOD).

Trending north east, BH's 45, 46 and 47 show the anticipated ground conditions below the footprint of Unit 2, which is typical of the large structure units proposed on the IAMP Phase ONE development (approximately 290m length and 130m width in plan). These exploratory holes were drilled along the south western limb of the infilled glacial valley and they demonstrate the change in the depth to bedrock surface which occur across this feature, see Drawing 60283414-M015-ACM-L1-DR-GE_003.

In BH46 the Pelaw Clay (b5) was proved below topsoil to 2.2m (36.36mOD). The Pelaw Clay was underlain by laminated clay (b2) of the Tyne and Wear Complex, described as stiff greyish brown slightly sandy slightly gravelly clay with subangular to rounded, fine to coarse gravel of sandstone, mudstone, siltstone and coal to 7.9m (30.66mOD). The superficial soils were again underlain by rocks inferred to be part of the Pennine Middle Coal Measures, initially described as completely weathered siltstone (a3(w)) to 8.2m (30.36mOD), in turn underlain by an interbedded sequence of very weak and weak siltstone or mudstone (a2) to 14.86m (23.7 mOD). Below 14.86m, medium strong to strong, unweathered, grey fine grained sandstone with closely to medium spaced planar, smooth, clean fractures orientated between 20 and 30° was proved to the base of the hole at 30.6m (7.96mOD).

A similar soil sequence was proved in BH45 drilled approximately 290m north east of BH46, although the drift soils are noticeably thicker than found in BH51 to the south west as this hole is inferred to be located much closer to the centre of the buried valley, as shown on Drawing 60283414-M015-ACM-L1-DR-GE_003 . Topsoil (e1) was proved from ground level to 0.25m (35.63 to 35.38mOD) underlain by Pelaw Clay (b5) to 1.2m (34.43mOD). These soils are again underlain by laminated clay (b2) described as firm greyish brown slightly sandy slightly gravelly silty clay of intermediate plasticity (b2) which was proved to 9.5m (26.13mOD), underlain by greyish brown slightly sandy clayey silt (b4) to 10.7m (24.93mOD). Further lacustrine soils of the Tyne and Wear Complex described as soft, slightly sandy slightly gravelly silty clay of intermediate plasticity (b2) was proved to 12.7m (22.93mOD). The lacustrine soils were underlain by a thin layer of Lower Glacial Till (b1) described as stiff brown slightly sandy slightly gravelly clay of low plasticity with subangular to rounded, fine to coarse gravel of sandstone, mudstone and coal to 13.6m (22.03mOD), with rocks of the Pennine Middle Coal Measures inferred to be present to the base of the hole at 19.1m (16.53mOD).

A thin localised layer of made ground described as brown slightly sandy slightly gravelly clayey topsoil with subangular to rounded, fine to coarse gravel of sandstone, mudstone, coal and brick was proved from ground level to 0.6m (36.54 to 35.94mOD) in BH47 drilled approximately 150m south east of BH45. This is interpreted to constitute reworked or ploughed topsoil resulting from agricultural use as opposed to made ground associated with a specific historical or potentially contaminative land use. In this hole, the Lower Glacial Till present above the bedrock is noticeably thicker than BH45 which demonstrates the variable thickness of laminated clays deposited in the glacial valley. Similar thin localised deposits of made ground were encountered at ground surface in BH13, TP's 9, 10 and 11.

These holes demonstrate that the lacustrine Tyne and Wear Complex soils thicken markedly north east across the proposed footprint of Unit 2. This change in ground conditions will require detailed consideration in foundation design and may result in a marked change in pile foundation lengths and capacities over the extent of the structure, which may be particularly important where piles are designed to be end bearing in competent bedrock which is anticipated to vary in depth. Similarly, variable depths of Pelaw Clay (b5) and laminated clay (b2) superficial soils will need to be considered for ground bearing floor slabs, which may alternatively need to be piled in order to achieve serviceability limits over variable depths of drift. In addition to the anticipated variability in the depth to bedrock, there is variability noted in the thickness of rock surface weathering, rock strength, rock type (e.g. sandstone (a1), mudstone (a2) or siltstone (a3)) which will require consideration as part of pile foundation design, particularly in the assessment of pile rock socket lengths required to achieve adequate pile toe capacity.

Trending north east similar ground conditions and thick deposits of Tyne and Wear Complex superficial soils were proved below the proposed footprint of Units 7, 8 and 9.

The thickest depth of drift soils were proved in BH14 immediately north east of the Phase ONE development boundary. In this hole drift was proved from ground level to 20.33m (37.09 to 16.76 mOD). BH13 was drilled outside of the Phase ONE area and shows the north east extent of the infilled valley, where drift soils again begin to reduce in thickness with bedrock proved at shallower depth at 8.16m (28.84 mOD).

CPT's 14, 15, 22, 31C, 32B and 33 were pushed close to the alignment of Section A-A' with cone resistance (q_c) and friction ratio (f_r) used to estimate undrained shear strength in fine (cohesive) soils and provide an interpretation of ground conditions for correlation and comparison with those proved in adjacent boreholes. The CPT test data traces (q_c, f_r and S_u) are shown on the geological sections with the other exploratory holes, see Drawings 60283414-M015-ACM-L1-DR-GE-005 to 007. A fr value of 2% is taken as the division between coarse (granular) and fine (cohesive) soils, as summarised in Table 4. The ground conditions and strata proved have been broadly inferred based on the following assumptions:

Table 4. Strata Interpretation from CPT Data Output

Strata Division	Strata Type	Friction Ratio (%)	Cone Resistance (MPa)
Fine	Pelaw Clay (b5)	>2%	<5MPa
Fine	Tyne and Wear Complex – Laminated Clay (b2)/ Silt (b4)	>2%	<5MPa
Fine	Lower Glacial Till (b1)	>2%	>5MPa
Coarse	Sand (b3)	<2%	>5MPa
N/A	Rock (a1 –a4)	N/A	>15MPa

All CPT's terminated due to 'refusal' resulting from elevated cone resistance suggesting very stiff soils or bedrock. With the exception of CPT14, there is generally good correlation with the CPT termination depth and depth of bedrock proved through cable percussive drilling and/or rotary coring. The ground conditions inferred from the CPT traces shown on Section A-A' and summarised in Table 5 below.

Table 5. CPT Data – Inferred Ground Conditions and Observations – Section A-A'

CPT Number	Ground Conditions and Observations
CPT14	CPT 14 has proved Pelaw Clay (b5) from near ground level (37.2mOD) to around 32.5mOD, underlain by interbedded sands (b3) and clays to around 30.5mOD. Laminated clay (b5) is proved to 24 mOD, underlain by Lower Glacial Till (b1). The strata proved generally correlates well with the ground conditions proved in BH25. However, CPT 14 terminated at 15m (21.8mOD) within the Lower Glacial Till, whereas these soils were proved to 17.33m (19.24mOD) in BH25. The till was described in BH25 as stiff, dark brown slightly sandy slightly gravelly clay subangular to subrounded, fine to coarse gravel of sandstone, mudstone and coal. It is considered likely that this CPT terminated on a boulder or cobbler obstruction within the Lower Glacial Till above bedrock.
CPT 15	CPT 15 indicates the presence of Pelaw Clay (b5) from near ground surface (36.8mOD) to approximately 34.5mOD, underlain by laminated clay (b2) to around 24.0mOD. A thin band of Lower Glacial Till (b1) was proved between 24.0 and 23.0mOD where the cone terminated on strata inferred to be bedrock. The ground conditions inferred from CPT 15 correspond well with those proved in BH16. A plot illustrating material types and undrained shear strength inferred from the CPT data against depth is shown on Figure 1, further discussed in Section 6 of this report.
CPT22	CPT 22 proved Pelaw Clay (b5) from ground surface to around 31.0mOD. An interbedded sequence of clay and sands (b3) is shown between 31.0 and 28.0mOD, in turn underlain by laminated clay (b2) of the Tyne and Wear Complex to approximately 22.5 mOD. The Lower Glacial Till (b1) was proved from 22.5 to approximately 21.0mOD, where bedrock is inferred on termination of the CPT. The strata inferred from the CPT corresponds well BH24, although the ability of the CPT to identifying soil microfabric features such as thin interbedded bands of sands and clays is demonstrated. A plot illustrating material types and undrained shear strength inferred from the CPT data against depth is shown on Figure 2, further discussed in Section 6 of this report.
CPT31C	CPT31C shows the presence of Pelaw Clay (b5) from near ground level (39.07 mOD) to around 35.0mOD. The CPT plot identifies a thin sand band at around 35mOD; this was not unexpected as localised water bearing sand bands are anticipated to occur widelywithin the Tyne and Wear Complex. However, between 35.0 and 30.0mOD, the CPT trace suggests the presence of the Lower Glacial Till (b1), which does not correspond to BH46, where soils are interpreted to comprise Tyne and Wear Complex Laminated Clay (b2). However, the CPT termination depth corresponds well with the depth of bedrock proved in BH46.
CPT32B	The CPT indicates the presence of Pelaw Clay (b5) from ground surface (37.13mOD) to approximately 34mOD, underlain by Tyne and Wear Complex Laminated Clay (b2) to 30.0mOD. The CPT trace indicates the presence of the Lower Glacial Till (b1) below 35.0mOD, with bedrock inferred at approximately 25.2mOD, which corresponds with the strata proved in BH47. A plot illustrating material types and undrained shear strength inferred from the CPT data against depth is shown on Figure 4, further discussed in Section 6 of this report.
CPT33	The depth to bedrock corresponds with the depth proved in BH46.

Groundwater strikes and rise as well as the results of groundwater monitoring from installed standpipe piezometers is shown on the geological section. Groundwater is also plotted against depth and level on Figures 5 and 6, further discussed in Section 6.11 of this report.

5.1.2 Geological Section B-B'

Geological Section B-B' is orientated north east to south west and is parallel to Section A-A cutting through the centre and east side of the site. This section also shows the superficial soils to thicken markedly towards the north east as it crosses the centre of the infilled glacial valley, see Drawings 60283414-M015-ACM-L1-DR-GE_003 and 005. Geological section B-B' crosses the footprint of Units 1, 3, 4, 5 and 6. In this section, it is not intended to provide a detailed description of the soils proved or variation in rockhead level as this detail was provided in the discussion of conditions for Section A-A' (see Section 5.1.1 above). A comparison between the two sections on Drawing 005 confirms that a similar stratigraphy was encountered. From geological section B-B' the following is inferred:

- There is a significant variation in the thickness of superficial soils and depth to bedrock over the proposed footprints of Units 1 and 3; this will require further consideration as part of detailed design and will probably have an impact on floor slabs and structure foundations, especially if pile foundations are proposed.
- A significant thickness of Pelaw Clay and Tyne and Wear Complex high plasticity clay is anticipated below the proposed footprints of Units 4, 5 and 6. This will also be a consideration in foundation (including pile) design and may also restrict the type and proximity of landscaping planting such as trees close to the proposed buildings. The minimum foundation depths for buildings will need to take account of the shrinkage/ swelling potential of these soils.
- Variation in depth to bedrock, rock type, strength and weathering below the proposed footprints of Units 4, 5 and 6 (for consideration in foundation (including pile) design).
- CPT's 19 and 22 all terminated at depths well above the level of the bedrock proved by cable percussive and rotary drilling. These CPT's are interfered to have terminated on boulder obstructions within the Lower Glacial Till (b1). Construction of piles using continuous flight auger (CFA) techniques through this material may therefore be problematic.
- CPT 26 suggests laminated clay soils may extend to 25mOD; this level is slightly below than the base of these soils proved in BH's 46 and 47. A plot illustrating material types and undrained shear strength inferred from CPT 26 plotted against depth is shown on Figure 3, further discussed in Section 6 of this report.
- Groundwater monitoring of standpipe piezometers and standpipes again shows water levels to be close or just below ground level. This will have significant impacts on structure foundation, road pavement and highway run-off drainage design as well as impacting the methods used to support temporary excavations and impacting on the trafficability of earth moving plant during construction at the wettest and coolest times of the year.
- There is variability within the bedrock sequence identified in exploratory holes, particularly in BH's 17, 27 and 28. In BH28 assumed zones of core loss (AZCL), coal (a4) and possible coal workings (a4(work)) have been identified. Possible coal workings were also encountered in BH30, located outside of the IMAP site boundary, approximately 200m south east of BH28. Of note is that in BH's 28 and 30 the coal is shown to be interbedded with layers of soft to firm gravelly clay. These layers may represent poor rotary drilling recovery, however, the possibility of unrecorded coal workings subsequently infilled with goaf (fine cohesive materials0 following roof collapse or before abandonment cannot be ruled out, particularly below the footprints of Units 4, 5 and 6.

5.1.3 Geological Sections C-C', D-D' and E-E'

Geological Sections C-C', D-D' and E-E' are provided on Drawings 60283414-M015-ACM-L1-DR-GE_006 and 007. Geological section C-C' extends across the footprint of Units 1 and 2, with D-D' orientated through the centre of the proposed balancing ponds and includes the footprint of Units 4 and 5. Geological section E-E' is relevant to Units 5, 6 and 7. In this section, it is not intended to provide a detailed description of the soils proved as this detail was provided in the discussion of conditions for Section A-A' (see Section 5.1.1 above).

The purpose of these sections is to provide information on the anticipated ground conditions below the proposed development units for use in detailed design of foundations and infrastructure (by others) as part of future phases of scheme development. However, the following summary observations are made:

- Section C-C' is orientated north west – south east on the south west portion of the Phase ONE development area. The section is orientated across the south western limb of the infilled glacial valley, see Drawings 60283414-M015-ACM-L1-DR-GE_003 and 004. The depth of superficial soils and bedrock level is shown to vary below the footprint of Units 1, 2 and 3. This change in ground conditions will require further consideration as part of detailed design of floor slabs and structure foundations, particularly if piles are to be installed to support the buildings. A preliminary design assessment is presented in Section 6.14.

- Section D-D' is orientated north west – south east, broadly through the centre of the Phase ONE development area and includes the proposed balancing ponds. The eastern half of this section crosses the centre of the infilled glacial valley. The ground investigation indicates the balancing ponds are likely to be excavated within Pelaw Clay (b5). Ground investigation data suggests the depth to rock head below the footprint of Unit 4 may be relatively consistent, but may decrease moving north over the footprint of Units 8 and 9, see Drawing 60283414-M015-ACM-L1-DR-GE_003. Data suggest rock head to be around 15m below units 8 and 9 and around 20m below Unit 4. Rock head deepens towards the south east along the alignment of the infilled valley from around 21 to 15 mOD.
- Section E-E' is orientated north west – south east through the north east portion of the Phase ONE development area and the centre of the infilled glacial valley. As a result, ground investigation data suggests the depth to rockhead below Units 5, 6 and 7 will be relatively consistent below the footprint of the structures at around 17 to 18m, see Drawing 60283414-M015-ACM-L1-DR-GE_003. However, depth to rock head may be more variable below Unit 7.
- Monitoring of groundwater in standpipe piezometers and standpipes shows water levels to be close or just below ground level; this is an important potential constraint which needs careful consideration during the design of structure foundations, road pavements and drainage and temporary excavations. Depending on the time of year when bulk earthworks are undertaken, this could also result in trafficking problems during construction, resulting in restrictions on the size of earthworks moving plant as well as reducing efficiency below normal expected productivity.

5.1.4 Geological Section F-F'

Geological Section F-F' is provided on Drawing 60283414-M015-ACM-L1-DR-GE_007. The section is drawn along the alignment of the Primary Access Road into the Phase ONE development area. The purpose of this section is to provide information on the anticipated ground conditions below the proposed road for use in detailed design of the road pavement and associated highway drainage (by others) as part of future phases of scheme development.

The alignment of the road is provided on SYSTRA Highway General Arrangement Drawing IAMP_ONE-SYS-HGN-ZA1-DR-D-01-001-S0-P04 dated 16/01/18 included in Appendix A. Longitudinal sections showing the proposed road elevation and anticipated cut and fill have not been provided. However, significant earthwork cuttings or embankments are not anticipated so for the purposes of this preliminary assessment it is assumed that the road is to constructed at grade.

Assuming the road is at grade or in shallow cutting, the geological section shows the subgrade over the whole length of the road will comprise Pelaw Clay (b5). Groundwater monitoring of standpipe piezometers and standpipes shows water levels to be at or just below ground level. The high water table will need to be considered in assessment of long term equilibrium CBR conditions and it will also have an impact on the trafficability of earthworks moving plant during construction. Preliminary design recommendations are provided in Section 6.14 of this report.

6. Ground Conditions and Material Properties

This section of the report describes the material properties for each strata encountered during the ground investigation. The geotechnical parameters and material properties provided in this GIR are used to inform a preliminary assessment of the ground conditions for the purposes of preliminary engineering assessment and to inform detailed design to be undertaken by others as part of future phases of development.

For the purposes of this report, material properties and test data are discussed for each stratigraphic unit identified in Section 5 and summarised in Table 3. Materials have been grouped on the plots based on sample description and are identified using the strata codes described in this report and shown on the geological sections presented on Drawings 60283414-M015-ACM-L1-DR-GE_005 to 007.

Geotechnical plots presented as Figures 7 to 62 show the in-situ and laboratory test data obtained from the Draft Dunelm Preliminary SI Report, February 2018 [Ref. 10].

Although the site is relatively flat, data is plotted against both depth below ground (mbgl) and ordnance datum level (mOD) for completeness. The range of depths (0 to 28m) and levels (40 to 10mOD) are kept consistent on all plots to aid visual assessment of variation in depth and thickness of each material type relevant to the total depth of superficial soils and rockhead levels proved.

The geotechnical parameters presented are considered 'moderately conservative' and have been derived from in-situ and laboratory testing and empirical correlations supplemented by engineering judgement, where necessary.

The results have been used to provide characteristic geotechnical design values for each material type for use in design. Proposed values are summarised in Table 20.

The methodology adopted in assessment and material properties plotted include:

- Atterberg Limits and natural water (moisture) contents are presented against depth and level. In addition, Atterberg Limits are presented on Casagrande plasticity charts. This data has been used to calculate consistency index (I_C) for fine soils in accordance with BS EN ISO 14688-2:2004+A1:2013 [Ref. 3] from which a consistency term is derived; this can then be compared against equivalent consistency term assigned to the stratum on the borehole log.
- Gradings obtained from particle size distribution analyses are plotted on grading charts based on strata type. The samples were tested in accordance with BS1377-2:1990 [Ref. 11].
- The shape of the grading curves has been classified in accordance with Table 2 of BS EN ISO 14688-2:2004+A1:2013 [Ref. 3] based on C_u, and C_c, calculated from the laboratory grading curves. Uniformity coefficient and coefficient of curvature are provided for coarse (granular) soils.
- Standard penetration tests (SPT) results are plotted against depth and level. Uncorrected SPT ‘N’ values obtained from the full 300mm test drive are shown as well as any extrapolated values derived from partially completed tests limited to a maximum value of 100.
- Undrained shear strength (c_u) values are plotted against depth. The data includes undrained shear strengths measured from hand shear vanes undertaken during the excavation of borehole inspection pits and trial pits. The data also includes c_u values measured from unconsolidated undrained laboratory triaxial tests. For fine soils c_u has also been estimated from the uncorrected SPT ‘N’ values in accordance with the empirical correlation $c_u = f_1$*N (kPa), as devised by Stroud and Butler (1975) [Ref. 12]. Values typically used for 'f_1' generally range between 4 and 6, depending on soil plasticity. For the purposes of this assessment an ‘f_1’ value of 4.0 is adopted for the fine grained soils. Undrained shear strengths for fine soils have also been estimated using the empirical relationship based on Liquidity Index (I_L), as proposed by Wroth and Wood (1978) [Ref. 13]. This has since been modified to consider research undertaken by Barnes and Staples (1988) [Ref. 14]. It is noted that BS EN 1997-1:2004+A1:2013 (Eurocode 7) [Ref. 3] states that design values should be based on direct measurements of shear strength.
- Cone resistance (q_c) and friction ratio (f_r) as well as the inferred ground conditions and strata proved are shown on the geological sections. Undrained shear strengths calculated from CPT’s shown on the

geological sections are estimated using correlation $S_u = (q_c - \sigma_{vo})/N_k$ where σ_{vo} is total in situ vertical stress and N_k is an empirical cone factor. A value of N_k of 20 is adopted for the fine soils on the site which is considered conservative.

- Bulk density determinations were made in the laboratory as part of the undrained triaxial testing on fine soils and during compaction testing on prepared coarse soils. These laboratory results are augmented with recommendations detailed on Figures 1 and 2 of BS8004: 2015 (Ref 16).
- Calculated values of constant volume effective angle of shearing resistance (φ'_{cv}) are plotted against depth and level. Characteristic values for fine soils were calculated based on measured plasticity indices using an empirical relationship in BS8002: 2015 (Ref. 15), which is based on an equation proposed by Santamarina and Diaz-Rodriguez (2003) (Ref. 16).
- One-dimensional consolidation (oedometer) testing was undertaken during the ground investigation. Values of compression index (Cc) and swelling index (Cs) derived from one-dimensional consolidation (oedometer) testing are plotted against depth. This data has been augmented by indices determined from correlation with index properties in accordance with the empirical relationship with liquid limit proposed by Skempton (1944) (Ref. 17). Cs values are estimated as a proportion of the inferred Cc_{LL} values in accordance with Sladen and Wrigley (1983) (Ref. 18).
- Curves of dry density against moisture content obtained from laboratory compaction tests undertaken using the BS 2.5kg rammer method are presented for bulk disturbed samples on the Pelaw Clay (b5) that was present close to ground surface across the IAMP ONE site. The 0, 5 and 10% air void lines are calculated based on an average measured ρ_s of 2.53Mg/m^3 and are shown for comparison with the compaction curves. The ρ_s of 2.53Mg/m^3 is slightly lower than a typical value of 2.65Mg/m^3 suggesting some organic material may be present.
- Pont load testing and unconfined compressive strength testing on rock is plotted against both depth and level. The point load strength test is frequently used to determine crushing strength through established empirical relationships like those proposed by Broch, E and Franklin, JA (1972) (Ref. 20). Johnston (1991) (Ref. 21) suggests typical multiplication factors to convert ($I_{s(50)}$) to UCS; typically values between 15 and 25 are applied to diametral tests. Gannon, JA, Masterton, GGT, Wallace, WA and Muir Wood, D (1999) (Ref. 22) states that the development of site specific or formation specific correlations between UCS and $I_{s(50)}$ are essential. Test data indicates a value of 23 is appropriate for preliminary assessment.

A summary of the in situ and laboratory testing undertaken on the various strata encountered within the Phase ONE development area are provided as tables with minimum, maximum and average values presented which have been used in the derivation of characteristic geotechnical parameters provided for preliminary design.

6.1 Topsoil (e1)

Topsoil (e1) was encountered BH's 14 to 17, 24 to 25, 39 to 40, 45 to 46, 49, 50 to 52 and in TP's 35 to 38. The topsoil was described as brown or dark brown slightly sandy slightly gravelly clayey topsoil with subangular to rounded, fine to coarse gravel of sandstone, mudstone and coal. The topsoil was typically between 0.1 and 0.35m thick, although it was 0.4 and 0.45m thick in TP37 and BH14, respectively. The thicker deposits of topsoil are most likely a result of deep agricultural ploughing.

The results of in-situ and laboratory tests undertaken in the Topsoil (e1) are summarised in Table 6.

Table 6. Summary of In-situ and Laboratory Test Results - Topsoil (e1)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Hand shear vane, $c_{u\ HV}$ (kPa)	4	24	26	25

6.2 Made Ground - Topsoil (d1 to d5)

6.2.1 Made Ground Topsoil (d1)

Made ground topsoil (d1) described as dark brown slightly clayey, slightly gravelly sandy topsoil with subangular to subrounded, fine to coarse gravel of sandstone, mudstone, limestone, concrete and brick was proved in BH's 13, 28, 31, 35, 38, 47, CPT's 28, 29, TP's 10, 13, 16, 16A, 19 and TPS's 02 and 04. The soils were proved to be typically between 0.2 and 0.3m thick, although thicker deposits of between 05 and 0.6m were proved in BH's 28, 31, 35 and 37. There is no discernible deposition pattern for these soils which are interpreted to be reworked topsoil which contain occasional fragments of man-made materials such as brick and concrete. They are inferred to be present due to deep agricultural ploughing as opposed to any particular former historical or potentially contaminative land use.

The results of in-situ and laboratory tests undertaken in the Made Ground Topsoil (d1) are summarised in Table 7.

Table 7. Summary of In-situ and Laboratory Test Results – Made Ground Topsoil (d1)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Hand shear vane, $c_{u\ HV}$ (kPa)	1	25	-	-

6.2.2 Made Ground Topsoil (d2)

Made ground topsoil (d2) described as dark brown slightly sandy slightly gravelly clayey topsoil with angular to subrounded, fine to coarse gravel of sandstone, occasional mudstone, rare coal, glass, ceramic, clay tile and pot was proved locally in TP's 25, 30, 31 and TPS's 01 and 03. These made ground surface soils are differentiated due to the presence of tile, glass and ceramics although there is no pattern to their deposition and they are also again inferred to be associated with deep agricultural ploughing with the presence of ceramics and tile most likely a result of damage to installed land drains over time.

6.2.3 Made Ground Topsoil with Organic Matter (d3) and with Ash (d4)

Made ground (d3) (Topsoil with organic matter) and (d4) (cohesive made ground with ash) were proved locally within the whole of the IAMP site area investigated but were not proved within any of the exploratory holes excavated in the Phase ONE development area and are therefore not discussed further.

There were no in-situ or laboratory tests undertaken on made ground topsoil (d3) or cohesive made ground with ash (d4).

Thicker deposits of cohesive made ground (d5) described as orangish brown slightly sandy gravelly clay of intermediate to high plasticity with subangular to subrounded, fine to coarse gravel of coal, sandstone and brick were proved locally in BH's 28, 31 and 35 from 0.5m to 3.0m (35.25 to 32.75mOD), 0.5 to 1.7m (34.68 to 33.48mOD) nd 0.5 to 1.65m (33.96 to 32.81mOD), respectively. Historical OS mapping and site walkover does not indicate any specific land use that can be attributable to the presence of these soils. It is considered possible that these materials represent natural soils and that the brick gravel has either dropped down from surface during drilling or actually represents natural red sandstone within the Pelaw Clay (b5). It is considered likely that these soils are mis-logged. However, consideration could be given to determine the presence/ absence of these soils as part of additional detailed ground investigation (to be undertaken by others) as part of further phases of design. BH28 was drilled adjacent to the proposed position of Unit 5, whereas BH's 31 and 35 are below the proposed drainage and balancing ponds/ outfalls which are to be constructed north west - south east through the approximate centre of the Phase ONE development area.

The results of in-situ and laboratory tests undertaken in the Made Ground – Cohesive (d5) as well as calculated geotechnical parameters adopting the methodologies described under Section 6 above are summarised in Table 8.

Table 8. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Made Ground – Cohesive (d5)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Uncorrected SPT 'N' value (blows/300mm)	1	16	-	-
Natural Water Content, w (%)	5	21	28	25
Liquid Limit, w_L (%)	3	46	62	52
Plastic Limit, w_P (%)	3	16	23	19
Plasticity Index, I_p (%)	3	24	43	32
Percentage passing 425μm sieve (%) #2	3	98	100	99
Weight Density (Mg/m^3)	2	1.94	2.01	1.98
Dry Density (Mg/m^3)	2	1.52	1.66	1.59
BS 8002:2015 φ' cv,k degrees **(calculated)**	3	21.6	24.8	23.3
Consistency index (I_C) **(calculated)**	3	0.79	0.83	0.82
Particle size distribution: % passing 63μm sieve (%)	3	81	84	83
Hand shear vane, c_u HV (kPa)	0	-	-	-
c_u HV (kPa) Triaxial	0	-	-	-
c_u (kPa) W_L **(calculated from liquidity)**	3	57.3	69.7	65.5
C_u SPT N (kPa) 'f_1' = 4.0 **(calculated)**	1	64	-	-
C_cWL = 009(LL-10) **(calculated)**	3	0.32	0.47	0.38
C_sWL **(calculated)**	3	0.046	0.067	0.054

6.2.4 Particle Size Distribution

Particle size distribution curves for the cohesive made ground (d5) are shown on Figure 7. Testing was undertaken on samples from BH28 at 2.0m, BH31 at 0.8m and BH35 at 0.6m. The plot shows a tight range of well graded curves with all of these exhibiting more than 80% passing the 2mm sieve (ranging between 96 and 100%). The soils tested exhibit more than 80% passing the 63μm sieve, producing gradings typical of the Pelaw Clay, discussed in Section 6.4 below. The gradings undertaken on soils logged as made ground suggest the soils are either derived from the Pelaw Clay (b5) or have mis-logged and are actually representative of natural fine (cohesive) soils.

Values of uniformity coefficient (C_U) and coefficient of curvature (C_C) were not reported due to the high proportion of fines, (percentage of material passing 63µm sieve) which was present.

6.2.5 Unit Weight

An average bulk density of 1.98 Mg/m^3 is obtained from laboratory testing. A characteristic value of 2.0 Mg/m^3 is adopted, which lies close to the middle of the range of weight density suggested for medium strength fine soils in accordance with Figures 1 and 2 of BS8004: 2015 (Ref. 15).

6.2.6 Moisture Content and Atterberg Limits

Atterberg Limits tests were undertaken on three samples of Cohesive Made Ground (d5) recovered from BH's 28, 31 and 35. Plasticity data is plotted on a Casagrande plasticity chart on Figure 8 which indicates the clays are of intermediate and high (CI and CH) plasticity.

Natural moisture content, liquid and plastic limits plotted against depth and level are shown on Figures 9 and 10, respectively. There is no trend with depth which would be anticipated given the relatively shallow thickness of the material proved and the limited data set due to the localised distribution of these soils across the site.

6.2.7 Consistency

Values of consistency index (I_C) between 0.79 and 0.83 indicate the material to be of firm consistency.

6.2.8 Undrained Shear Strength

Undrained shear strength calculated from a single uncorrected SPT 'N' value and three liquidity indices derived from the moisture content and Atterberg Limits are plotted against depth and level on Figures 11 and 12. Test data indicates a characteristic c_u value of 60kPa to be appropriate.

6.2.9 Drained Shear Strength

Values of constant volume effective angle of shearing resistance (φ'_{cv}) calculated from correlation with plasticity index range between 21.6 and 24.8°. The results are plotted against depth and level of Figures 13 and 14. A characteristic φ'_{cv} value of 23° is adopted for design.

6.3 Alluvium (c1 to c2)

Localised alluvial soils were proved adjacent to the River Don as part of the ground investigation works undertaken for the whole of the IAMP site. Alluvial soils were not encountered within the Phase ONE development area and are therefore not discussed further in this report.

However, it is noted that a 250m length of the western site boundary the site follows the alignment of a tributary of the River Don (Usworth Burn), converging with the River Don at the corner of this area. When the preliminary GI was designed, no development was proposed at this location. However, balancing ponds are now proposed in this area and the presence of alluvial soils would be anticipated within the watercourse floodplain limits. Further ground investigation is recommended to determine the composition, thickness and engineering properties of the alluvial soils.

6.4 Pelaw Clay (b5)

Natural soils comprising Pelaw Clay (b5) described as soft, firm or stiff orange brown, reddish brown, greyish brown, brown or dark brown, mottled light grey, slightly sandy slightly gravelly clay of intermediate and high plasticity with subangular to rounded, fine to coarse gravel of sandstone, mudstone and coal were proved in all exploratory holes formed within the Phase ONE development area. The soils were occasionally recorded as laminated and noted to become stiff with depth. The soils were proved at depths from 0.1m (38.46mOD) to 3.1m (33.34mOD) extending down to between 0.4m (35.47mOD) and 7.0m (28.75mOD). The maximum thickness of these soils was 5.95m (BH14).

The results of in-situ and laboratory tests undertaken on the Pelaw Clay (b5) as well as calculated geotechnical parameters adopting the methodologies described under Section 6 above are summarised in Table 9.

Table 9. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Pelaw Clay (b5)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Uncorrected SPT 'N' value (blows/300mm)	28	7	100*****	15.5
Natural Water Content, w (%)	59	13	41	22
Liquid Limit, w_L (%)	46	31	85	45
Plastic Limit, w_P (%)	46	15	32	20
Plasticity Index, I_p (%)	46	14	53	25.5
Percentage passing 425µm sieve (%) #2	46	73	100	94.6
Weight Density (Mg/m^3)	13	1.96	2.22	2.07
Dry Density (Mg/m^3)	13	1.50	1.90	1.73
Compaction - Maximum Dry Density (Mg/m^3)	9	1.61	1.74	1.69
Compaction – Optimum Moisture Content %	9	9	21	14
BS 8002:2015 φ' cv,k degrees **(calculated)**	46	20	28	24
Consistency index (I_C) **(calculated)**	45	0.04	1.29	0.89
Particle size distribution: % passing 63µm sieve (%)	23	49	91	70.35
Hand shear vane, c_u HV (kPa)	38	31	126	76
c_u HV (kPa) Triaxial	13	19	180	106
c_u (kPa) W_L **(calculated from liquidity)**	46	2	250**	117
C_u SPT N (kPa) 'f_1' = 4.0 **(calculated)**	28	35	250**	78
C_c WL = 009(LL-10) **(calculated)**	46	0.19	0.68	0.32
C_sWL **(calculated)**	46	0.027	0.096	0.045

* SPT N values extrapolated to a maximum of 100 blows, excluded from average value presented.

** Cu extrapolated to a maximum value of 250 kPa.

6.4.1 Particle Size Distribution

Particle size distribution curves for the Pelaw Clay (b5) are shown on Figure 15. Testing was undertaken on 28 samples at depths ranging between 1.2 and 4.4m. The gradings fall into a tight envelope with the samples containing a wide range of particle sizes, typical of well graded soils. All of the curves exhibit more than 80% passing the 2mm sieve (ranging between 82 and 100%).

Most of the soils tested (25 out of the 28 samples) exhibit between 60 and 91% passing the 63µm sieve, indicative of a high fines content and intermediate to high plasticity clays. Three of the 28 tests contained a lower percentage passing the 63µm sieve (49 to 55%) and may represent slightly coarser less plastic soil layers present within the Pelaw Clay (b5). These gradings were noted to include minor proportions of sand and fine and medium gravels, sufficient for them to classify as Class 2A/ 2B materials in accordance with the HASHW Series 600 Earthworks Specification [Ref. 27]. However, the majority of the Pelaw Clay (b5) would classify as Class 2D (silty cohesive fill) by virtue of grading, see Section 6.14.4 below.

One grading sample plots as an evenly graded soil comprising silt and sand. The sample was obtained from TP31 at 4.4m. The sample is described on the laboratory test sheet as brown, slightly clayey, slightly sandy silt. This corresponds to the exploratory hole log which indicates that grey clayey slightly sandy silt was proved at the base of the hole, at a similar depth of 4.5m.

Values of uniformity coefficient (C_U) and coefficient of curvature (C_C) were generally not reported due to the high proportion of fines, (percentage of material passing 63µm sieve) which was present. Values of C_U of 19 and C_C of 0.45 were only reported for one of the 28 samples of Pelaw Clay (b5); this was from BH39 at 3.4m, and the result is indicative of a multi-graded soil. The sample was recovered from strata described as firm greyish brown silty, slightly sandy clay of intermediate plasticity on the exploratory hole log; however the soil is described as brown, slightly gravelly, slightly sandy, clayey silt on the laboratory test sheet.

It is considered likely that the samples obtained from both TP31 and BH39 are representative of localised thin bands of silt rich material or discrete silty layers within the Pelaw Clay (b5), which were not easy to identify during cable percussive drilling.

6.4.2 Unit Weight

An average bulk density of 2.07 Mg/m^3 is obtained from laboratory triaxial tests undertaken to determine undrained shear strength. A characteristic ρ value of 2.1 Mg/m^3 is adopted, which lies towards the upper end of the range of weight densities suggested for medium strength fine soils in accordance with Figures 1 and 2 of BS8004: 2015 (Ref 15).

6.4.3 CBR and MCV

Compaction testing was undertaken on seven samples of Pelaw Clay (b5). Individual compaction curves obtained from each test are shown on Figure 16. The 0, 5 and 10% air void lines are calculated assuming a ρ_s of 2.53Mg/m^3, based on an average of measured values from laboratory tests, are shown for comparison with the compaction curves.

The plot shows that the these soils can be separated into two 'groups' of material which generally reflect the percentage of fines passing the 63µm sieve as described above. The maximum dry density values achieved ranged between 1.61 and 1.74 Mg/m^3 at optimum moisture contents of 9.1 to 10.8% (Group 1) and between 1.73 and 1.61 Mg/m^3 at optimum moisture contents between 16 and 21% (Group 2).

Natural moisture contents measured within the Pelaw clay (b5) at between 0.3 and 2.0m depth (33.7 to 38.2 m OD) fall in a wide range, with values between 14 and 38% (average 22%). In comparison, the as received moisture contents on the soils tested for compaction varied between 13 and 21%. The optimum moisture contents indicate the material is wet of optimum and precludes re-use in earthworks using moisture content criteria. A preliminary earthworks acceptability assessment based on the site wide moisture contents obtained in the Pelaw Clay (b5), suggests approximately half of this material would not be acceptable for re-use without pre-treatment. Limits of acceptability based on undrained shear strength should be considered as part of future phases of design, with c_u values between 50 and 200kPa typically accepted as lower and upper limits of undrained shear strength within which adequate compaction may be achieved with modern plant. It should also be noted that the moisture content of the near surface soils changes during the year due to variations in soil moisture deficit balance resulting from fluctuations in air temperature and precipitation. It is likely that the proportion of acceptable material could be much lower than suggested if bulk earthworks are undertaken during the typically wetter winter and spring periods of the year.

Six laboratory samples of Pelaw Clay (b5) were selected for CBR, MCV and undrained shear strength earthworks acceptability relationship testing.

Figure 17 shows the results of CBR test results plotted against measured moisture contents for each point of the compaction test curves. Data plotted includes measured CBR at the top and bottom of the mould. The data plot shows two trend lines, with the upper trend (shown black) considered representative of material containing a minor proportion of sand and gravel, and the lower trend (shown red) inferred to be typical of the behaviour of the finest parts of the Pelaw Clay (b5). These latter soils are likely to govern the behaviour of soils at road pavement formation and are therefore considered most critical for the purposes of this assessment. Data shows that the minimum required undrained CBR of 2.5% was achieved at soil moisture contents of 20% or lower, with undrained CBR increasing to approximately 10% at a moisture content of approximately 17%. However, the data shows when the soil moisture content reaches 21.5% the undrained CBR falls to less than 2%. As detailed above, site wide moisture contents within the Pelaw Clay (b5) fall in a wide range between 14 and 38% (with an average of 22% obtained from the 59 tests undertaken).

Over 70% of the moisture contents obtained in the Pelaw Clay (b5) were shown to exceed 20% suggesting that a significant proportion of Pelaw Clay soils on the site will exhibit an undrained CBR value below 2.5% and require formation treatment (thickened subgrade, dig out and replacement, improvement or strengthening) prior to construction of the bound pavement layers. This is further discussed in Section 6.14.7 of this report.

Figure 18 shows the results of average undrained shear strength from laboratory hand vanes within the CBR moulds plotted against the moisture contents measured at each point of the compaction curves. The lower trend line (shown red) is again considered to represent the most critical design case. The data shows that a moisture content of approximately 20% correlates with an undrained shear strength of 50kPa and therefore equates to an undrained CBR of 2.5% (as described above). However, a large proportion of the laboratory hand vane tests indicated higher undrained shear strengths exceeding 120kPa. An average undrained shear strength of >75kPa is suggested from site wide testing, however approximately 20% of the dataset produced c_u values below 50kPa. It is also noted that the CPT data indicated undrained shear strengths of around 50kPa, see Figures 1 to 4. The undrained shear strength of the material is discussed in more detail in Section 6.7.6 below. However, the data suggests a minor but significant proportion of the Pelaw Clay (b5) will exhibit CBR values below 2%. It is also noted that the ground investigation fieldwork was undertaken from late July to late November 2017, a relatively warm and drier part of the year when water levels would be expected to be relatively depressed. It is therefore likely that if construction is undertaken during the wetter parts of the year, the proportion of Pelaw Clay (b5) unable to achieve CBR 2% may be greater than estimated from the laboratory test data.

Figure 19 shows the results of average undrained shear strength from laboratory hand vane testing plotted against the average undrained CBR readings obtained. The purpose of this assessment is to estimate a specific correlation between undrained shear strength and undrained CBR specific to the Pelaw Clay (b5) present on this site. Data suggests a CBR correlation of ${}^{Cu}/_{20}$ is applicable. This correlation is slightly more favourable than a correlation of CBR~ ${}^{Cu}/_{23}$ published for glacial soils which is often used to provide general design guidance [Ref. 19].

Figure 20 is a plot of undrained shear strength and Moisture Condition Value (MCV) with Figure 21 presenting MCV against undrained CBR values, respectively. The purpose of these plots is to allow an alternative method for assessment of compaction acceptability criteria during construction (as opposed to undrained shear strength detailed above). The data indicates a MCV of around 8 correlates to a minimum undrained shear strength of 50kPa and a minimum undrained CBR of 2.5%. It is noted that a proportion of the test data shows a MCV below 8, again indicating that approximately 20% of the Pelaw Clay (b5) may exhibit an undrained CBR of less than 2%.

6.4.4 Moisture Content and Atterberg Limits

Atterberg Limits tests were undertaken on 46 samples of Pelaw Clay (b5) recovered at depths ranging between 0.5 and 5.0m. Plasticity data is plotted on a Casagrande plasticity chart on Figure 22 which indicates that the clays are generally of intermediate to high (CI and CH) plasticity. There are however a number of outliers which plot on both the low and high side of the main data body. Two samples (BH39 0.50m and TP19 0.40m) indicate clay of very high (CV) plasticity and five samples classify as clay of low (CL) plasticity. The spread of the results confirms the variable plasticity characteristics of this soil. Natural moisture content, liquid and plastic limits are plotted against depth and level on Figures 23 and 24, respectively.

Natural moisture contents within the Pelaw clay (b5) between 0.3 and 5.0m depth (33.7 to 38.2 m OD) fall in a wide range between 13 and 41% (average 22%). The most plastic soils are evident within 2.2m of ground surface. The development of highly plastic fine soils close to ground surface is believed to be due to post depositional weathering resulting from surface drainage changes, oxidation and leaching of carbonates. Numerous technical publications have suggested the development of thin highly plastic layers close to ground surface is typical of soils deposited in, or, altered under periglacial conditions across north east England. The data shows a trend of reducing plasticity (both LL and PL) with depth.

6.4.5 Consistency

Values of consistency index (I_C) between 0.04 and 1.29 (average 0.89) were calculated from the tests described in Section 6.4.4. The average I_C value indicates the material to be of stiff consistency. However, a lower average I_C value (of 0.75) indicative of firm soils was obtained from test data obtained at between 2.5 and 5m depth. The variation in I_C provides evidence of desiccation or weathering effects resulting in higher strength soils close to ground surface (as described above).

6.4.6 Standard Penetration Tests

Twenty eight uncorrected SPT's undertaken within the Pelaw Clay (b5) are plotted against depth and level on Figures 25 and 26, respectively. SPT 'N' values between 1.2 and 6.0m (29.75 to 38.51 mOD) range between 7 and 38 blows (and produce an average 15.5). The data is indicative of firm, stiff and very stiff soils, with the average 'N' value consistent with firm soils. There is no apparent trend of SPT 'N' values with depth or level.

A single SPT undertaken in BH51 at 3.0m (36.6 mOD) terminated after 50 blows (values > 50 have been calculated by linear extrapolation to a maximum value of 100). This test is thought to be due to a cobble or boulder obstruction and is not considered representative of the material consistency as a whole. As a result, this test result is excluded from the calculation of the average 'N' value.

6.4.7 Undrained Shear Strength

Undrained shear strengths have been directly measured in the field using a hand vane and in the laboratory during undrained laboratory triaxial testing. The direct strength measurements are augmented by c_u values inferred from correlation with SPT 'N' values and liquidity indices (as described in Section 6.0). The data are plotted against depth and level on Figures 27 and 28, respectively. Undrained shear strength in the Pelaw Clay (b5) indicates a wide range of results. There is no apparent trend of increasing strength with depth.

Undrained shear strengths estimated from correlation with SPT 'N' values range between 35 and 190kPa (excluding a extrapolated value of 250kPa). An average c_u value of 78kPa is inferred.

Undrained shear strengths obtained from 38 in-situ hand vane testing carried out in the Pelaw Clay (b5) at depths between 0.4 and 4.3m (31.17 and 39.10mOD) indicate c_u ranging between 31 and 126kPa, with an average value of 76kPa. These measured strengths correlate well with c_u values inferred from SPT N results. The results indicate c_u values between 50 and 100kPa between ground level (GL) and 2.5m, although lower undrained strengths of between 31 and 83kPa are indicated below 2.5m to 4.3m (average 57kPa). The higher strengths may be due to desiccation and seasonal drying of shallow soils immediately below ground surface.

A total of 13 undrained triaxial tests were undertaken in the laboratory on undisturbed samples of Pelaw Clay (b5) recovered at depths between 1.2 and 5.0m (30.75 to 37.36mOD). Undrained shear strengths measured by triaxial testing range from 19kPa (BH28 at 5.0m, 30.75 mOD) to 180kPa (BH47 at 1.2m, 36.54 mOD) with an average value of 106kPa obtained. However, there appears to be a distinct reduction in strength with depth (test data shown red) with undrained shear strengths below 2.5m varying between 19 and 110kPa (average 68kPa). The triaxial testing again provides evidence of higher strengths in the upper 2.5m, which is attributed to desiccation/ geo-chemical changes and seasonal drying close to existing ground.

Significant scatter is apparent in the values obtained from undrained shear strength calculated from correlation with liquidity data. Values of c_u in the Pelaw Clay (b5) estimated from correlation with 46 liquidity indices indicates undrained shear strength in the range of 2 to truncated values exceeding 250kPa, with an average of 117kPa. However, an average undrained shear strength estimated from test results below 2.5m is calculated to be 76kPa, again showing a reduction of strength which occurs with depth within the Pelaw Clay.

However, in light of the variable nature of the dataset, a cautious characteristic c_u value of 70kPa is assumed for design. This value is based on the lower shear strengths measured in the Pelaw Clay below 2.5m, neglecting the higher c_u values measured close to ground surface.

Figures 1 to 4 show estimated undrained shear strengths calculated from CPT data using correlation $s_u = (q_c - \sigma_{vo})/N_k$ where σ_{vo} is total in situ vertical stress and N_k is an empirical cone factor. A value of N_k of 20 is adopted for the fine soils on the site which is considered conservative.

Data shows undrained shear strength in the Pelaw Clay to vary between 50 and 150kPa from ground surface to 2m, reducing to 50kPa or lower between 2 and 4m with lower values inferred at the interface with the underlying Laminated Clay (b2). Consideration may be given in detailed design to adopt a more favourable value of N_k of 15 at specific structure locations following a data review specific to each location.

However, for the purposes of preliminary design based on data obtained across the whole depth range, a characteristic design c_u value of 50 kPa is recommended for the Pelaw Clay.

6.4.8 Drained Shear Strength

Values of constant volume effective angle of shearing resistance (φ'_{cv}) calculated from correlation with plasticity index range between 20 and 28° (average 24°). The results are plotted against depth and level on Figures 29 and 30. The data shows a trend of increasing effective friction angle with depth corresponding to the reduced plasticity with depth trend described in Section 6.4.4 above.

Effective stress laboratory testing comprising consolidated undrained triaxial compression tests were scheduled on selected samples; however laboratory non-conformance notices were received typically due to recovered sample splitting due to silt layers resulting in insufficient remaining sample for testing.

A characteristic φ'_{cv} value of 24° is adopted for design.

6.4.9 Consolidation

Compressibility and swelling indices (C_c and C_s) estimated from an empirical relationship with liquid limit are plotted against depth and level on Figures 31 and 32. C_s data is limited to values estimated as proportion of the calculated C_c as outlined earlier. Calculated C_c values ranged from 0.19 to 0.62 (average 0.32).

The data shows a reduction in C_c values with depth bgl (due to the reduction in liquid limits with depths). The data suggests a C_c of 0.35 is appropriate for preliminary design above 2.5m bgl reducing to 0.25 at greater depth. C_s data inferred from C_c lies between 0.027 and 0.095, with a characteristic value of 0.050 appropriate for design.

6.5 Tyne & Wear Complex – Silt (b4)

Bands and layers of silt (b4) were encountered either below the Pelaw Clay (b5) described above, or within the laminated clays (b2) further described below.

These soils are typically described as dark grey clayey sandy silt or soft and firm greyish brown slightly sandy slightly clayey silt. The soils were proved in BH's 16, 38, 45 and 49 at depths ranging from 3.6 to 9.5 (26.13 to 33.21mOD) and extended down to depths of 3.9 to 10.7m (24.93 to 32.71mOD). Silt was also proved in trial pits at depths at depths ranging from 3.5 to 4.5 (30.67 to 33.51 mOD) and proven to 4.3 to 5.0m (30.17 to 32.51m OD). The silt was generally confined to thin layers between 0.3 and 1.3m thick (average 0.8m).

The results of the limited in-situ and laboratory tests undertaken on the Silt (b4) as well as calculated geotechnical parameters adopting the methodologies described under Section 6 above are summarised in Table 10. Given the localised presence of discrete silt layers and very limited data set, separate geotechnical plots are not presented for these soils.

Table 10. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Silt (b4)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Natural Water Content, w (%)	1	17	-	-
Liquid Limit, w_L (%)	1	41	-	-
Plastic Limit, w_P (%)	1	21	-	-
Plasticity Index, I_p (%)	1	20	-	-
Percentage passing 425µm sieve (%) #2	1	93	-	-
BS 8002:2015 φ' cv,k degrees **(calculated)**	1	28	-	-
Consistency index (I_C) **(calculated)**	1	1.2	-	-
Particle size distribution: % passing 63µm sieve (%)	2	79	81	80
Hand shear vane, c_u HV (kPa)	1	31	-	-
c_u (kPa) W_L **(calculated from liquidity)**	1	>250*	-	-
C_c WL = 009(LL-10) **(calculated)**	1	0.28	-	-

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
C_sWL **(calculated)**	1	0.04	-	-

* Not Applicable due to coarse grained fraction

6.5.1 Particle Size Distribution

Particle size distribution curves for the Silt (b4) are shown on Figure 33. Testing was undertaken on two samples from TP's 10 and 19, both recovered at a depth of 3.5m. The plot shows these soils to be well graded to gap graded with both exhibiting more than 80% passing the 2mm sieve (ranging between 97 and 98%). The soils tested exhibit 79 and 81% passing the 63µm sieve, indicative of a high fines content and classify as Class 2D (silty cohesive fill) by virtue of grading in accordance with the HASHW Series 600 Earthworks Specification [Ref. 27]. However, these soils are not widely encountered across the site and will not be intercepted by the proposed at grade or shallow cut earthworks but may be locally present in deep drainage or balance pond excavations.

Values of uniformity coefficient (C_U) and coefficient of curvature (C_C) of 20 and 0.41 were reported for only one of the two samples (from TP10 at 3.5m). The results indicate the sample tested to be gap graded.

As further geotechnical testing is limited to a single test it is not proposed to present the results graphically, the following characteristic values are assumed for geotechnical design.

- A characteristic bulk density of value of 1.9Mg/m^3 is adopted, which lies within the weight density limits for medium strength fine soils suggested in Figure 1 and 2 of BS8004: 2015 [Ref. 15]).
- A characteristic undrained shear strength (c_u) of 30kPa is assumed based on a single hand vane test result where the silt contained sufficient fines to behave as a cohesive soil.
- A characteristic value of constant volume effective angle of shearing resistance (φ'_{cv}) of 28° is adopted based on a single measured plasticity index of 20% calculated from the empirical relationship detailed in BS8004: 2015 [Ref. 15] and Section 6 above.

6.6 Tyne & Wear Complex – Glacial Sand (b3)

Localised bands or pockets of sand (b3) were encountered either associated with the silt layers (b4) described above or as discrete lenses within the laminated clays (b2) further described below. These soils were not widely encountered across the site.

These soils are described as medium dense brown or grey clayey fine to medium sand and were proved in BH's 16 at depths of 7.0 to 7.5m (30.83 to 30.33mOD) and between 8 to 9m (29.83 to 28.83mOD, respectively) and in BH17 between 16.0 and 17.0m (20.46 to 19.46mOD). Sand was also proved at shallower depth from 3.2 to 3.5m (32.45 to 32.15mOD) in TP19, where it was present directly above a silt layer (b4) which was proved from 3.5m depth to the base of the hole at 4.3m bgl. It is possible that the 'sand' described in BH17 at depth may actually represent completely weathered bedrock.

The results of the limited in-situ and laboratory tests undertaken on sand (b3) are summarised in Table 11.

Table 11. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Sand (b3)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Uncorrected SPT 'N' value (blows/300mm)	2	13	39	26
Particle size distribution: % passing 63µm sieve (%)	1	64	-	-

A particle size distribution curve for a single sample of soil logged as medium dense sand (b4) is shown on Figure 34. The testing was undertaken on soil recovered from BH16 at a depth of 8.0m for classification purposes. The plot shows the soil to have a similar grading to the glacial silt (b4) described above. The laboratory described the soil tested as brown slightly clayey silt, which does not correspond the strata description given on the exploratory hole log. It is also noted that an interbedded sequence of sand (7.0 to 7.5m), silt (7.5 to

8.0m) and sand (8.0 to 9.0m) were proved in BH16 and therefore the soils from which this sample was taken may have been mislogged.

The description of the soils and gradings are considered typical of variable coarse grading into fine glacio-fluvial soils and highlight the difficulty in distinguishing discrete strata types present in relatively thin localised bands within the glacial sequence. In addition, given the presence of groundwater within the boreholes, the samples may be compromised through a washout of fines (silt) resulting in the recovered sample appearing to be coarser (sandier) than the in-situ condition.

For the purposes of preliminary design, based on the particle size distribution testing, characteristic geotechnical design values for the glacial sands (b3) are assumed to be the same as those provided for the glacial silts (b4).

6.7 Tyne & Wear Complex – Laminated Clay (b2)

Natural soils comprising Laminated Clay (b2) are described as either soft, firm or stiff thinly laminated brown or grey silty clay of high plasticity with light brown and grey silt dustings/ partings noted on laminae surfaces, or as soft, firm or stiff laminated brown or greyish brown, slightly sandy, slightly gravelly clay of intermediate plasticity. Gravel is subangular to subrounded, fine to medium of coal and sandstone and was proved in one trial pit and in the majority of the boreholes within the Phase ONE development area (BH's 13 to 17, 24, 25, 28, 31, 35, 38 and 45 to 48). These soils were not proved in the south west corner of the site outside of the infilled glacial valley described in Section 5, where the Pelaw Clay (b5) is directly underlain by the Lower Glacial Till (b1).

The soils were proved at depths from 1.2 to 15.0m (31.65 to 21.46 mOD) down to depths ranging between 5.35 to 17.9m (35.47 to 19.19 mOD). The Laminated Clay (b2) soils were proved to a maximum thickness of 13.8m (BH31), which is close to the centre and drilled through the deepest part of the infilled glacial valley, as shown on Drawing 60283414-M015-ACM-L1-DR-GE-003.

The results of in-situ and laboratory tests undertaken on the Laminated Clay (b2) as well as calculated geotechnical parameters adopting the methodologies described under Section 6 above are summarised in Table 12.

Table 12. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Laminated Clay (b2)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Uncorrected SPT 'N' value (blows/300mm)	59	5	100	14
Natural Water Content, w (%)	49	10	39	24
Liquid Limit, w_L (%)	37	27	64	44
Plastic Limit, w_P (%)	37	11	27	20.4
Plasticity Index, I_p (%)	37	9	38	23.6
Percentage passing 425µm sieve (%) #2	37	81	100	97
Weight Density (Mg/m^3)	12	1.88	2.12	2.01
Dry Density (Mg/m^3)	12	1.36	1.85	1.63
BS 8002:2015 φ' cv,k degrees **(calculated)**	37	22	30	25
Consistency index (I_C) **(calculated)**	37	0.39	1.42	0.84
Particle size distribution: % passing 63µm sieve (%)	9	48	99	77
Hand shear vane, c_u HV (kPa)	2	34	53	44
c_u HV (kPa) Triaxial	12	22	75	49
c_u (kPa) W_L **(calculated from liquidity)**	37	9	250	83
C_u SPT N (kPa) 'f_1' = 4.0 **(calculated)**	59	20	250**	62

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
C_c WL = 009(LL-10) **(calculated)**	37	0.15	0.49	0.31
C_sWL **(calculated)**	37	0.022	0.069	0.044

* SPT ‘N’ values extrapolated to a maximum of 100 blows, average value presented excludes extrapolated values of 53 and 100.

** Cu extrapolated to a maximum value of 250kPa.

6.7.1 Particle Size Distribution

Particle size distribution curves for the Laminated Clay (b2) are shown on Figure 35. Testing was undertaken on nine samples at depths ranging between 2 and 15m. With the exception of one test, the plot falls into a tight envelope characteristic of uniformly graded soils with all curves exhibiting more than 80% passing the 2mm sieve (ranging between 98 and 100%). These soils exhibit between 65 and 99% passing the 63µm sieve, indicative of high fines content and intermediate to high plasticity clays. These soils classify as Class 2D (silty cohesive fill) by virtue of grading in accordance with the HASHW Series 600 Earthworks Specification [Ref. 27]. The two coarsest of the eight similar gradings are more typical of silt and may represent silt bands within the laminated clay succession.

One sample plots as well graded material more typical of glacial till. The sample was obtained from BH45 at 1.2m. It is noted that firm greyish brown slightly sandy slightly gravelly silty clay of intermediate plasticity was recorded from 1.2 to 9.5m in BH45. It is considered likely that either the soils nearer to ground surface have been mis-logged in this hole, or possible the sample depth has been mislabelled on site as the test result is actually representative of the underlying Lower Glacial Till (b1). It is also noted that the sample includes a large piece of gravel influencing the shape of the curve and suggests that insufficient sample was available for testing.

These soils general occur at depth below the Pelaw Clay (b5) and are unlikely to be generally intercepted by earthworks on the site apart from deep excavations required to form basements, drainage or balancing ponds. They will however be intercepted by piled foundations at depth.

Values of uniformity coefficient (C_U) and coefficient of curvature (C_C) were not reported due to the high proportion of fines (percentage of material passing 63µm sieve), which was present in the soils tested.

6.7.2 Unit Weight

An average bulk density of 2.01 Mg/m^3 is obtained from triaxial testing undertaken to determine undrained shear strength in the laboratory. A characteristic value of 2.0 Mg/m^3 is adopted, which lies close to the centre of the range of weight densities suggested for medium strength fine soils in accordance with Figures 1 and 2 of BS8004: 2015 [Ref 15].

6.7.3 Moisture Content and Atterberg Limits

Atterberg Limits tests were undertaken on thirty seven samples of Laminated Clay (b2) recovered at depths ranging between 2 and 17.5m (18.9 to 36.36mOD). Plasticity data is plotted on a Casagrande plasticity chart on Figure 36 which indicates clays are generally of intermediate and high (CI and CH)plasticity, with a seven (~20% of the dataset) plotting as clays of low (CI) plasticity.

Natural moisture content, liquid and plastic limits are plotted against depth and level on Figures 37 and 38, respectively. It is noted that the low plasticity soils were encountered in two discrete zones, between 6.0 and 7m (23 to 24mOD) and at 14m (20mOD) depth. It is considered likely that these soils were probably recovered from close to the interface between the Laminated Clay (b2) and underlying Lower Glacial Till (b1). The depth of the interface between these materials changes sharply across the site along the margins of the infilled glacial valley. In addition, interdigitation or interlensing has often resulted in the development of complex sequences of laminated clays and glacial till within buried valleys in the North East of England. The highest plasticity soils are noted to occur between 7 and 13m (~30 to 23mOD).

Natural moisture contents within the Laminated Clay (b2) between 2 and 17.5m depth fall in a wide range between 10 and 39% (average 24%).

6.7.4 Consistency

Values of consistency index (I_C) between 0.39 and 1.42 (average 0.84) were calculated from the tests described in Section 6.4.3. The average I_C value indicates the material to be of stiff consistency. However, I_C values between 0.39 and 0.75 (average 0.66) are obtained in BH's 16, 17, 24, 25, 28 and 48 at depths between 6 and 15m (21.46 to 29.57mOD), indicative that firm soils are present at depth within the Laminated Clay (b2).

6.7.5 Standard Penetration Tests

Fifty nine uncorrected SPT's undertaken within the Laminated Clay (b2) are plotted against depth and level on Figures 39 and 40, respectively. SPT 'N' values between 2 and 17.5m depth (18.88 to 36.6mOD) range between 5 and 41 blows (average 14), indicative of low to very high strength soils. The average 'N' value is indicative of medium strength soils. Extrapolated SPT 'N' values by linear extrapolation are recorded in BH46 at depths of 2.2m (100 blows) and 7.0m (53 blows). The value of 100 blows and is thought to represent a cobble or boulder obstruction present at shallow depth whilst the deeper SPT at 7m is also noted to have been undertaken close to rockhead and may have affected by the presence of weathered bedrock.

6.7.6 Undrained Shear Strength

Undrained shear strength has been measured from undrained laboratory triaxial testing and hand shear vanes and inferred by correlation with SPT 'N' values and liquidity indices. The data are plotted against depth and level on Figures 41 and 42, respectively. Undrained shear strength in the Laminated Clay indicates a wide range of results. There is an apparent trend of increasing strength with depth as shown.

Undrained shear strengths obtained from correlation with SPT 'N' values range between 20 and 164kPa (excluding extrapolated SPT values described in Section 6.7.5 above). An average c_u value of 56kPa is inferred.

Undrained shear strength obtained from two in-situ hand vane testing carried out in the Laminated Clay (b2) obtained at shallow depths between 3.6 and 4.6m (32.91 and 33.91 m OD) indicate c_u of 34 and 53kPa, with an average value of 43.5kPa. These strengths correlate well with the values inferred from SPT 'N' results.

A total of 12 undrained laboratory triaxial tests were undertaken on undisturbed samples of Laminated Clay (b2) recovered at depths between 5 and 15m (21.46 and 31.57mOD). Undrained shear strengths measured by triaxial testing range from 22kPa (BH35 at 7.0m, 27.46 mOD) to 75kPa (BH45 at 17.0m, 28.623 mOD) with an average value of 49kPa obtained. However, the lowest strengths were obtained at depths between 7 and 11m (28.12 to 26.83mOD) with c_u values lying in the range of 22 to 36kPa. The lower strengths measured over these depths are shown by a dashed blue line on Figure 41.

Significant scatter is apparent in c_u values obtained from correlations with liquidity data. Undrained shear strength in the Laminated Clay (b2) estimated from correlation with 37 liquidity indices indicates undrained shear strength in the range of 9 to truncated values exceeding 250kPa, with an average value of undrained shear strength of 83kPa. However, the average strength estimated from test results below 8m bgl is calculated to be 70kPa. This suggests the trend of reducing strength is also reflected in c_u values estimated from correlation with liquidity data.

Figures 1 to 3 show estimated undrained shear strengths calculated from CPT data using correlation $s_u = (q_c - \sigma_{vo})/N_k$ where σ_{vo} is total in situ vertical stress and N_k is an empirical cone factor. A value of N_k of 20 is adopted for the fine soils on the site which is considered conservative.

Data shows undrained shear strength in the Laminated Clay (b2) to be generally around 50kPa (occasionally lower) over the full depth of the laminated clays proved.

The information obtained from all datasets is shown to variable, however, an average characteristic c_u value of 50kPa is assumed for preliminary design. Lower values of 30kPa may be applicable at between 7 and 11m depth at specific structure locations. The variation of undrained shear strength within the Laminated Clay (b2) should be considered as part of future scheme development and detailed design in conjunction with the assessment of data specific to the structure/ infrastructure position.

6.7.7 Drained Shear Strength

Values of constant volume effective angle of shearing resistance (φ'_{cv}) calculated from correlation with plasticity index range between 22 and 30° (average 25°). The results are plotted against depth and level of Figures 43 and 44. There is no distinct trend with depth.

Effective stress laboratory testing comprising consolidated undrained triaxial compression tests were scheduled on selected samples; however laboratory non-conformance notices were received typically due to recovered sample splitting due to silt layers, resulting in insufficient intact sample for testing.

A characteristic φ'_{cv} value of 25° is adopted for design.

6.7.8 Consolidation

Compressibility and swelling indices (C_c and C_s) estimated from an empirical relationship with liquid limit are plotted against depth and level on Figures 45 and 46. C_s data is limited to values estimated as proportion of the calculated C_c as outlined earlier. Calculated C_c values ranged from 0.15 to 0.5 (average 0.31).

The data shows increased values of C_c between 6.0 and 10m bgl (26 to 30.0m OD) within the highest plasticity soils present within the Laminated Clay (b2) over these depths. The data suggests a C_c of 0.35 is appropriate for preliminary design. C_s data inferred from C_c lies between 0.022 and 0.069, with a characteristic value of 0.05 appropriate for design.

6.8 Lower Glacial Till (b1)

Natural soils comprising the Lower Glacial Till (b1) described as stiff or very stiff brownish grey or dark brown slightly sandy slightly gravelly clay with subangular to subrounded, fine to coarse gravel of sandstone, mudstone, siltstone and coal was proved in BH's 13 to 16, 24, 25, 28, 31, 39, 40, 45 and 47 to 50. These soils were proved below the Laminated Clay (b2) directly overlying the Upper and Middle Pennine Coal Measures rocks.

Within the boreholes the soils were proved at depths from 3.1 to 19.78m (34.99 to 17.31mOD) extending to between 4.6 to 20.33m (34.37 to 16.76 mOD). The maximum thickness of till was proved in BH25 (5.23m), BH47 (6.8m), BH48 (6.2m) and BH50 (7.5m). The thickest till deposits were proved within the sides of infilled glacial valley (see Drawing 60283414-M015-ACM-L1-DR-GE-003) below the laminated clay (b2) but the material thinned noticeably towards the centre of the glacial valley (as shown on Geological Section B-B' Drawing 60283414-M015-ACM-L1-DR-GE-005). Lower Glacial Till (b1) was also proved at shallower depths directly below the Pelaw Clay (b5) in BH's 40, 49 and 50 along the margins of the buried valley.

Soils typical of the Lower Glacial Till were also recorded in one trial pit (TP16A) between 2.1m and the base of the hole at 4.6m (33.77 to 31.27mOD). The soils are described as very stiff, dark brown, slightly sandy, slightly gravelly clay with angular to subangular, fine to coarse gravel of mudstone and coal. It is considered likely that these soils have been mislogged and actually represent the Pelaw Clay (b5).

The results of in-situ and laboratory tests undertaken on the Lower Glacial Till (b1) as well as calculated geotechnical parameters adopting the methodologies described under Section 6 above are summarised in Table 13.

Table 13. Summary of In-situ, Laboratory Test Results and Calculated Parameters: Lower Till (b1)

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Uncorrected SPT 'N' value (blows/300mm)	46	11	100*	57
Natural Water Content, w (%)	21	7.6	28	18
Liquid Limit, w_L (%)	20	23	38	31
Plastic Limit, w_P (%)	20	13	24	16
Plasticity Index, I_p (%)	20	4	19	15
Percentage passing 425µm sieve (%) #2	20	65	100	86

Test Description	No. of Tests	Minimum Value	Maximum Value	Average Value
Weight Density (Mg/m^3)	1	2.01	2.01	2.01
Dry Density (Mg/m^3)	1	1.59	1.59	1.59
BS 8002:2015 φ' cv,k degrees **(calculated)**	19	26	30	27
Consistency index (I_C) **(calculated)**	20	0.14	1.65	0.87
Particle size distribution: % passing 63µm sieve (%)	3	59	92	76
c_u HV (kPa) Triaxial	1	38	38	38
c_u (kPa) W_L **(calculated from liquidity)**	20	3***	250**	115
C_u SPT N (kPa) 'f_1' = 4.0 **(calculated)**	46	44	250**	122***
C_c WL = 009(LL-10) **(calculated)**	20	0.12	0.25	0.18
C_sWL **(calculated)**	20	0.017	0.036	0.017

* SPT N values extrapolated to a maximum 100 blows.

** Cu extrapolated to a maximum value of 250kPa.

*** Not considered representative of in situ shear strength, average does not include extrapolated shear strengths for more than 50 SPT blows.

6.8.1 Particle Size Distribution

Particle size distribution curves for the Lower Glacial Till (b1) are shown on Figure 47. Testing was undertaken on three samples taken at depths ranging between 3.2m (BH50) and 13.2m (BH45). The gradings show uniformly graded soils which all exhibit more than 80% passing the 2mm sieve (ranging between 82 and 100%). These soils exhibit between 59 and 92% passing the 63µm sieve, indicative of high fines content and intermediate to high plasticity clays. These soils classify as Class 2A/B (general cohesive fill) or 2D (silty cohesive fill) by virtue of grading in accordance with the HASHW Series 600 Earthworks Specification [Ref. 27].

These soils general occur at depth and are unlikely to be excavated during earthworks undertaken on the site but they will however be intercepted by piled foundations at depth.

Values of uniformity coefficient (C_U) and coefficient of curvature (C_C) were not reported due to the high proportion of fines, (percentage of material passing 63µm sieve) which was present in the soils tested.

6.8.2 Unit Weight

An bulk density of 2.01 Mg/m^3 is obtained from a single triaxial testing undertaken to determine undrained shear strength in the laboratory. A characteristic value of 2.0 Mg/m^3 is adopted, which lies within the centre of a range of weight densities suggested for high strength fine soils in accordance with Figures 1 and 2 of BS8004: 2015 [Ref 15].

6.8.3 Moisture Content and Atterberg Limits

Atterberg Limits tests were undertaken on 20 samples of Lower Glacial Till (b1) recovered at depths ranging between 3.2 and 16.45m (34.89 to 20.12mOD). Plasticity data is plotted on a Casagrande plasticity chart on Figure 48 which indicates clays are of low (CL) plasticity although two samples are a little more plastic, classifying as intermediate (CI) plasticity clay soils.

Natural moisture content, liquid and plastic limits are plotted against depth and level on Figures 49 and 50. There is no trend in the data with depth or level and similar results are obtained over the full depth range over which the Lower Glacial Till (b1) was tested.

Natural moisture contents within the Lower Glacial Till (b1) between 3.2 and 16.45m depth fall in a wide range between 8 and 28% (average 18%).

6.8.4 Consistency

Values of consistency index (I_C) between 0.14 and 1.65 (average 0.87) were calculated from the tests described in Section 6.4.3. The average I_C value indicates the material to be of stiff consistency. Lower I_C values of 0.14, 0.44, 0.31 and 0.5 indicative of very soft or soft soils were calculated for samples obtained in BH's 35, 40, 47 and 50 at depths of 13.0, 5.7, 9.0 and 4.2m, respectively. One of these samples were noted to be close to the interface with the overlying laminated clays (BH35) and one sample is coincidental with a silt band at 4.2m (BH50). The low consistency index values obtained from the other two samples may be attributable to sample disturbance and very soft and soft consistency is not considered representative of overall in-situ conditions.

6.8.5 Standard Penetration Tests

Fifty nine uncorrected SPT's undertaken within the Lower Glacial Till (b1) are plotted against depth and level on Figures 51 and 52, respectively. SPT 'N' values between 3.2 and 18.7m (16.48 to 35.77 mOD) range between 11 and 50 blows (average 30), indicative of firm to very stiff soils. The data shows an increase with depth/ reducing level. SPT 'N' values calculated by linear extrapolation are recorded in 22 of the 59 SPT's undertaken. These truncated tests are indicative of boulder and cobble obstructions which were commonly encountered during drilling and are not considered representative of the in-situ density or mass strength of the soil as a whole.

6.8.6 Undrained Shear Strength

Undrained shear strength has been measured by undrained laboratory triaxial testing and this data is augmented by c_u values inferred from correlation with SPT 'N' values and liquidity indices. The data are plotted against depth and level on Figures 53 and 54, respectively. Undrained shear strength in the Lower Glacial Till (b1) indicates a wide range of results. There is scatter in the data and there is no distinct trend with depth or reduced level.

Undrained shear strengths inferred from correlation with SPT 'N' values range between 44 and 200kPa (excluding extrapolated values derived from SPT's which produced 50 blows, as these are considered to reflect the presence of cobbles and boulders with the Lower Glacial Till (b5) described in Section 6.8.5 above). An average c_u value of 122kPa is inferred.

Undrained shear strength was not possible using in-situ hand vane testing due to the depth at which the soils were encountered.

Only one undrained triaxial test was undertaken on an undisturbed sample of Lower Glacial Till (b1) recovered at 4.2m (33.89 mOD) from BH50. A c_u value of 38 kPa is reported by the laboratory. The soil was described as stiff greyish brown silty slightly sandy clay of low plasticity on the exploratory hole log with a SPT 'N' value of 17 suggesting it to be medium strength. This sample coincides with a band of silt obtained at this depth (as described in Section 6.8.4 above) and it is inferred that the test was influence by the presence of silty material and not representative of the in-situ soil strength.

Significant scatter is apparent in c_u values obtained from correlation with liquidity data. Undrained shear strength in the Lower Glacial Till (b1) estimated from correlation with 20 liquidity indices indicates undrained shear strength in the range of 3 to truncated values exceeding 250kPa, with an average value of 115kPa. The average c_u value calculated from liquidity data correlates well with undrained shear strengths inferred from in-situ SPT 'N' tests.

Figures 1 to 4 show estimated undrained shear strengths calculated from CPT data using correlation $s_u = (q_c - \sigma_{vo})/N_k$ where σ_{vo} is total in situ vertical stress and N_k is an empirical cone factor. A value of N_k of 20 is adopted for the fine soils on the site which is considered conservative. Data shows undrained shear strength in the Lower Glacial Till (b1) to rise rapidly from around 50kPa (medium strength) and to greater than 150 kPa (very high strength) towards the base depths of the soils tested.

Based on all the datasets assessed, an average characteristic undrained shear strength of 120kPa is assumed for preliminary design. For detailed design, a more favourable trend of increasing shear strength could be adopted, such as $c_u = 25 + 10(z)$, where z is metres below ground level.

6.8.7 Drained Shear Strength

Values of constant volume effective angle of shearing resistance (φ'_{cv}) calculated from correlation with plasticity index on 19 samples range between 26 and 30° (average 27°). The results are plotted against depth and level of Figures 55 and 56. There is not distinct trend with depth.

Effective stress laboratory testing comprising consolidated undrained triaxial compression tests was scheduled on selected samples; however laboratory non-conformance notices were received typically due to the recovered samples being fissured due to its high strength resulting in insufficient intact sample remaining for testing.

A characteristic φ'_{cv} value of 27° is adopted for design.

6.8.8 Consolidation

Compressibility and swelling indices (C_c and C_s) estimated from an empirical relationship with liquid limit are plotted against depth and level on Figures 57 and 58. C_s data is limited to values estimated as proportion of the calculated C_c as outlined earlier. Calculated C_c values ranged from 0.12 to 0.25; the average of 0.18 is assumed to represent a characteristic design value.

C_s data inferred from C_c lies between 0.017 and 0.036, with the average of 0.026 appropriate as a characteristic value for design.

6.9 Bedrock (a1 to a3)

All the cable percussive boreholes were extended by rotary coring into the underlying Pennine Upper or Middle Coal Measures, an interbedded sequence of sandstone (a1), mudstone (a2), siltstone (a3) and coal (a4). The surface of the rock was proved to be weathered (denoted (w)) with the average thickness of surface weathering shown to be 0.4m within the sandstone (a1(w)), 0.6m within mudstone (as(w)), and 0.9m within siltstone (a3(w)).

The depth at which bedrock was proved varies widely across the site at depths ranging from 2.1 to 29.6m (37.6 to 8.49mOD) proved to depths between 2.4 and 31.2 (37.3 to 6.89mOD). The surface of the bedrock follows the contours of the infilled glacial valley as shown on Drawing 60283414_M015_GEO_DR_003 and also illustrated on the geological cross sections A-A' to F-F' as shown on Drawings 005 to 007.

An assessment of the depth and level to bedrock will be required for each structure as part of further phases of detailed design. The depth to bedrock will be dependent on the location of the structure relative to the position of the infilled glacial valley. A variable depth of drift cover above the bedrock is to be anticipated across the plan area of all proposed structures and variable pile lengths are likely to be required to support the building loads.

Rock core quality indices are plotted against depth and level on Figures 59 and 60. The data comprises of total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD), all of which were recorded during logging of rock core recovered from the investigation. RQD is a measure of rock quality, which may be described as Excellent for RQD between 90 and 100%, Good for RQD between 75 and 90%, Fair for RQD between 50 and 75% and Poor for RQD between 25 and 50% and Very Poor between 0 and 25%.

Rock quality indices for each rock type are summarised in Table 14 below. The weathered surface of the bedrock (w) is generally designated poor or very poor, as is the mudstone (a2) and areas identified as potentially fractured rock associated with past possible coal workings (a4work). The sandstone (a1) and siltstone (a3) are designated as fair.

Table 14. Rock Quality Indices

Rock Description	Strata Code	TCR % Min – Max (Average)	SCR % Min – Max (Average)	RQD % Min – Max (Average)	Quality [#1]
Weathered Sandstone - Very dense brown, yellow or reddish grey sandy GRAVEL. Gravel is angular to subangular, fine to coarse of sandstone.	a1(w)	100-100 (100)	9-100 (59)	0-91 (48)	Poor
Sandstone – Weak, partially weathered, orange brown fine, predominantly medium to coarse, micaceous SANDSTONE. Fractures are sub-horizontal, planar, smooth with dark red	a1	33-100 (98)	18-100 (89)	0-100 (70)	Fair

Rock Description	Strata Code	TCR % Min – Max (Average)	SCR % Min – Max (Average)	RQD % Min – Max (Average)	Quality [#1]
staining. or: Medium strong, partially weathered, light grey fine grained SANDSTONE. Fractures are medium spaced sub-horizontal, planar, smooth, undulose, clean.					
Weathered Mudstone - Firm to stiff light grey slightly sandy gravelly CLAY. Gravel is angular to subangular, fine to coarse of mudstone lithorelicts. or: Very dense light grey or reddish brown, slightly clayey, slightly sandy, GRAVEL. Gravel is angular to subangular, fine to coarse of mudstone.	a2(w)	86-100 (99)	37-10 (72)	0-84 (14)	Very Poor
Mudstone – Extremely weak or very weak distinctly weathered light grey MUDSTONE. Fractures are very closely to closely spaced, sub-horizontal, planar, smooth, clean with dark grey discolouration on fracture surface.	a2	46-100 (96)	9-100 (78)	0-100 (45)	Poor
Weathered Siltstone - Very dense light grey sandy GRAVEL. Gravel is angular to subangular of siltstone. or: Stiff, grey mottled brown slightly sandy, slightly gravelly CLAY. Gravel is angular to subangular of mudstone and siltstone.	a3(w)	100-100 (100)	0-92 (44)	0-92 (23)	Very Poor
Siltstone – Very weak, weak or medium strong dark grey SILTSTONE with bands of mudstone. Fractures are very closely to closely spaced, sub-horizontal planar, smooth, clean.	a3	90-100 (99)	0-100 (87)	0-100 (65)	Fair
Coal - Very weak black COAL. Frequently randomly orientated interlocking fractures.	a4	100-100 (100)	40-100 (78)	29-71 (55)	Fair
Potential Coal Workings.	a4(work)	89-100 (97)	0-92 (41)	0-92 (38)	Poor

Notes:
#1: based on average RQD value.

6.9.1 Standard Penetration Tests

All standard penetration tests undertaken on bedrock were terminated before completion with 'N' values > 50 calculated by linear extrapolation. The results are summarised in Table 15 below.

Table 15. Summary of SPT N' value (extrapolated blows/300mm) in Bedrock

Rock Type	Strata Code	No. of Tests	Minimum Value	Maximum Value	Average Value
Weathered Sandstone	a1(w)	6	100	300	227
Sandstone	a1	2	333	500	417
Weathered Mudstone	a2(w)	4	86	214	139
Weathered Siltstone	a3(w)	7	25	75	288
Siltstone	a3	3	79	1500	581
Coal	a4	1	74	-	-

There is no clear trend with depth delineating the thickness of surface weathering at rockhead and therefore the results are not plotted against depth or level. Within the weathered mudstone (a2(w)), weathered siltstone (a3(w)) and coal (a4) approximately 50% of the results fall below 100 blows indicative of completely weathered rock.

The remaining tests in unweathered sandstone (a1) and siltstone (a3) were terminated before completion with 'N' values > 50 calculated by linear extrapolation with the values generally well above 100 blows indicative of more competent bedrock.

6.9.2 Unconfined Compressive Strength and Point Load Testing

Uniaxial compressive strength (UCS) tests were carried out on eleven selected rock core samples recovered from BH's 15, 17, 31, 35, 38, 39, 45, 47, 50 and 51. Point load index tests were carried out on smaller core specimens recovered from all rotary cores holes undertaken within the Phase ONE development area. Both axial and diametral tests were undertaken. The testing strategy adopted was to undertake point load tests at 1.0m intervals over the first 5.0m of recovered core (5 tests per borehole) supplemented with one UCS test over the first 5m of recovered core in each hole where suitable sample recovery allowed.

The point load strength test is frequently used to determine crushing strength through established empirical relationships like those proposed by Broch, E and Franklin, JA (1972) [Ref. 20]. Johnston (1991) [Ref. 21] suggests typical multiplication factors to convert ($I_{s(50)}$) to UCS; typically values between 15 and 25 are applied to diametral tests. Gannon, JA, Masterton, GGT, Wallace, WA and Muir Wood, D (1999) [Ref. 22] states that the development of site specific or formation specific correlations between UCS and $I_{s(50)}$ are essential. UCS values have been calculated from both axial and diametral point load indices using a correlation factor of 23; this value was chosen as it provides a good fit for the higher strength point load test results values measured from UCS testing in the laboratory. It is noted that lower strengths obtained from point load testing may represent lower bound design values, as only unweathered intact rock core samples was suitable for UCS testing which is inherently considered more representative of upper bound design strengths.

Rock strength is plotted against depth and level on Figures 61 and 62. The results of the laboratory tests undertaken is summarised in Table 16 below.

Table 16. Unconfined Compressive Strength

Rock Type	Strata Code	No. of Tests	Point Load (MPa) Min. – Max. (Average)	No. of Tests	UCS (MPa) Min. – Max. (Average)
Sandstone	a1	80	0-149.5 (21.0)	8	24.9-53.0 (33.0)
Weathered Mudstone	a2(w)	2	2.3 – 2.3 (2.3)	-	-
Mudstone	a2	47	0-87.4 (8.0)	3	6.0-32.2 (16.0)
Weathered Siltstone	a3(w)	4	0-2.3 (0.6)	-	-
Siltstone	a3	23	0-110 (16.5)	-	-

The exploratory hole logs, RQD and rock compressive strength data indicated that the bedrock is noted to include weathered zones which exhibit a lower compressive strength at surface across the site. However, some of the weathering may have been formed by the actions of cable percussive boring tools when bedrock was encountered close to borehole termination depths.

For the purposes of preliminary design (for example, the calculation of pile rock socket capacity) the weathered mudstone (a2(w)) and weathered siltstone (a3(w)) are considered to behave as residual soils and exhibit geotechnical properties similar to intact soils. An undrained shear strength of 250kPa is assumed together with a value of φ'_{cv} of 26°.

For design, unconfined compressive strength may also be inferred from field descriptions of rock material strength given on the exploratory hole logs as detailed in Table 25 of BS5930: 2015 [Ref. 23].

The surface of the sandstone (a1) bedrock was typically described as weak (5 - 25 MPa), the mudstone (a2) as extremely weak (0.6 - 1.0 MPa) and the siltstone (a3) as very weak (1 - 5 MPa) or weak. With the exception of siltstone (a3), unconfined compressive strengths inferred from field descriptions are generally consistent with the average values obtained from point load laboratory testing.

It is recommended the lower bound values provided in Table 25 of BS5930 [Ref. 23] are adopted for preliminary design, with additional assessment to be considered for individual structure or foundation positions as part of further phases of development and detailed design.

6.10 Coal and Potential Mine Workings (a4 and a4(work))

The locations where coal (a4) or potential coal workings were proved is shown on the Geotechnical Constraints drawing 60283414-M015-ACM-L1-DR-GE_004 and are summarised in Table 17 below. For the purposes of this assessment, all holes drilled on the IMAP site (i.e. including areas outside of Phase ONE within the overall IAMP DCO boundary) are shown on the constraints drawing and summarised in Table 17 below.

Table 17. Coal and Potential Coal Workings

Exploratory Hole	Depth from (m)	Depth to(m)	Level from (mOD)	Level to (mOD)	Thickness (m)	Strata Code	Description
BH22	10.8	10.87	29.89	29.82	0.07	a4	Black COAL.
BH22	11.13	11.21	29.56	29.48	0.08	a4	Black COAL.
BH23	10.26	10.34	29.53	29.45	0.08	a4	Black COAL
BH27	22.3	22.5	13.35	13.15	0.2	a4	Very weak to weak black COAL.
BH28	16.45	17.2	19.3	18.55	0.75	a4(work)	Firm grey, locally mottled brown, slightly sandy slightly gravelly CLAY. Gravel is angular to subangular, fine to coarse of mudstone lithorelicts.
BH28	18.6	18.98	17.15	16.77	0.38	a4(work)	Firm grey slightly sandy slightly gravelly CLAY. Gravel is angular to subangular, fine to coarse of mudstone lithorelicts.
BH28	18.98	19.05	16.77	16.7	0.07	a4	Very weak black COAL. Frequent randomly orientated interlocking fractures.
BH28	19.35	19.45	16.4	16.3	0.1	a4	Very weak black COAL. Frequently randomly orientated interlocking fractures.
BH28	24.5	24.85	11.25	10.9	0.35	a4(work)	Soft to firm grey slightly gravelly sandy CLAY. Gravel is angular to subangular, fine to coarse of mudstone.
BH28	25.07	25.2	10.68	10.55	0.13	a4	Very weak black COAL. Frequent randomly orientated interlocking fractures.
BH30	16.9	19.47	18.72	16.15	2.57	a4(work)	Soft to firm light grey slightly sandy gravelly CLAY with frequent subrounded cobbles of sandstone. Gravel is angular to subangular, fine to coarse of sandstone.
BH30	19.47	19.59	16.15	16.03	0.12	a4	Very weak to weak black COAL. Frequent randomly orientated interlocking fractures.
BH30	19.59	19.8	16.03	15.82	0.21	a4(work)	Soft to firm light grey slightly sandy slightly gravelly CLAY. Gravel is angular to subangular, fine to coarse of sandstone and coal.

Exploratory Hole	Depth from (m)	Depth to(m)	Level from (mOD)	Level to (mOD)	Thickness (m)	Strata Code	Description
BH30	19.8	19.93	15.82	15.69	0.13	a4	Very weak black COAL. Frequent randomly orientated interlocking fractures.
BH32	19.82	19.97	16.33	16.18	0.15	a4	Very weak to weak black COAL.
BH32	21.68	21.94	14.47	14.21	0.26	a4	Very weak to weak black COAL.
BH36	17.62	17.74	18.34	18.22	0.12	a4	Very weak to weak black COAL.
BH38	19.3	19.6	16.44	16.14	0.3	a4	Stiff black slightly sandy, slightly gravelly CLAY. Gravel is angular to subangular, fine to coarse of mudstone. Assumed to be thin coal.
BH49	6.75	6.88	32.46	32.33	0.13	a4	Black COAL.

It is noted that there are no mapped coal seam subcrops in the vicinity of the coal (a4) or potential unrecorded worked coal seams (a4(work)) proved, although the site is shown to be intersected by a number of mapped geological faults as shown on Drawing 60283414-M015-ACM-L1-DR-GE_004.

The Hylton Castle seam is shown to subcrop north of the Phase ONE boundary, between Downhill Lane and West Pastures but is not expected to underlie the site if the position of the coal mapped by the BGS is broadly accurate. The Top and Bottom Hebburn Fell Seams are shown to subcrop approximately 1.1 km south west of BH49 and 1km south east of the Phase ONE site boundary. These seams are expected to underlie the whole of IAMP ONE.

It is therefore most considered likely that seams proved represent unnamed coals present within the top of the Pennine Middle Coal Measures Formation and the base of the overlying Pennine Upper Coal Measures Formation. This illustrated by the strata on the geological section shown on Drawing 009 between the named Hylton Castle and Top Hebburn Fell seams.

Of note is that in BH’s 28 and 30 the coal is shown to be interbedded with layers of soft to firm gravelly clay. These layers may represent the presence of thin mudstone bands within individual coal seams or the effect of poor rotary drilling recovery. However, the possibility of unrecorded coal workings subsequently infilled with fine cohesive materials following roof collapse cannot be ruled out.

Over these areas, further ground investigation to assess this risk to proposed structure foundations and other infrastructure is recommended as part of further phases of development and design.

6.11 Groundwater

During site works, groundwater strikes were only encountered in approximately 30% of the boreholes and trial pits; a summary of groundwater strikes and rises is included as Table 18 below.

Table 18. Groundwater Strikes

Exploratory Hole	Strike Depth (mbgl)	Rise after 20 minutes	Strata Code	Strata
BH13	8.10	3.6	b1	Lower Glacial Till
BH14	10.50	9.96	b2	Laminated Clay
BH16	13.10	12.1	b2	Laminated Clay
BH16	6.00	4.9	b2	Laminated Clay
BH17	16.30	14.2	b3	Glacial Sand

Exploratory Hole	Strike Depth (mbgl)	Rise after 20 minutes	Strata Code	Strata
BH35	18.80	-	a3	Siltstone
BH45	13.20	3.4	b1	Lower Glacial Till
BH47	15.30	11.8	a1	Sandstone
BH48	18.00	5.3	a1(w)	Weathered Sandstone
TP19	3.80	-	b4	Silt
TP31	4.70	-	b4	Silt
TP35	4.50	-	a3(w)	Weathered Siltstone
TP36	3.40	-	b5	Pelaw Clay
TP36	3.40	2.1	b5	Pelaw Clay
TP37	2.30	-	a1(w)	Weathered Sandstone
TP38	3.90	3	a3(w)	Weathered Siltstone

Shallow groundwater strikes were only encountered in the Pelaw Clay (b5) in TP 36 at 3.4m rising to 2.1m. It is anticipated that most shallow excavations will be formed in the Pelaw Clay, although long term equilibrium levels also need to be considered as detailed below. Where bedrock was proved to be shallow (in the south west portion of the site), groundwater strikes were encountered at its surface between 2.3 and 4.5m depth, although significant rises were not recorded over a 20 minute period.

Long term groundwater monitoring readings are plotted against depth and level on Figures 5 and 6. The plot is colour coded to the strata type shown on the data plot figures. It is noted that groundwater level rise to equilibrium conditions occurred rapidly on site, typically within a few days of placement of the instrument. Therefore, although groundwater readings were routinely taken as site works progressed, only the results of the four monitoring visits taken 17th December 2017, 12th January, 26th January and 9th February 2018 are presented. It is noted that in the majority of installations irrespective of the strata in which they were installed, long term groundwater depths are within 1.5m of ground surface. Slightly deeper water levels (1.69 and 2.19m from ground surface) are suggested from two standpipe piezometers installed in BHs 13 and 47 within the bedrock. However, it is possible that water levels in these installations not yet reached equilibrium at the time of the last reading in February 2018.

The results are summarised in Table 19 below.

Table 19. Summary of Groundwater Monitoring Depth and Level

Strata Code At installation Depth	Strata	Exploratory Hole	Depth (m) Min. Max (Average)	Level (mOD) Min – Max
b5	Pelaw Clay	BH's 28, 35, 38, 49, 51 and 52	0.23-1.30 (0.60)	34.17 – 39.98
b3	Glacial Sand	BH's 16 and 17	0.99 – 1.38 (0.99)	34.81 – 36.84
b2	Laminated Clay	BH's 14, 15, 25, 31, 45, 46 and 48	0.25 – 2.25 (0.90)	33.82 – 37.81
b1	Lower Glacial Till	BH's 39 and 40	0.37 -0.85 (0.62)	36.99 – 38.31
a3	Siltstone	BH's 13 and 50	0.54 – 3.83 (2.19)	33.17 – 37.55
a2	Mudstone	BH24	0.72 – 0.86	34.62 – 34.76

Strata Code At installation Depth	Strata	Exploratory Hole	Depth (m) Min. Max (Average)	Level (mOD) Min – Max
			(0.78)	
a1	Sandstone	BH47	1.54-1.84 (1.67)	34.7 to 34.97

For preliminary design purposes groundwater levels should be assumed to be at or close to ground surface.

6.12 Soakaway Testing

Soakaway tests were carried out in accordance with BRE365 in TPS 01 to 04 at depths between 2.0 and 2.1m. All tests were undertaken in the Pelaw Clay (b5). Coefficient of permeability of 5.33 X10^{-7}, 0 (zero), 0 (zero) and 3.52 X10^{-7} are reported. However, in all tests it is noted that there was 'insufficient change in head and lack of infiltration to accurately calculate infiltration rate. Quoted rate should be regarded as indicative only'. The test results are included in Appendix D of the Dunelm Factual Report [Ref. 10].

6.13 Summary of Characteristic Design Values

A review of information provided in Sections 5 and 6 indicate the following characteristic geotechnical design values can be assumed for preliminary design.

Table 20. Characteristic Design Values and Geotechnical Parameters

Strata	Strata Code	Bulk Unit Weight *** $\gamma_{b,k}$ Mg/m^3	Undrained Shear Strength $c_{u,k}$ kN/m^2	Effective Cohesion c'_k kN/m^2	Constant Volume Effective Angle of Shearing Resistance $\varphi'_{cv,k}$ °	Compression Index C_C	Swelling Index C_s	UCS (MPa)
Made ground	d1-d5	2.00	50	0	23	0.4	0.057	N/A
Pelaw Clay	b5	2.10	50	0	24	0.35 / 0.25 #1	0.050	N/A
Silt	b4	1.90	30	0	28	0.28	0.040	N/A
Glacial Sand	b3	1.90	N/A	N/A	28	N/A	N/A	N/A
Laminated Clay	b2	2.00	50	0	25	0.35	0.050	N/A
Lower Glacial Till	b1	2.00	120	0	27	0.18	0.026	N/A
Weathered Sandstone	a1(w)	2.10	N/A	N/A	N/A	N/A	N/A	0.60
Sandstone	a1	2.10	N/A	N/A	N/A	N/A	N/A	5.0-25.0
Weathered Siltstone	a2(w)	2.10	N/A	N/A	26	N/A	N/A	0.25
Siltstone	a2	2.10	N/A	N/A	N/A	N/A	N/A	5.0-25
Weathered Mudstone (assume cohesive soils)	a3(w)	2.10	N/A	N/A	26	N/A	N/A	0.25
Mudstone	a3	2.10	N/A	N/A	N/A	N/A	N/A	1.0-5.0
Coal	a4	1.80	N/A	N/A	N/A	N/A	N/A	0.25

Notes:

#1: C_C of 0.35 applicable to depths above 0.35.

6.14 Preliminary Design Assessment

6.14.1 Introduction

The following sections of this report provide preliminary geotechnical and geo-environmental assessment for the purposes of feasibility design and are intended to provide support to a number of technical teams who are developing the overall site masterplan and assessing design risks and costs for the scheme. Design assumptions provided do not constitute a detailed design, which it is assumed will be developed as part of further phases of assessment and later in construction.

The site conditions, geotechnical parameters and material properties provided in this GIR have been used to inform the preliminary design assessment. The comments provided are based on interpretation of the documentary records obtained to date and the findings of the ground investigation completed across the whole of the IAMP site between 21st July and 29th November 2017, as included within the Draft Factual SI Report prepared by DGE [Ref. 10].

The ground conditions at the site are not considered to pose a contamination risk to the site's end users. However, due care will be required to ensure that the construction works do not have any negative impacts on the underlying Pennine Upper and Middle Coal Measures bedrock which are both classified as a Secondary A Aquifer.

In order to comply with the Construction Design Management (CDM) Regulations 2015 [Ref. 24], the Principal Designer will have to be informed of the results of this study and future phases of design. Appropriate precautions must be taken by construction workers to ensure that they are not affected by site contamination.

6.14.2 Aggressive Ground Conditions

In accordance with BRE Special Digest (SD1), 3rd Edition [Ref. 25], the potential for sulfate content on buried concrete in contact with the soil and groundwater at a site is classified on the basis of the sulfate content expressed as SO_4, mobility of groundwater, the acidity and form of concrete.

Water soluble sulfate and pH determinations were undertaken on 33 samples as summarised in Table 21 below.

Table 21. Summary of BRE Ground Aggressivity testing

Strata	Strata Code	Number of Tests	pH Min – Max (Average)	Water soluble sulfate as SO_4 (mg/l) Min – Max (Average)	Water soluble chloride (mg/l) Min – Max (Average)
Topsoil	e1	2	6.2-6.3 (6.25)	11.0-14.8 (12.9)	-
Made ground - Cohesive	d5	2	8.1-8.2 (8.15)	110-180 (149)	7.1-12 (9.55)
Made ground – Topsoil with brick	d2	1	7.9	40.0	12.63
Made ground – Topsoil with glass, ceramics and pottery	d1	1	6.1	16.2	-
Pelaw Clay	b5	21	6.5-8.5 (8.11)	8.9-393 (99.4)	5-23 (10.6)
Silt	b4	1	8.4	107	9.4
Laminated Clay	b2	5	8.2-8.6 (8.36)	45-206 (136)	6.3-13 (9.7)

All soil samples tested proved water soluble sulfates less than 500mg/l and pH values were all greater than 5.5. All strata present at the site are indicated to have an ACEC class of DS-1 AC-1.

Highways Agency DMRB BD12/01 – Design of Corrugated Steel Buried Structures [Ref. 26] indicates that corrosion is unlikely at chloride concentrations less than 50ppm (50mg/l) at pH values ranging from 6 to 9. Chloride aqueous extract determinations were undertaken as part of the ground aggressivity test suite. Chloride concentrations were low, ranging between 5.1 and 23mg/l.

6.14.3 Mining Summary

A Coal Authority Mining Report was obtained by Mott MacDonald (Ref. 4). It is noted that the Coal Authority search area undertaken at that time extends outside the IAMP DCO boundary and significantly beyond the Phase ONE development area.

The report indicates that the property is in the likely zone of influence from underground coal workings in 8 seams of coal at 190m to 570m depth, and last worked in 1981. Any ground movement from these coal workings should have stopped by now. The site is in an area where the Coal Authority believe that there is coal at or close to the surface and the coal may have been worked at some time in the past. The potential presence of coal workings at or close to the surface should be considered prior to any site works or future development activity.

Ground investigation has not identified worked seams below most of the site. However, the ground conditions proved in BH's 28 and 30 shows the coal to be interbedded with layers of soft to firm gravelly clay, see Geotechnical Constraints Drawing 60283414-M015-ACM-L1-DR-GE-004 and Geological Section B-B' on Drawing 005. Therefore, the possibility of unrecorded coal workings below this area of the site, and in particularly below the footprint of Units 4, 5 and 6, taken from the AJA Architects development layout in Appendix A, cannot be ruled out based on the preliminary information obtained.

Further ground investigation to assess this risk to proposed structures is recommended as part of further phases of development and detailed design.

6.14.4 Earthworks and Excavations

Detailed earthwork proposals, cut and fill requirements or indicative earthwork sections were not available at the time of writing this report. It is assumed that an Earthwork Specification and calculation of earthwork materials cut/ fill balance for the scheme will be addressed by others. However, as the site is relatively level, it is not anticipated that significant depths of cut and fill will be formed during the development.

Construction of the proposed highway as shown on Systra Highways General Arrangement Drawing IAMP_ONE-SYS-HGN-ZA1-DR-D-01-001-S)-PO4 dated 16/01/18 included in Appendix A indicates construction of low embankment earthworks will be required.

More extensive earthwork embankments will be required to the north within the overall IAMP development area where crossing of the River Don and A19(T) are proposed, however, these areas are however outside of the Phase ONE development area considered as part of this report.

Earthworks should be undertaken in accordance with the HASHW Series 600 Earthworks Specification (Ref. 27). The classification system adopted for site won earthwork materials is in accordance with HASHW Clause 601, Tables 6/1 and 6/2. The compaction criteria for various material classes are defined in Clause 612 and Table 6/4.

Highways Works (HASHW) (Ref. 27), Series 600, defines materials into Class 1 (granular) or Class 2 (cohesive) material. Class 1 material has less than 15% fines (i.e. material finer than or passing a 63µm sieve) and Class 2 has more than 15% material passing the 63µm sieve. Further sub divisions of Classes 1 and 2 are again defined primarily on grading. Acceptability criteria are set to ensure adequate material strength and the achievement of minimum degrees of relative compaction to limit any subsequent volume changes.

For Classes 2A, 2B, 2C and 2D fills, which exhibit cohesion, acceptability criteria are based on undrained shear strength (c_u). A range of c_u between 50 and 200kPa is typically assumed with the minimum limit defined from consideration of trafficability and long term consolidation aspects. The maximum limit is intended to be that at which satisfactory compaction can be achieved in terms of air voids content in the completed fill.

Shallow earthworks are most likely to be within the Pelaw Clay (b5) which was widely encountered below topsoil across most of the site. The lower plastic soils present within the Pelaw Clay sequence were noted to include a proportion of sand and fine and medium gravels, indicative of Class 2A/ B materials. However, these soils are unlikely to be separated from the overall material during bulk earthworks on site and these soils would, overall, classify as Class 2D (silty cohesive fill) by virtue of grading. However, limits of acceptability based on undrained shear strength (c_u) would need to be considered as part of future phases of design, as undrained shear strength testing has shown these soils to be at or close to the lower limit of strength acceptability. It is noted that the

Pelaw Clay is likely to lose strength rapidly in periods of prolonged wet weather or where these soils are disturbed by trafficking.

Given the high plasticity of the Pelaw Clay (b5), careful consideration will be required in assessment of these soils particularly for use in low cutting or embankment construction. A maximum slope angle of 1 vertical to 3.5 horizontal (1V:3.5H) is considered appropriate for the purposes of preliminary design and assessment of earthwork balance quantities. Steeper embankment slopes could be adopted assuming these were constructed from imported materials with more favourable geotechnical strength (c', φ') parameters in terms of effective stress. Imported materials will need to be classified in accordance with a specific Earthworks Specification developed for the scheme based on Appendix 6/1 of the HASHW Series 600 Earthworks Specification.

Temporary excavations within localised made ground (d1-d4) and Pelaw Clay (b5) will be required. Superficial deposits may be loose and variable in nature, are likely to be unstable and, dependent upon depth, may require continuous support. Alternatively, temporary excavation faces will have to be battered back to a safe angle as determined on site. Shallow groundwater at or close to ground level is anticipated across the site.

Excavations extending below ground level are likely to encounter groundwater inflows particularly from coarse soils or water bearing granular layers within fine (clay, silt) and after prolonged periods of wet weather. Such materials will require continuous support. For shallow excavation below groundwater, pumping from sumps in the base of excavations may be feasible. For deeper excavations, sheet pile cut off walls to control inflow and base instability may be required.

6.14.5 Drainage and Balancing Ponds

The proposed drainage is shown on Systra Drainage General Arrangement Drawing IAMP_ONE-SYS-HDG-ZA1-DR-D-05-002-D2-P01 dated 31/01/18 included in Appendix A. Drainage networks, outfalls and storage options for the site are being developed by others as part of further phases of development and detailed design.

However, as noted above, excavations extending below ground level are likely to encounter groundwater inflows particularly from coarse soils or water bearing granular layers within fine (clay, silt) and after prolonged periods of wet weather. Such materials will require continuous support.

Assessment and interpretation of soakaway testing is to be undertaken by the specialist drainage designer as part of detailed design. Soakaway testing has shown the Pelaw Clay (b5) to be of low permeability in the order of $(X)10^{-7}$ m/s or less which is typical for un-fissured clays and clay silts (which contain >20% clay). Groundwater monitoring has shown equilibrium water levels to be at or close to ground level; this combined with the measured permeability's indicates that soakaways are unlikely to be a feasible drainage option within the Pelaw Clay at the site.

As long term groundwater level is shown to be at or around ground level, long term uplift pressures may be generated by pore water pressures within soils constrained beneath the pond base/ liner and provision of a permanent thickened/ deepened cover layer may need to be considered.

6.14.6 Services

With the exception of potential constraints in forming excavations as described above, it is not anticipated that there will be any unusual geotechnical constraints affecting service installations.

There is no evidence of potential risk to water services, for example due to low concentrations of hydrocarbons within the localised made ground. However, results of the assessment should be agreed with the Water Utility company (Northumbrian Water Ltd) in order to confirm the requirements for potable water supply pipes.

Further specific ground investigation is recommended along the routes of any proposed services as part of further phases of development and detailed design.

6.14.7 Pavements

According to HD25/94 [Ref. 28] (now partially superseded by Interim Advice Note 73/06 (2009)) [Ref. 29], CBR values for imported granular (coarse) backfill material must exceed 15%. However, new pavement construction at the interface with the A1290 and proposed access road/ s will need to be tied into existing pavement subgrade and capping layers.

Pelaw Clay (b5) is widely present across the site below topsoil at pavement foundation level. The Pelaw Clay was described as soft, firm or stiff orange brown, reddish brown, greyish brown, brown or dark brown, mottled light grey, slightly sandy slightly gravelly clay of intermediate and high plasticity with subangular to rounded, fine to coarse gravel of sandstone, mudstone and coal.

Particle size distribution tests show these soils contain a significant proportion of fines, see Section 6.4 and soils classify as Class 2D (silty cohesive fill) in accordance with the HASHW Series 600 Earthworks Specification (Ref. 27). Laboratory CBR testing indicates a significant proportion of these soils exhibit a CBR of less than 2.5%.

Assuming average construction conditions and a high water table, a CBR value of <2.5% should be assumed for construction costing and pavement design. Under 76/03, the minimum permitted Design CBR is 2.5%. Where a subgrade has a lower CBR it is considered unsuitable support for a pavement foundation and must be permanently improved. The document advises using one of the following options:

- *'The material at the surface can be removed and replaced by a more suitable material. If the depth of relatively soft material is small, it can be replaced in its entirety, although it may only be necessary to replace the top layer. The thickness removed will typically be between 0.5 and 1.0m.*
- *Although the new material may be of better quality, the new Design CBR should be assumed to be equivalent to 2.5%, in order to allow for effects of any softer underlying material and the potential reduction in the strength of the replacement material to its long-term CBR value.*
- *If the soil is cohesive, a lime (or similar) treatment may be appropriate, subject to soil suitability being demonstrated.*
- *For certain conditions, the incorporation of a geosynthetic material into the foundation design may be advantageous.*
- *Assuming the soil is reasonably permeable, a deeper than normal drainage system may be considered, together with a system of monitoring the improvement expected. Design of the main foundation may then be based on the conditions are achievable in the time available subject to consideration of the long-term equivalent CBR value'.*

It is noted that pavement optimisation may be considered in accordance with 73/06, following an assessment of anticipated traffic frequency and loading. The thickness of the granular sub base construction below the bound pavement could be further optimised during detailed design by incorporating geogrid reinforcement. Such reinforcement layers could be placed into the unbound granular sub base construction to provide a reinforced flexible pavement over the Pelaw Clay present at formation level.

6.14.8 Structure Foundations

For geotechnical design, the National Annex for Eurocode 7 BS EN 1997-1:2004+A1:2013 (Geotechnical Design Part 1 – General Rules) (Ref. 30) and Eurocode BS EN 1990-2002 (Basis of Structural Design) (Ref. 31) are adopted in order to define characteristic values for use in foundation design. The accompanying National Annex (Ref. 31) sets out that from the three Design Approach options given in Eurocode 7, only Design Approach 1 (DA1) is to be used in the United Kingdom. DA1 has two possible load combinations, DA1-1 and DA1-2.

DA1-1 (Combination 1) involves applying partial factors to actions (loads) or the effects of actions (dead loads/ surcharges) whilst using unfactored values for the soil parameters and earth resistance.

However, it is noted that at this preliminary development stage details of proposed structure actions (axial and/ or lateral load conditions) or structure specific serviceability limits are not available. It is presumed that these are to be developed as part of further phases of detailed design by others. **Without known load conditions and serviceability limits it is not possible to provide foundation design recommendations compliant with Eurocode 7 and the British Standard Code of Practice for Foundations.**

Therefore, the following discussion is intended to provide general foundation recommendations only and will need to be developed once structure loads and serviceability limits are known.

Lightly loaded structures may be founded on shallow spread foundations or raft foundations bearing within the natural succession below any made ground (d1-d5) and below the depth of influence of any seasonal, climatic or vegetation effects.

Given the thickness of soft and firm Pelaw Clay (b5), the underlying soft (low strength) and firm (medium strength) laminated clays (b2), and firm and stiff glacial till (b1) over carboniferous bedrock, for larger heavily loaded structures and floor slabs piled foundations are proposed.

Significant variation in rockhead level has been proved on the site across the area crossed by the infilled buried glacial valley as shown on Drawings 60283414-M015-ACM-L1-GE-003 and 004. Variation in rockhead level below the proposed development units is also illustrated on the geological sections included as Drawings 60283414-M015-ACM-L1-GE-005 to 007. Allowance would need to be made for varying pile lengths and pile capacity across the footprint of proposed structure units.

At this preliminary design stage, pile cap level, possible pile layout, pile group effects, laterals loads, settlement and tension requirements etc. are unknown and pile foundations would need to be assessed for serviceability as part of further detailed design. Detailed calculations will need to be prepared in accordance with Eurocode 7 when anticipated pile loads, layout, and serviceability limits are defined, as Eurocode 7 requires both soil parameters and loads are factored.

Driven piles are technically feasible in the ground conditions present across the site although there is a risk that obstructions may be encountered especially within the Lower Glacial Till (b1) if the intention is to extend them to end bear on bedrock. Driven piles have the disadvantage of inducing significant noise and ground vibrations during driving which may have an adverse effect on nearby farmstead properties, although they are located significant distances from the IAMP ONE development site.

The use of conventional bored piles is considered practical. However, temporary casing may be required to support the pile bores in water bearing Laminated Clays (b2) which are interbedded with thin lenses of glacial sands (b3) and silts (b4) below groundwater level. Pile bores would therefore need to be filled with water or drilling mud to balance external water pressures to avoid base disturbance during drilling. Allowance should be made for placing concrete by tremie.

Continuous flight auger (CFA) piles have the advantage that they induce less intense ground vibrations and temporary casing is not needed because the bore is continuously supported by soil or concrete on the auger. This means that production rates are generally higher than those achieved by conventional boring. CFA is considered technically the most favourable piling option.

The construction of CFA piles will require careful supervision and monitoring in order to ensure that the design capacity is achieved. Pile load tests will be required to prove pile performance and integrity tests should be specified on working piles to confirm there no workmanship or quality issues.

6.14.9 Floor Slabs

Proposed building column loads are unknown. If a ground bearing floor slab is proposed, it is recommended that settlement calculations are completed as part of detailed design, in accordance with Eurocode 7, with reference to anticipated load conditions and serviceability limits required by the end user. Allowance should be made for the formation to be proof rolled to limit potential for differential settlement where lightly loaded ground bearing floor slabs are proposed.

Floor slabs subject to higher loads or stringent serviceability limits may need to be piled to carry the loads into most competent strata underlying the site, notably the Lower Glacial Till (b1) or underlying bedrock (a1- a3). It is noted that the depth to bedrock varies significantly across the width of the infilled glacial valley, as discussed in Section 5.

Therefore the design will need to allow for varying pile lengths across the footprint of the proposed structure floor slabs.

6.14.10 Unexploded Ordnance (UXO)

An aerodrome identified as RAF Usworth was present south of the site since 1916. The area has been redeveloped since 1984 and is now predominantly occupied by developments associated with the Nissan UK car manufacturing plant. The proximity of the aerodrome to the site raises the risk that Unexploded Ordnance (UXO) may be present. As a result, further specific desk study was recommended and undertaken by Zetica in 2016. An extract of the report identifying specific UXO risk is included in Appendix D of this GIR.

CPT Magnetometer Testing for UXO (CPTM-04 to 09) was undertaken within the Phase ONE site to target risk areas previously identified by Zetica at the drilling locations only in order to satisfy risk to drilling staff in order to comply with the Construction Design Management (CDM) Regulations 2015 [Ref. 24].

A copy of the CPT-M traces is included in Appendix D of the Dunelm Factual Preliminary SI Report [Ref. 10]. UXO risk was not identified at the exploratory hole positions as part of the ground investigation works.

However, as these tests only target a small volume of the ground in relation to the size of the area where this risk may be present, it remains over these areas as only the exploratory hole positions have been cleared. This particularly where earthworks or excavations are proposed and therefore this risk should be included in the Detailed Design and Construction Risk Registers for the project.

6.14.11 Preliminary Contamination Risk Assessment

Conceptual Site Model

This section of the report comprises an analysis of the chemical testing undertaken in the ground investigation.

The obligations for a Developer are set out in The National Planning Policy Framework, March 2012, Department for Communities and Local Government, ISBN: 978-1-4098-3413-7:

> *120. To prevent unacceptable risks from pollution and land instability, planning policies and decisions should ensure that new development is appropriate for its location. The effects (including cumulative effects) of pollution on health, the natural environment or general amenity, and the potential sensitivity of the area or proposed development to adverse effects from pollution, should be taken into account. Where a site is affected by contamination or land stability issues, responsibility for securing a safe development rests with the developer and/ or landowner.*

In order to make an assessment of construction, environmental and human health risks a conceptual model needs to be developed for the site. This requires an examination of the 'Source-Pathway-Receptor' linkages to define construction, human health and environmental risk associated with existing and future conditions. The first step of the model development is to identify the contaminants of concern from possible sources and potential receptors on and around the site.

The risk assessment is based on guidance provided in CIRIA C552 - Contamination Land Risk Assessment, A Guide to Good Practice [Ref. 36]. The risk assessment is performed in accordance with the precautionary principle, in which a pathway is assumed to exist unless there is reasonable contrary evidence. The risk associated with each source-receptor linkage is a product of the probability that a significant pathway exists and the severity of the potential impact. For preliminary risk assessment the adopted method for risk evaluation is a qualitative method and involves classification of:

- magnitude of the potential consequence (severity) of risk (Table 6.3 - CIRIA 552), classified as: Severe, Medium, Mild, Minor.
- magnitude of the probability (likelihood) of risk occurring (Table 6.4 - CIRIA 552), classified as High Likelihood, Likely, Low Likelihood, Unlikely.

It will be assumed that chronic impact on human health may be a consequence if concentrations of substances from the industry or process generating the contamination could reasonably be expected to exceed a soil screening value for the identified receptor. This means that the consequence of there being a contaminant linkage depends on both the source and the receptor characteristics. In rare circumstances there could also be an acute impact on human health, however this normally requires a highly contaminative industry or process and a sensitive receptor. The probability of a consequence arising depends upon the properties of the pathway. The assessment is used to identify significant source-pathway-receptor linkages by combining the consequence of exposure to different receptors and sources with the probability of exposure. This is calculated in accordance with Table 6.5 – CIRIA 552, reproduced below:

		CONSEQUENCE			
		Severe	**Medium**	**Mild**	**Minor**
PROBABILITY	**High Likelihood**	Very high risk	High risk	Moderate risk	Moderate/ low risk
	Likely	High risk	Moderate risk	Moderate / low risk	Low risk
	Low Likelihood	Moderate risk	Moderate/ low risk	Low risk	Very low risk
	Unlikely	Moderate/ low risk	Low risk	Very low risk	Very low risk

Receptors

The receptors identified for potential impact from development and use of the site is as follows;

- Members of the General Public (visitors/ neighbours)
- Site Users (staff)
- Ground Workers and Service Maintenance Staff
- Building Materials/ Water Services
- Coal Measures Secondary (A) Aquifer
- Surface Water Courses
- Fauna & Flora in landscaped areas

For the purpose of the assessment it is assumed that the Phase ONE Site will be developed as a Commercial land use as defined in the DEFRA CLEA exposure model. The sensitive human health receptor for this land use is an adult female worker. Soil screening for the sensitive receptor will also be adequate for protection of members of the general public, ground workers and service maintenance personnel assuming standard PPE and adequate hygiene facilities are provided for the latter two cases. Conservative soils screening values have been selected from the LQM/ CIEH S4UL thresholds for a commercial end use for a Sandy Loam with a Soil Organic Matter Content of 1%.

Water quality screening values have been taken where available from regulatory standards and guidance for protection of surface freshwater receptors e.g. EQS, in preference to Drinking Water Standards, since the local Coal Measures strata are regarded principally as a pathway to surface water rather than a sensitive receptor with regard to water resources.

Risk from chemical attack on building materials is included in the geotechnical risk assessment.

Contaminant Sources

Evidence of Ordnance Survey mapping indicates the site to be agricultural land since at least 1862. The east boundary of the site alongside the A1290 was formerly a railway line up until the 1970's. There is local development indicated on the eastern boundary of the site at the Hylton Lane Level crossing and depot, which from 1921 to 1980 was location of the Three Horse Shoes (Public House). The area to the east of Phase ONE was Officer's quarters for the Usworth RAF Airbase, subsequently Sunderland Airport; however no direct impact of this is anticipated across the (former) railway line onto the site itself. No significant contaminative history has been identified for the site with respect to the relatively insensitive commercial land use.

There is no indication of made ground being present on site away from the railway/ road embankment and former Three Horse Shoes depot/ public house; however historical mapping does not offer a continuous record of past activity. Made ground may be present due to overspill from the railway of ballast and embankment fill, import of materials for improvement of farm tracks, drainage or raising of low-lying areas. Railway embankments are commonly constructed of colliery spoil or ash and clinker from associated industry. Imported fills would likely comprise similar wastes generated in the locality and also general demolition rubble. The most likely contaminants of made ground are heavy metals/ metalloids, sulphate, acids or alkalis associated with colliery spoil and ash, and PAHs and other hydrocarbons associated with combustion products, coal tars, oil and fuel. Spills of oil from electrical equipment may contain PCBs. Some railway trackside equipment contains asbestos, which is also commonly found in demolition rubble.

Made ground on the site is unlikely to be thick enough or have sufficient organic content to be a significant source of ground gas. Old workings of shallow coal seams if present below the site could be a source of hazardous mine gas or offsite landfill could be sources of hazardous gas. Natural organic deposits such as alluvium and peat may contain high methane concentrations but except in exceptional circumstances, low rates of on-going gas generation means the risk for creation of explosive atmospheres is low.

Natural soils may be contaminated by localised spills of agrochemicals. This is a plausible if unlikely risk to ground workers; however it would also be intrinsically difficult to find any such hotspots therefore the most practical solution is avoidance of soils showing visual or olfactory evidence of contamination. Agrochemicals are likely to be present at low concentrations in all soils however toxicity of chemicals used on crops is generally short-lived and unlikely to be an appreciable risk from either short-term or long-term exposure.

Due to the greenfield nature of the site and the insensitivity of the proposed land use to contamination, the scope of investigation for contamination was therefore limited to obtaining a broad characterisation of the soil lithology to identify anthropogenic content, and testing of a few representative samples of each soil type encountered to confirm presence or absence of widespread unexpected contamination.

Pathways

Members of the general public and site users are unlikely to have much direct exposure to site soils, however the CLEA exposure model assumes pathways via soil ingestion, dermal contact and inhalation of soil dust are active for site users via contact with soil during work breaks. Ground workers may be exposed during construction; however standard health and safety measures are likely to mitigate this risk with the possible exception of exposure to fibrous asbestos if present.

Exposure to soil vapours is only likely if there has been an undocumented spill of hydrocarbons. The main risk from vapours in soils occurs from chronic indoor exposure to site users and risk to ground workers within confined spaces via inhalation. Volatiles and dissolved organics may also permeate plastic water pipes and expose site users to chemicals, although often the main complaint is impact on the taste of potable water.

Shallow coal seams are likely to be flooded, and the intervening saturated low-permeability drift deposits are not expected to allow significant migration of gas into building foundations. Migration of landfill gas from the historical landfill c.400m northwest of the site is unlikely to occur due the distance, low permeability of the drift and high water table. A summary of the pre-investigation risk assessment is provided in Table 22 below.

Table 22. Preliminary Risk Assessment

AGROCHEMICALS *Herbicides* *Pesticides* *Fertilisers*	**1. Dermal contact** **2. Ingestion of contaminated soils** **3 . Inhalation of dust** **4. Inhalation of soil vapour** **5. Ingestion of contaminated water** **6. Plant uptake, ingestion** **7 . Migration of mine/ ground gases to confined spaces** **8. Leaching, migration, of contaminants to groundwater** **9. Leaching, runoff to surface water** **10. Contact with building materials** **11. Permeation of water pipes**	General Public (1,2,3,4,5)	None	n/a	n/a
		Site Users (1,2,3,4,5)	None	n/a	n/a
		Ground Workers (1,2,3,4,5)	Medium	Unlikely	Low
		Fauna & Flora (6)	Minor	Low likelihood	Very low
		Building materials (10)	Minor	Low likelihood	Very low
		Water pipes (11)	Medium	Unlikely	Low
		Groundwater (8)	Mild	Low likelihood	Low
		Surface Water (9)	Mild	Low likelihood	Low
MADE GROUND **RAILWAY (BALLAST/FILL)** **IMPORTED FILL** **FARM TRACKS** *Heavy metals / metalloids* *PAHs, TPH* *Asbestos*		General Public (1,2,3,4,5)	Medium	n/a	n/a
		Site Users (1,2,3,4,5)	Medium	Unlikely	Low
		Ground Workers (1,2,3,4,5)	Medium	Low likelihood	Moderate/Low
		Fauna & Flora (6)	Minor	Unlikely	Very low
		Building materials (10)	Mild	Low likelihood	Low
		Water pipes (11)	Medium	Unlikely	Low
		Groundwater (8)	Mild	Low likelihood	Low
		Surface Water (9)	Mild	Low likelihood	Low
HAZARDOUS GASES **IMPORTED FILL** **OFFSITE LANDFILLING** **COAL MINING** *Carbon Dioxide* *Methane* *Hydrogen Sulphide* *Soil vapours*		General Public (7)	Severe	Unlikely	Moderate/ Low
		Site Users (7)	Severe	Unlikely	Moderate/ Low
		Ground Workers (7)	Severe	Unlikely	Moderate/ Low
		Water pipes (11)	Medium	Unlikely	Low

The only appreciable risk (Moderate/Low) from direct exposure to soil is anticipated to be within the construction phase of the development when ground workers may be exposed to unexpected contamination on the site. This risk can be adequately mitigated through construction health & safety management on site. The main objective of the soil investigation will be to confirm that the site is largely Greenfield and hence no risk would be anticipated to site users or controlled waters.

The risk from ground gases or mine gases is Moderate/ Low resulting from combination of a severe risk (methane explosion) leading to possible loss of life, and a probability rating of Unlikely (an improbable event which is by no means certain in the long term and less likely in the short term). Accordingly gas monitoring has been conducted in order to evaluate this risk further.

Soil Screening

Investigation of Phase ONE was carried out as part of a wider investigation for the IAMP area. In total eighteen boreholes including two re-drill of BH16 and ten trial pits including one re-dig were completed within the Phase ONE area. This represents twenty-three locations with an average spacing of approximately 250m, taking into account co-location of exploratory holes. This is considered to be acceptable for a Greenfield site and an insensitive commercial land use. It is obvious however that undocumented development or waste disposal on the site may be missed with a large spacing, although the consequences for development from contamination would not necessarily be significant given the high soil screening thresholds and lack of any obvious signs of contamination having affected the agriculture.

No significant anthropogenic contamination was encountered in the exploratory holes in Phase ONE. Made Ground was described by the AEG site engineer in BH28 and BH31, but as discussed in the soils section, this is believed by AECOM to be natural soil. The fragments of red brick are most likely to be sandstone gravel. Otherwise the soils are exactly as described in other holes, and there is no difference in the soil gradings or chemical analysis compared with soils in other parts of the site.

Eleven soil samples were selected for testing including two samples from the deeper “made ground” in BH28 and BH31 at 0.5mbgl and 0.7mbgl, respectively;

> BH24(0.1-e1), BH28(0.1-d1), BH28(0.5-d5), BH31(0.1-d1), BH31(0.7-d5), BH38(0.1-d1), BH45(0.1-e1), BH46(0.2-e1), BH47(0.1-d1), BH48(0.1-e1), TP10(0.2-d1)

The other samples were taken within topsoil at a depth of 0.1mbgl or 0.2mbgl, which is typically logged within Phase ONE by AEG as either;

> (e1) “Dark brown slightly sandy slightly gravelly clayey TOPSOIL. Gravel is subangular to rounded, fine to coarse of sandstone, mudstone and coal.“
>
> (d1) “Brown slightly sandy slightly gravelly clayey topsoil with some rootlets present. Gravel is angular to subrounded, fine to coarse of coal and sandstone.”

No difference was found in chemical composition between the two descriptions of Topsoil and Made Ground provided by AEG therefore all samples have been classified together as Topsoil, although it could be argued that BH28@0.5m and BH21@0.7m should be classified as Pelaw Clay (b5).

A summary of chemical testing of total concentrations has been provided in Table 23 below against the adopted standards. Full details with laboratory certificates are provided in the Dunelm Factual Preliminary SI Report [Ref. 10]. No exceedances of the thresholds for a Commercial landuse were identified.

Table 23. Soil Screening – Total Concentrations of Potential Contaminates in Topsoil (11 samples)

Parameter	Units	Commercial Threshold Licence S4UL3064	Topsoil Range Phase ONE (minimum-maximum)
Organic Matter	%	(>1)	(1.8 to 6.4)
pH - Automated**	pH Units	9	(6.1 to 8.3)
Total Cyanide	mg/kg	1200	(<1)
Free Cyanide	mg/kg	150	(<1)
Total Phenols (monohydric)	mg/kg	440	(<1)
Asbestos (Soil Screening)	(-)	(trace)**	None Detected
Arsenic (aqua regia extractable)	mg/kg	640	(6.6 to 21)
Barium (aqua regia extractable)*	mg/kg	22000	(110 to 270)
Boron (water soluble)	mg/kg	240000	(1 to 2.1)
Cadmium (aqua regia extractable)	mg/kg	190	(<0.2 to 0.3)
Chromium (hexavalent)	mg/kg	33	(<4)

Parameter	Units	Commercial Threshold Licence S4UL3064	Topsoil Range Phase ONE (minimum-maximum)
Chromium (aqua regia extractable)	mg/kg	8600	(22 to 38)
Copper (aqua regia extractable)	mg/kg	68000	(17 to 89)
Lead (aqua regia extractable)	mg/kg	2300	(22 to 170)
Mercury (aqua regia extractable)	mg/kg	58	(<0.3)
Nickel (aqua regia extractable)	mg/kg	980	(15 to 44)
Selenium (aqua regia extractable)	mg/kg	12000	(<1)
Zinc (aqua regia extractable)	mg/kg	730000	(49 to 130)
Benzene	µg/kg	27	(<0.001)
Toluene	µg/kg	56000	(<0.001)
Ethylbenzene	µg/kg	5700	(<0.001)
p & m-xylene	µg/kg	5900	(<0.001)
o-xylene	µg/kg	6600	(<0.001)
MTBE (Methyl Tertiary Butyl Ether)	µg/kg	7900	(<0.001)
TPH-CWG - Aliphatic >EC5 - EC6	mg/kg	3200	(<0.001)
TPH-CWG - Aliphatic >EC6 - EC8	mg/kg	7800	(<0.001)
TPH-CWG - Aliphatic >EC8 - EC10	mg/kg	2000	(<0.001)
TPH-CWG - Aliphatic >EC10 - EC12	mg/kg	9700	(<1)
TPH-CWG - Aliphatic >EC12 - EC16	mg/kg	59000	(<2)
TPH-CWG - Aliphatic >EC16 - EC21	mg/kg	1600000	(<8)
TPH-CWG - Aliphatic >EC21 - EC35	mg/kg	1600000	(<8 to 12)
TPH-CWG - Aliphatic > EC35 - EC44	mg/kg	1600000	(<8.4)
TPH-CWG - Aromatic >EC5 - EC7	mg/kg	26000	(<0.001)
TPH-CWG - Aromatic >EC7 - EC8	mg/kg	56000	(<0.001)
TPH-CWG - Aromatic >EC8 - EC10	mg/kg	3500	(<0.001)
TPH-CWG - Aromatic >EC10 - EC12	mg/kg	16000	(<1)
TPH-CWG - Aromatic >EC12 - EC16	mg/kg	36000	(<2)
TPH-CWG - Aromatic >EC16 - EC21	mg/kg	28000	(<10)
TPH-CWG - Aromatic >EC21 - EC35	mg/kg	28000	(<10 to 17)
TPH-CWG - Aromatic > EC35 - EC44	mg/kg	28000	(<8.4)
Naphthalene	mg/kg	190	(<0.05 to 0.32)
Acenaphthylene	mg/kg	83000	(<0.05)
Acenaphthene	mg/kg	84000	(<0.05)
Fluorene	mg/kg	63000	(<0.05)
Phenanthrene	mg/kg	22000	(<0.05 to 0.55)
Anthracene	mg/kg	520000	(<0.05 to 0.09)
Fluoranthene	mg/kg	23000	(<0.05 to 0.83)

Parameter	Units	Commercial Threshold Licence S4UL3064	Topsoil Range Phase ONE (minimum-maximum)
Pyrene	mg/kg	54000	(<0.05 to 0.55)
Benz(a)anthracene	mg/kg	170	(<0.05 to 0.51)
Chrysene	mg/kg	350	(<0.05 to 0.48)
Benzo(b)fluoranthene	mg/kg	44	(<0.05 to 0.66)
Benzo(k)fluoranthene	mg/kg	1200	(<0.05 to 0.35)
Benzo(a)pyrene	mg/kg	35	(<0.05 to 0.58)
Indeno(1,2,3-cd)pyrene	mg/kg	500	(<0.05 to 0.31)
Dibenz(a,h)anthracene	mg/kg	3.5	(<0.05 to 0.11)
Benzo(ghi)perylene	mg/kg	3900	(<0.05 to 0.31)

(*AECOM AGAC, **pH/ asbestos customary limits)

Hazardous Gas

Four gas instruments have been monitored on four occasions between 20/12/17 and 9/02/18 within the IAMP site, two of which, BH28 and BH35 fall within Phase ONE. Results for BH32 and BH36 provide background information for the general geological conditions and offsite migration however they are not as relevant to the site. Results of the gas monitoring are provided in the appendices of this report.

No coarse (granular) soils were intercepted at shallow depth and all instruments were installed within Pelaw Clay – b5 (logged as cohesive Made Ground - d5 in BH28). Clays are relatively impermeable although vertical fissuring could provide preferential flow paths in unsaturated stiff clays at shallow depth.

Atmospheric pressure at the time of visits was between a high pressure of 1028mbar on the first visit (falling trend) and a low pressure of 1001mbar (rising trend). No visits were conducted at a time of low and falling pressure. Water levels within the boreholes were generally above the response zone of their respective instruments (1.5mbgl and 1.0mbgl in the cases of BH28 & BH35, and 1.0mbgl for BH32 & BH36) with the singular exception of BH32 in the first visit (1.09mbgl). This means that with one exception, the gas instruments were not directly in contact with soil gas. Saturation of the clay strata will also limit advective (pressure-driven) gas flow and reduce diffusive migration of ground/ mine gas to insignificant rates.

No initial or steady borehole gas flow rates were detected above the instrument level of detection believed to be 0.1 l/hr, although given the high water levels any measured flows would have been unreliable.

Concentrations of methane were below the detection limit of 0.1%v/v. The highest concentration of carbon dioxide detected within the Phase ONE area was 0.1%v/v (BH28 on 12/12/17 and 12/01/2018 and BH35 on 9/02/2018). The highest carbon dioxide concentration overall was 0.3%v/v BH36 on the final visit.

No VOCs were detected by PID and carbon monoxide and hydrogen sulphide concentrations were also below detection limits (not given).

Based on the gas measurements taken, the CIRIA Characteristic Situation is CS1 for a Gas Screening Value of 0.0003, and no gas protection is indicated. In certain respects the gas monitoring has been sub-optimal since none of the visits occurred at a time of low and falling barometric pressure. However due to the high water levels and cohesive soils it is not considered that further monitoring is necessary.

Controlled Waters

No risk is anticipated for controlled waters (groundwater or surface water) from the soils encountered in the investigation. A preliminary screening of the risk for generation of leachate from topsoil has been undertaken using the 2:1 or 10:1 Liquid/ Solid (L/S) leaching stage undertaken for Waste Acceptance Criteria testing. Leachable inorganic components, total dissolved organic carbon and phenol have been compared with screening values for surface water (primarily freshwater EQS), or Drinking Water Standards in the absence of EQS. PNEC values have been calculated according to UKTAG guidance for a Dissolved Organic Carbon concentration of 10mg/l, which is considered to be reasonable for screening purposes given that no dilution has been assumed for leachate. The short-term leaching concentration based on the 2:1 L/S stage has been used in preference except in cases where the laboratory has only reported the 10:1 L/S stage, which is more characteristic of long term conditions. The results of the screening have been summarised in Table 24.

The limit of detection exceeded the screening limit for cadmium (2:1 L/S extract only), mercury and phenol index (monohydric phenols) as highlighted in green within the table, however none of these is indicated to be present at significant total concentrations and presence of these substances at leachable concentrations is not anticipated.

Marginal exceedance of leachable copper in BH46@0.2m (0.042mg/l compared with a screening value of 0.04mg/l) and chromium in BH45@0.1m (0.0068mg/l compared with a screening value of 0.0047mg/l) has occurred for the 2:1 L/S extracts as highlighted in pink within the table. The corresponding 8:1 L/S extracts are much lower at <0.001mg/l for chromium and 0.0086mg/l for copper, which suggests these exceedances are due to "first flush" rather than continual leaching. Since no mixing or other dilution has been assumed and the other samples comply, albeit they are similar in magnitude, it is considered that average concentrations will be acceptable, and no impact is likely following normal rates of attenuation to the watercourse (assumed to be x 10 or higher).

The topsoil samples do not appear to contain significant concentrations of leachable metals or anions chloride and sulphate. Total concentrations of TPH/ PAH organics are also low therefore no risk is anticipated to controlled waters from leaching.

Table 24. Leachate Screening

Parameter (mg/l)	10:1 Extract	2:1 Extract	Screening Value	Source (PNEC after UKTAG)
Arsenic	(<0.0011 to 0.0021)	(<0.01)	0.05	WFD England/Wales 2015 - Freshwater Standards
Barium	(0.004 to 0.0433)	(0.0069 to 0.11)	1.3	WHO DWG 2017
Cadmium	(<0.0001)	(<0.0005)	0.00025	WFD England/Wales. 2015 - AA-EQS Inland
Chromium	(<0.0004 to 0.0032)	(<0.001 to 0.0068)	0.0047	WFD England/Wales 2015 - Freshwater Standards
Copper	(0.0039 to 0.034)	(0.019 to 0.042)	0.04	PNEC (Boavailability for DOC of 10mg/l)
Mercury	(<0.0005)	(<0.0015)	0.00007	WFD England/Wales. 2015 - MAC-EQS Inland
Molybdenum	(<0.0004 to 0.0025)	(<0.003)	0.07	WHO DWG 2017
Nickel	(<0.0003 to 0.0018)	(<0.001 to 0.0051)	0.02	PNEC (Boavailability for DOC of 10mg/l)
Lead	(<0.001 to 0.0091)	(<0.005 to 0.01)	0.012	PNEC (Boavailability for DOC of 10mg/l)
Antimony	(<0.0017 to 0.0028)	(<0.005)	0.005	WS Regs 2016 (Eng/Wal)
Selenium	(<0.004)	(<0.01)	0.01	WS Regs 2016 (Eng/Wal)
Zinc	(0.0033 to 0.012)	(0.007 to 0.013)	0.033	PNEC (Boavailability for DOC of 10mg/l)
Chloride	(1.3 to 3.8)	(<4 to 11)	250	SEPA WAT-SG-53 Fresh EQS - AA - 2015
Fluoride	(0.45 to 1.2)	(0.4 to 1.1)	5	SEPA WAT-SG-53 Fresh EQS - AA - 2015
Sulphate	(1.9 to 41)	(4.8 to 18)	400	SEPA WAT-SG-53 Fresh EQS - AA - 2015
Phenol Index	(<0.01)	(<0.13)	0.0077	WFD England/Wales 2015 - Freshwater Standards

Waste Assessment

Materials that are surplus to requirement are classified as waste and subject to rules on waste classification. Natural soils from uncontaminated sites do not as a rule require chemical testing for waste classification. Notwithstanding based on the chemical testing undertaken the topsoil does not contain any Hazardous Properties. Results of WAC testing and screening against Inert, Stable Non-reactive Hazardous Waste and

Hazardous Waste thresholds for each of the eleven soil samples is provided in the Dunelm Investigation Factual Preliminary SI Report (Ref. 10).:

> BH24(0.1-e1), BH28(0.1-d1), BH28(0.5-d5), BH31(0.1-d1), BH31(0.7-d5), BH38(0.1-d1), BH45(0.1-e1), BH46(0.2-e1), BH47(0.1-d1), BH48(0.1-e1), TP10(0.2-d1)

With the exception of Total Organic Carbon (TOC) of 3.5%w/w in BH24@0.1m and 3.6%w/w in BH31@0.1m compared to the Inert Limit of 3%w/w, all WAC parameters meet the WAC criteria for Inert Waste. In the case of TOC there may be flexibility for the landfill when Dissolved Organic Carbon is less than 500mg/l, which appears the case, although values of <0.001mg/l provided for the 10:1 L/S extracts do not square with the range of 12-19mg/l for the 2:1 L/S extracts. However it should also be considered that average TOC is only 2.4%.

Uncontaminated topsoil and subsoil can be used for landfill restoration without attracting landfill tax, or it may be possible to transfer uncontaminated soil off the site under the CL:AIRE Code of Practice. Another option could be to send topsoil to a licensed site for recovery. Given one of these options it is unlikely that topsoil will need to be landfilled and even more unlikely that it would attract one of the higher disposal costs for Non-hazardous or Hazardous waste.

Summary

The ground investigation provides confirmation that the Phase ONE site is greenfield.

No contamination was encountered in the ground investigation. Any made ground is likely to be localised to the boundary of the Site along the A1290. The proposed commercial land use is relatively insensitive to contamination therefore it is unlikely that contamination will be a significant constraint, however if visual or olfactory evidence of contamination were encountered, ground works should be made safe and stopped pending further investigation, risk assessment, remediation works and verification.

No gas protection is indicated based on the high water table and cohesive natural Pelaw Clay (b5) found within the Phase ONE site at shallow depth. This mitigates the generation, storage and migration of hazardous ground gases. The unsaturated zone was too thin to monitor with the installed gas instruments; however results of the monitoring support designation of the site as Characteristic Situation CS1, but this position should be reviewed subsequent to a fuller assessment of the risk of shallow mining for each new building and possible creation of preferential migration pathways for mine gas.

A previous study by Mott MacDonald in 2014 (Envirocheck O/N 56696506_1_1) suggests that no radon protection is required; however AECOM has not been commissioned to update this assessment.

Topsoil as would be expected contains Total Organic Carbon concentrations that exceed the Inert Waste limit of 3%w/w. This is not unusual for topsoil and should not provide any unusual difficulty for disposal if it were not possible to accommodate excavated topsoil on the site within the design.

7. Geotechnical Risk Register

A preliminary geotechnical risk register for the proposed improvement scheme is presented in Table 8 1. The register is in accordance with the format detailed in Highways Agency, HD22/08, Managing Geotechnical Risk (Ref. 2). The purpose of the register is to identify the risks and consequences of those risks together with measures to be undertaken to mitigate the risks. The recommendations provided are based on site observations, an interpretation of the GI information and historical records obtained to date.

The register includes details of the risks identified within the PB Geotechnical Constraints Report (Ref. 1) and the AECOM Geotechnical TBR (Ref. 6) and any subsequent risks identified after completion of GI works. This register will require regular review, and should be updated and revised as the project progresses. Risks and mitigation measures identified in this report should be considered in future detailed design (by others) and where required, discussed further during the reporting of geotechnical design calculations.

Where works are to be completed as part of the detailed design or construction, the risk rating 'After Control' scoring has not been included in the risk register, as this will be completed by others later during scheme development. The risk rating scoring system is based on Managing Geotechnical Risk DETR Partners in Technology Programme Institution of Civil Engineers Thomas Telford (2001) (Ref. 32), which is referenced in HD22/08 (Ref. 2).

In order to comply with the CDM Regulations 2015 (Ref. 24), the Principal Designer should be informed of the results of this study. It is recommended that a detailed Environmental Management Plan, Materials Management Plan (MMP) and Construction Specifications are implemented during the detailed design and construction phases.

It is considered that the following risks should be considered during detailed design:

- Potential unrecorded mine workings in particularly on the eastern portion of the site in the vicinity of Units 4, 5 and 6.
- Slope stability in shallow cuttings formed in the Pelaw Clay (b5). Preliminary design slope gradients of 1 vertical to 3.5 horizontal (1v:3.5h) are proposed.
- Re-use of Pelaw Clay (b5) in bulk earthworks particular in the construction of shallow embankments.
- Shallow groundwater is at or close to ground surface, resulting in the potential for surface water flooding generated from groundwater.
- The Pelaw Clay is of low permeability and is not suitable to form soakaways to discharge highway and development surface runoff as part of scheme drainage proposals.
- As long term groundwater level is shown to be at or around ground level, long term uplift pressures may be generated by pore water pressures within soils constrained beneath the pond base/ liner and provision of a permanent thickened/ deepened cover layer may need to be considered.
- Constraints in forming temporary and permanent excavations (e.g. basements) on site due to the presence of potentially fissured soil (Pelaw Clay (b5)) and shallow groundwater.
- CBR values of less than 2.5% at likely pavement foundation levels within the Pelaw Clay (b5) for consideration of pavement design and construction. Permanent formation improvement is likely to be required.
- Low undrained shear strengths measured or inferred in parts of the Pelaw Clay (b5) from ground surface and underlying Laminated Clay (b2) at depth.
- Variation in depth of superficial soils and rockhead over the site and over the footprint of individual structure units which will influence pile foundation design lengths and may affect serviceability limits for both structure foundations and floor slabs. Global failure, bearing capacity failure, deflection, deformation and/ or differential settlement of the proposed structures will need to be considered as part of further phases of development and detailed design.

- Unexploded ordnance (UXO) remains a risk in areas previously identified by survey and this risk should be included in the Detailed Design and Construction Risk Registers.
- Concrete attack due to high soils/ groundwater aggressivity; data indicates the ground risk is generally low and may be mitigated by adopting appropriate concrete classification.
- Contaminated soils; preliminary assessment of chemical test data does not highlight constraints to preclude the use of Made Ground within the proposed earthworks. However, it is recommended that made ground is not exposed on cutting slope faces to reduce the risk of run-off of potentially impacted leachate during construction.

For the purposes of this GIR, risk has been assessed with reference to ‘likelihood’, ‘severity’ and ‘risk rating’. Risk rating (R) = Likelihood (L) x Severity (S).

LIKELIHOOD (L)		SEVERITY (S)					RISK (R = L x S)	RESPONSE TO RISK
Very probable	5	Very High	5	Potential to halt project	OR	Potential for major claim or similar	17 to 25	Unacceptable: act now to prevent
Probable	4	High	4	Significant delay on overall project		Major impact on cost	13 to 16	Unacceptable: act now to prevent
Possible	3	Medium	3	Major delay on this task, but significant impact on overall project unlikely		Significant impact on cost	9 to 12	Early attention required
Unlikely	2	Low	2	Minor delay on this task, but significant impact on overall project unlikely		Minor impact on cost	5 to 8	Regular attention required
Negligible	1	Very Low	1	No significant impact on task or project		Negligible impact on cost	1 to 4	Monitor

Table 25. Geotechnical Risks

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
STRUCTURES (PROPOSED BUILDINGS)										
Collapse of building	Unknown soil strength Bearing capacity is lower than anticipated or variable Depth of Pelaw Clay and Laminated clay soils deeper or more variable than anticipated. Depth to rockhead and degree of rock surface weathering deeper / thicker than anticipated.	2	5	10	Collapse due to excessive deformation of the structure. Damage to adjacent highway drainage, road pavement and third party utilities.	TBC at detailed design.	Structure specific Ground Investigation Adequate design in light of ground conditions proved on site. Deep foundations used if required. Variable pile lengths required over structure units plan areas. Adequate design of pile rock socket length. Adequate design and allowance of whole life risk.	*	*	*

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
Collapse of building Differential settlement Deflection Deformation	Potentially unrecorded mine workings	2	5	10	Collapse due to excessive deformation of the structure.	TBC at detailed design. In particular Units 4, 5 and 6.	Risk to be mitigated through structure specific ground investigation. Inherent risks remains, to be included in the overall development cost and added to detailed design and construction risk registers.	1	5	5
Settlement (total) Differential settlement Deflection Deformation	Unknown soil strength. Bearing capacity is lower than anticipated or variable below proposed building. Depth of made ground or soils variable/ unknown. Depth of Pelaw Clay and Laminated clay soils deeper or more variable than anticipated. Depth to rockhead and degree of rock surface weather deeper / thicker than anticipated.	2	5	10	Structural damage due to excessive deformation. Damage to adjacent highway drainage, road pavement and third party utilities.	All - TBC at detailed design.	Structure specific Ground Investigation. Adequate design in light of ground conditions proved on site. Deep foundations used if required. Variable pile lengths required over structure units plan areas. Adequate design of pile rock socket length. Adequate design and allowance of whole life risk.	*	*	*

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
Differential settlement Deflection Deformation	Unknown soil strength. Bearing capacity is lower than anticipated or variable below proposed building. Depth of made ground or soils variable/ unknown. Depth of Pelaw Clay and Laminated clay soils deeper or more variable than anticipated. Depth to rockhead and degree of rock surface weather deeper / thicker than anticipated.	2	5	10	Structural damage of floor slabs due to excessive deformation and settlement	All - TBC at detailed design	Structure specific Ground Investigation. Adequate design in light of ground conditions proved on site. Pile foundations to be adopted for floor slabs if required dependent on serviceability limit requirements. Variable pile lengths required over structure units plan areas. Adequate design of pile rock socket length. Adequate design and allowance of whole life risk.			
Difficult foundation construction	Shallow groundwater encountered during foundation construction – Pile integrity and pile squeezing.	2	3	6	Delay in construction programme, disposal costs associated with groundwater.	All - TBC at detailed design	Structure specific Ground Investigation. Adequate design in light of ground conditions proved on site. Balance of water pressures at the toe of the pile during construction by adopting cfa or cased bored piles. Construction pile rig monitoring of auger torque and flight rotation rates to ensure concrete injections match spoil removal, thus avoiding potential integrity problems.			

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
Difficult foundation construction	Shallow groundwater encountered during foundation construction	2	3	6	Delay in construction programme, disposal costs associated with groundwater.	All - TBC at detailed design.	Adequate provision for groundwater pumping / cut off during construction	1	3	3
EARTHWORKS CUTTINGS AND EMBANKMENTS										
Slope failure	Slopes are too steep	3	3	9	Local collapse, damage to third party land/ utilities, road pavement and highway drainage.	All - TBC at detailed design.	Cuttings and embankments (if constructed from site won Pelaw Clay). Form side slopes at appropriate safe gradients. 1 vertical in 3.5 horizontal (preliminary design). Install slope drainage at the crest and toe to ensure groundwater control. Adequacy to be confirmed in detailed design. Import of materials with more favourable geotechnical parameters for embankment construction.	1	3	3
PAVEMENT										
Differential movement between Made Ground and fine (cohesive) deposits.	Low undrained shear strength and CBR below 2.5%.	3	3	9	Differential pavement settlement leading to uneven road surface, cracking and pavement breakup well before design life.	Site wide pavement.	Adequate design. Permanent formation improvement. Flexible pavement and incorporation of geogrid rotation across the potential movement boundary.	1	3	3
Difficult Excavation	Shallow groundwater across the site. Water bearing strata (Made Ground / superficial) exposed within excavations/ road pavement formation.	3	3	9	Difficult construction, extended construction programme, raised cost.	Scheme wide pavement.	Excavations to be formed to ensure positive fall of water into temporary and/ or permanent drainage. Temporary and permanent groundwater control (e.g. pumping) will be required with continuous support during excavation.	2	3	6

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
GENERAL – EARTHWORKS / ENVIRONMENT / UTILITIES										
Difficult construction conditions	Unstable soft ground, shallow groundwater levels, poor trafficking conditions. Depth of Pelaw Clay and Laminated clay soils deeper or more variable than anticipated.	4	3	12	Delay in construction programme, deterioration in site conditions. Groundwater inflow from granular strata. Unstable excavation walls/ slopes. Possible over-excavation.	Site wide.	Adequate pre-contract access enabling works, including matting and drainage measures. Adequate provision for groundwater control (pumping) within excavations during construction Significant obstructions were generally not encountered during GI works. Provision of temporary supports within excavations during construction or batter back temporary excavation faces to a safe angle of repose (to be determined on site).	2	3	6
Aggressive ground conditions	Concrete and/ or steel attack due to the presence of sulphates and acidic soil and groundwater conditions	3	2	6	Corrosion of buried steel leading to excessive structural deflection and/ or serviceability concerns. Reduction in concrete strength/ structural damage.	Site wide	Ground investigation and associated BRE testing. Adequate design. Ground investigation and testing indicates Sulfate Class DS-1 and aggressive chemical environment AC-1 will be required.	1	2	2
Buried services	New construction causes damage to existing buried infrastructure/ services.	3	3	9	Damage to existing buried services during construction. Difficult construction, extended construction programme, cost.	TBC at detailed design.	Adequate services survey/ drawings Service/ utility surveys of existing services to be undertaken. Hand dug inspection pits to confirm position, depth and status of known utilities to be impacted by construction. Possible diversion of utilities, to be confirmed at detailed design.	1	3	3
Pollution of environment	Surface water runoff Dust	3	3	9	Pollution of local environment, disturbance to adjacent site users/ residents.	Site wide	Implement good construction/ site management practices.	1	3	3

Identified Geotechnical Hazard/ Risk	Cause	Risk before Control			Consequence	Structure affected	Control Measures	Risk after Control		
		L	S	R (L*S)				L	S	R (L*S)
Contamination of controlled waters	Surface water runoff in to controlled waters. Piling into underlying Upper and Middle Pennine Coal Measures Secondary A aquifers.	3	2	6	Adverse impact on water quality, with resultant impact on wildlife; costs of clean up.	Site wide	Adequate construction process, Management and Waste Management Plan. Environmental Mitigation. Obtaining temporary discharge licenses to permit discharge to existing NWL sewer network.	1	2	2
High cost for disposal of soil arisings	Soils removed from site are contaminated	3	3	9	Delay in construction programme, increase in disposal costs.	Site wide	Site walkover survey and ground investigation has not indicated widespread deposits of made ground in the Phase ONE development area. The ground investigation encountered materials that would be classified as Inert or Non Hazardous for waste disposal purposes.	1	3	3
Man-made obstructions	Obstructions left from previous site uses	3	3	9	Delay in construction programme, increase in disposal costs. Difficult construction, extended construction programme, cost.	Site wide.	Adequate design. Ground investigation undertaken and man-made obstructions were not encountered. Adequate design and allowance for localised treatment during construction.	1	3	3

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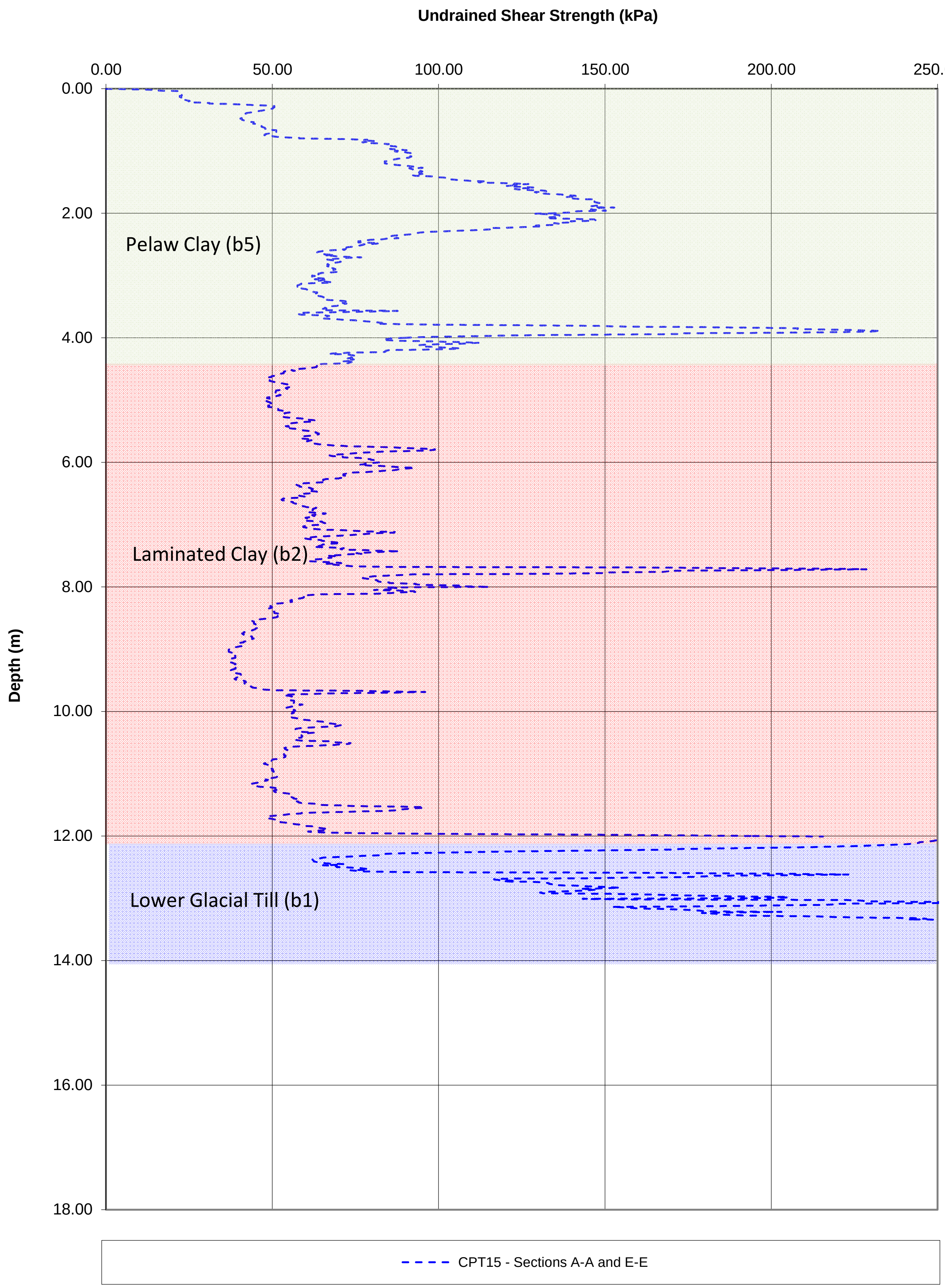
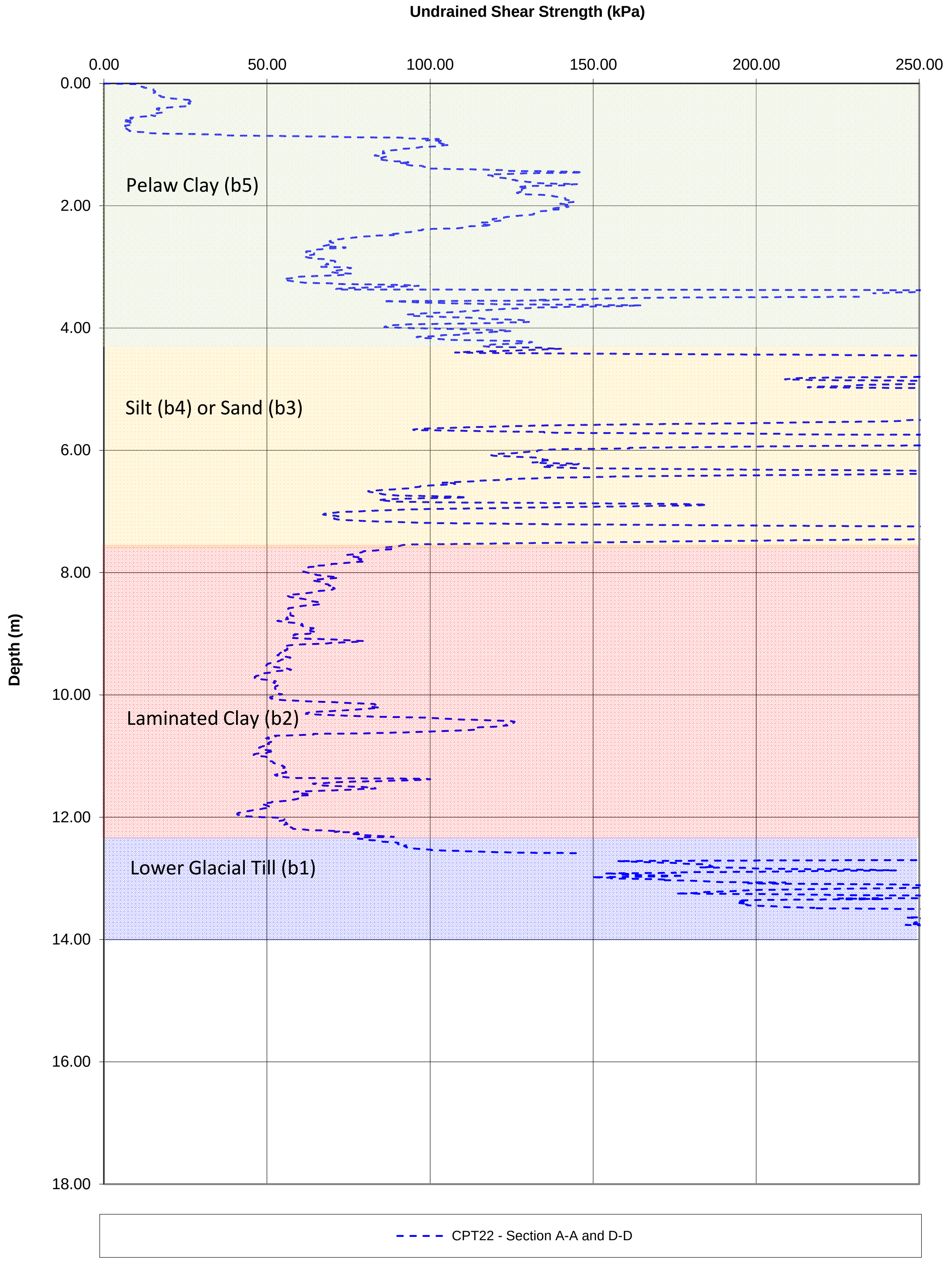
Figures



IAMP Ground Investigation - PHASE ONE	Sunderland City Council / AECOM Imagine it. Delivered.
Material Types and Undrained Shear Strength versus Depth Section A-A and E-E - CPT 15 - Nk = 20	60283414_M015_GEO_RT_001 Date: April 2018 Figure 1



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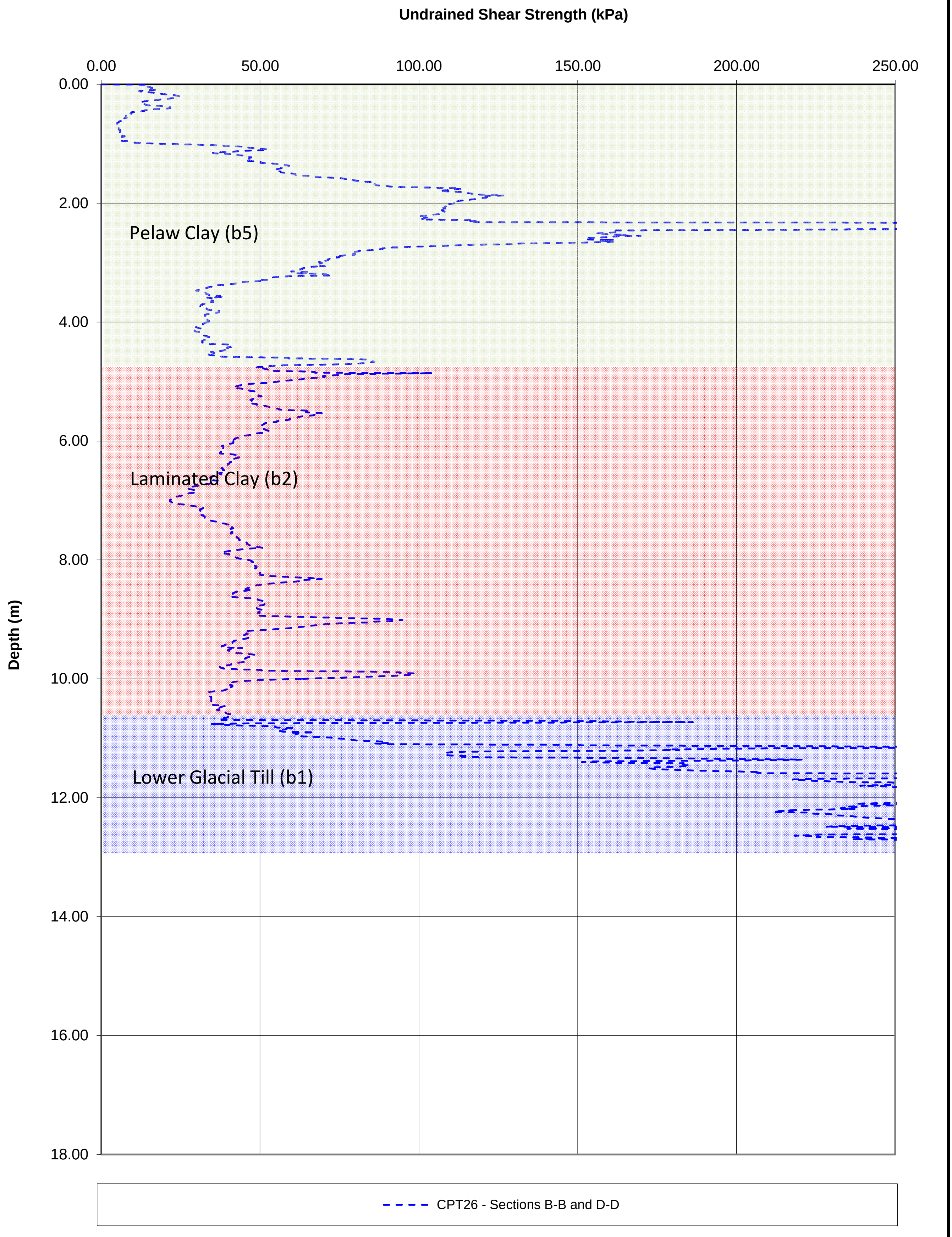
Material Types and Undrained Shear Strength versus Depth Section A-A and D-D - CPT 22 - Nk = 20	60283414_M015_GEO_RT_001 Date: April 2018 Figure 2



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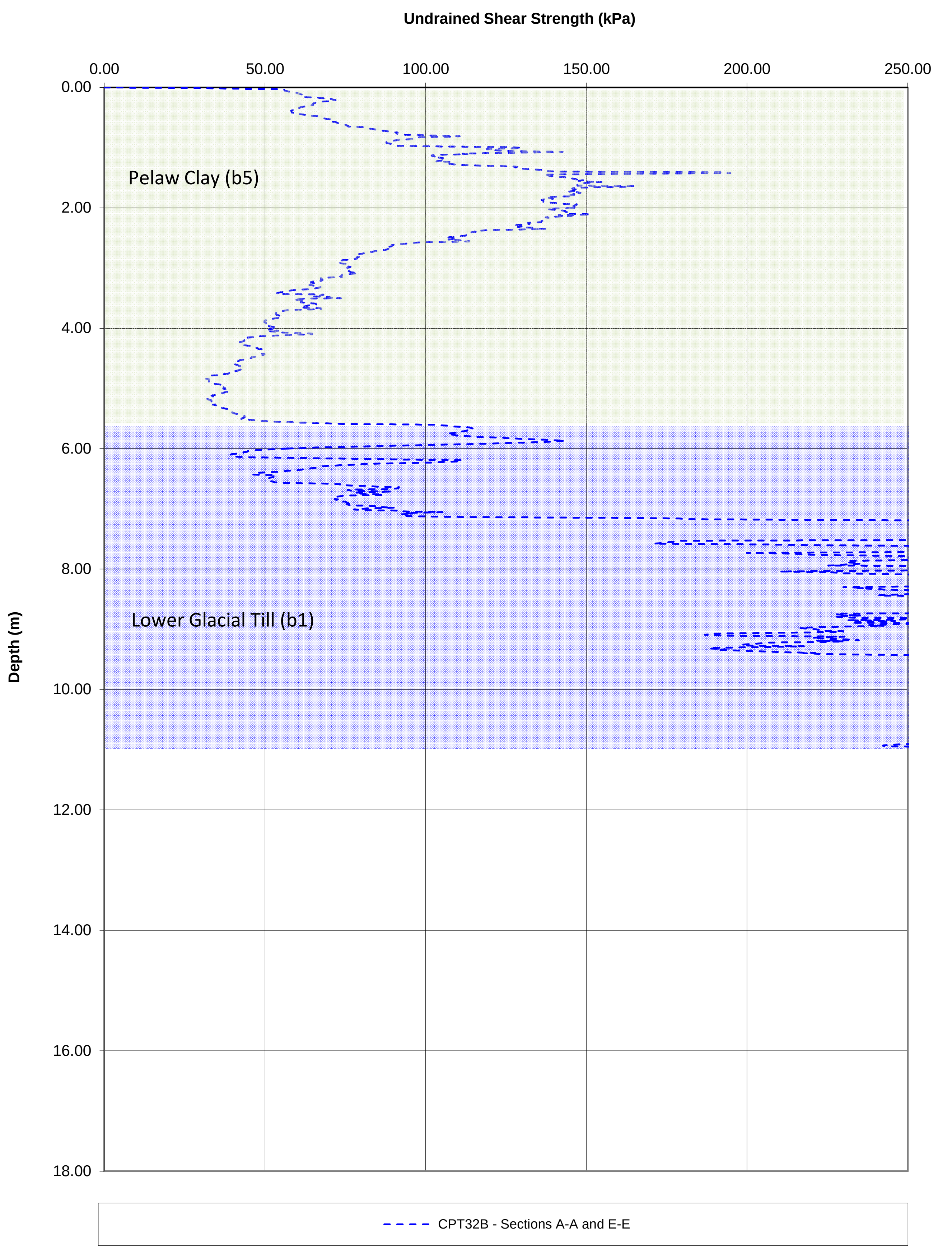
Material Types and Undrained Shear Strength versus Depth Section B-B and D-D - CPT 26 Nk = 20	60283414_M015_GEO_RT_001 Date: April 2018 Figure 3



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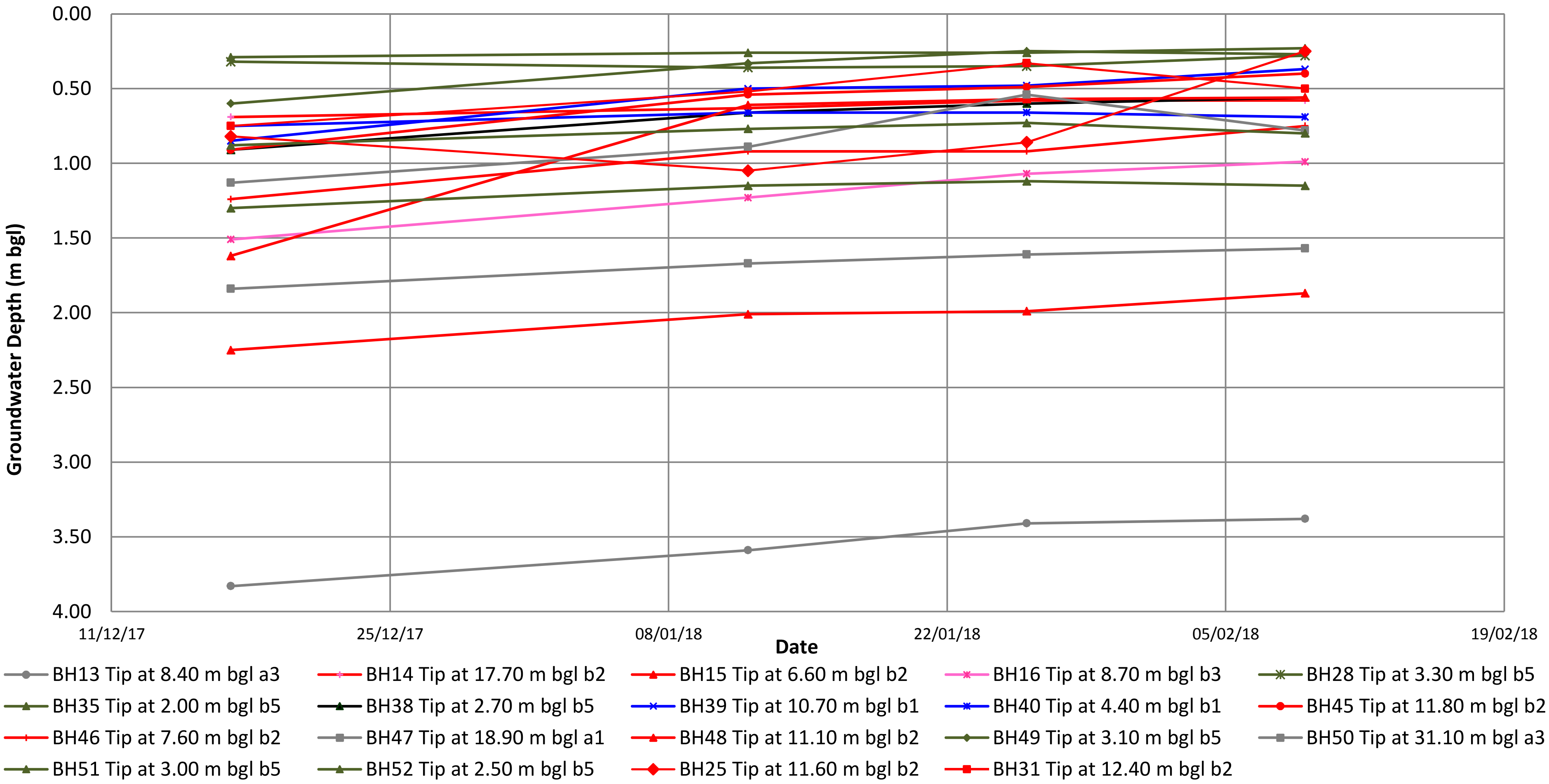

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IAMP Ground Investigation - PHASE ONE
Groundwater Depth versus Date

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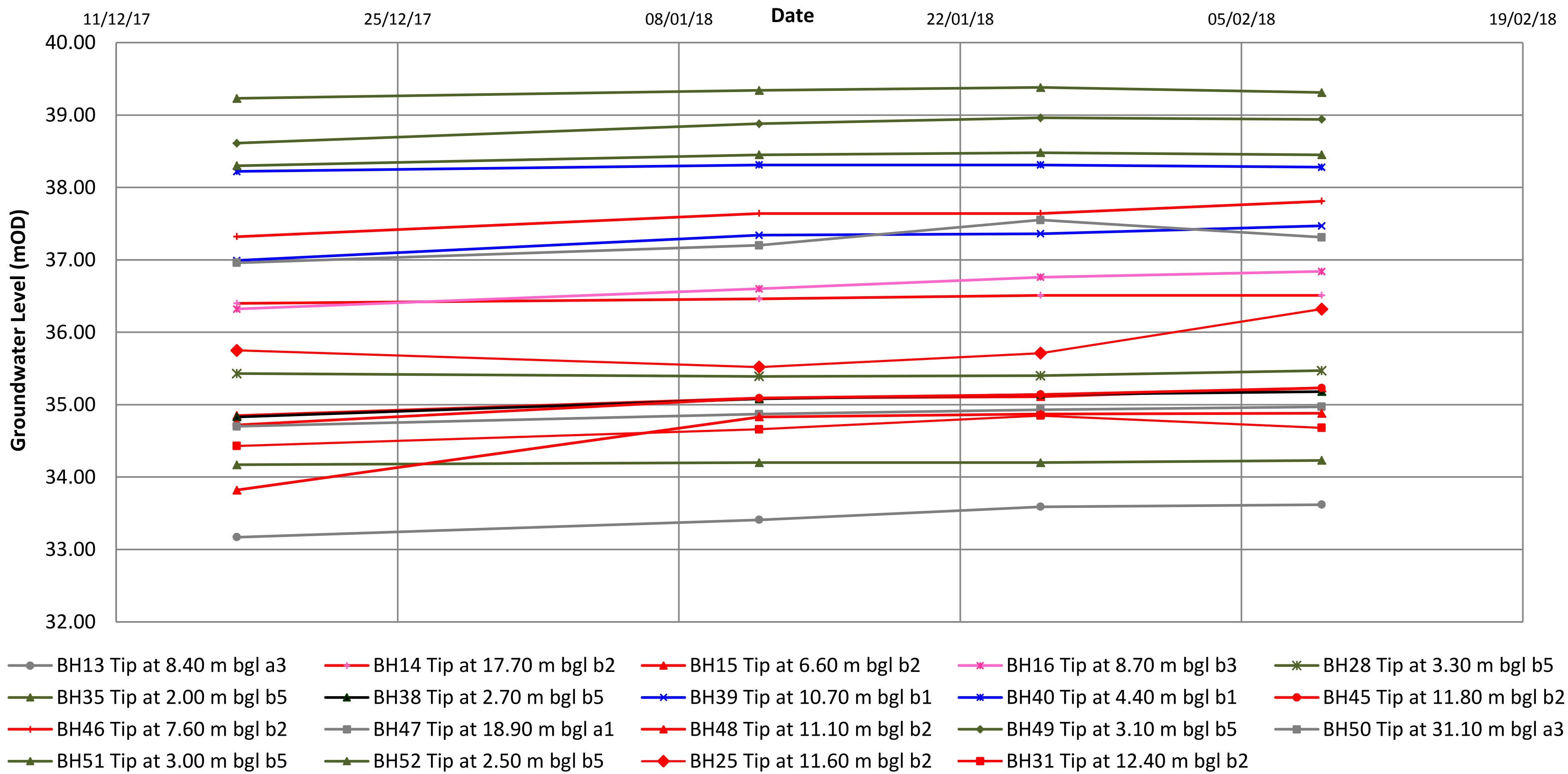
Figure 5



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IAMP Ground Investigation - PHASE ONE
Groundwater Level versus Date

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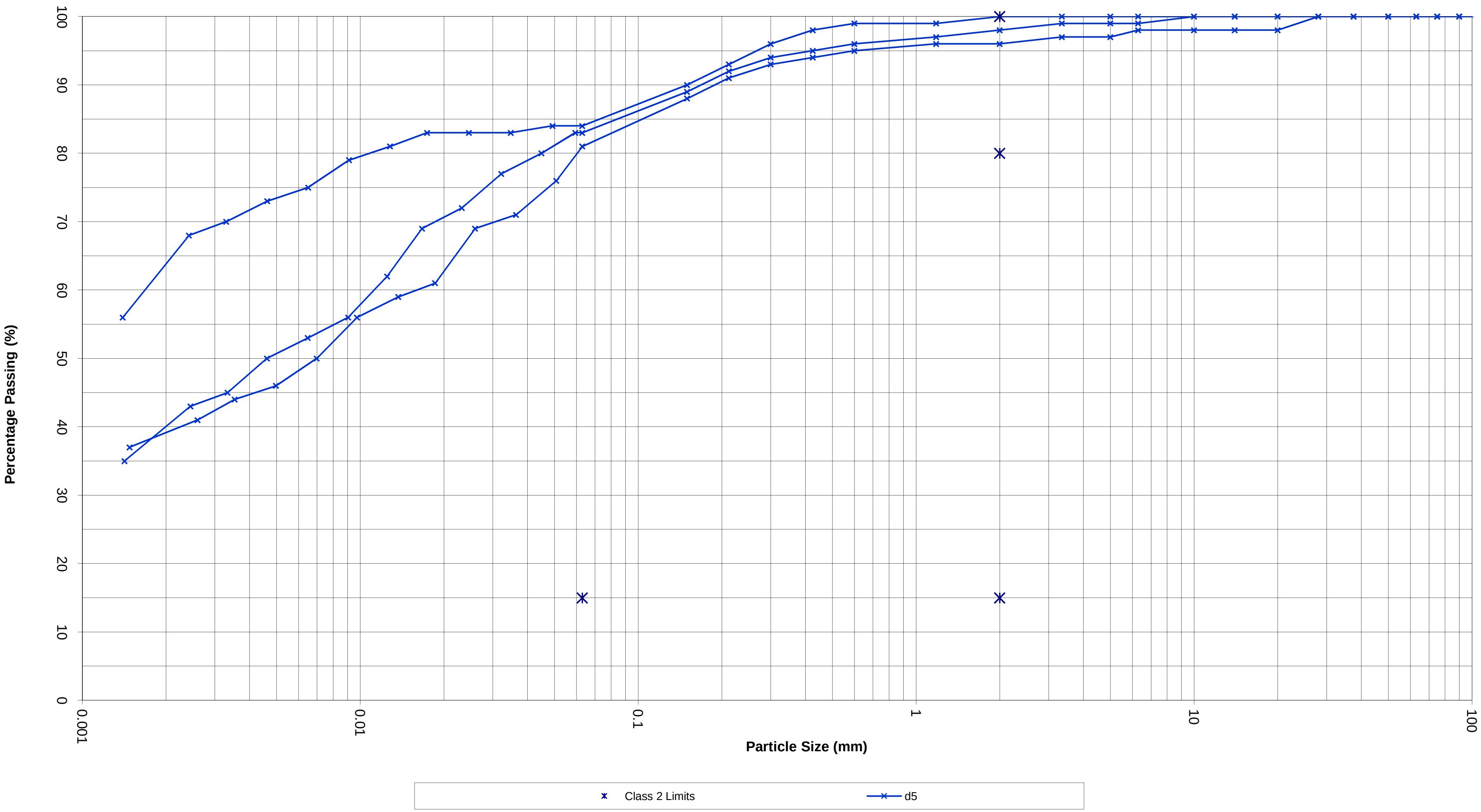
Figure 6



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IAMP Ground Investigation - PHASE ONE
Particle Size Distribution Curve - d5 MADE GROUND clay at depth

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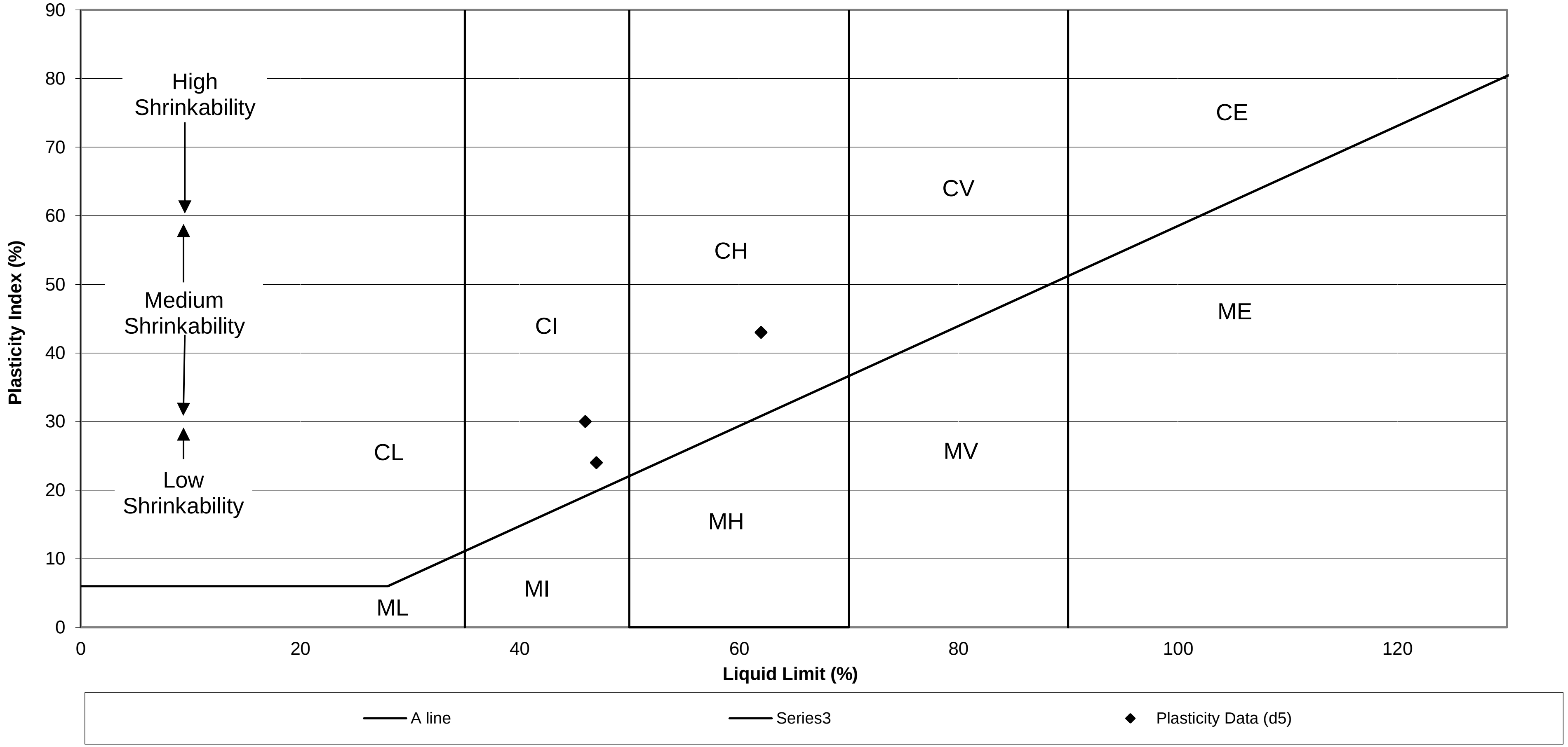
Figure 7



IAMP Ground Investigation - PHASE ONE
Casagrande Plasticity Chart - Made Ground d5

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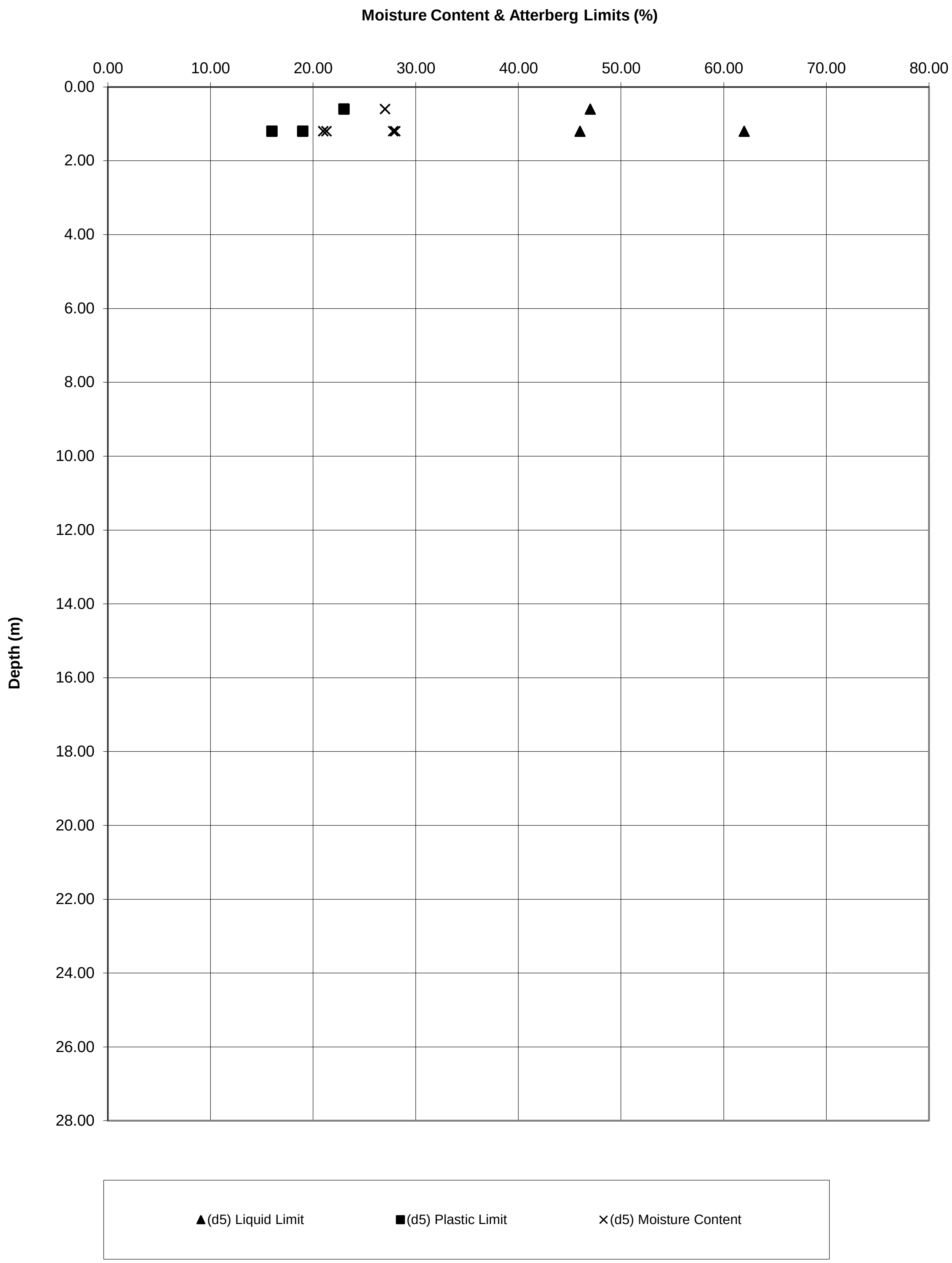
Figure 8



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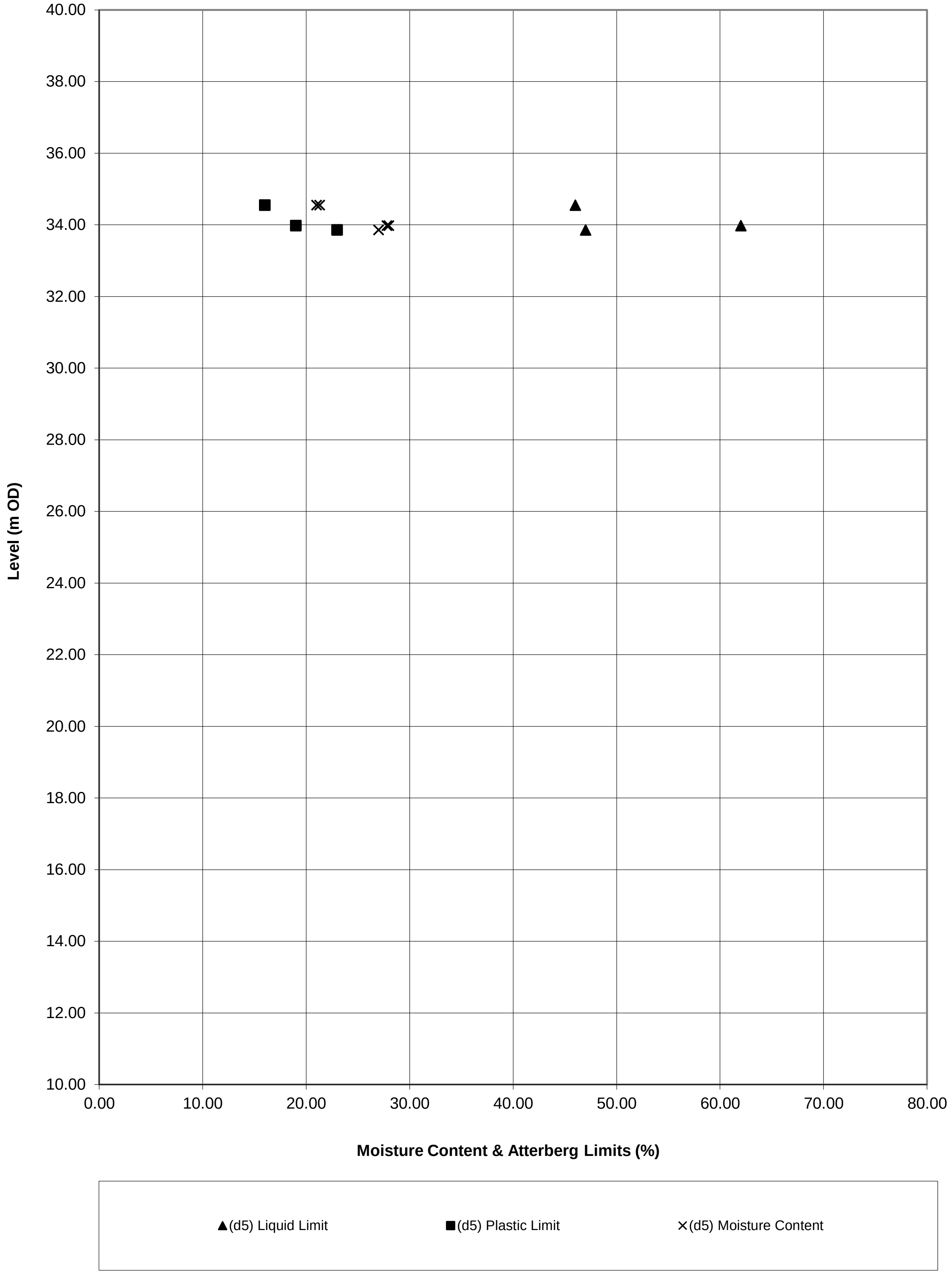
Moisture Content and Atterberg Limits versus Depth Made Ground d5	60283414_M015_GEO_RT_001 Date: April 2018 Figure 9



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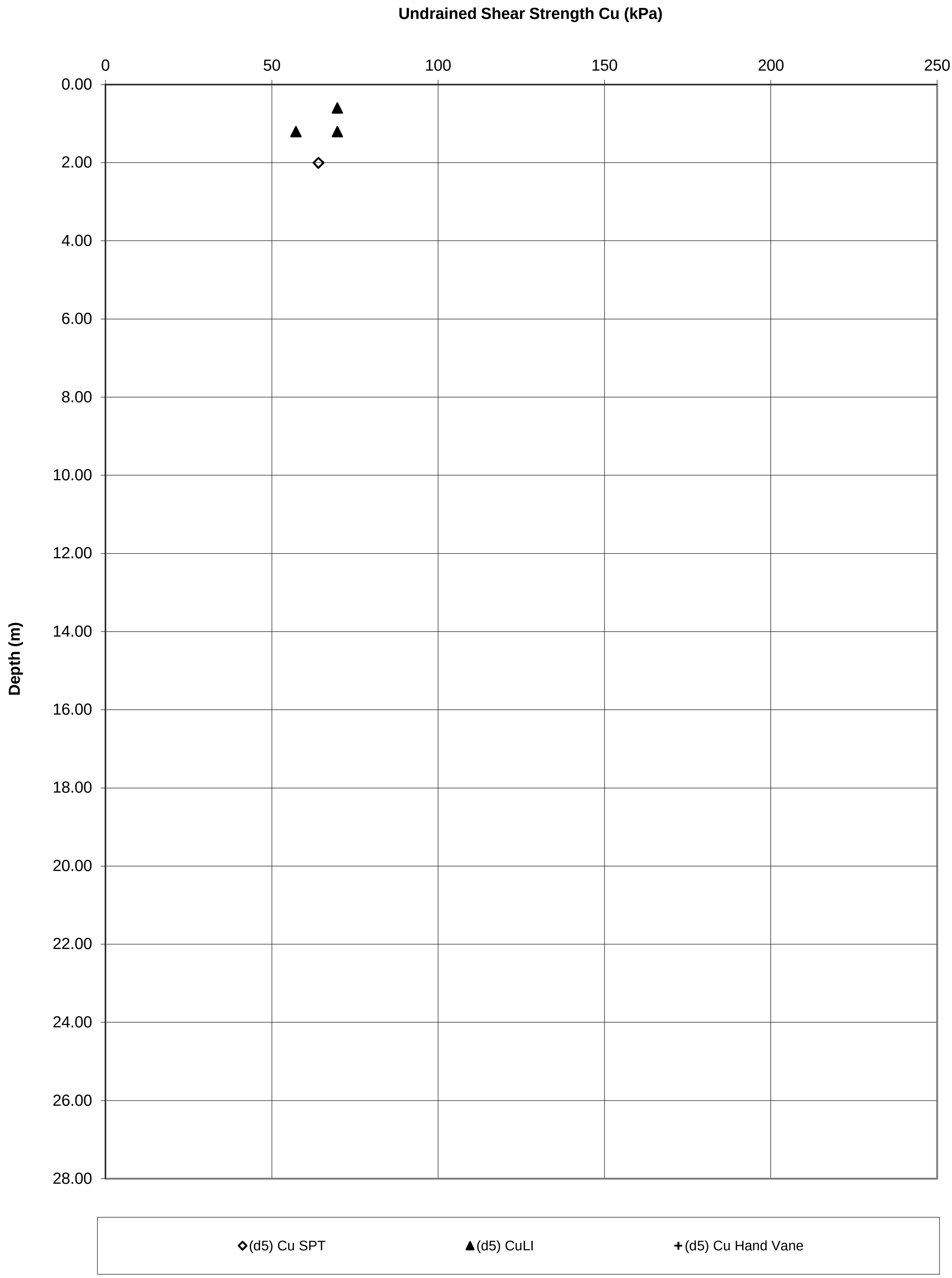
Moisture Content and Atterberg Limits versus Level Cohesive Made Ground d5	60283414_M015_GEO_RT_001	
	Date: April 2018	Figure 10



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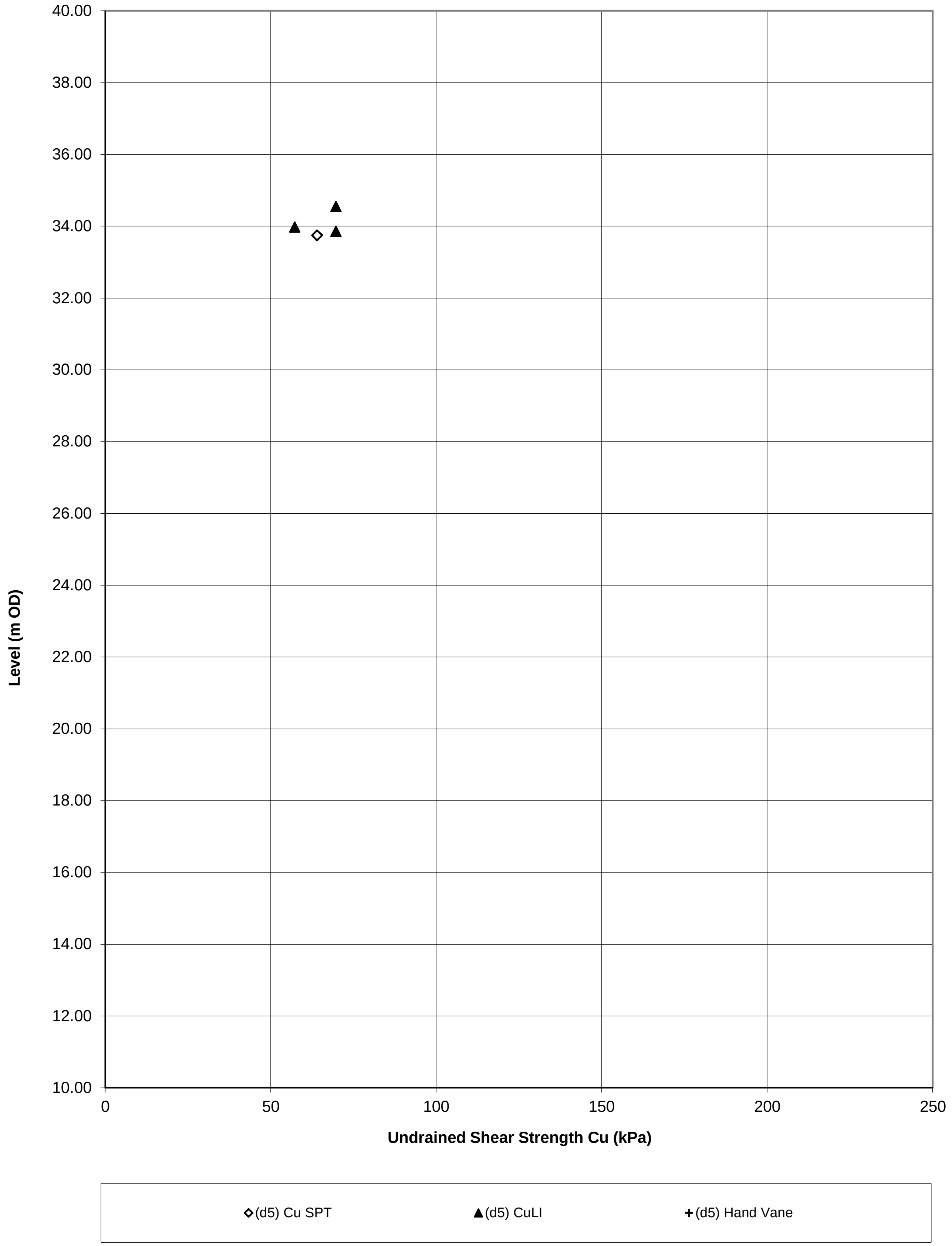
Undrained Shear Strength versus Depth Cohesive Made Ground d5	60283414_M015_GEO_RT_001 Date: April 2018 Figure 11



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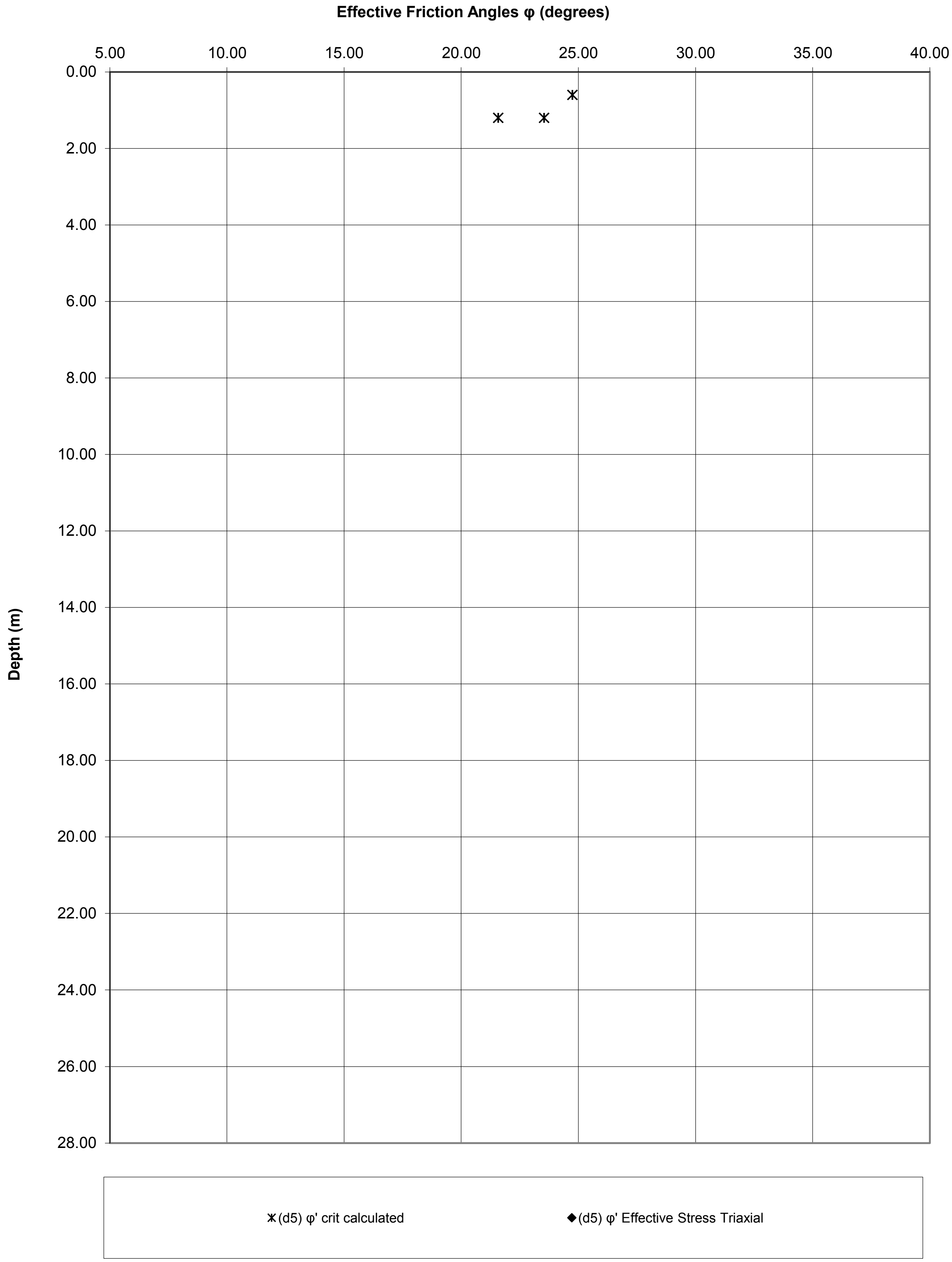
Undrained Shear Strength versus Level Cohesive Made Ground d5	60283414_M015_GEO_RT_001 Date: April 2018 Figure 12



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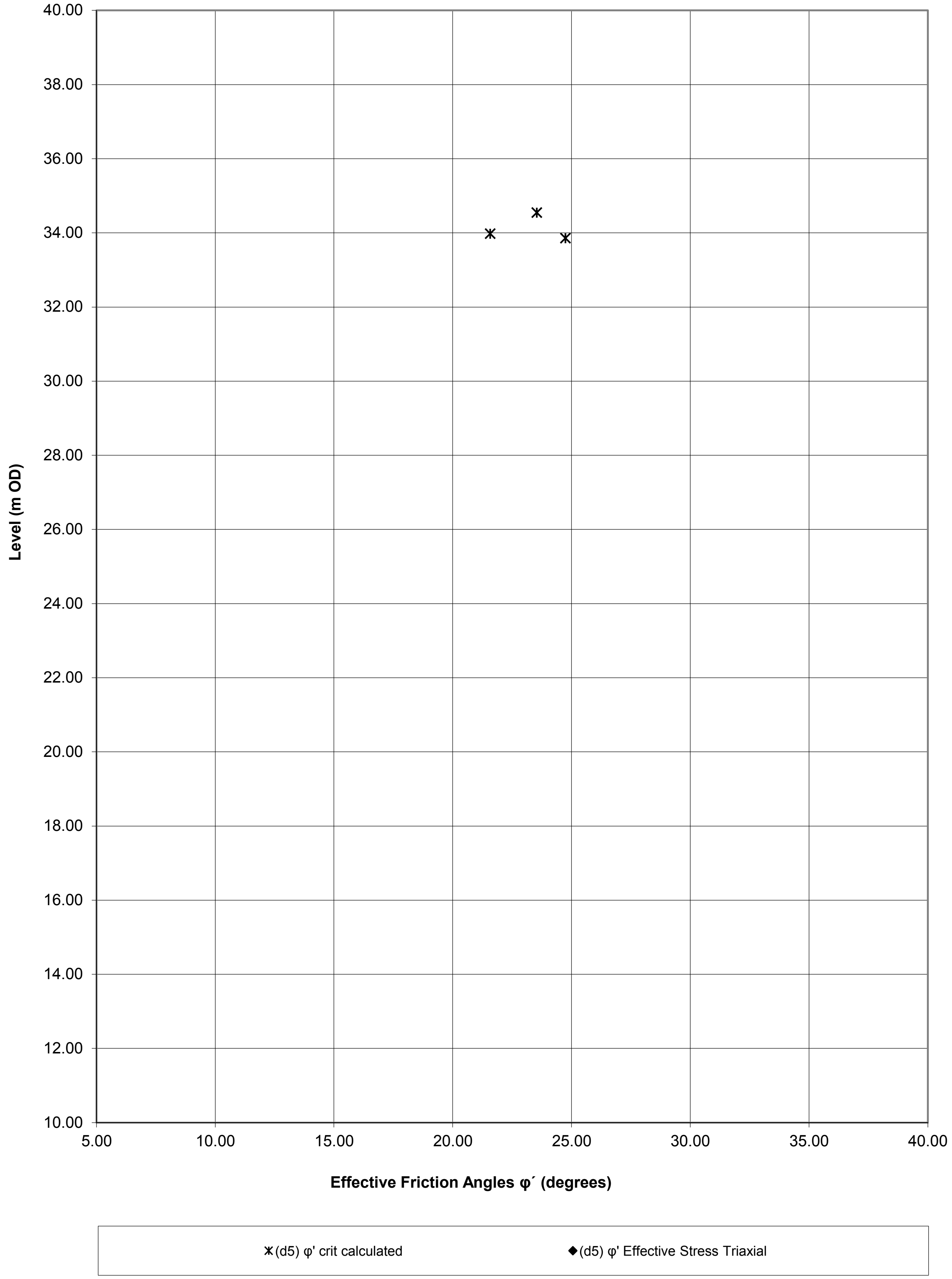
Effective Friction Angle versus Depth Cohesive Made Ground d5	60283414_M015_GEO_RT_001 Date: April 2018 Figure 13



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Effective Friction Angle versus Level
Cohesive Made Ground d5

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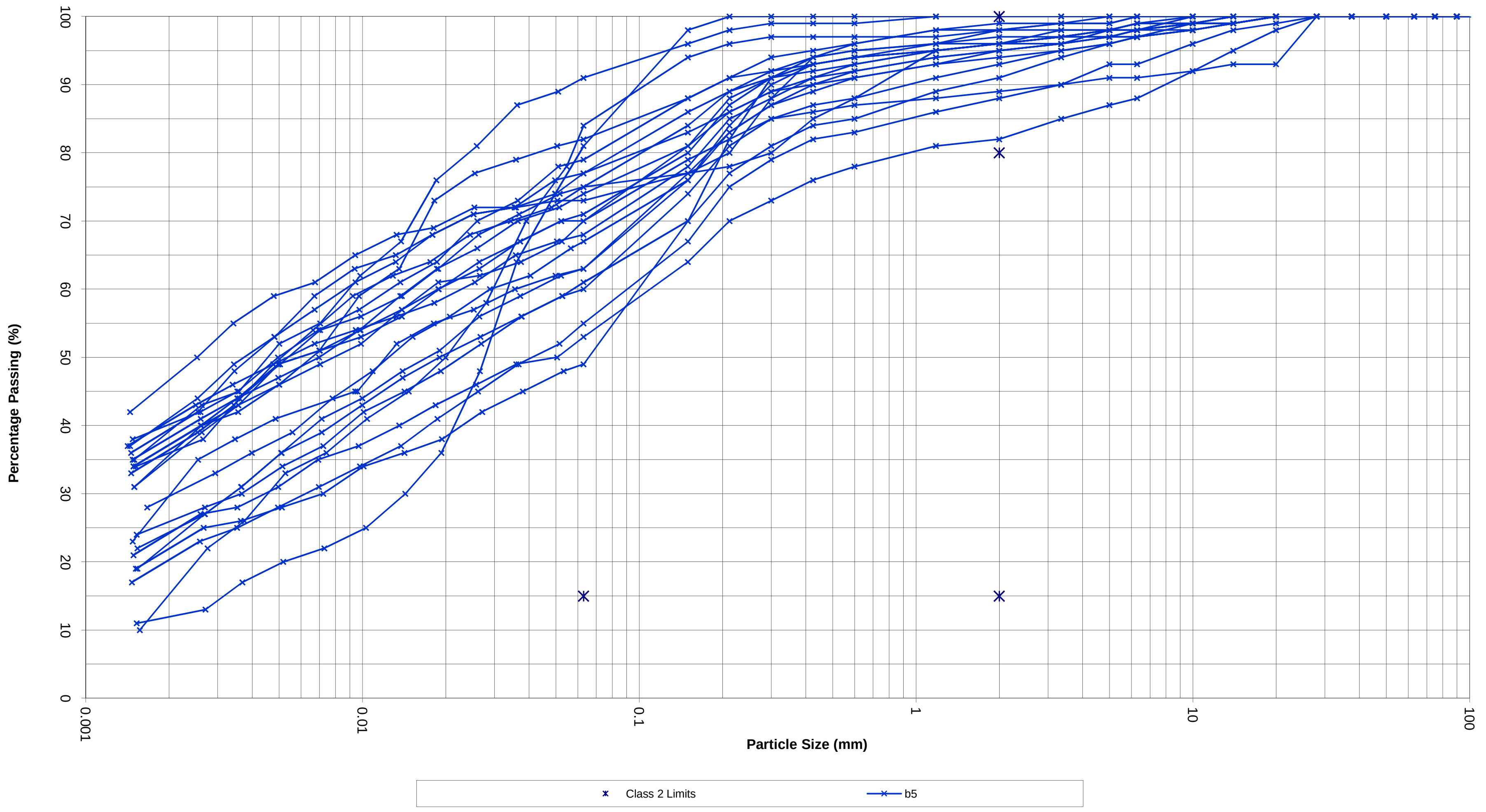
Figure 14



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IAMP Ground Investigation - PHASE ONE
Particle Size Distribution Curve - b5 Pelaw Clay

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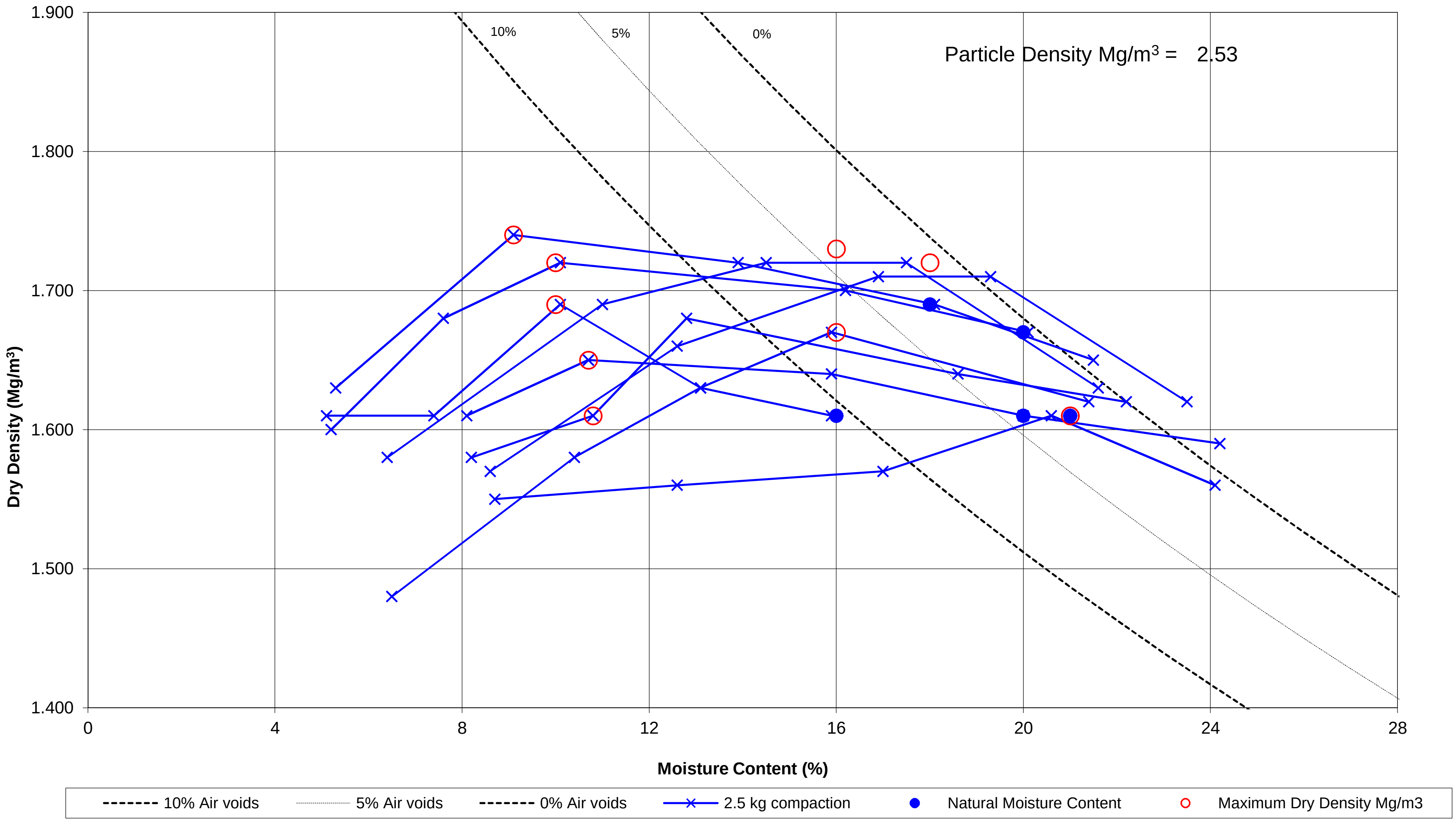
Figure 15



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Compaction Test and Moisture Density Relationship - b5 Pelaw Clay

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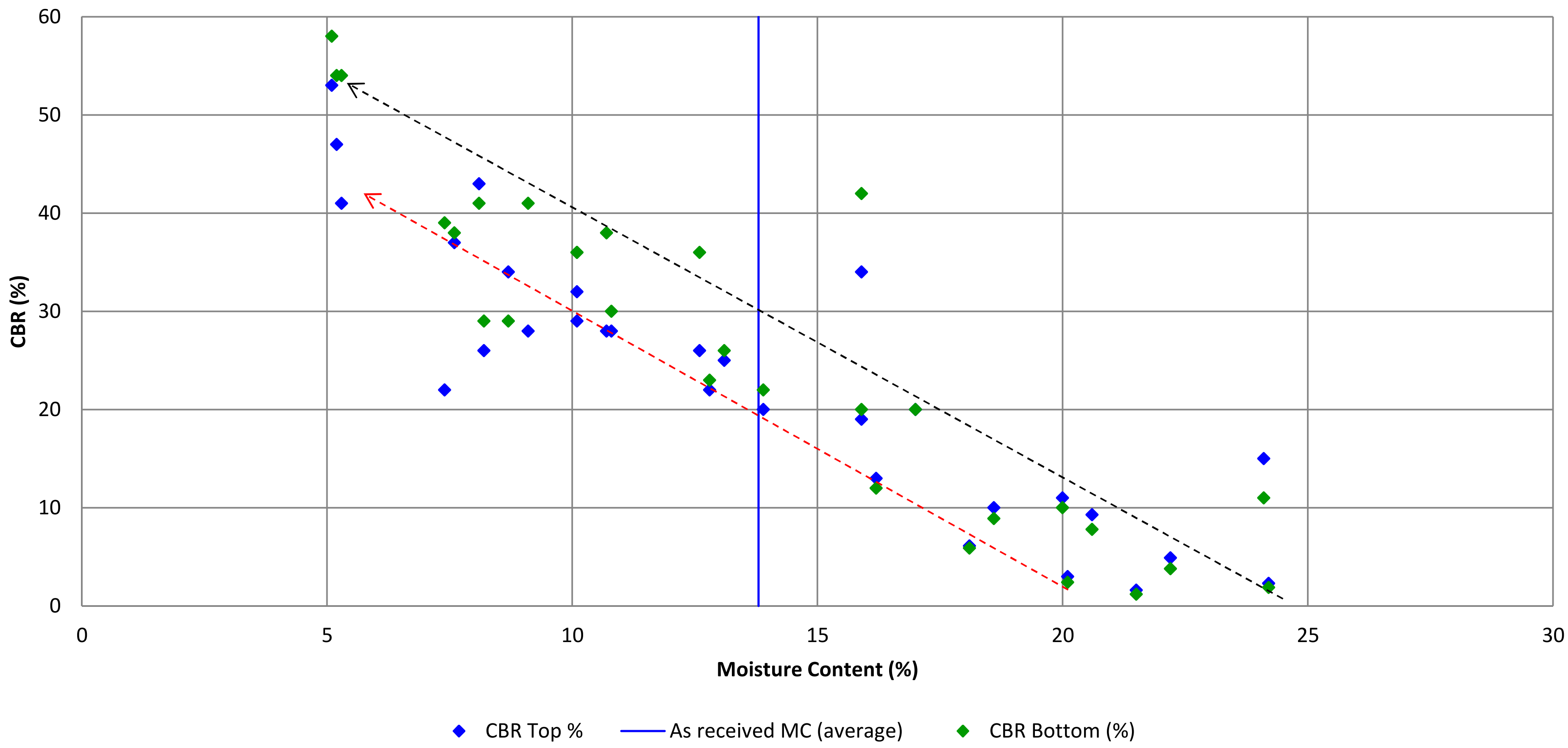
Figure 16



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IAMP Ground Investigation - PHASE ONE
CBR vs Moisture Content - Pelaw Clay (b5)

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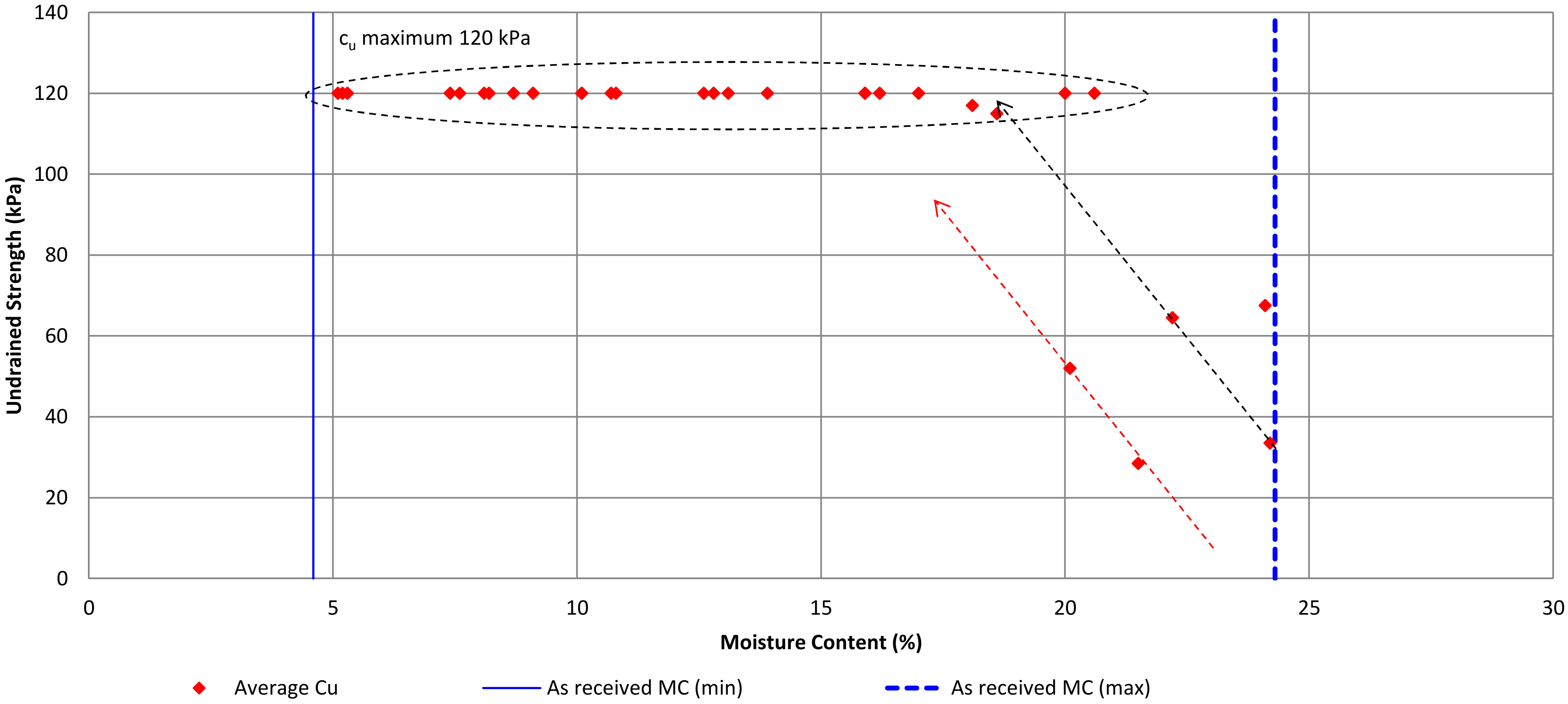
Figure 17



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IAMP Ground Investigation - PHASE ONE
Moisture Content vs Undrained Shear Strength - Pelaw Clay (b5)

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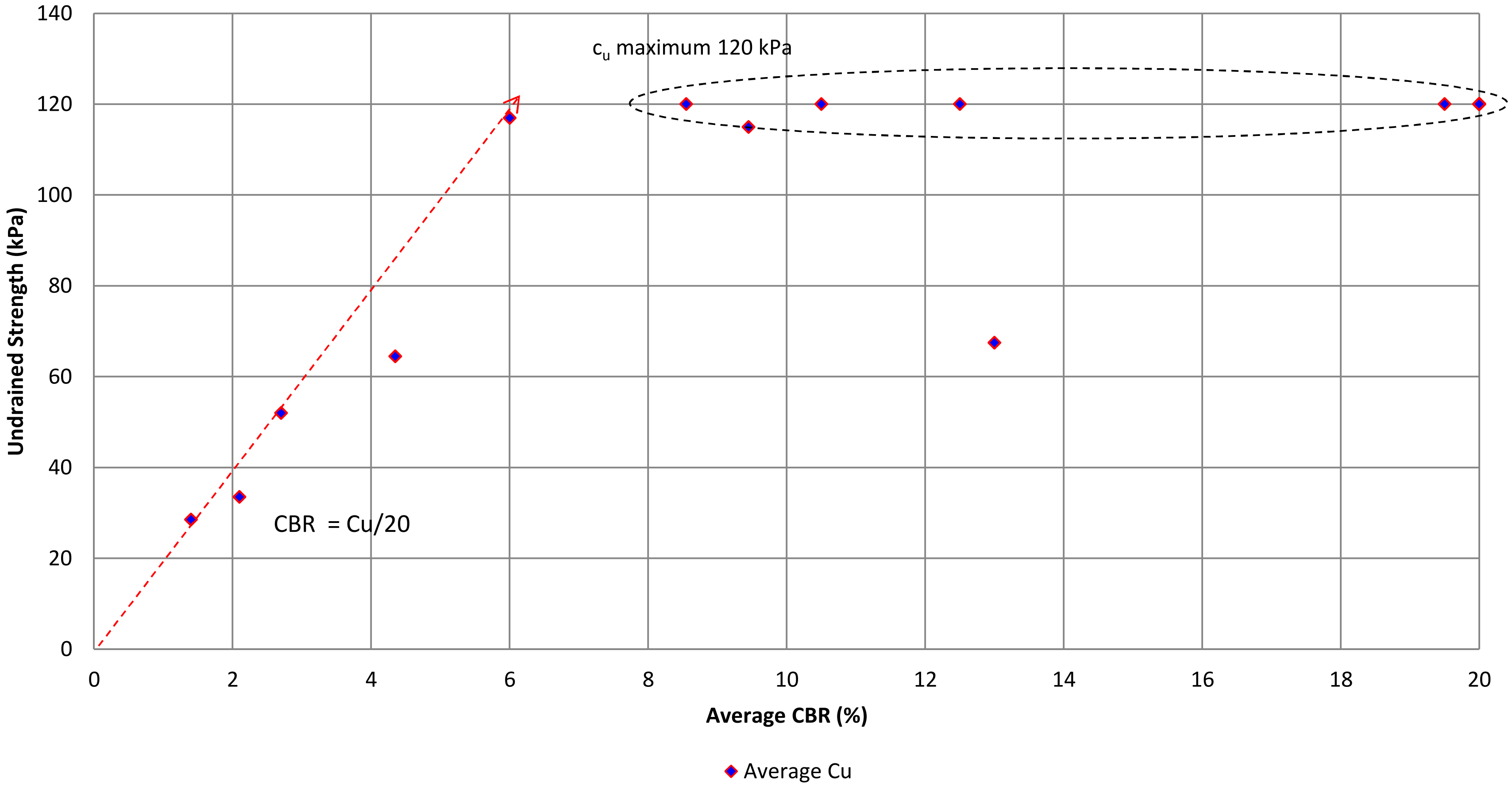
Date: April 2018

Figure 18



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IAMP Ground Investigation - PHASE ONE
CBR vs Undrained Shear Strength - Pelaw Clay (b5)

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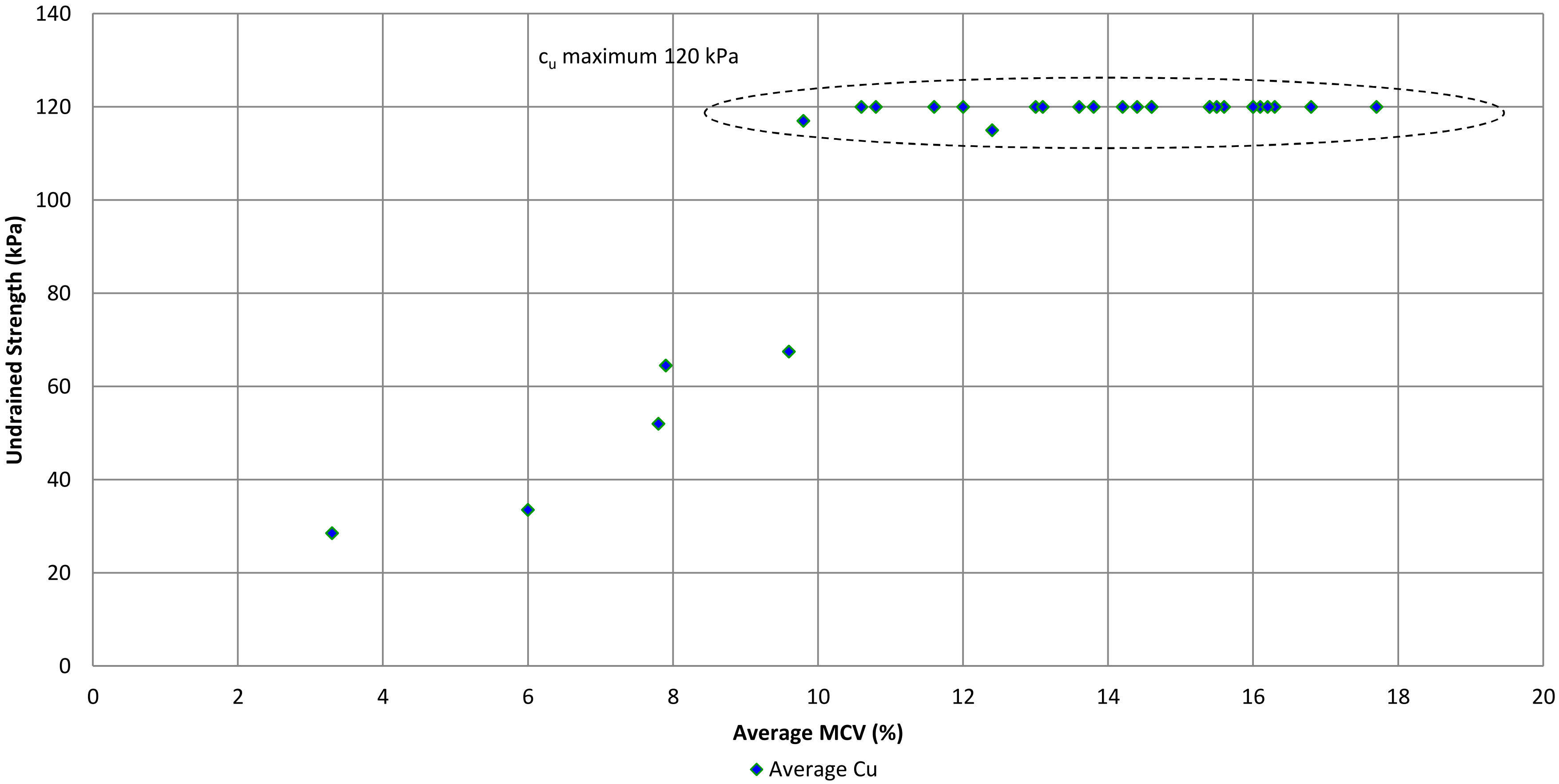
Date: April 2018

Figure 19



IAMP Ground Investigation - PHASE ONE
MCV vs Undrained Shear Strength - Pelaw Clay (b5)

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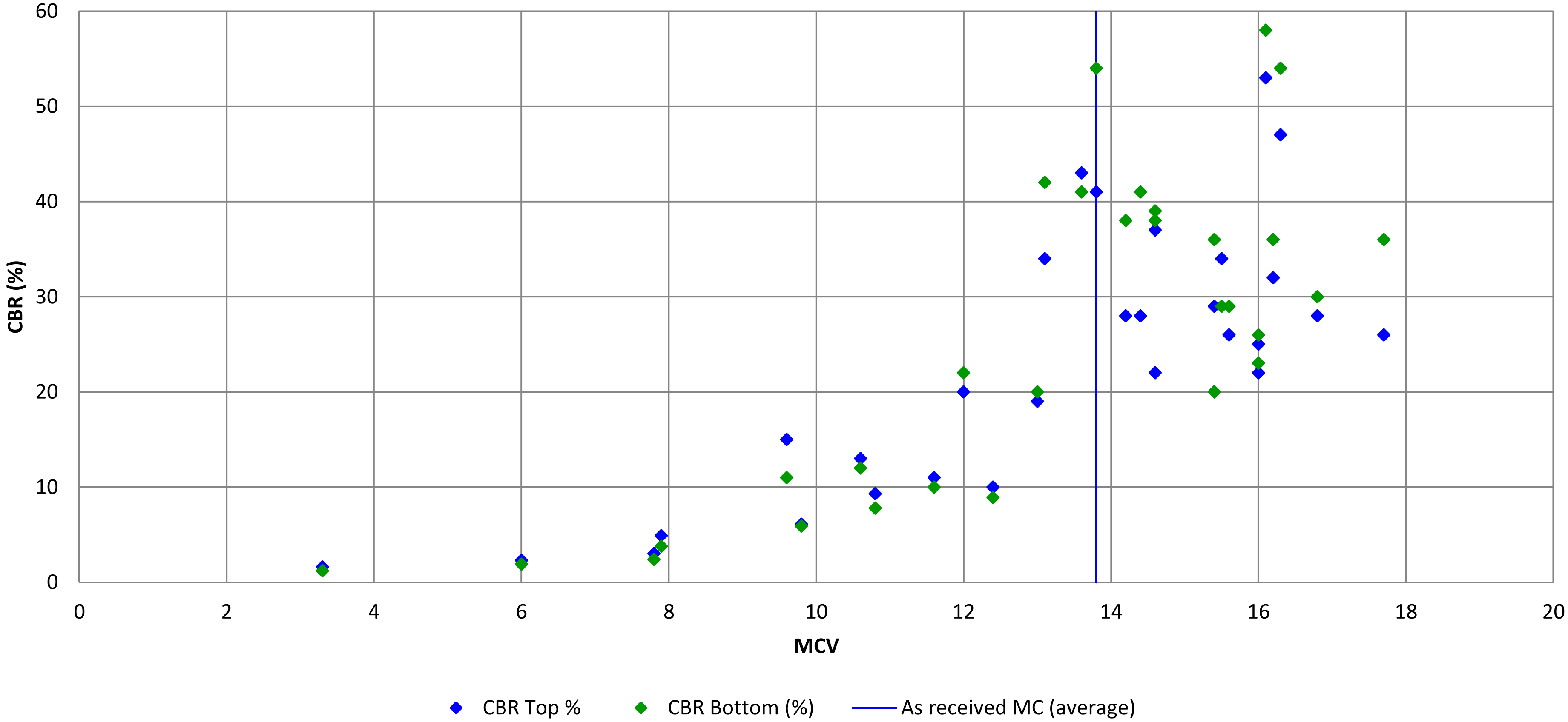
Date: April 2018

Figure 20



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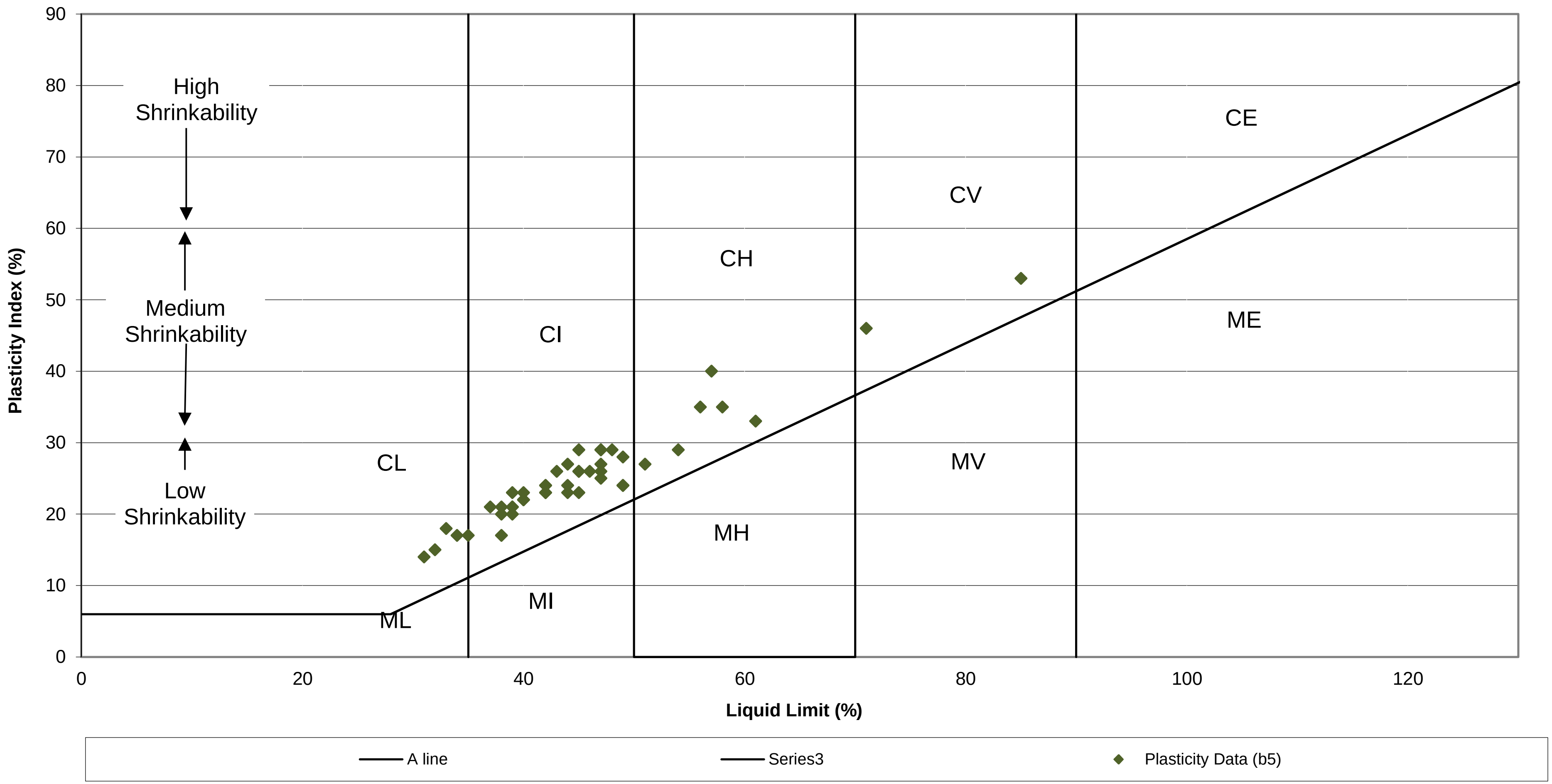
IAMP Ground Investigation - PHASE ONE MCV vs CBR - Pelaw Clay (b5)	60283414_M015_GEO_RT_001 Date: April 2018	Figure 21



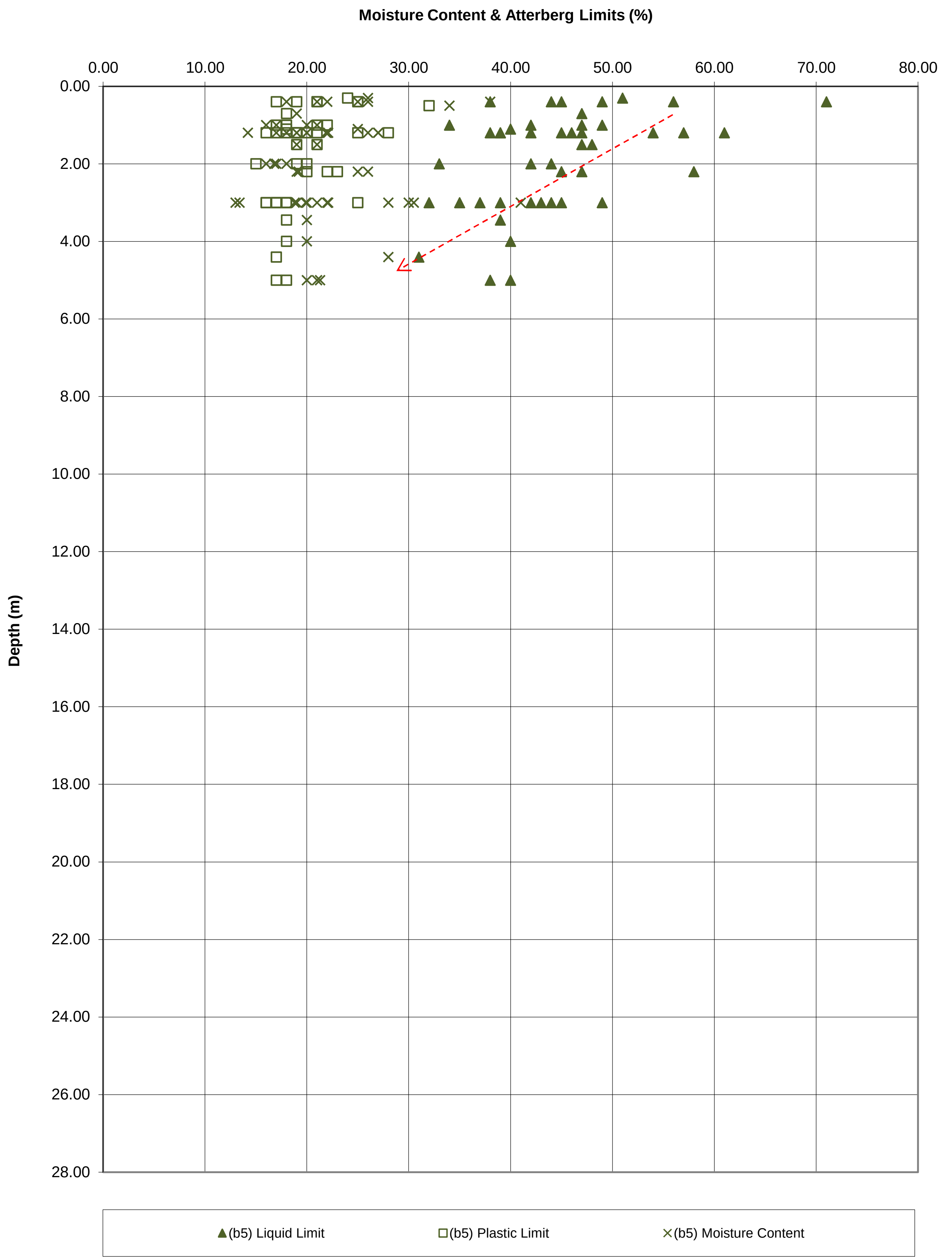
IAMP Ground Investigation - PHASE ONE
Casagrande Plasticity Chart - Pelaw Clay (b5)

Sunderland City Council

AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001
Date: April 2018

Figure 22



IAMP Ground Investigation - PHASE ONE

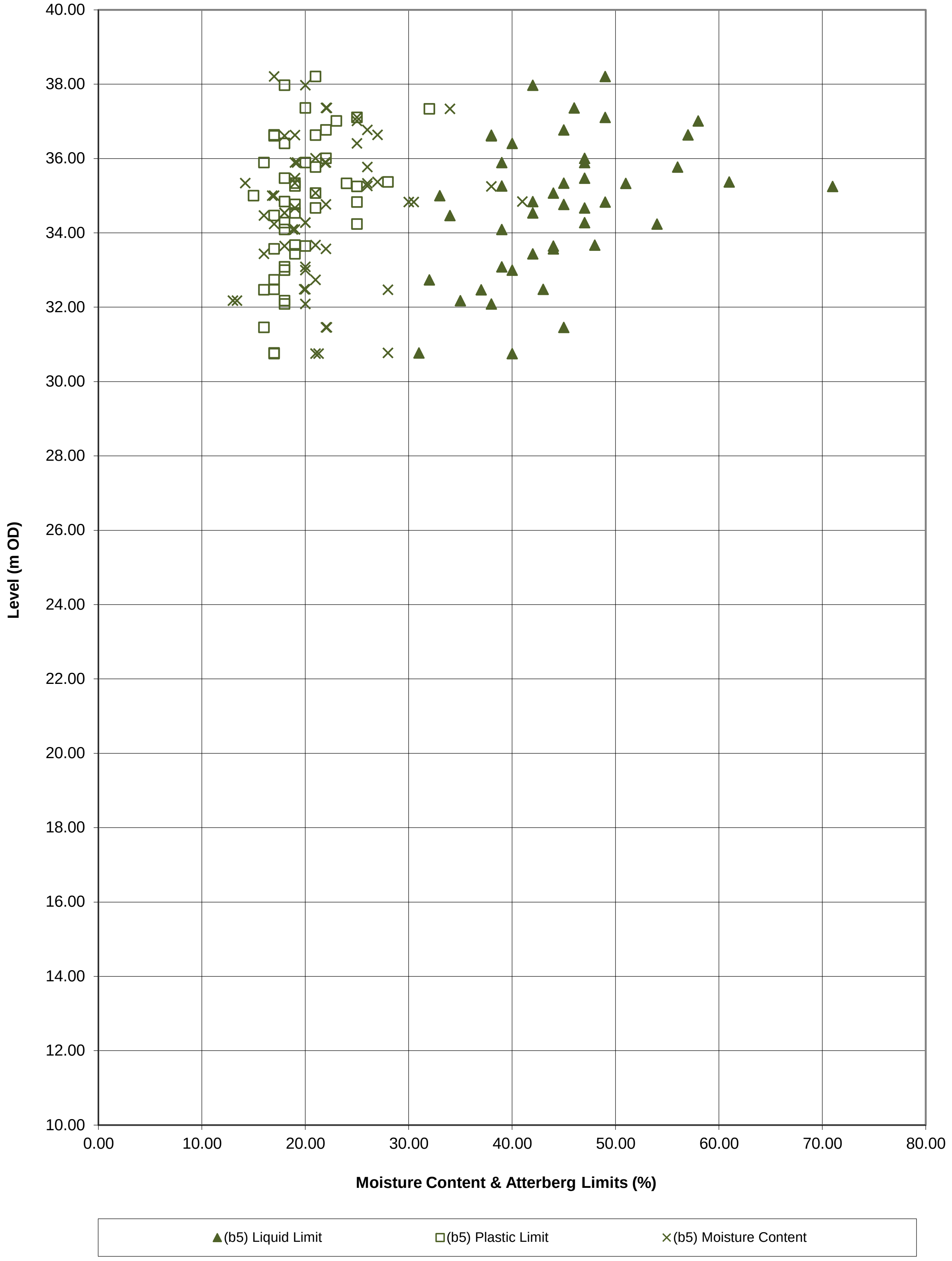
Sunderland City Council | AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

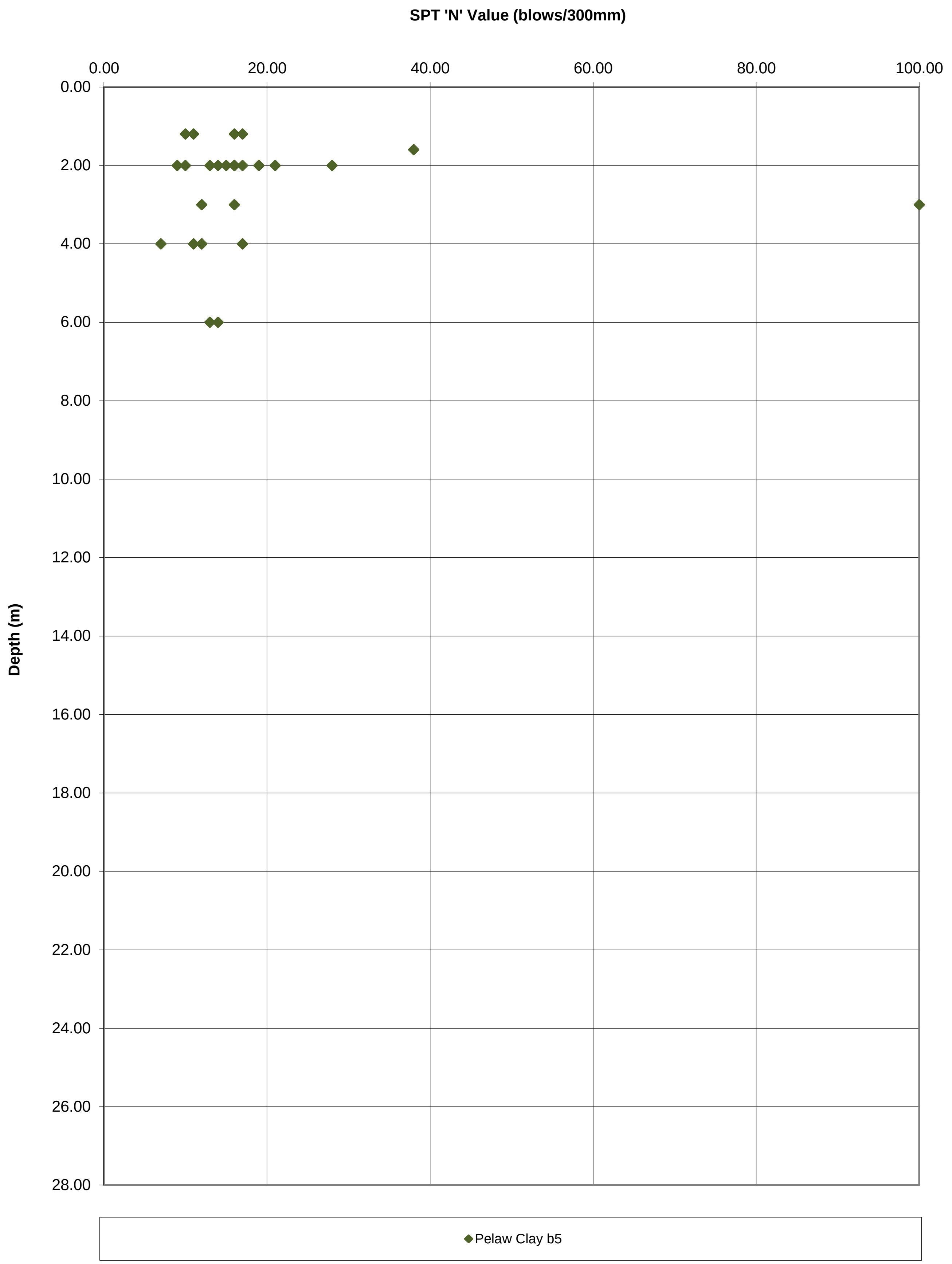
Moisture Content and Atterberg Limits versus Depth
Pelaw Clay (b5)

Date: April 2018

Figure 23



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Moisture Content and Atterberg Limits versus Level Pelaw Clay (b5)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 24



IAMP Ground Investigation - PHASE ONE

Sunderland City Council

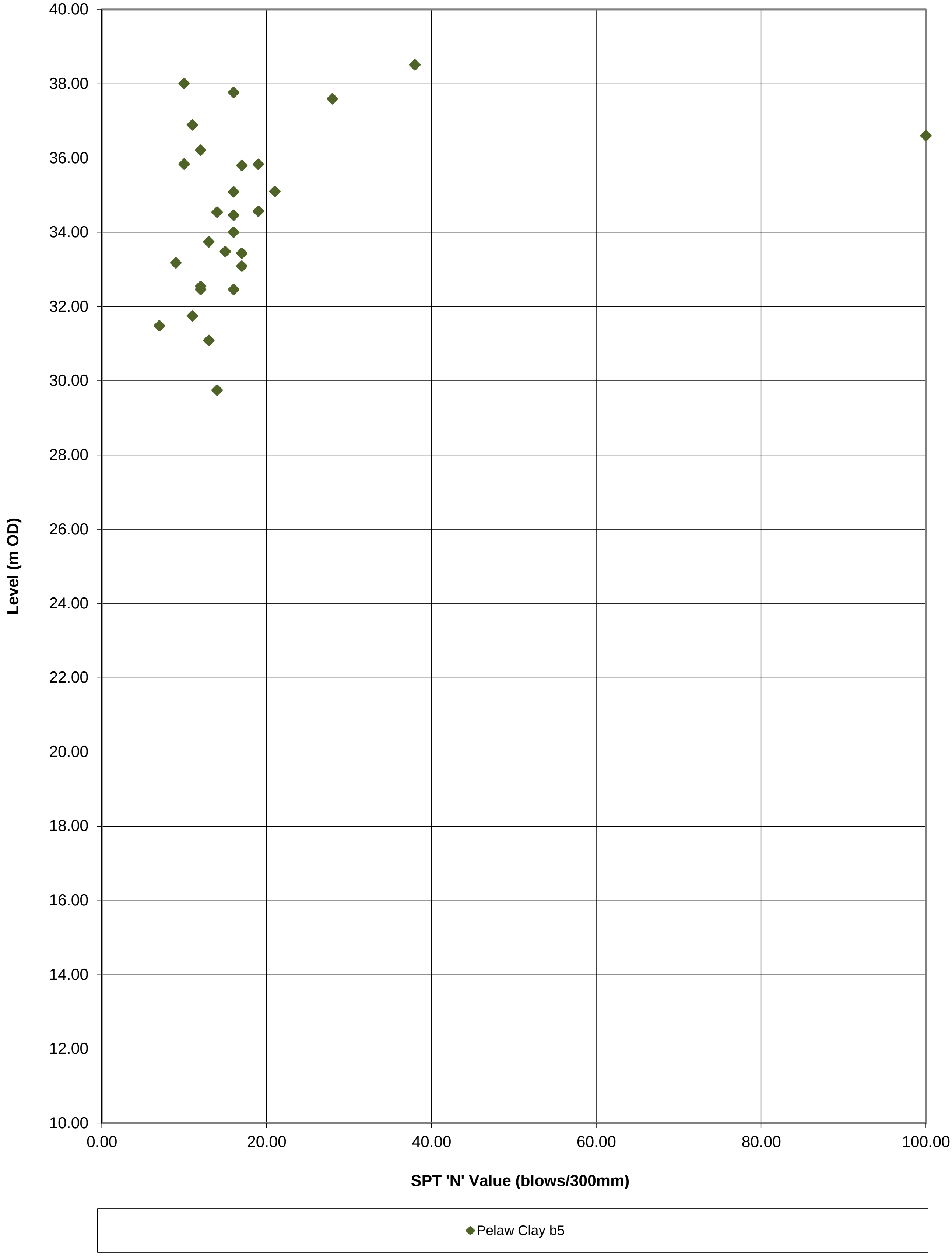
AECOM Imagine it. Delivered.

SPT 'N' Value versus Depth
Pelaw Clay (b5)

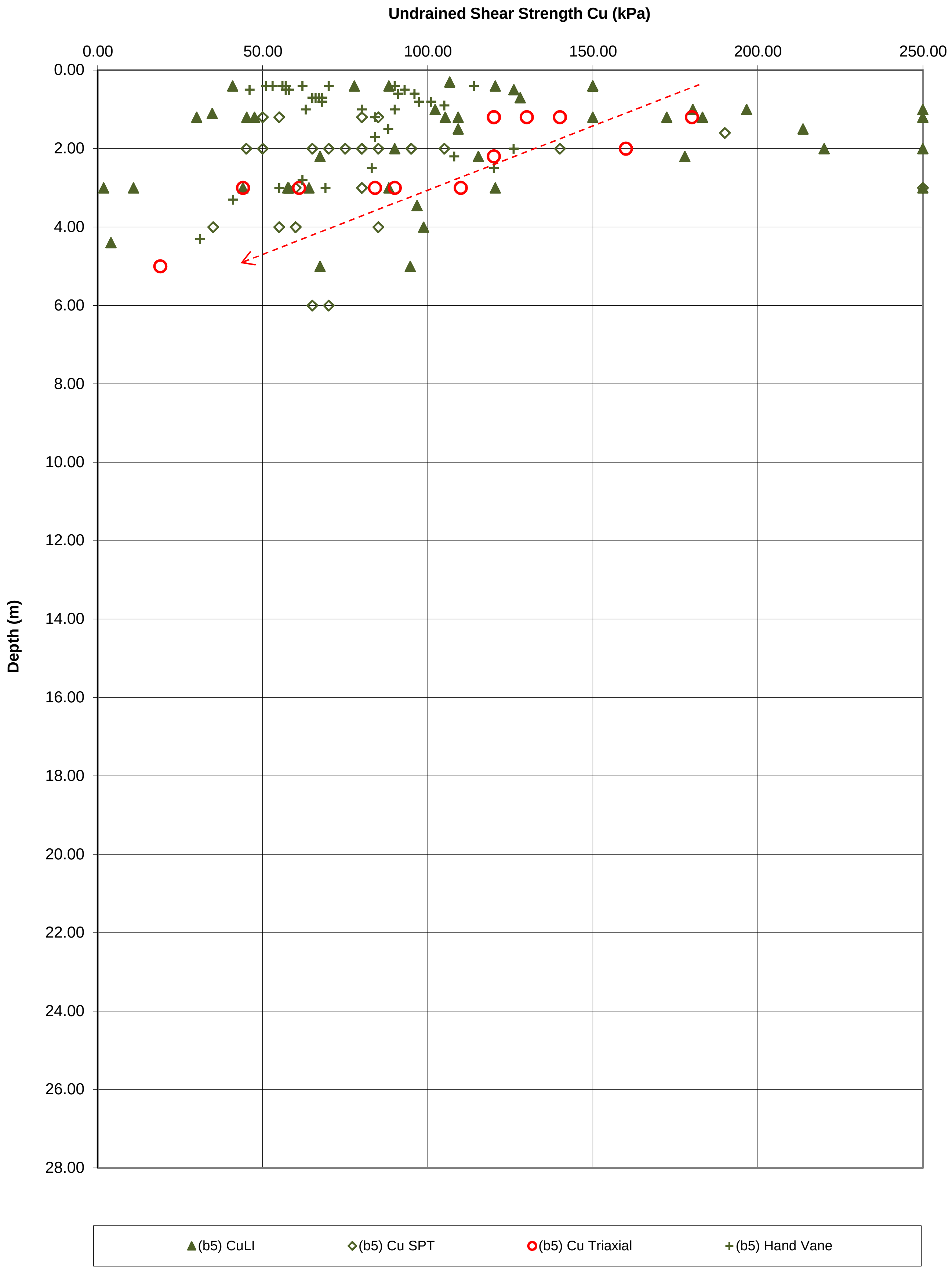
60283414_M015_GEO_RT_001

Date: April 2018

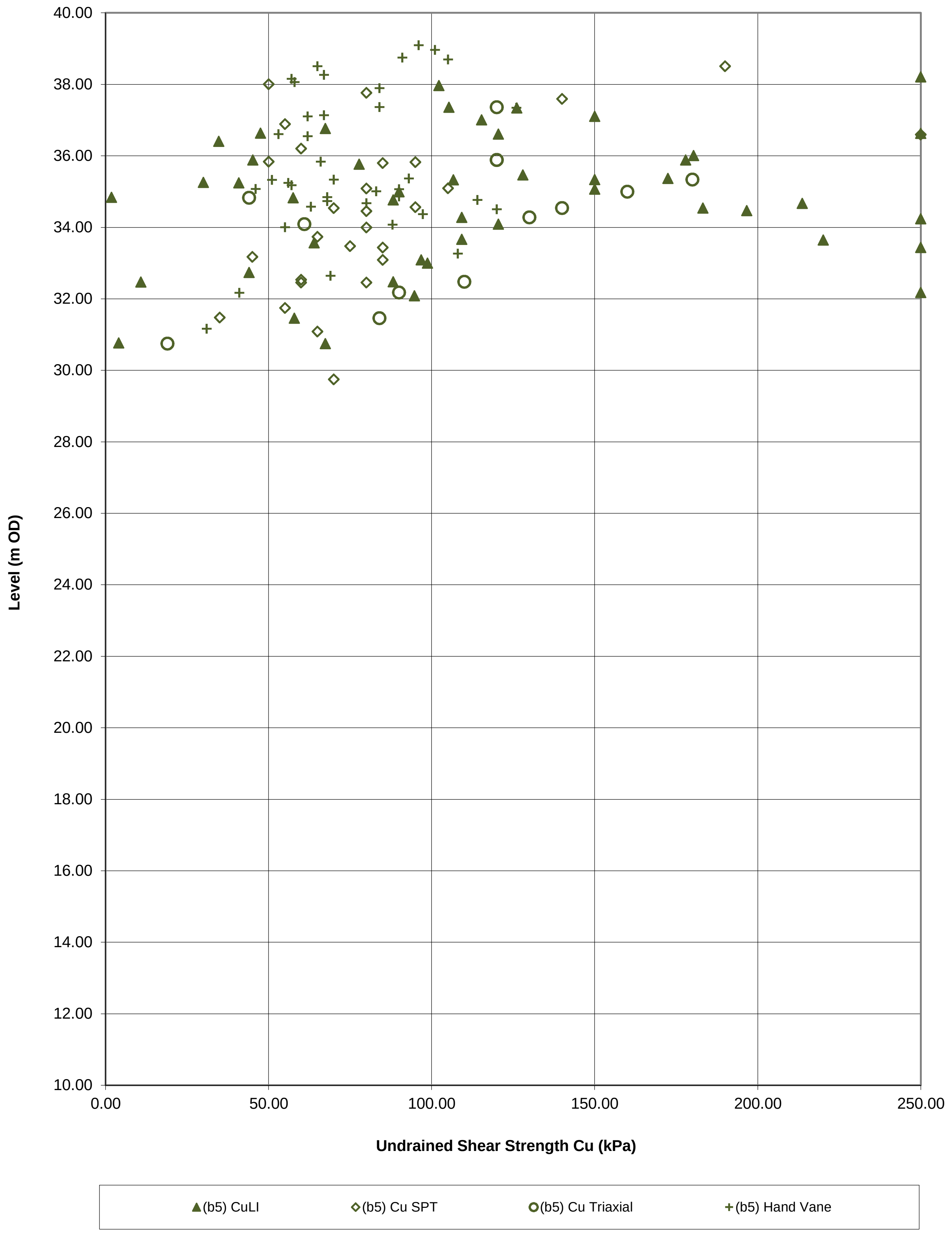
Figure 25



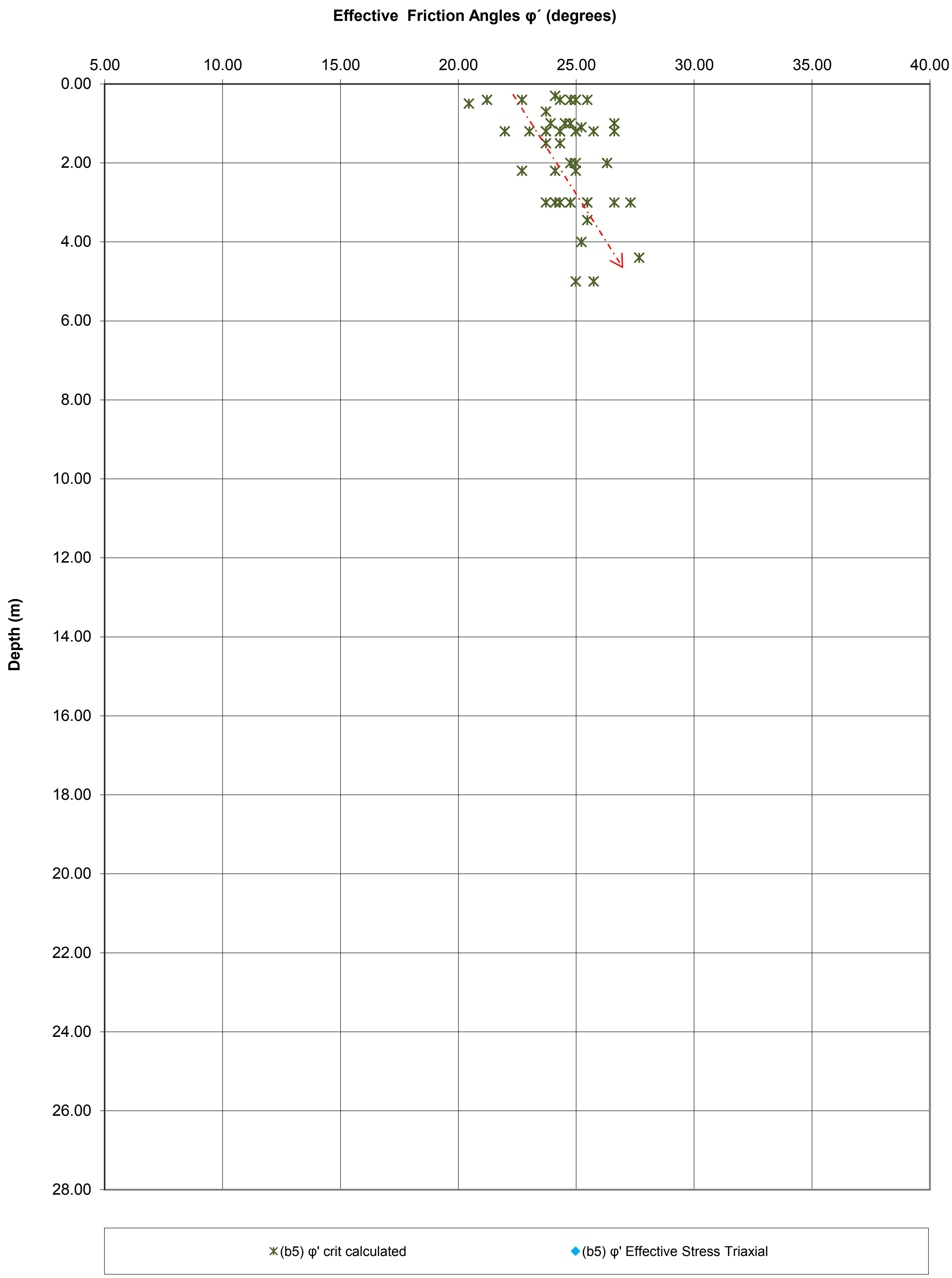
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
SPT 'N' Value versus Level Pelaw Clay (b5)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 26



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Undrained Shear Strength versus Depth Pelaw Clay (b5)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 27



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Undrained Shear Strength versus Level Pelaw Clay (b5)	60283414_M015_GEO_RT_001 Date: April 2018 — Figure 28



IAMP Ground Investigation - PHASE ONE

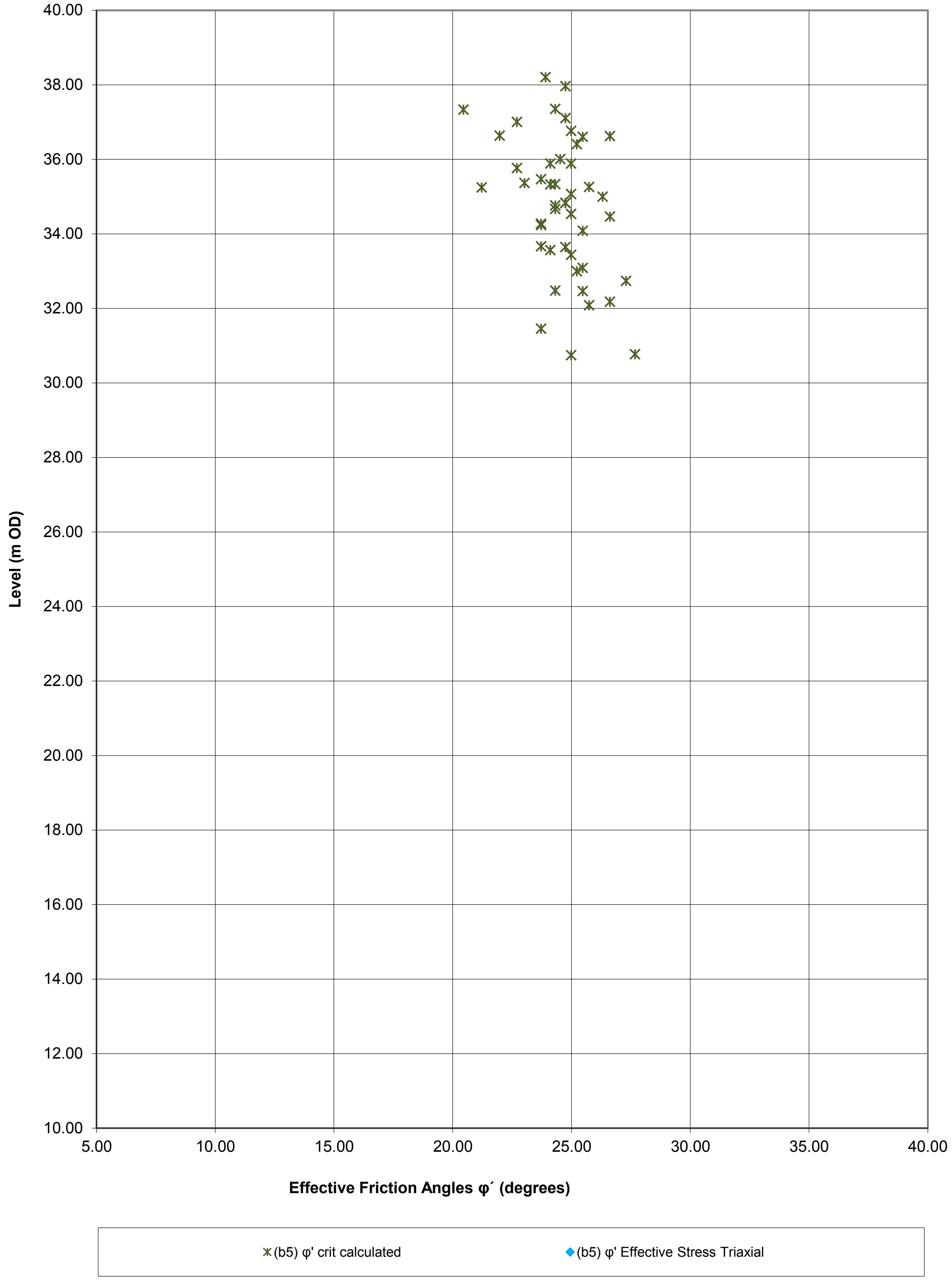
Sunderland City Council | AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

Effective Friction Angle versus Depth
Pelaw Clay (b5)

Date: April 2018

Figure 29



IAMP Ground Investigation - PHASE ONE

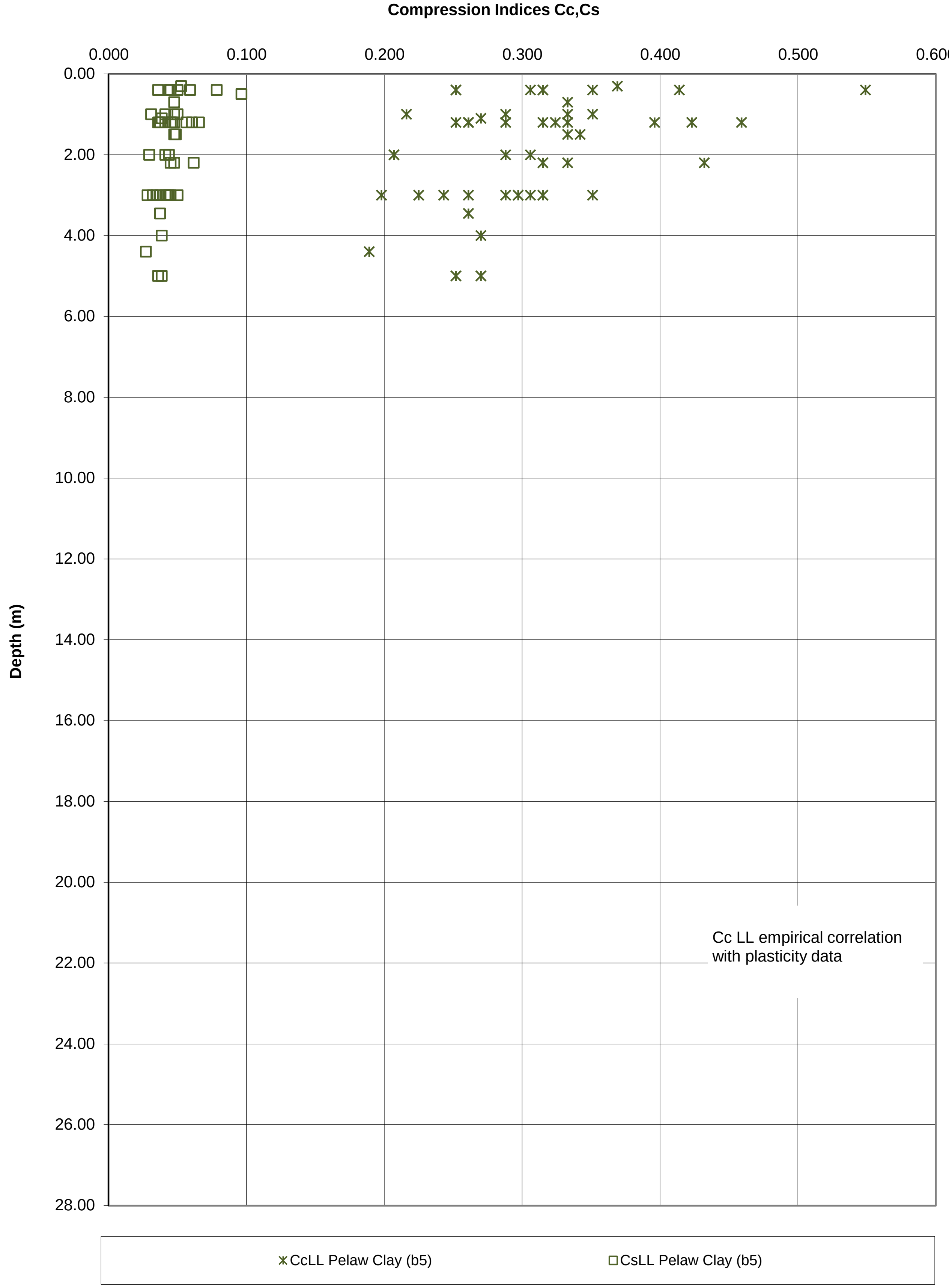
Sunderland City Council | AECOM Imagine it. Delivered.

Effective Friction Angle versus Level
Pelaw Clay (b5)

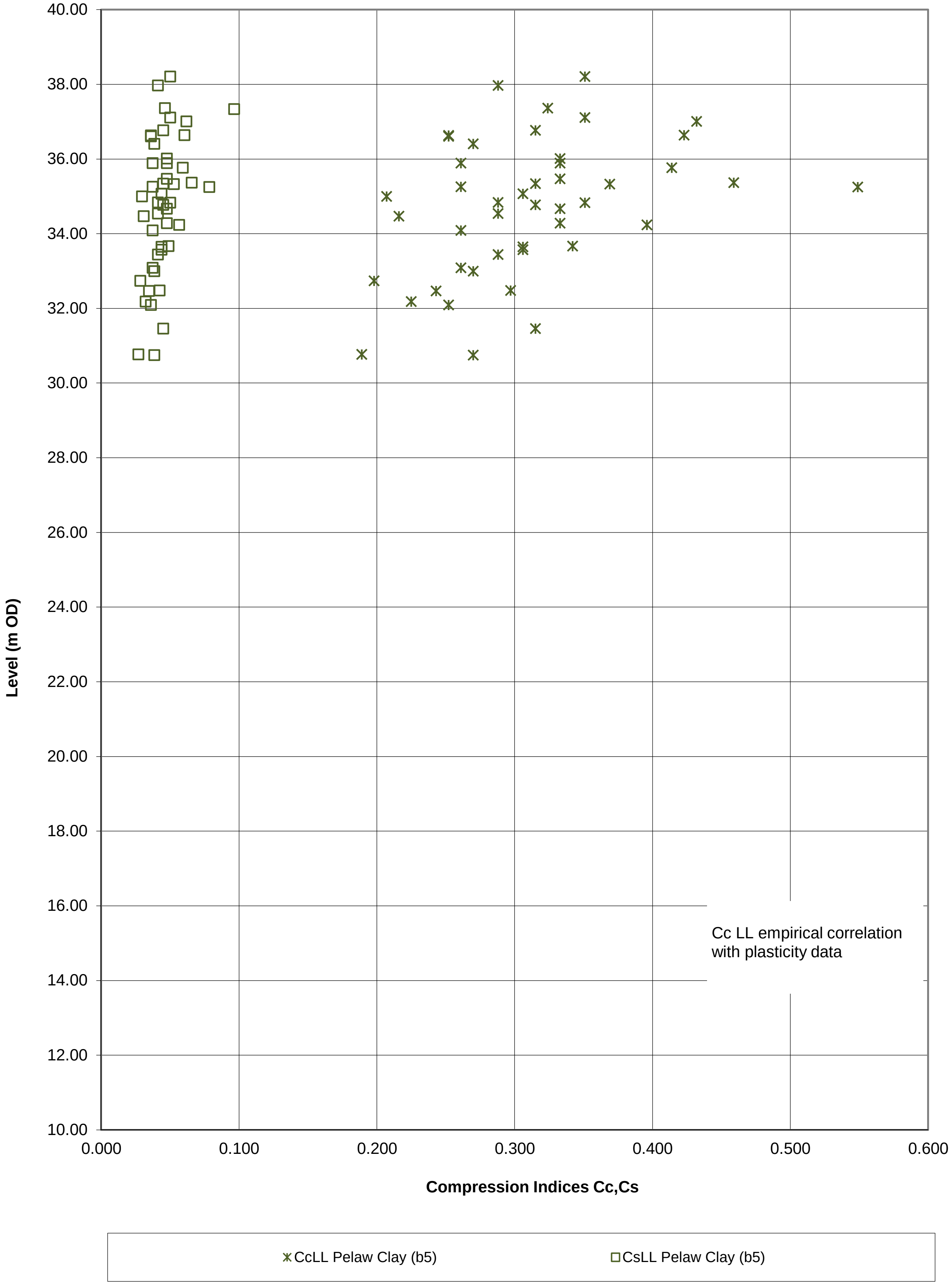
60283414_M015_GEO_RT_001

Date: April 2018

Figure 30



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.	
Compression Indices versus Depth Pelaw Clay (b5)	60283414_M015_GEO_RT_001	
	Date: April 2018	Figure 31



IAMP Ground Investigation - PHASE ONE

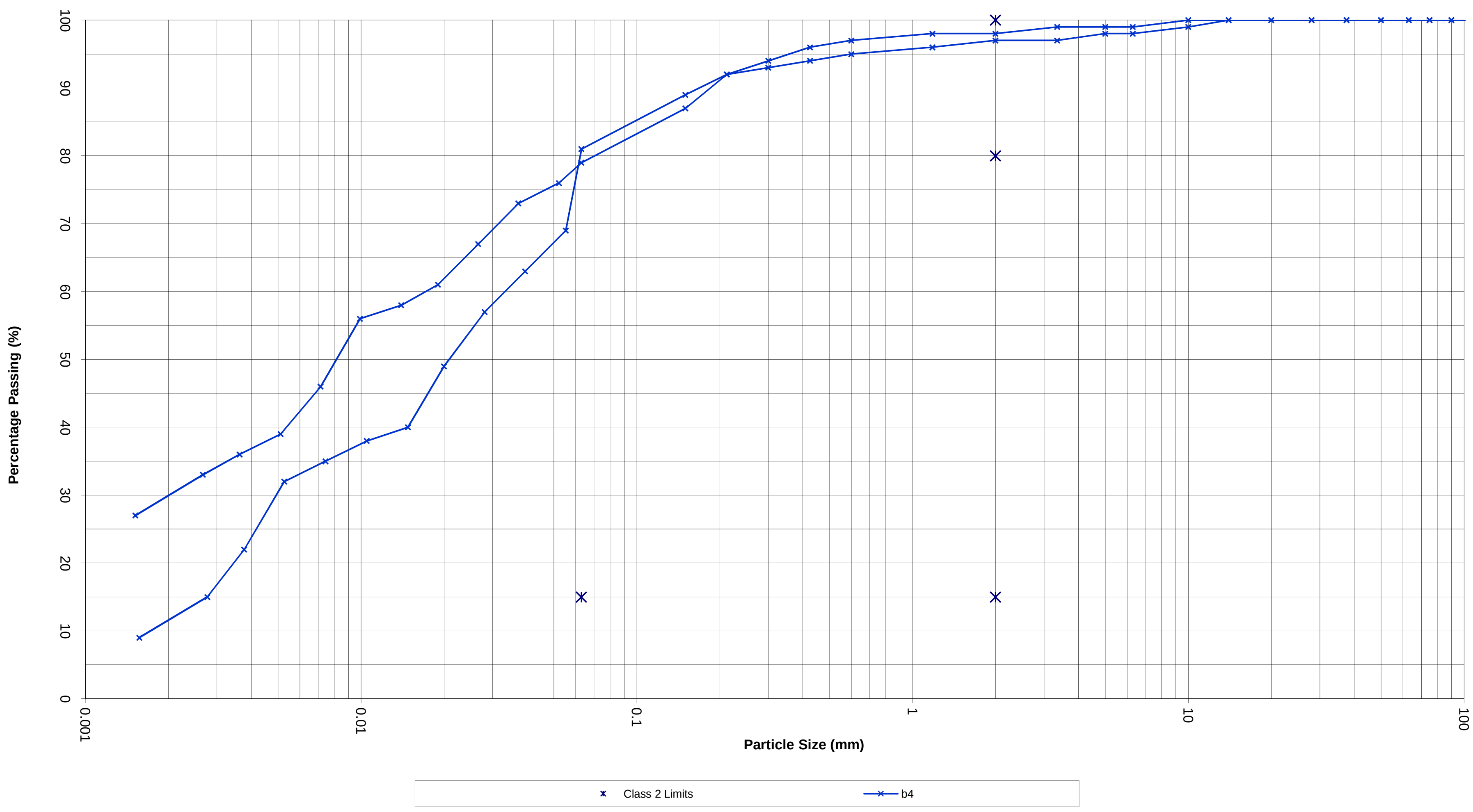
Sunderland City Council | AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

Compression Indices versus Level
Pelaw Clay (b5)

Date: April 2018

Figure 32



Sunderland City Council

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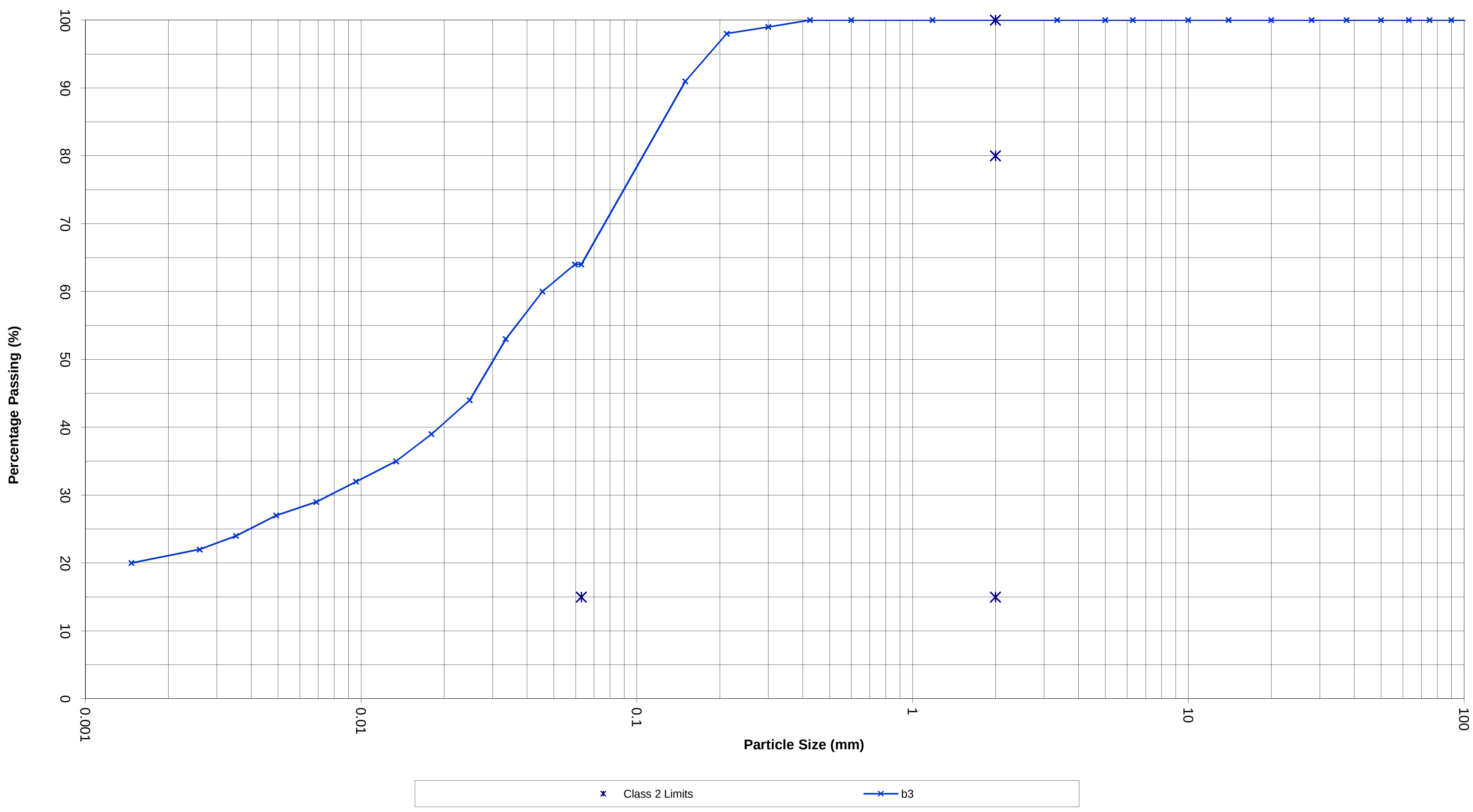
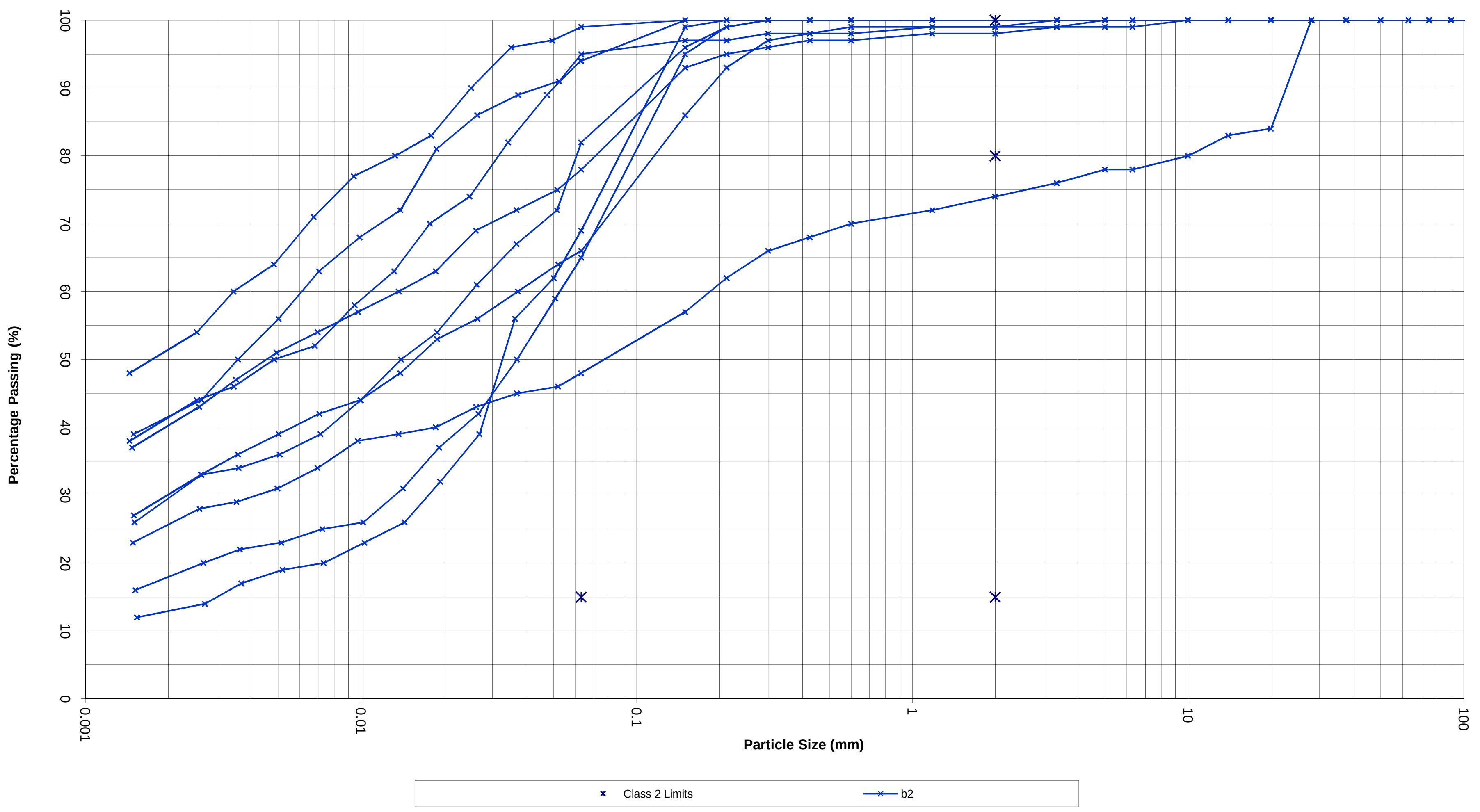
IAMP Ground Investigation - PHASE ONE
Particle Size Distribution Curve - b4 Silt

60283414_M015_GEO_RT_001
Date: April 2018

Figure 33



IAMP Ground Investigation - PHASE ONE Particle Size Distribution Curve - b3 Glacial Sand	60283414_M015_GEO_RT_001 Date: April 2018	Figure 34



Sunderland City Council

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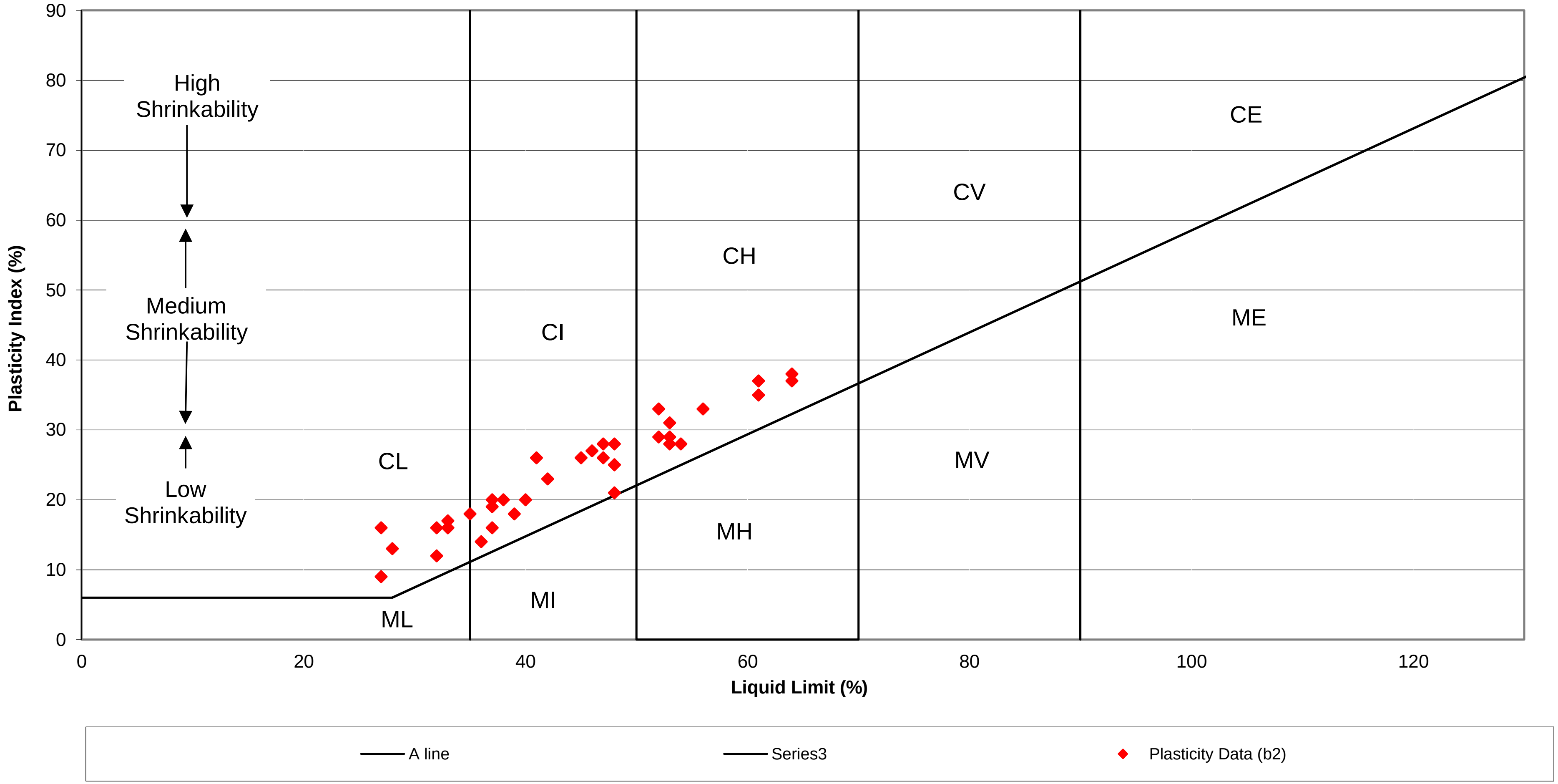
IAMP Ground Investigation - PHASE ONE
Particle Size Distribution Curve - b2 Laminated Clay

60283414_M015_GEO_RT_001

Date: April 2018

Figure 35



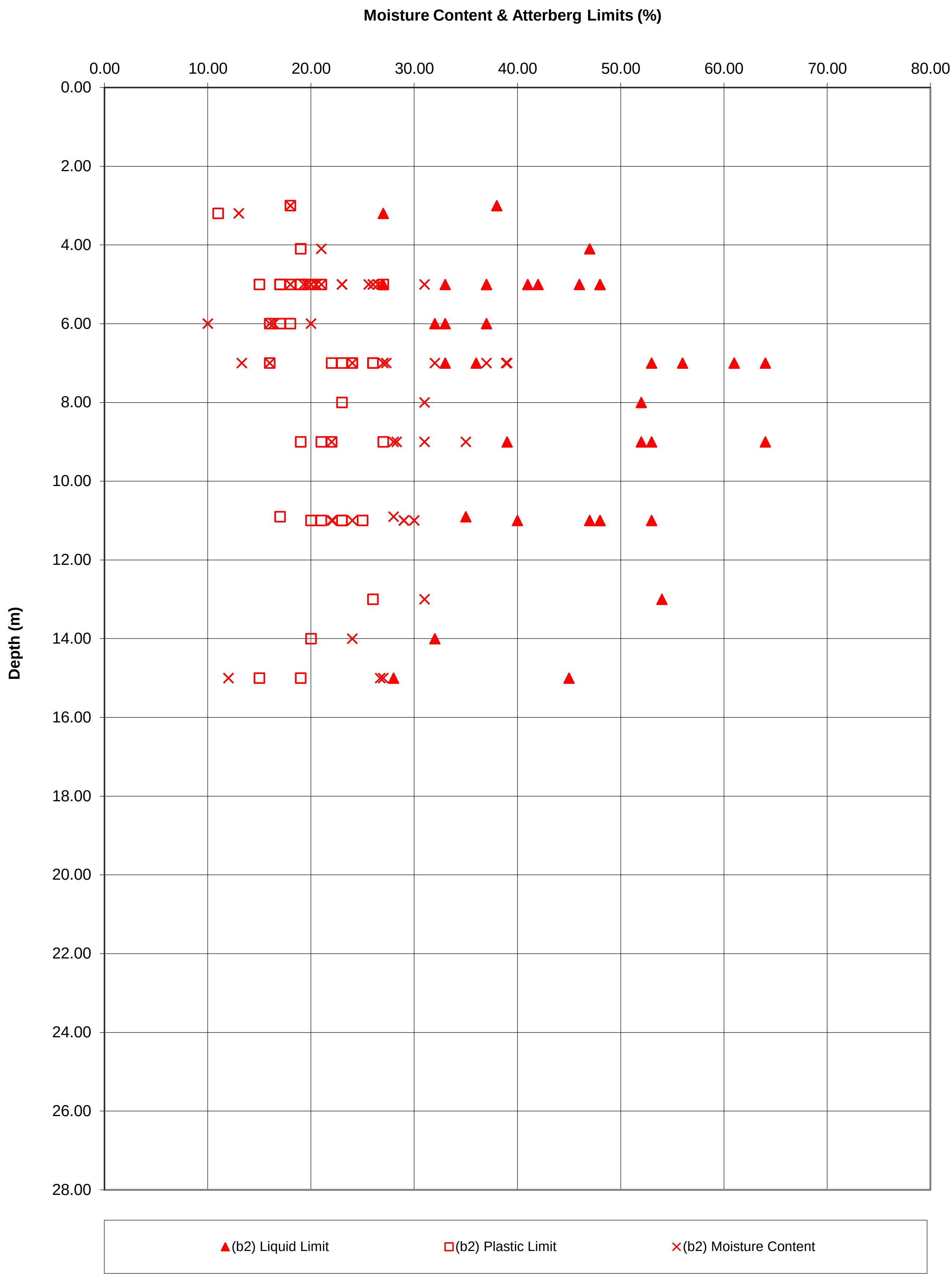
Sunderland City Council

AECOM Imagine it. Delivered.

IAMP Ground Investigation - PHASE ONE
Casagrande Plasticity Chart - Laminated Clay (b2)

60283414_M015_GEO_RT_001
Date: April 2018

Figure 36



IAMP Ground Investigation - PHASE ONE

Sunderland City Council

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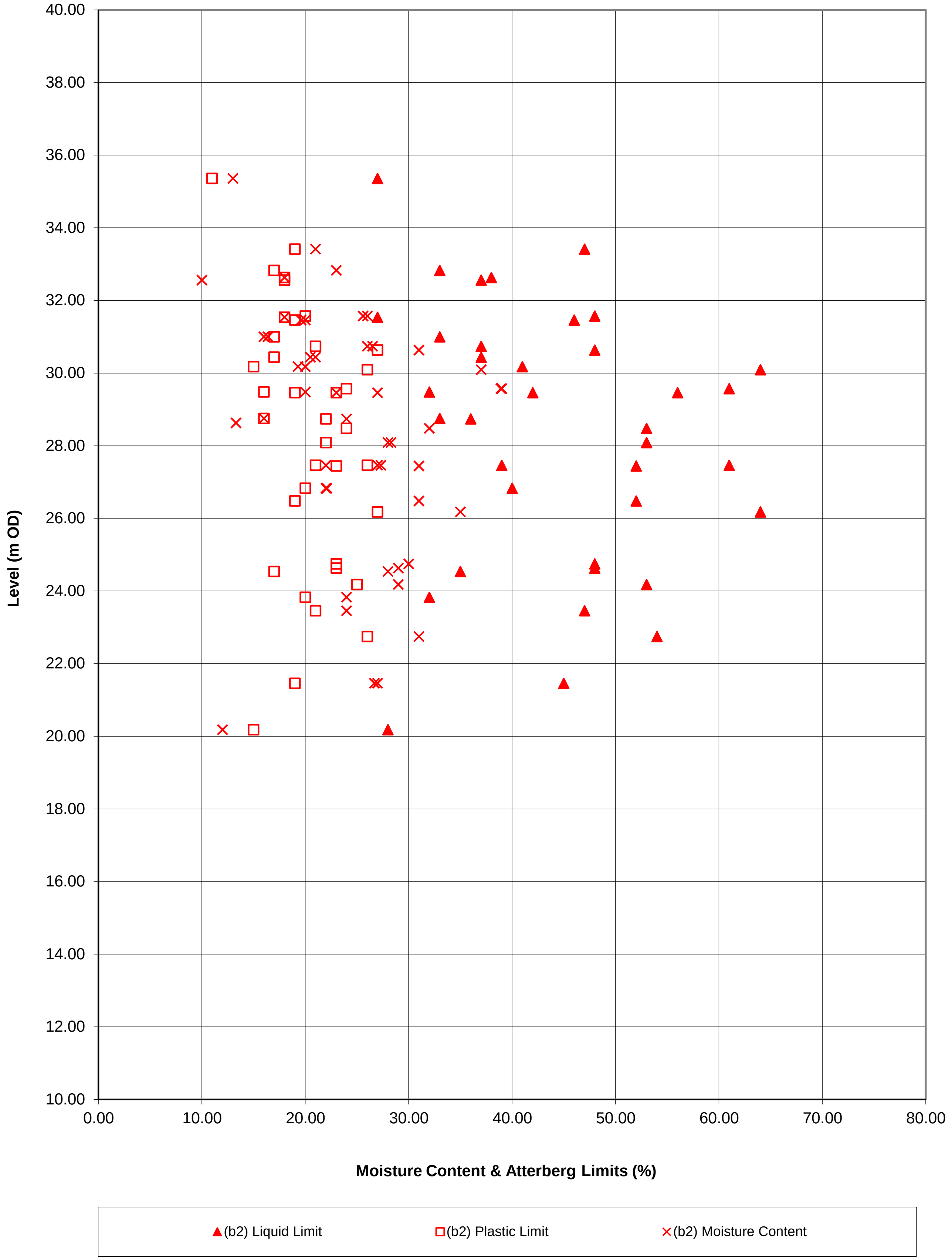
60283414_M015_GEO_RT_001

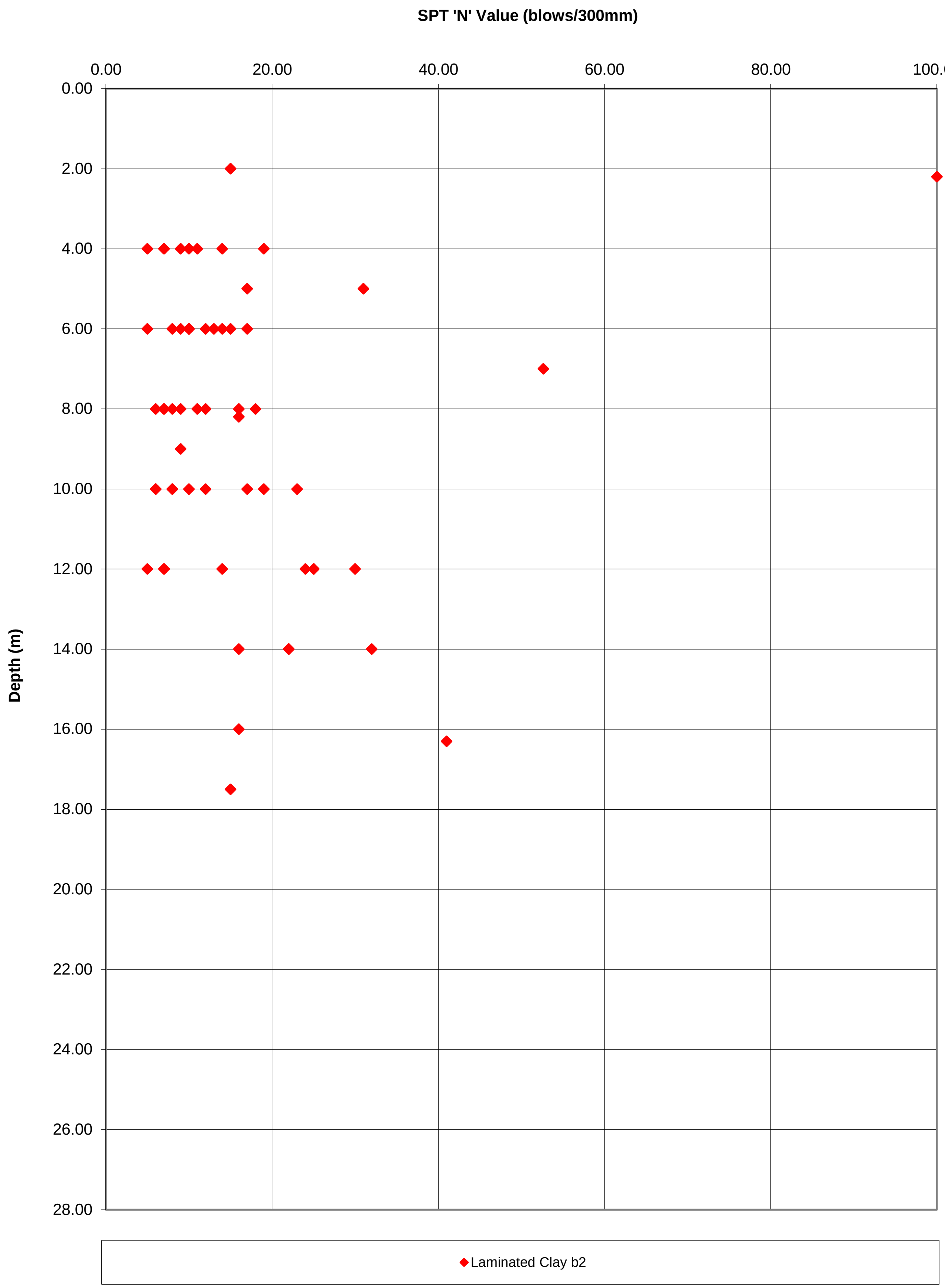
Moisture Content and Atterberg Limits versus Depth
Laminated Clay (b2)

Date: April 2018

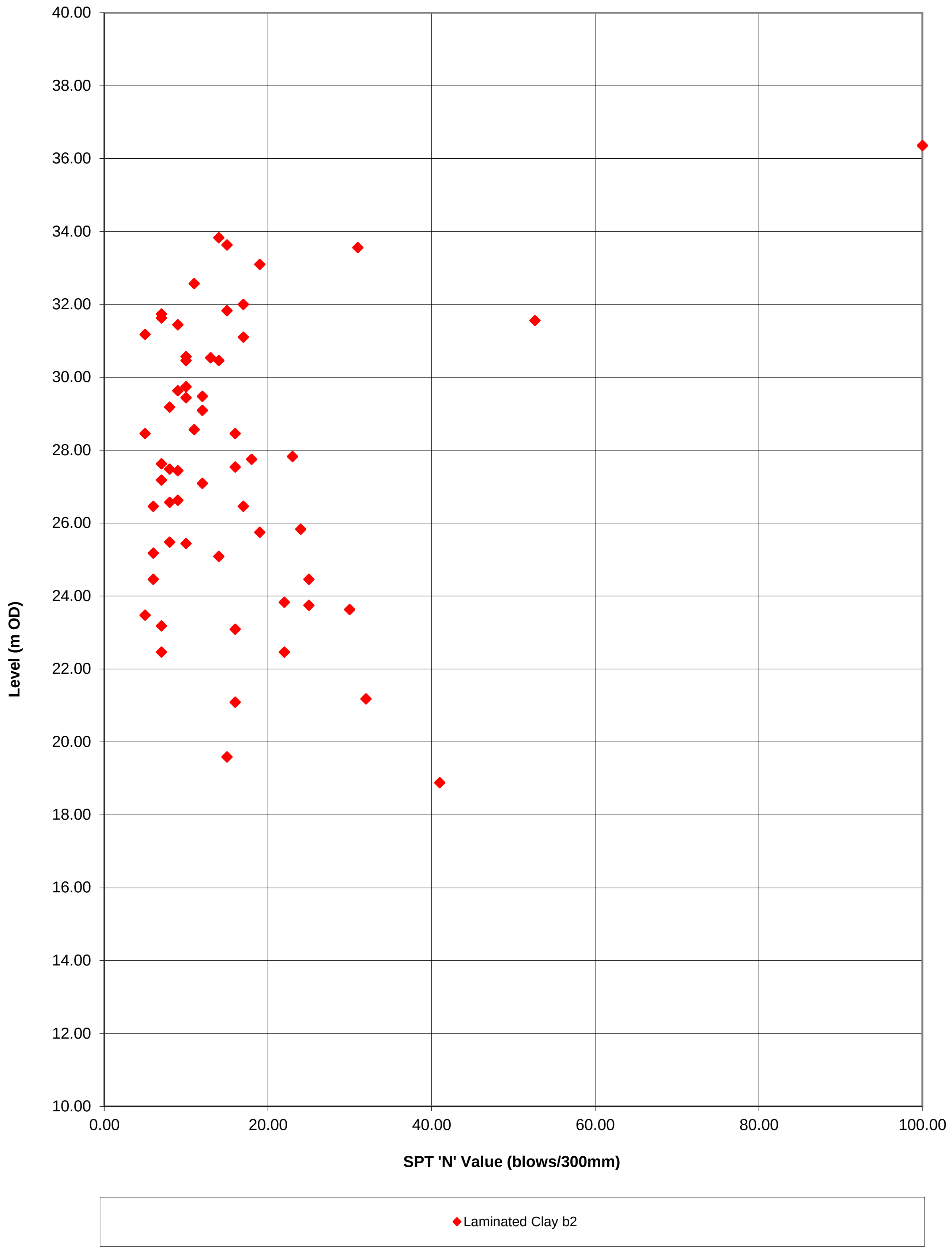
Figure 37



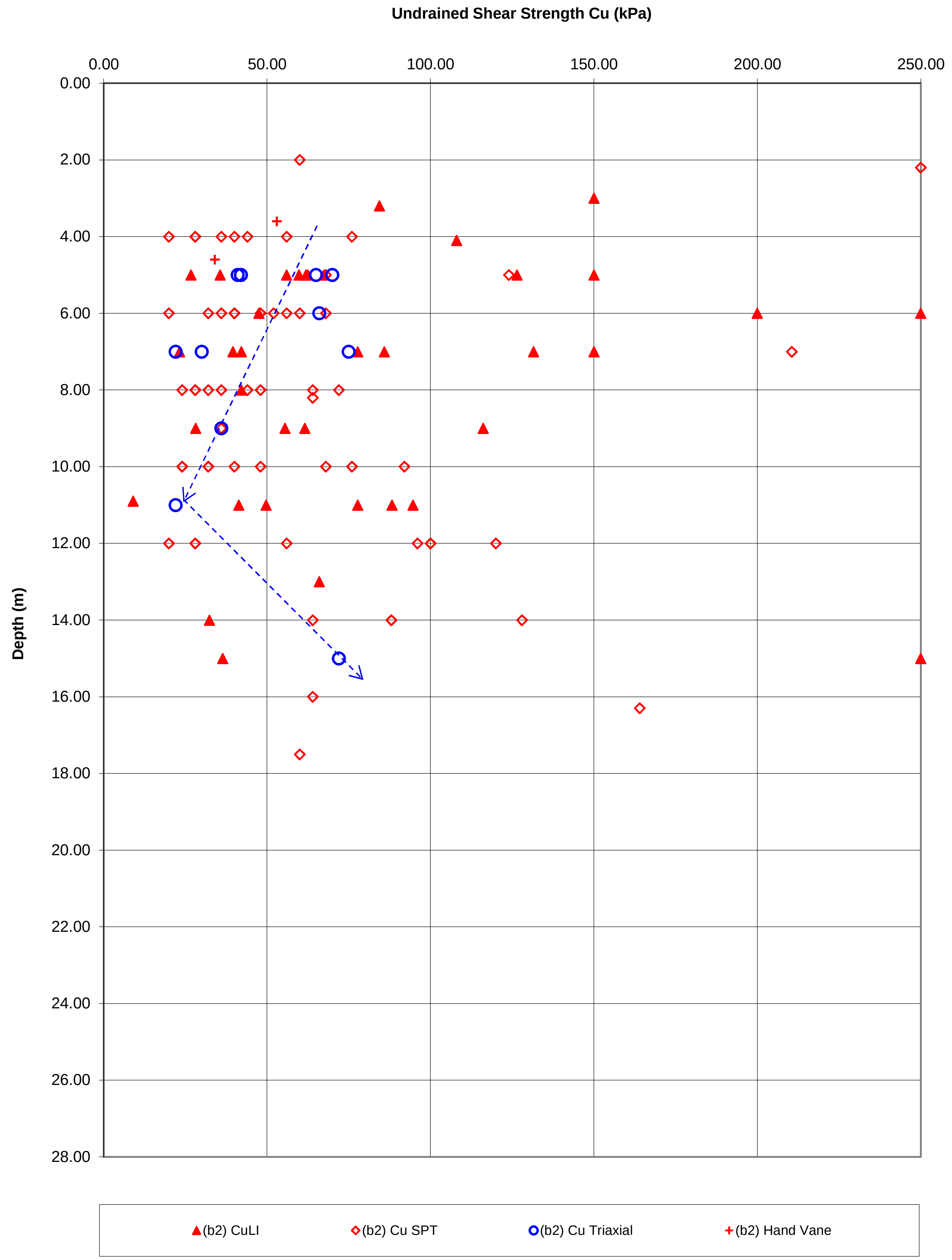
IAMP Ground Investigation - PHASE ONE	Sunderland City Council / AECOM Imagine it. Delivered.
Moisture Content and Atterberg Limits versus Level Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 38



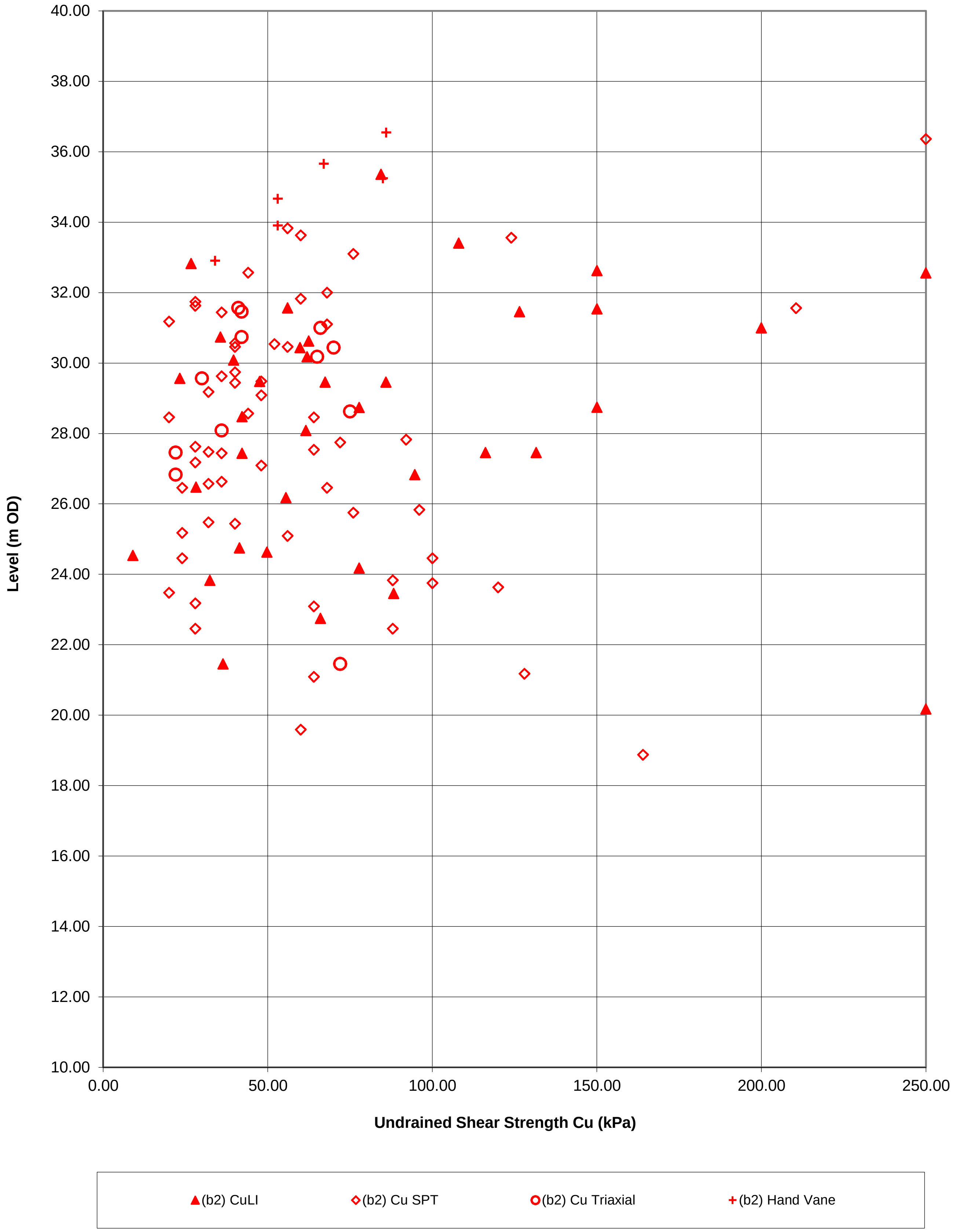
IAMP Ground Investigation - PHASE ONE	Sunderland City Council · AECOM Imagine it. Delivered.
SPT 'N' Value versus Depth Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 39



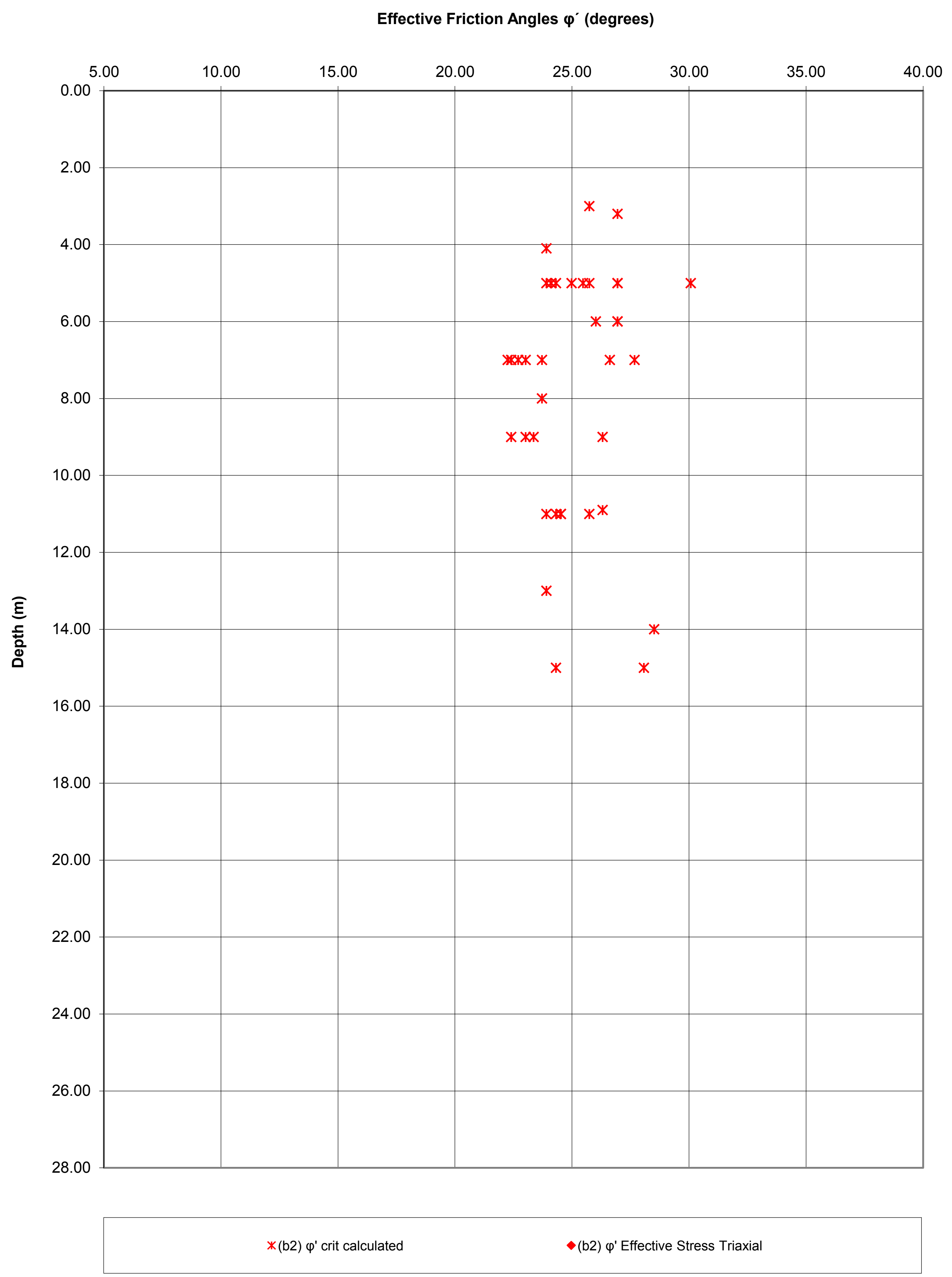
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
SPT 'N' Value versus Level Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 40



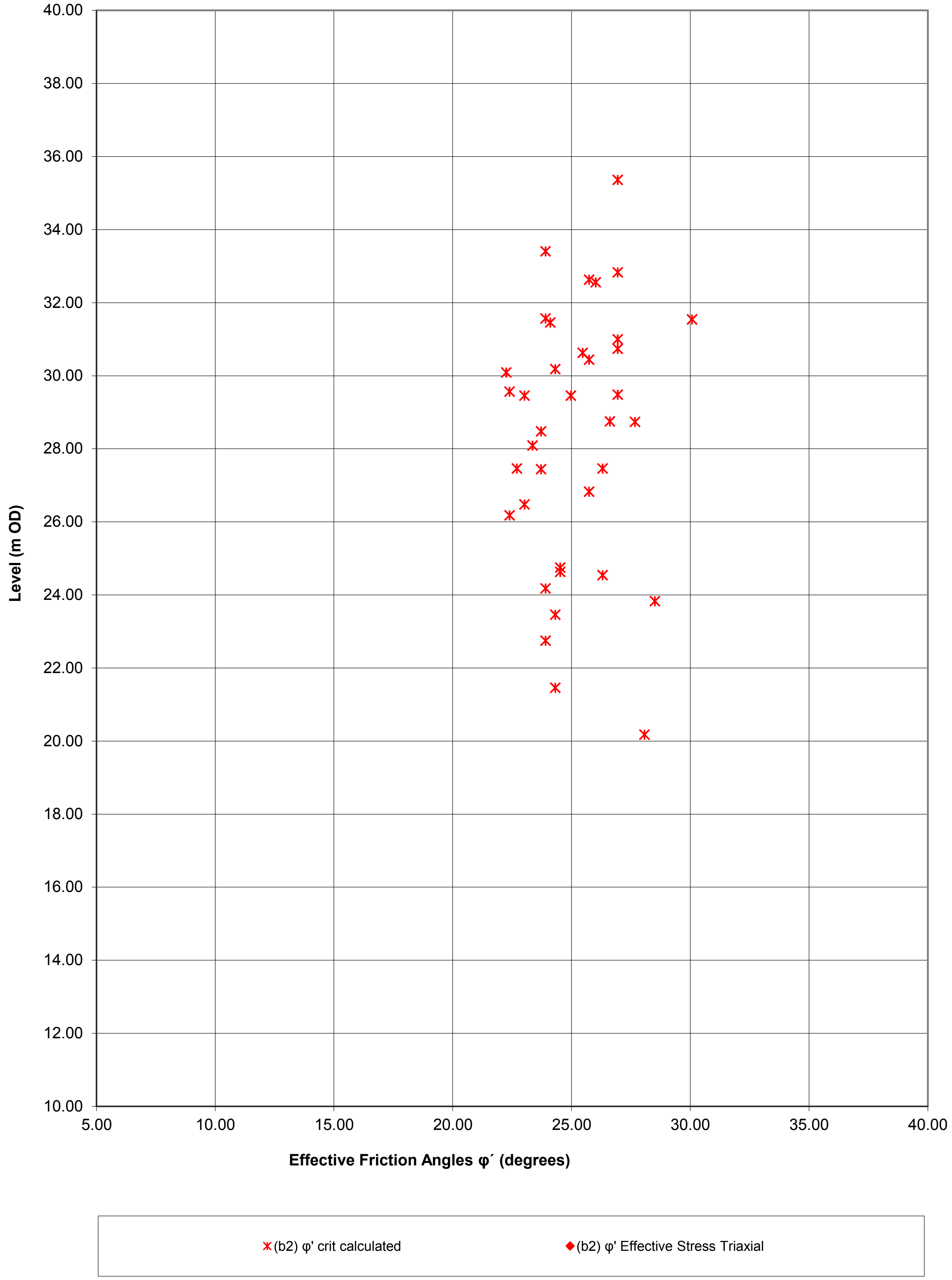
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Undrained Shear Strength versus Depth Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 41



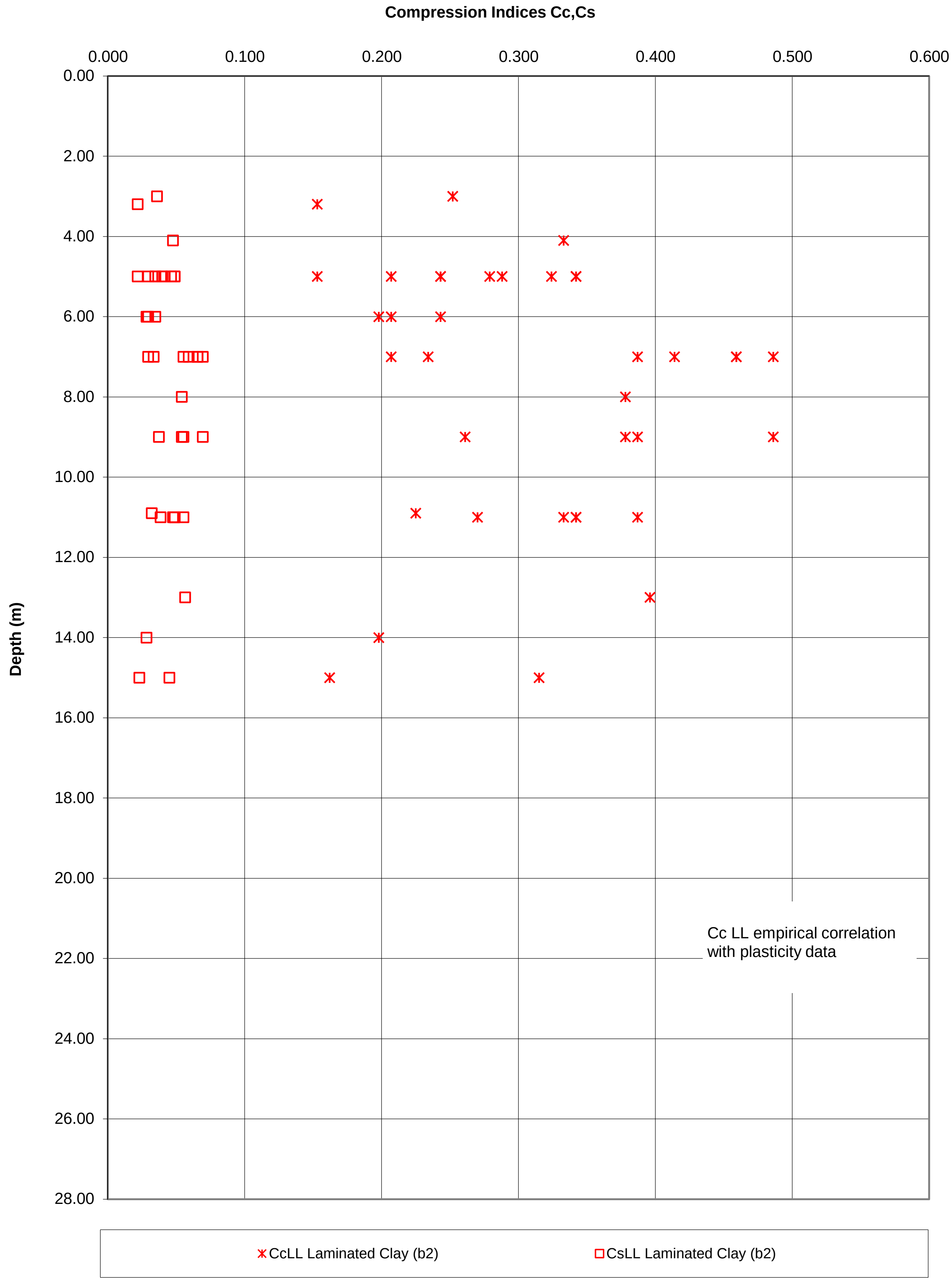
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Undrained Shear Strength versus Level Upper Laminated Clay (b1)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 42



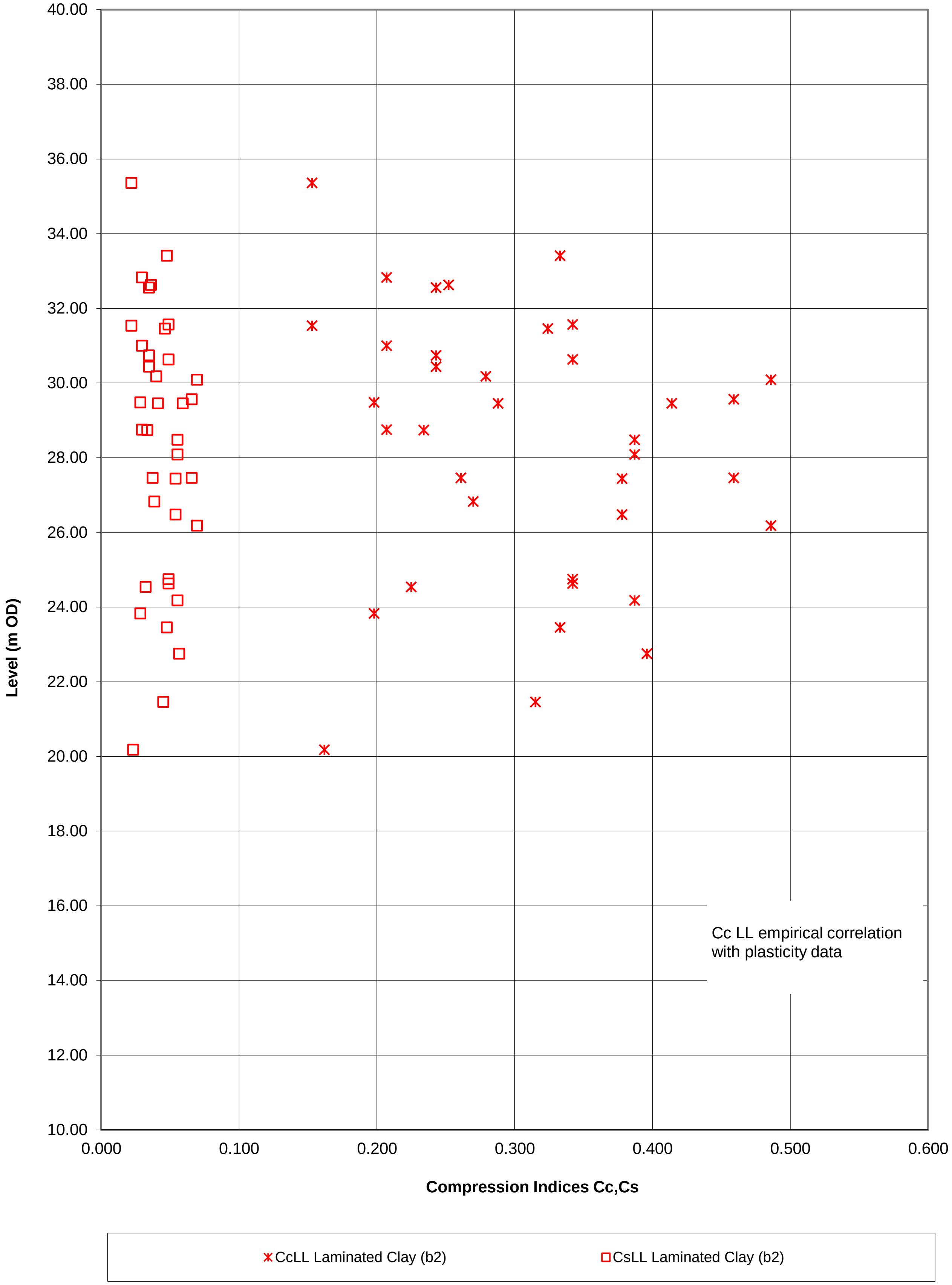
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Effective Friction Angles versus Depth Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 43



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Effective Friction Angles versus Level Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 44



IAMP Ground Investigation - PHASE ONE	Sunderland City Council · AECOM Imagine it. Delivered.
Compression Indices versus Depth Laminated Clay (b2)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 45



IAMP Ground Investigation - PHASE ONE

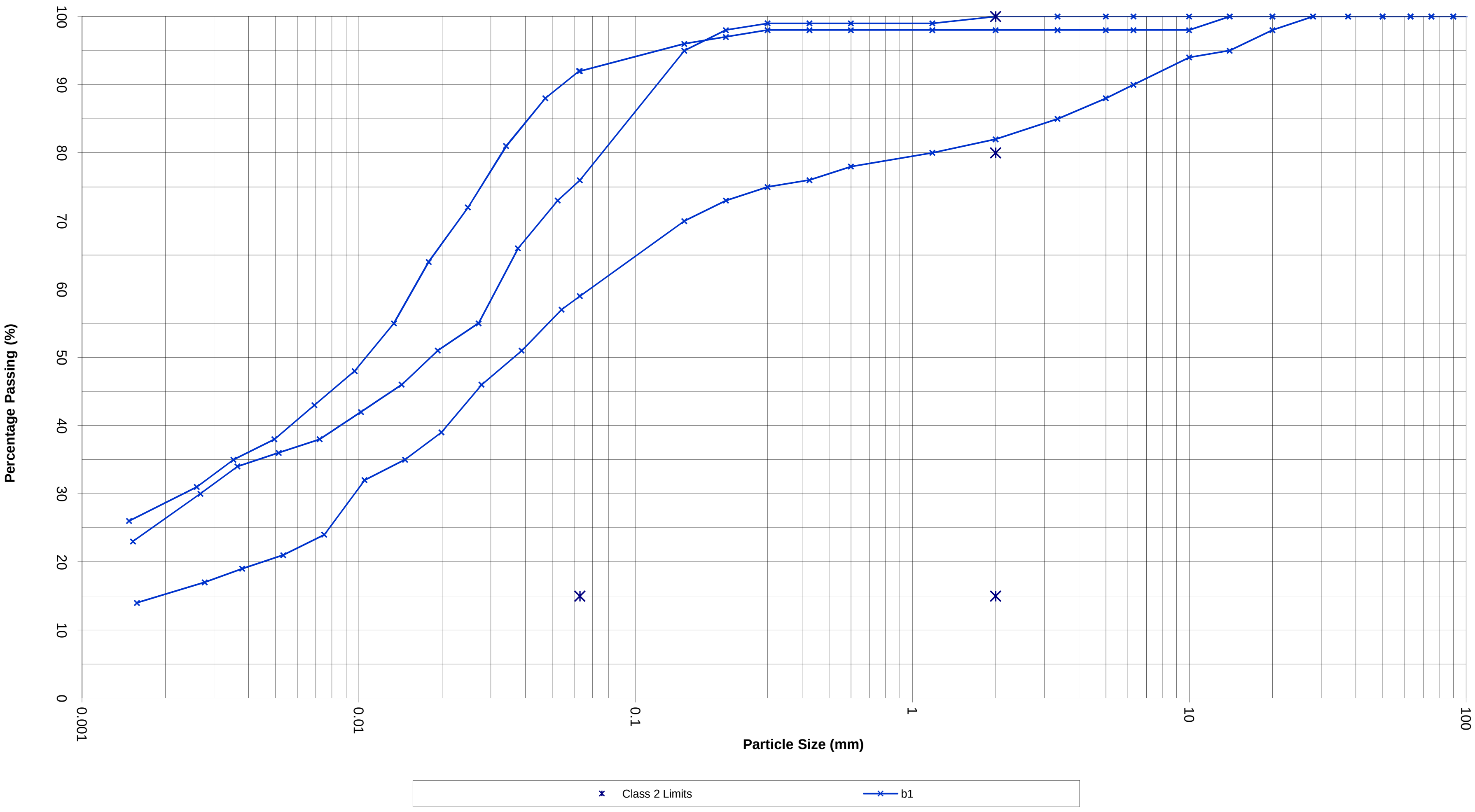
Sunderland City Council
AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

Compression Indices versus Level
Laminated Clay (b2)

Date: April 2018

Figure 46



Sunderland City Council

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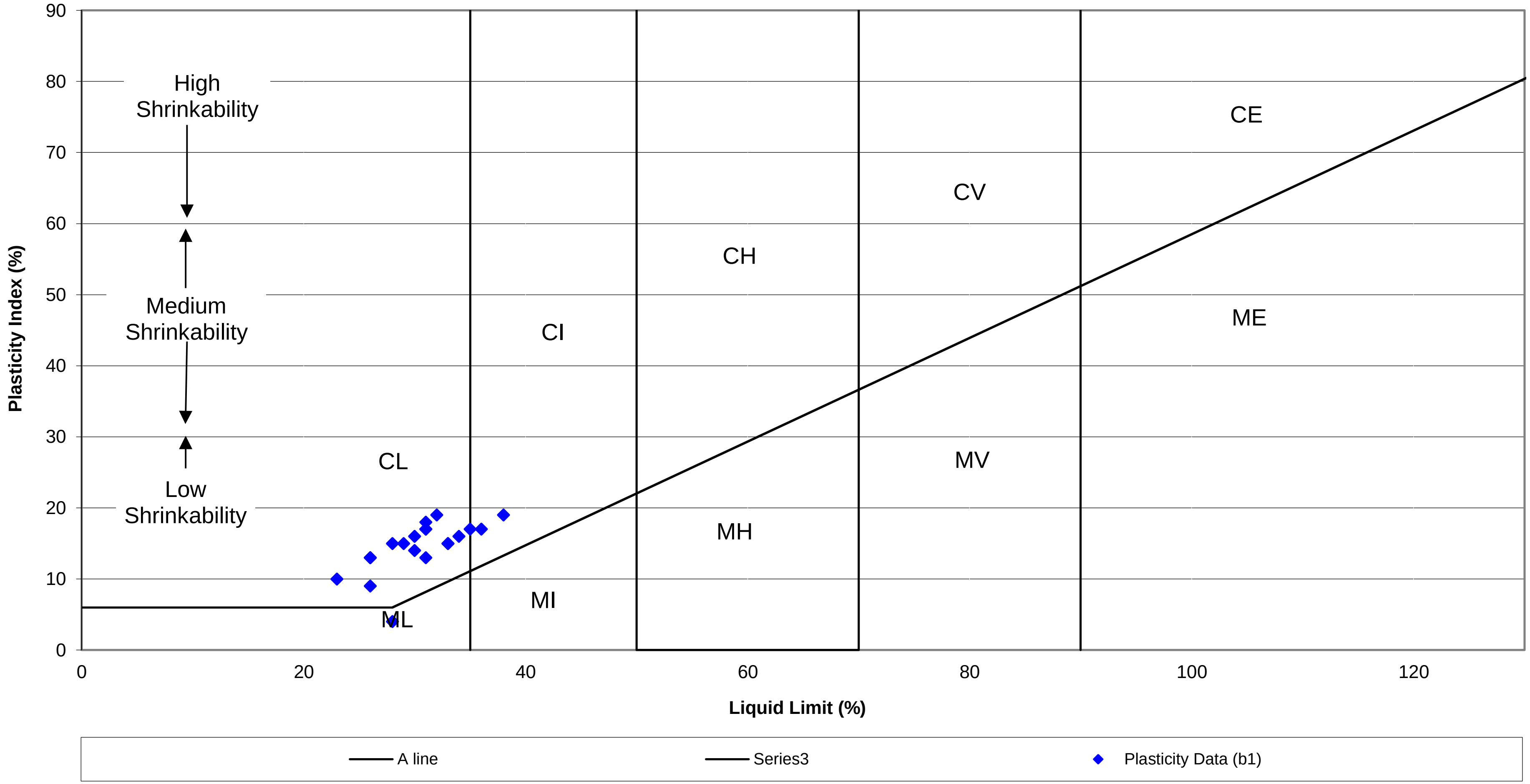
IAMP Ground Investigation - PHASE ONE
Particle Size Distribution Curve - b1 Lower Glacial Till

60283414_M015_GEO_RT_001
Date: April 2018

Figure 47



Sunderland City Council

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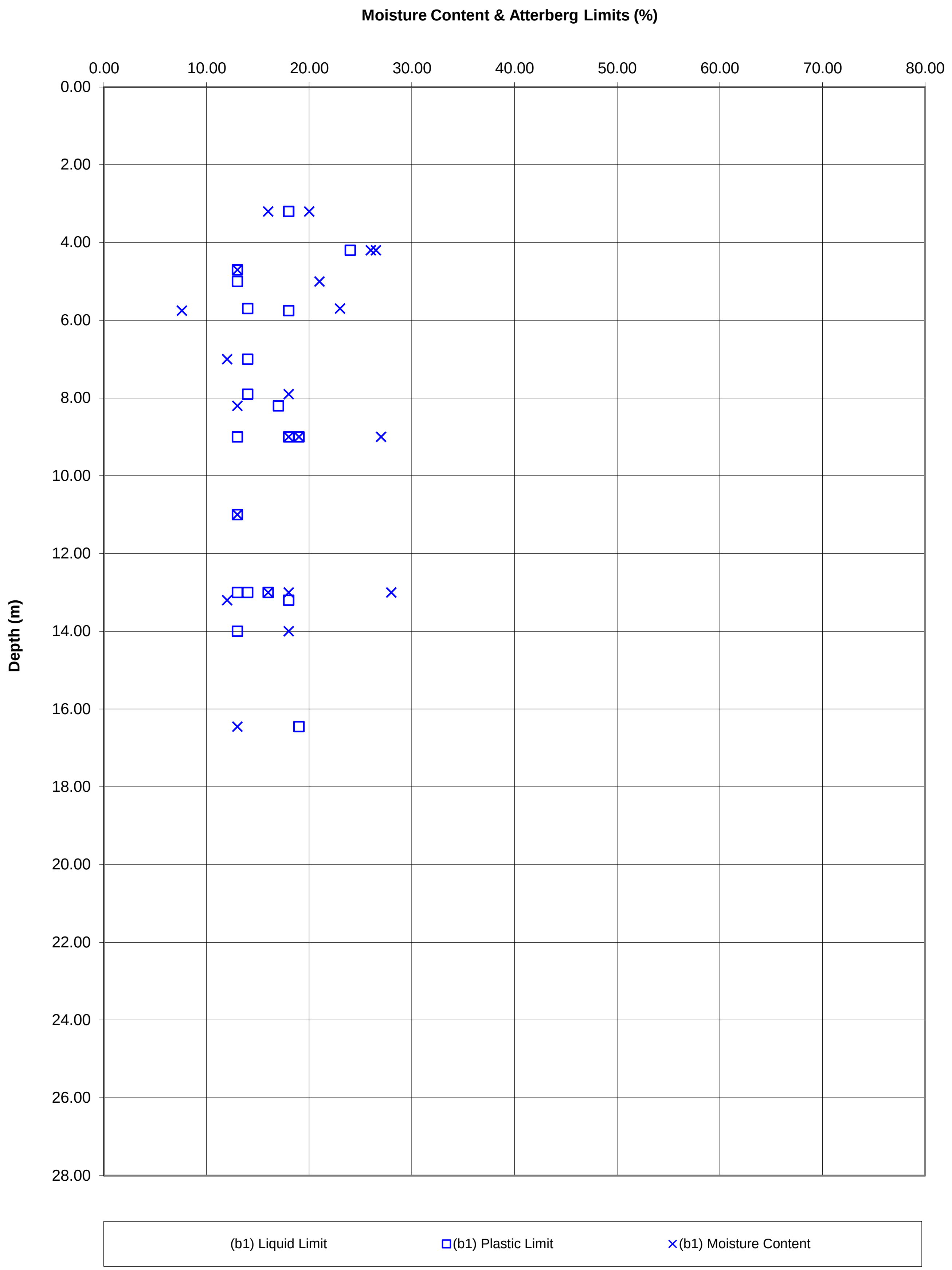
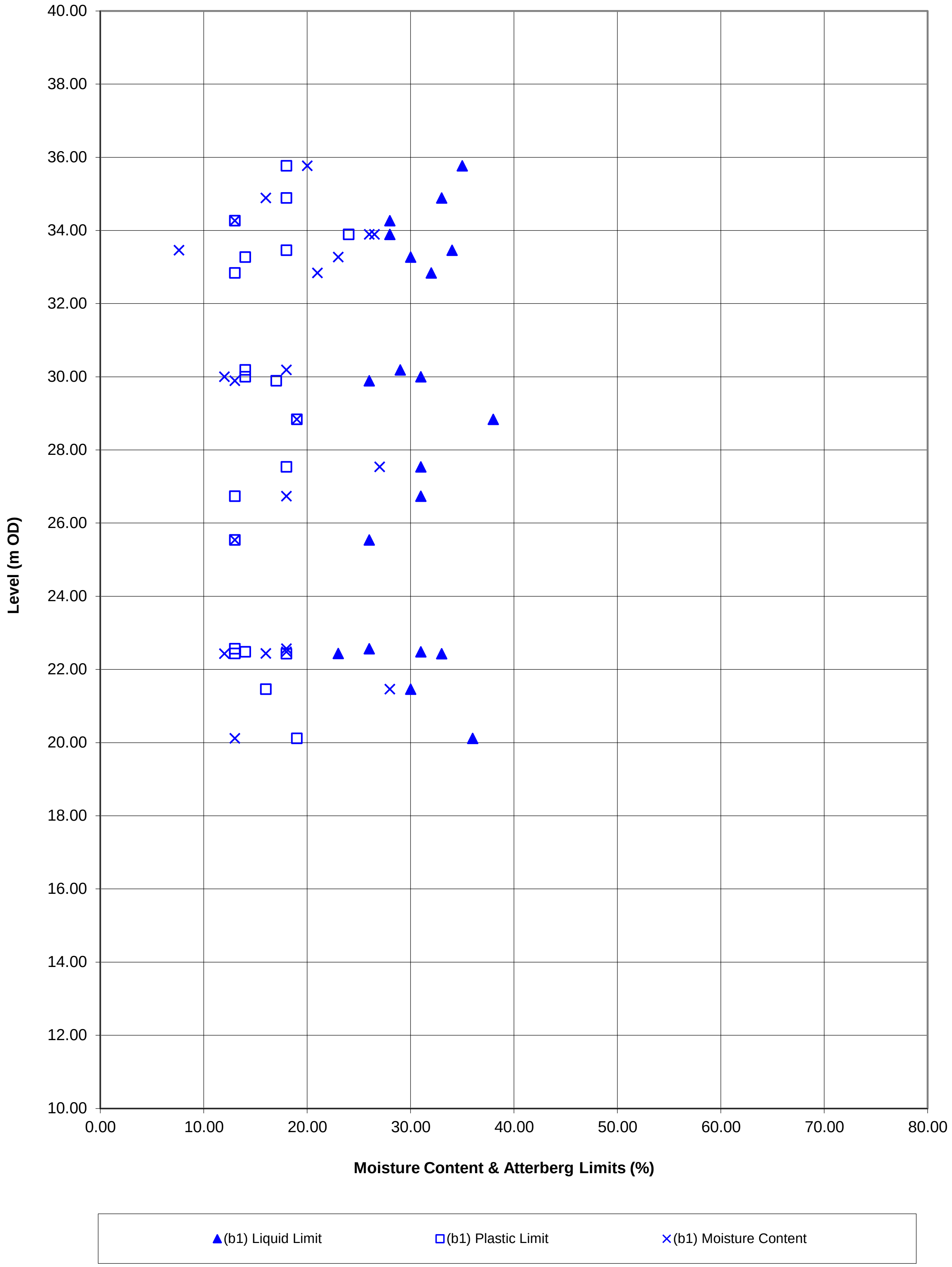
IAMP Ground Investigation - PHASE ONE
Casagrande Plasticity Chart - Lower Glacial Till (b1)

60283414_M015_GEO_RT_001
Date: April 2018

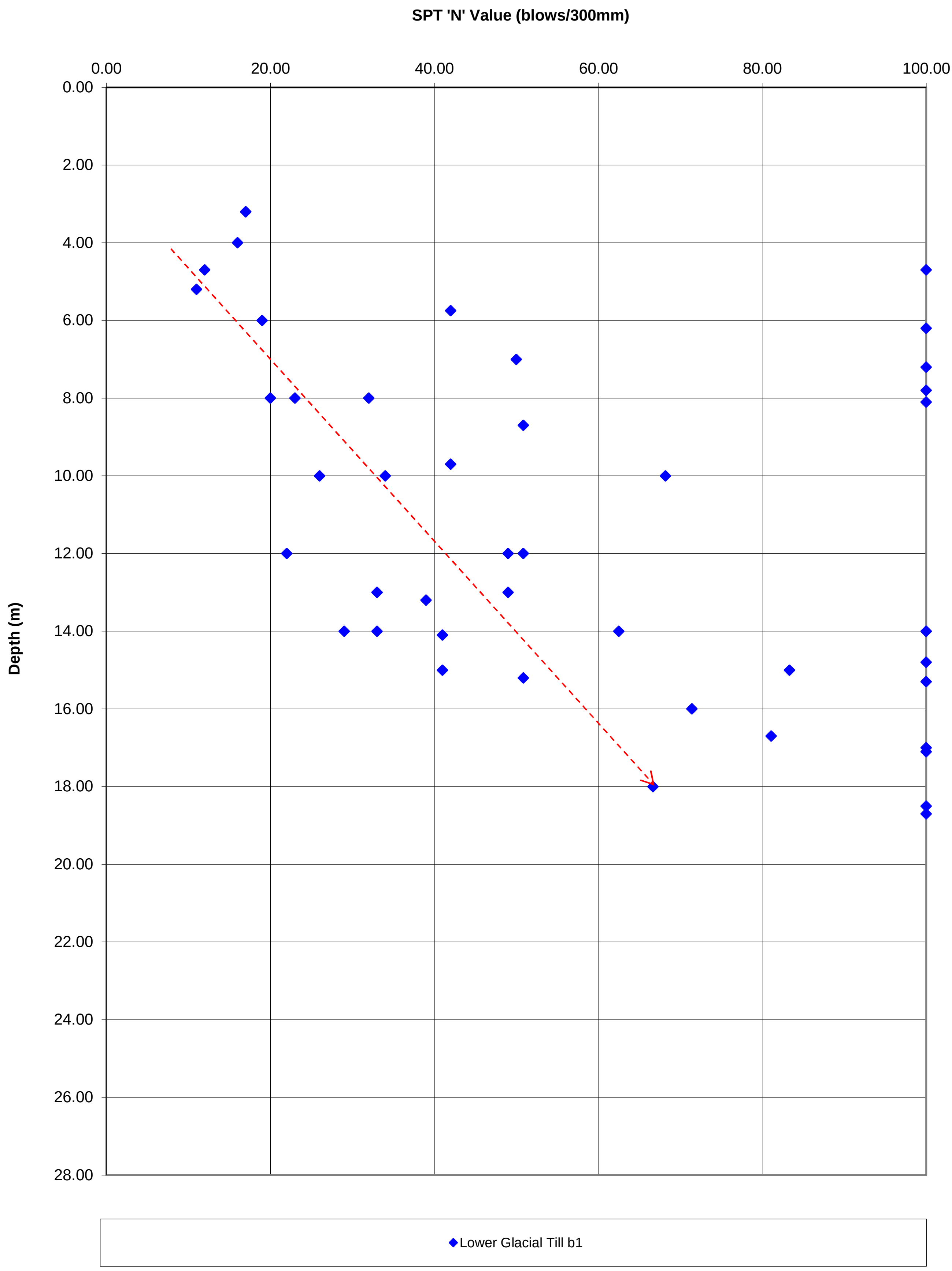
Figure 48



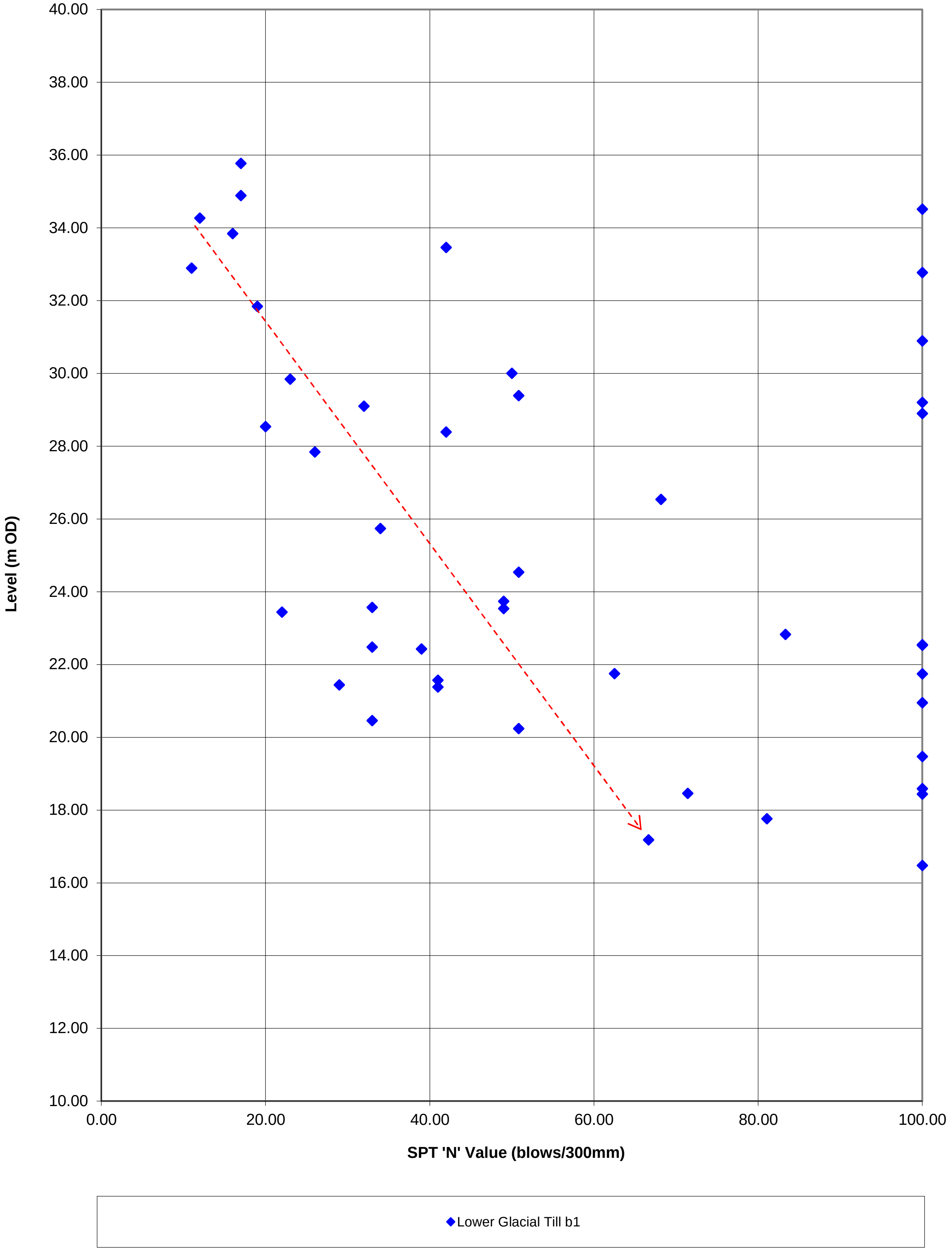
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Moisture Content and Atterberg Limits versus Depth Lower Glacial Till (b1)	60283414_M015_GEO_RT_001 Date: April 2018 — Figure 49



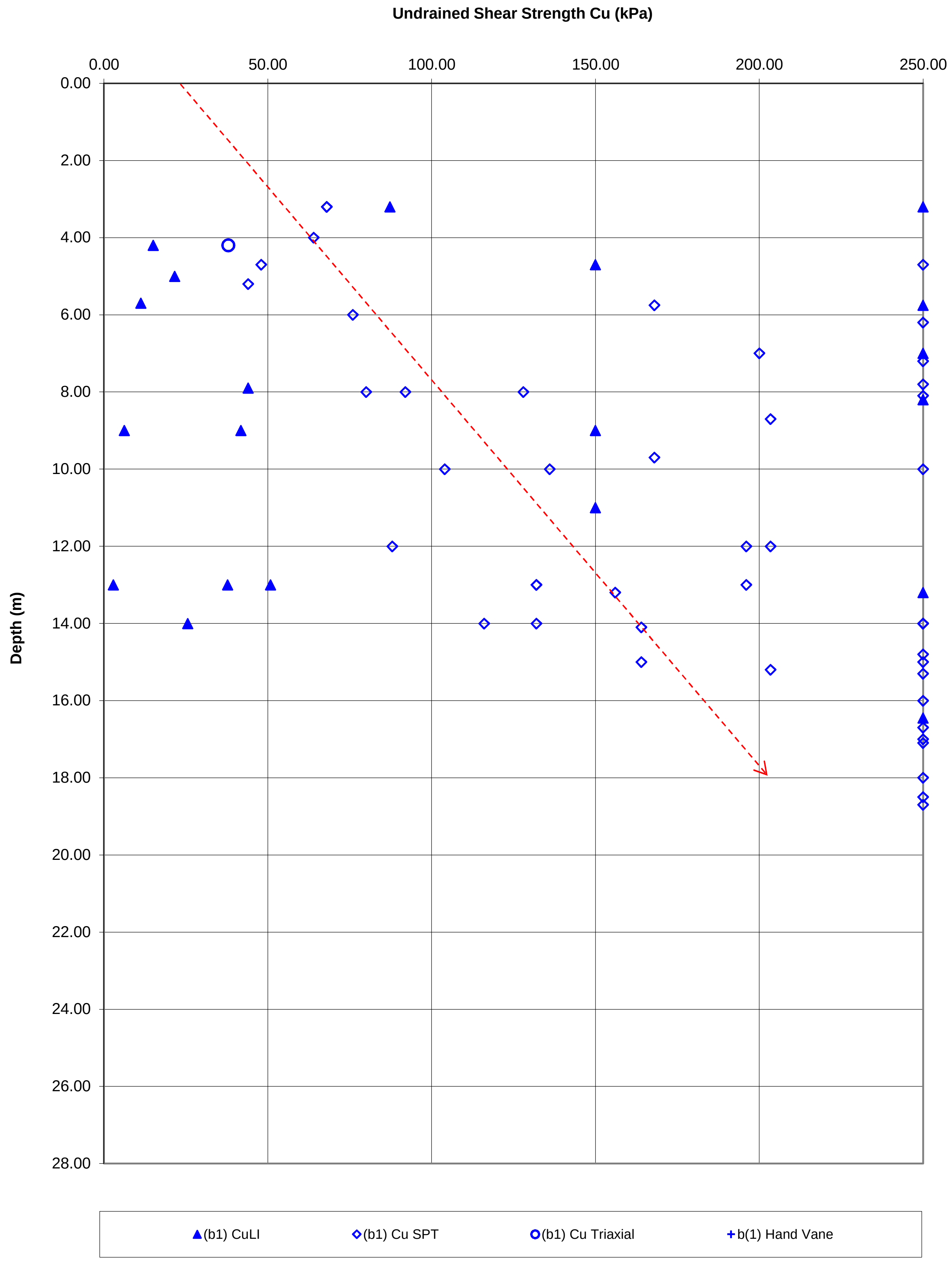
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Moisture Content and Atterberg Limits versus Level Lower Glacial Till (b1)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 50



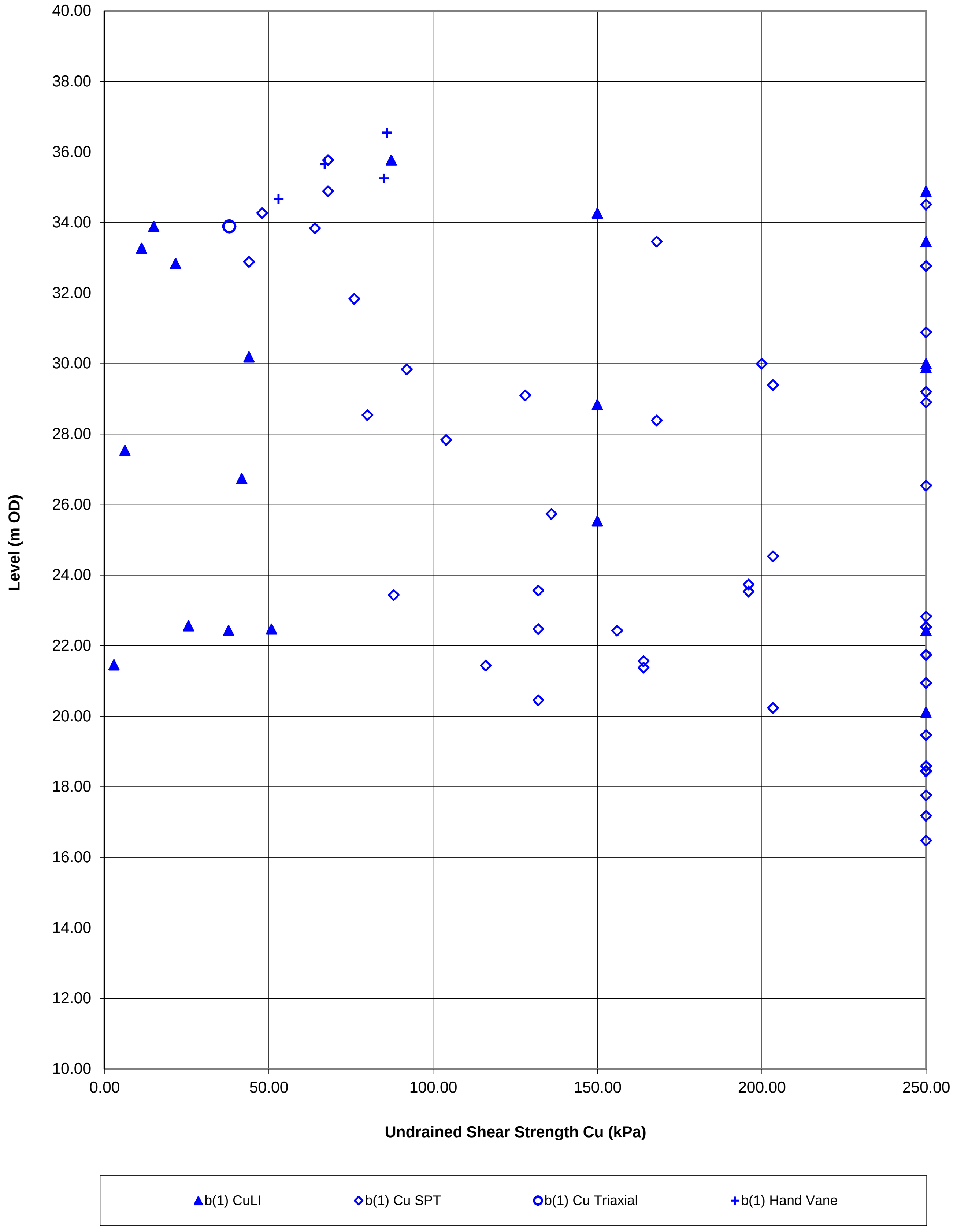
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.	
SPT 'N' Value versus Depth Lower Glacial Till (b1)	60283414_M015_GEO_RT_001	
	Date: April 2018	Figure 51



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
SPT 'N' Value versus Level Lower Glacial Till (b1)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 52



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Undrained Shear Strength versus Depth Lower Glacial Till (b1)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 53



IAMP Ground Investigation - PHASE ONE

Sunderland City Council

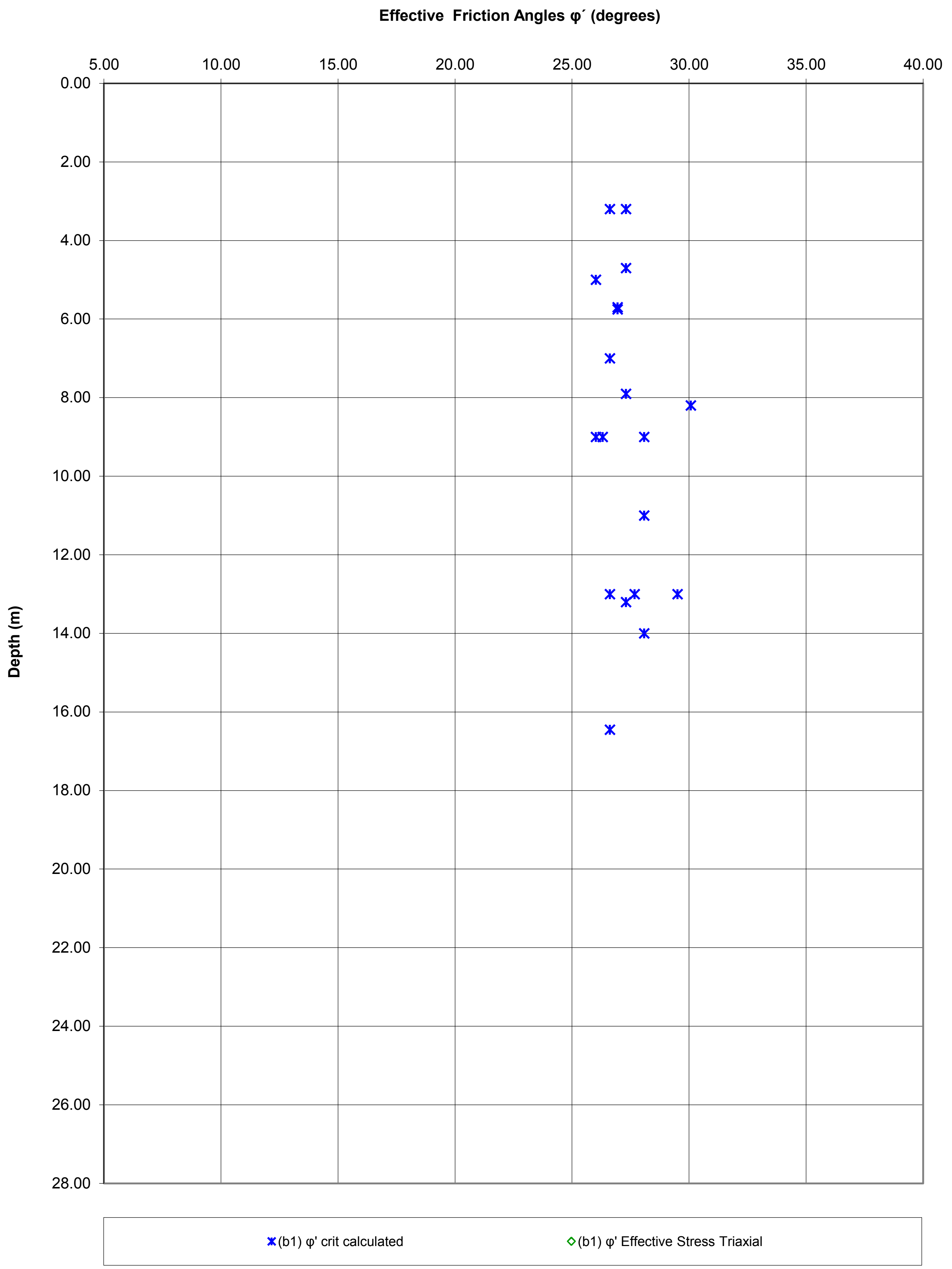
AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

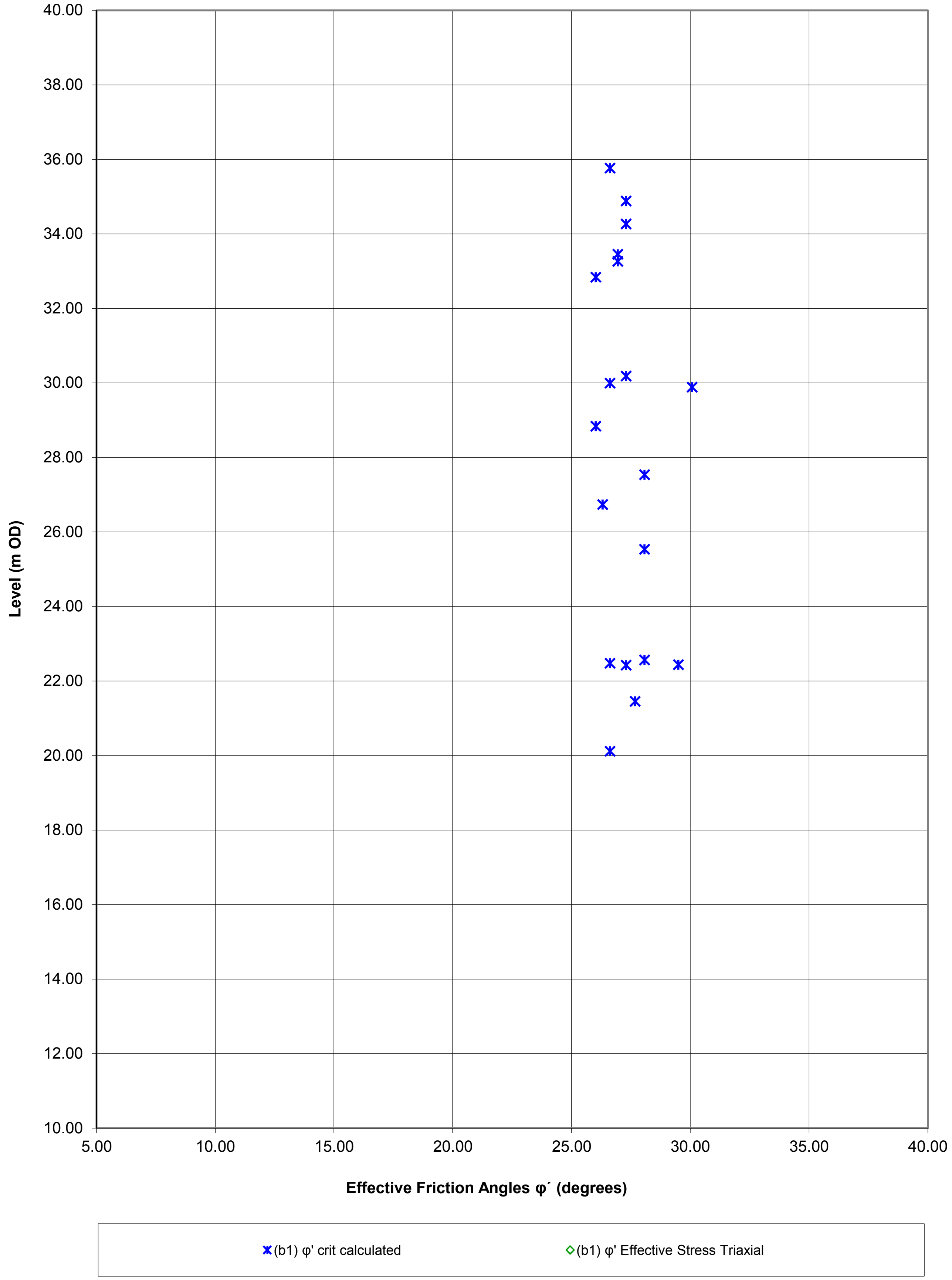
Undrained Shear Strength versus Level
Lower Glacial Till (b1)

Date: April 2018

Figure 54



IAMP Ground Investigation - PHASE ONE	Sunderland City Council · AECOM Imagine it. Delivered.
Effective Friction Angles versus Depth Lower Glacial Till (b1)	60283414_M015_GEO_RT_001 Date: April 2018 Figure 55



IAMP Ground Investigation - PHASE ONE

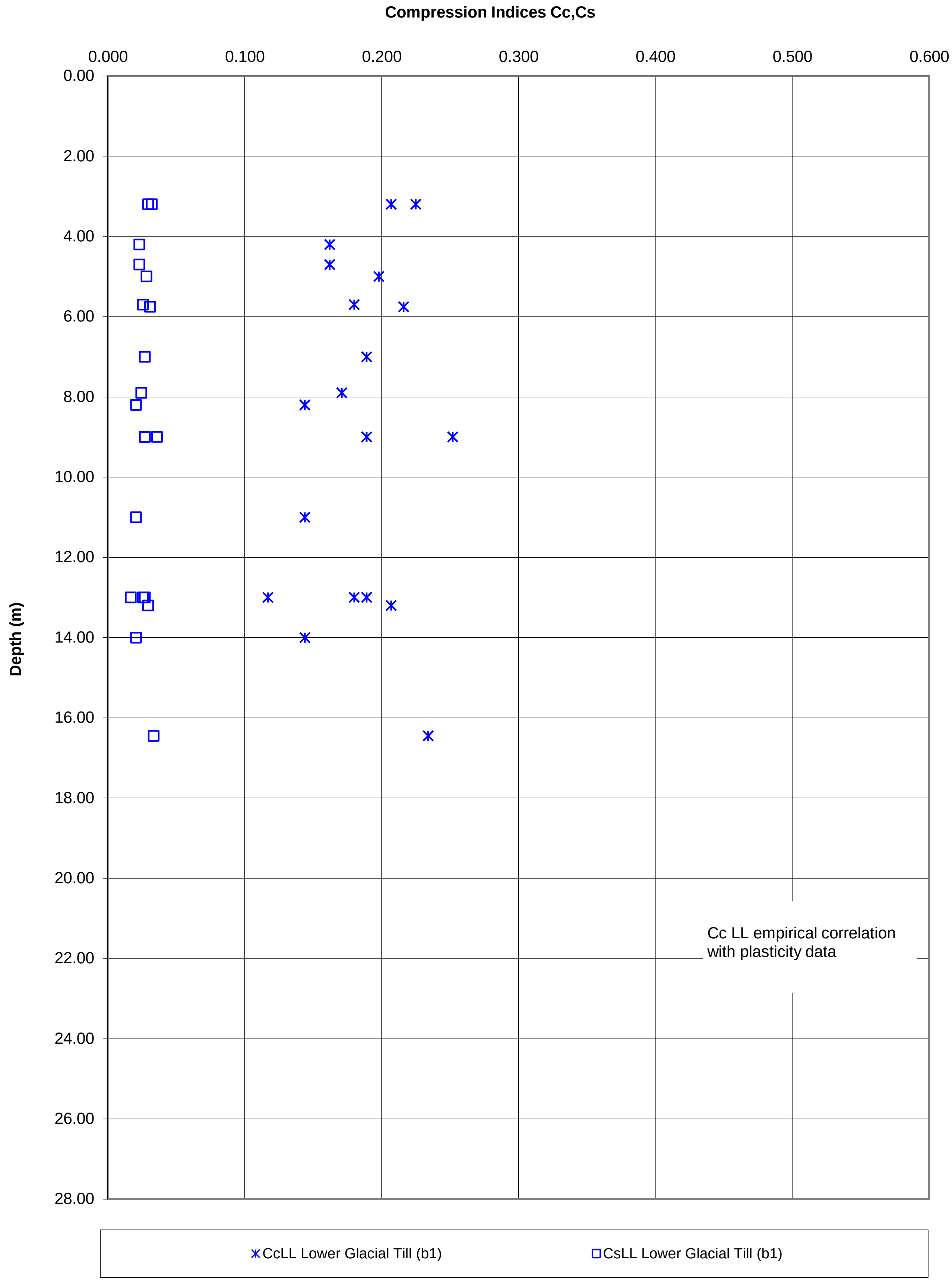
Sunderland City Council AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

Effective Friction Angles versus Level
Lower Glacial Till (b1)

Date: April 2018

Figure 56



IAMP Ground Investigation - PHASE ONE

Sunderland City Council

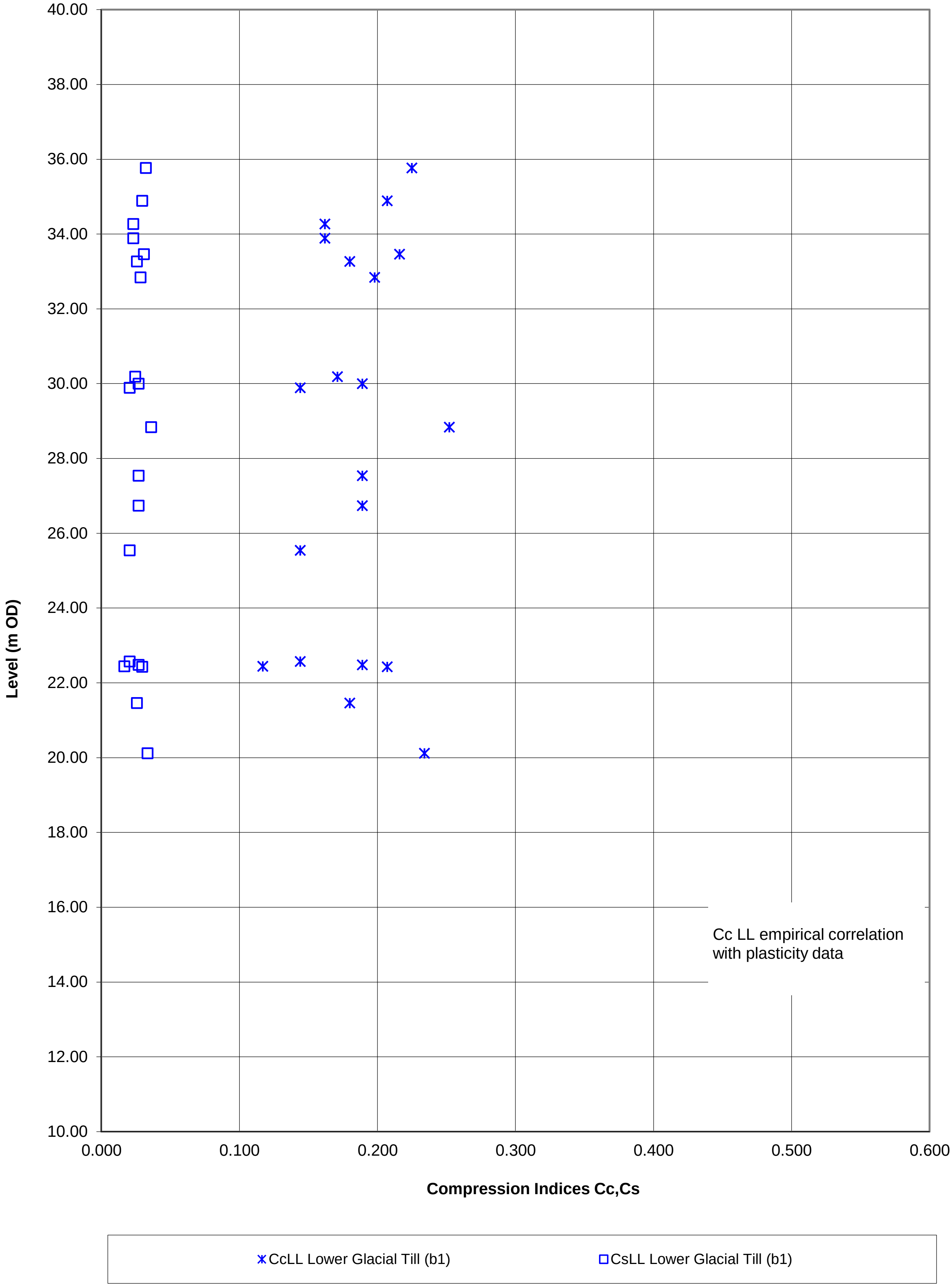
AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

Compression Indices versus Depth
Lower Glacial Till (b1)

Figure 57

Date: April 2018



IAMP Ground Investigation - PHASE ONE

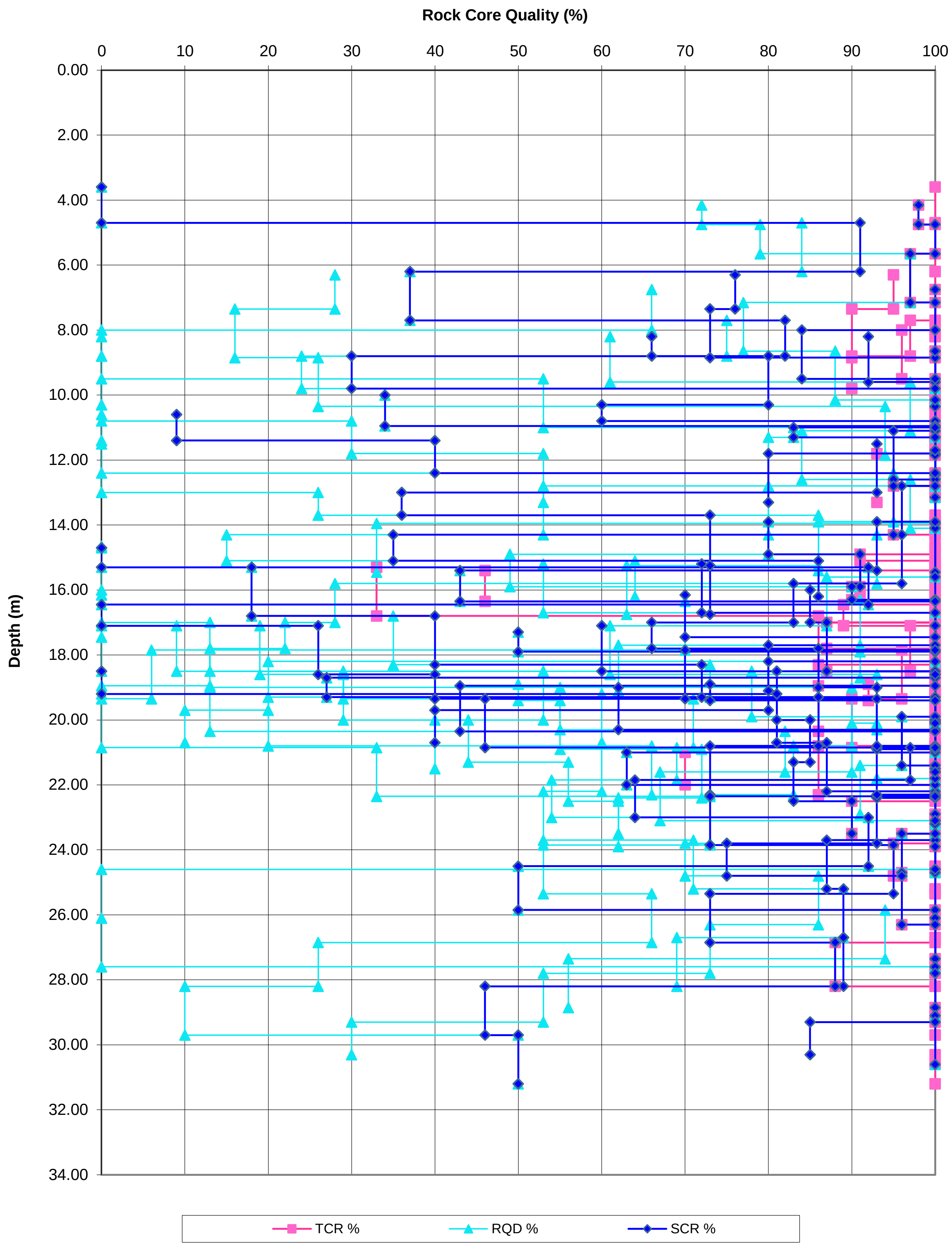
Sunderland City Council AECOM Imagine it. Delivered.

60283414_M015_GEO_RT_001

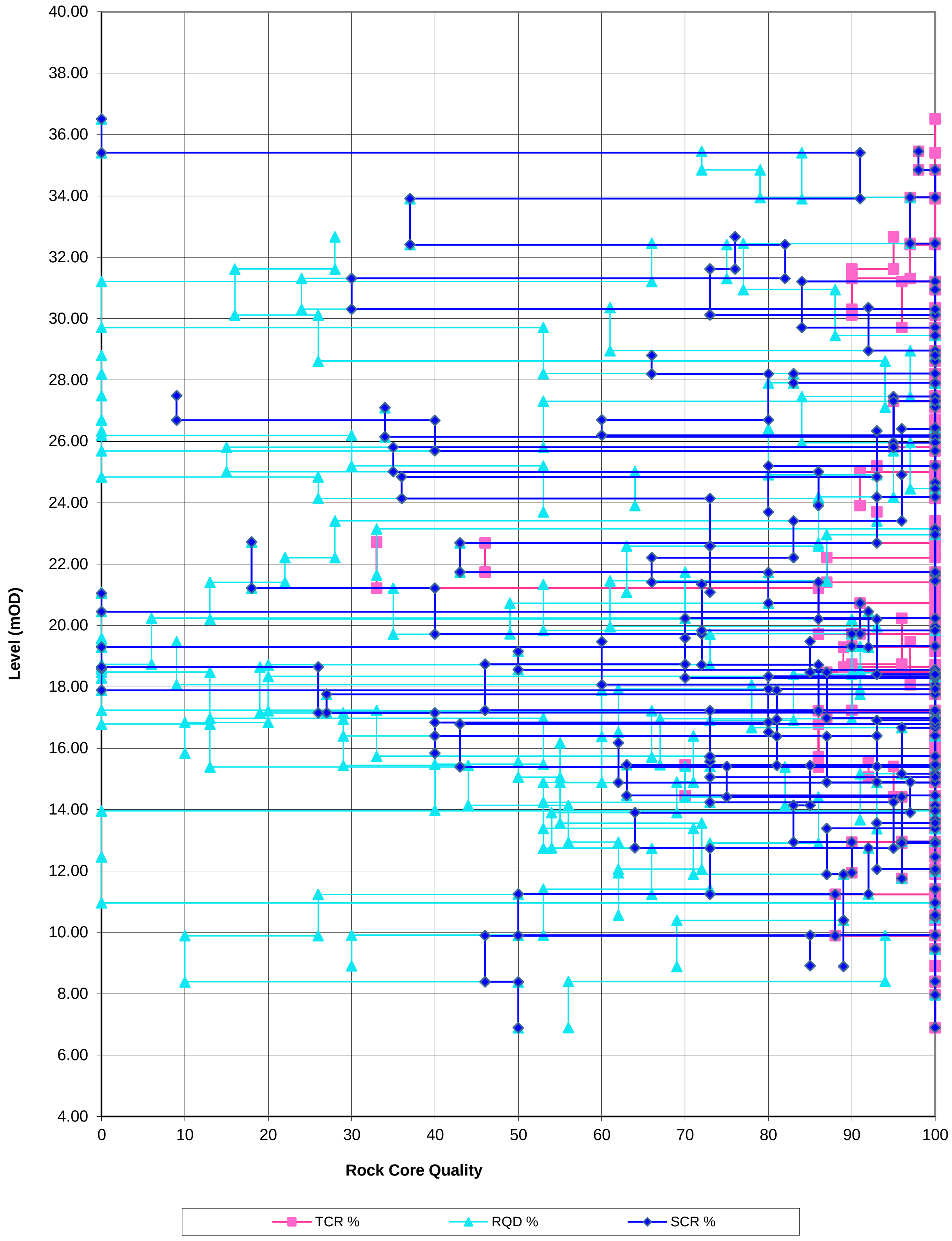
Compression Indices versus Level
Lower Glacial Till (b1)

Date: April 2018

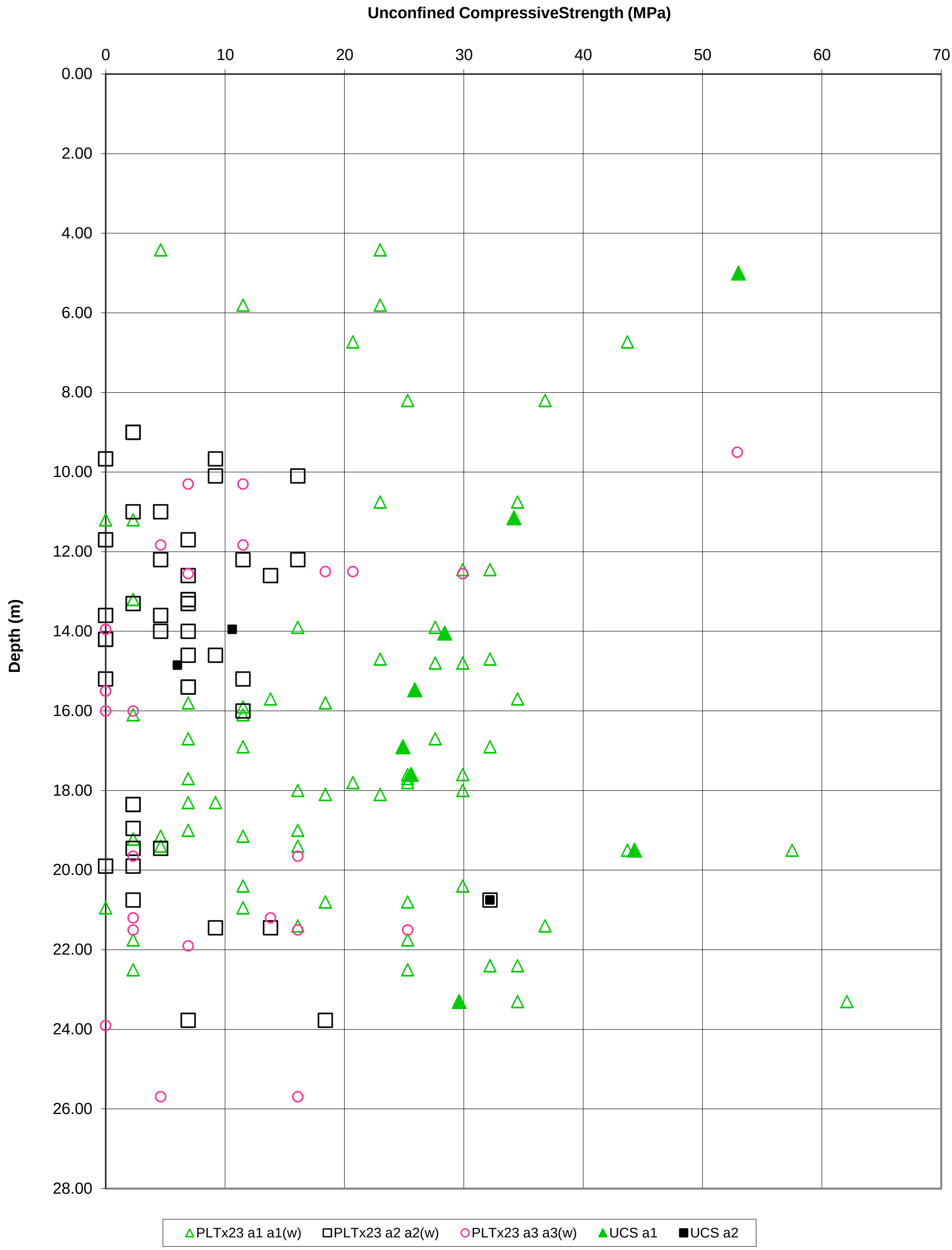
Figure 58



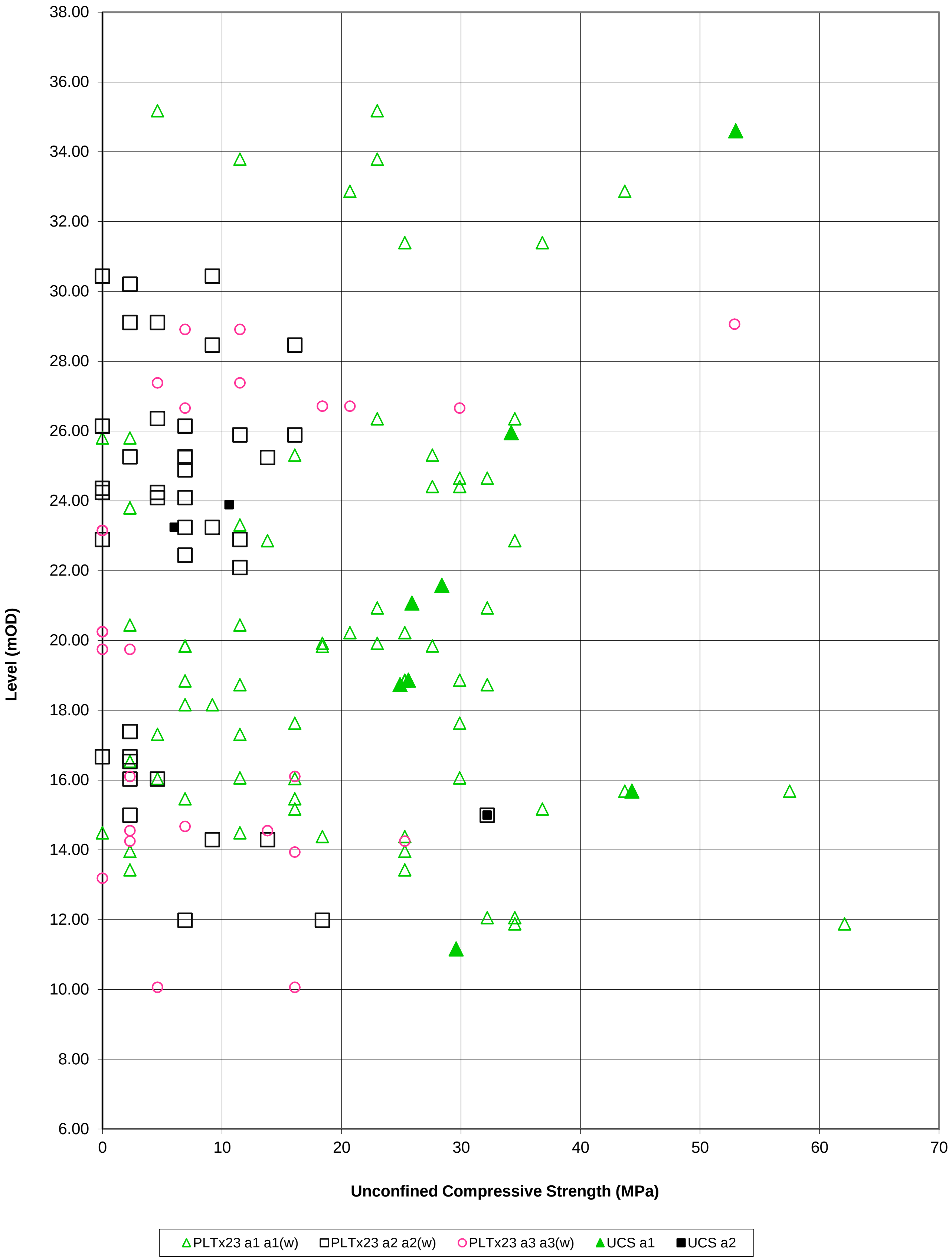
IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Rock Core Quality versus Depth	60283414_M015_GEO_RT_001 Date: April 2018 Figure 59



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Rock Core Quality versus Level	60283414_M015_GEO_RT_001
	Date: April 2018
	Figure 60



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Rock Strength versus Depth	60283414_M015_GEO_RT_001
	Date: April 2018
	Figure 61



IAMP Ground Investigation - PHASE ONE	Sunderland City Council AECOM Imagine it. Delivered.
Rock Strength versus Level	60283414_M015_GEO_RT_001
	Date: April 2018
	Figure 62

Drawings

60283414_M015_GEO_DR_001 - IAMP Site Location Plan (original red line boundary)

60283414_M015_GEO_DR_002 – Proposed Exploratory Hole Location Plan (original red line boundary)

60283414_M015_GEO_DR_005 - 1:50,000 Geological Map – Drift (original red line boundary)

60283414_M015_GEO_DR_006 - 1:50,000 Geological Map – Solid (original red line boundary)

60283414_M015_GEO_DR_007 - Agricultural Land Classification (original red line boundary)

60283414_M015_GEO_DR_008 - Soil Survey Map (original red line boundary)

60283414_M015_GEO_DR_009 - 1:10,000 Geological Map (original red line boundary)

60283414-M015-ACM-L1-DR-GE_001 – IAMP ONE Site Boundary

60283414-M015-ACM-L1-DR-GE_002 – IAMP ONE Masterplan - Exploratory Hole Location Plan

60283414-M015-ACM-L1-DR-GE_003 – IAMP ONE – Rockhead Contours

60283414-M015-ACM-L1-DR-GE_004 – IAMP ONE – Geotechnical Constraints

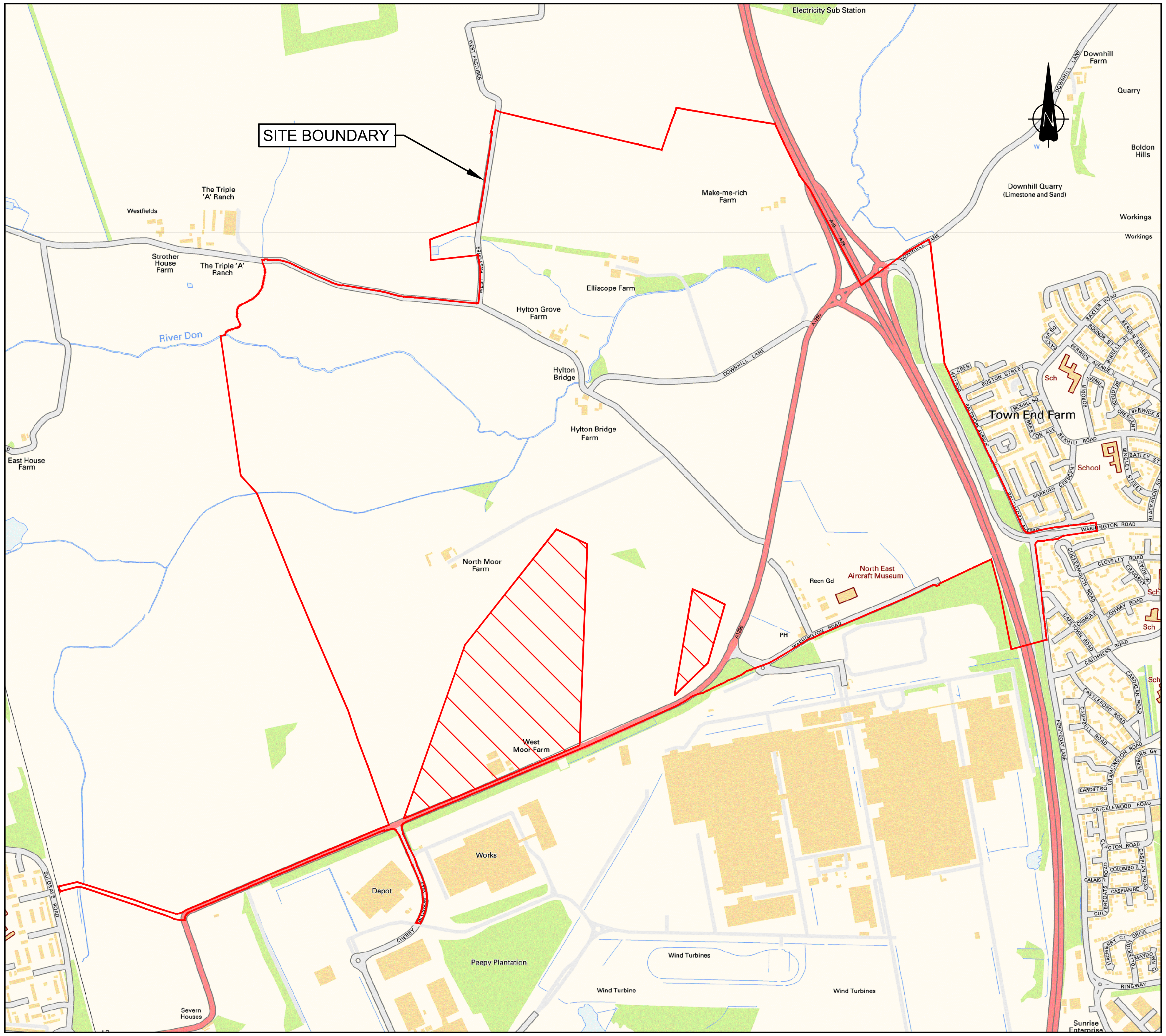
60283414-M015-ACM-L1-DR-GE_005 – IAMP ONE - Geological Sections A-A and B-B

60283414-M015-ACM-L1-DR-GE_006 – IAMP ONE - Geological Sections C-C and D-D

60283414-M015-ACM-L1-DR-GE_007 – IAMP ONE - Geological Sections E-E and F-F

Filename: F: PROJECTS MUNICIPAL INFRASTRUCTURE - NEPO FRAMEWORK LOT 6 M015 SCC IAMP GEOTECH ADVISOR AUTOCAD 001 - SITE LOCATION PLAN.DWG   Last saved by: ADDISONRP   Last Plotted: 11.23.2016 11:05 AM

Project Management Initials: Designer: RPA   Checked: CMC   Approved: SDM   ISO A3 297mm x 420mm



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SCALE 1:12 500

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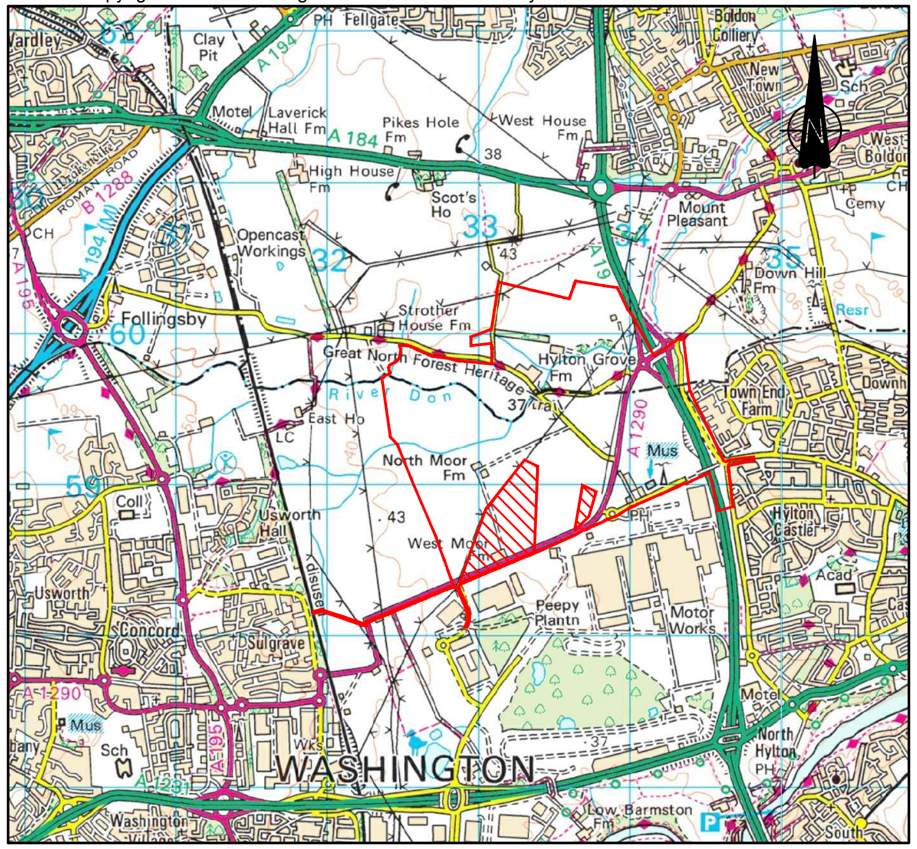


SCALE 1:50 000

KEY

RED LINE BOUNDARY (ARUP EIA)

SAFEGUARDED AREA

INTERNATIONAL ADVANCED MANUFACTURING PARK

Site Location Plan

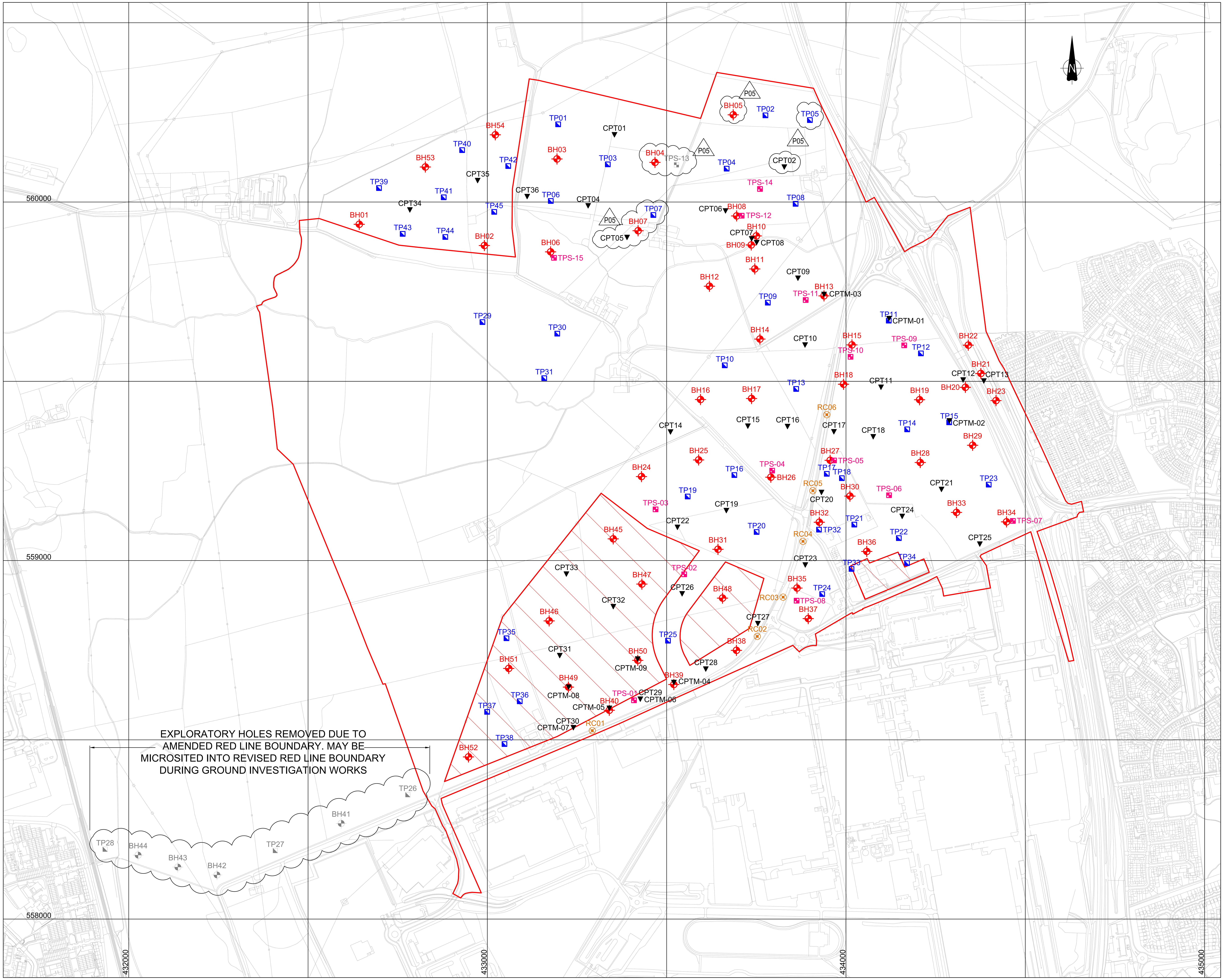
AECOM

Sunderland City Council   South Tyneside Council

Grid Reference: NZ 335 594
Drawing Number: 60283414_M015_GEO_DR_001

Project No: 60283414
Date November 2016

Project Management Initials: Designer: RPA Checked: CMC Approved: SDM ISO A1 594mm x 841mm



AECOM

NOTES

1. REFER TO DRAWING 004 SHEETS 1 TO 8 FOR 1:2,500 SCALE PRESENTATION.

EXPLORATORY HOLE KEY

BH##	PROPOSED BOREHOLE
TP##	PROPOSED TRIAL PIT
TPS##	PROPOSED TRIAL PIT SOAKAWAY TESTING
RC##	PROPOSED ROAD CORE (THROUGH EXISTING A1290)
CPT##	PROPOSED CPT
CPTM-##	PROPOSED CPT MAGNETOMETER
	RED LINE BOUNDARY
	SAFEGUARDED AREA

0 125 250 m

SCALE 1:5,000

ISSUE/REVISION

I/R	DATE	DESCRIPTION
P05	2017-10-18	LOCATIONS AMENDED
P04	2017-08-30	EARLY RELEASE LAND HOLES ADDED BH01&2 MOVED NORTH 3 X TP'S ADDED
P03	2017-07-11	EARLY RELEASE LAND HOLES ADDED
P02	2017-03-13	RED LINE BOUNDARY REVISED
P01	2016-11-22	PRELIMINARY ISSUE

CLIENT

South Tyneside Council

Sunderland City Council

PROJECT

INTERNATIONAL ADVANCED MANUFACTURING PARK

SHEET TITLE

PROPOSED EXPLORATORY HOLE LOCATION PLAN

CONSULTANT

AECOM
One Trinity Gardens
Newcastle upon Tyne
0191 224 6500 tel 0191 224 6599 fax
www.aecom.com

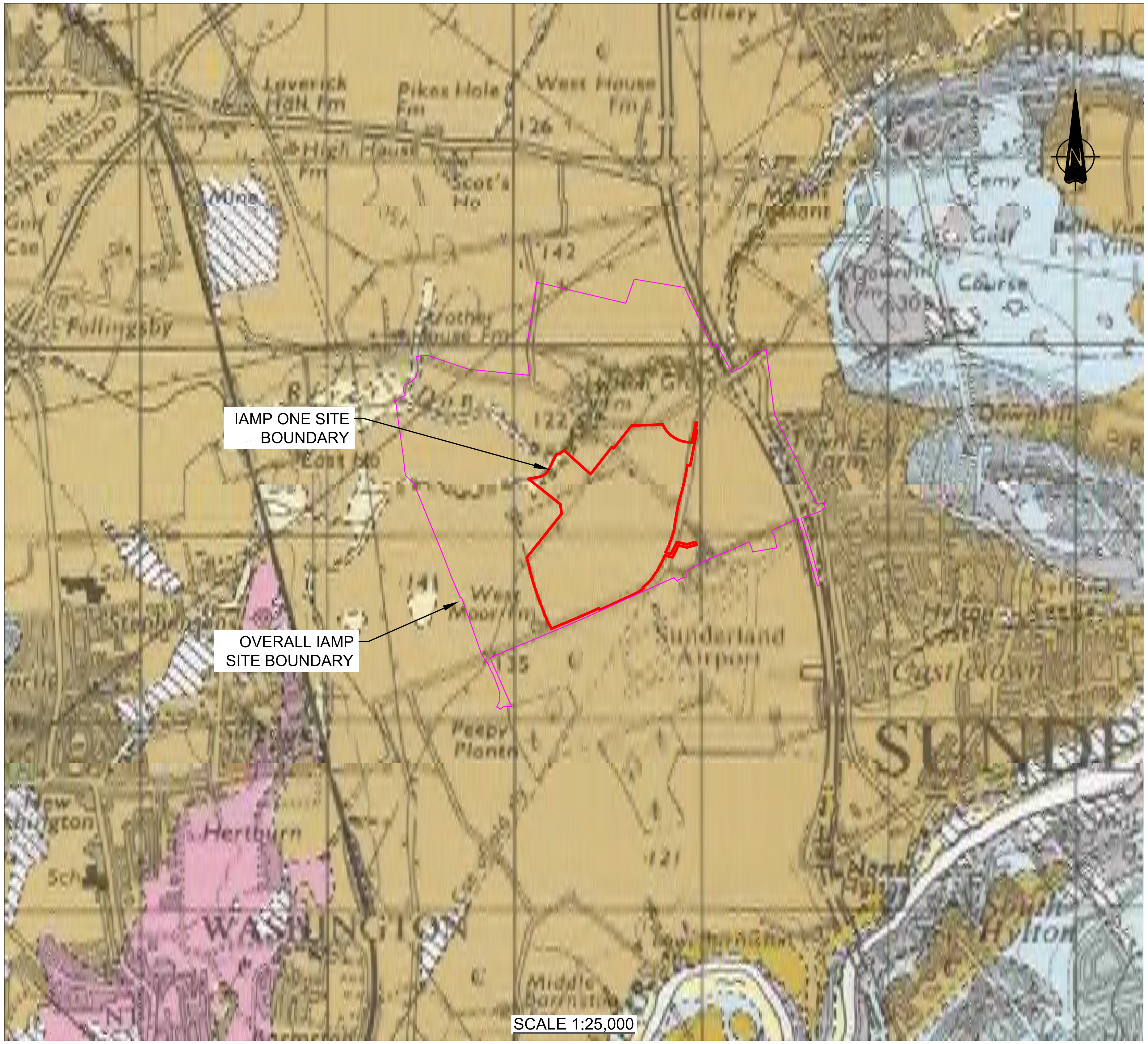
SHEET NUMBER

60283414_M015_GEO_DR_002 P05

Printed on ___% Post-Consumer Recycled Content Paper

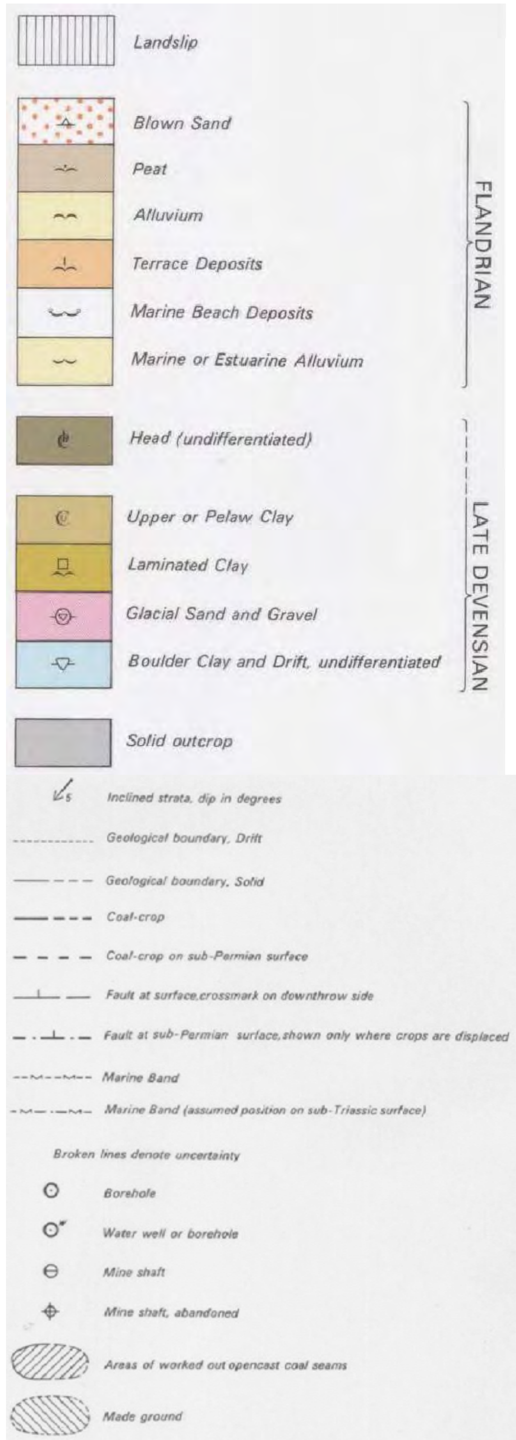
Filename: F:\PROJECTS\MUNICIPAL INFRASTRUCTURE - NEPO FRAMEWORK LOT 6\M015 SCC IAMP GEOTECH ADVISOR\AUTOCAD\005 - BGS 50K MAP - DRIFT_P001.DWG Last saved by: CARTERCM Last Plotted: 4/18/2018 12:41 PM

Project Management Initials: Designer: RPA Checked: CMC Approved: SDM ISO A3 297mm x 420mm



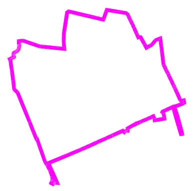
BASED UPON 1:50,000 MAP SHEET 21 - SUNDERLAND (SOLID WITH DRIFT, 1978), WITH THE PERMISSION OF THE BRITISH GEOLOGICAL SURVEY

GEOLOGICAL MAP KEY



KEY

IAMP ONE SITE BOUNDARY



OVERALL IAMP SITE BOUNDARY

INTERNATIONAL ADVANCED MANUFACTURING PARK

Geological Map - Drift

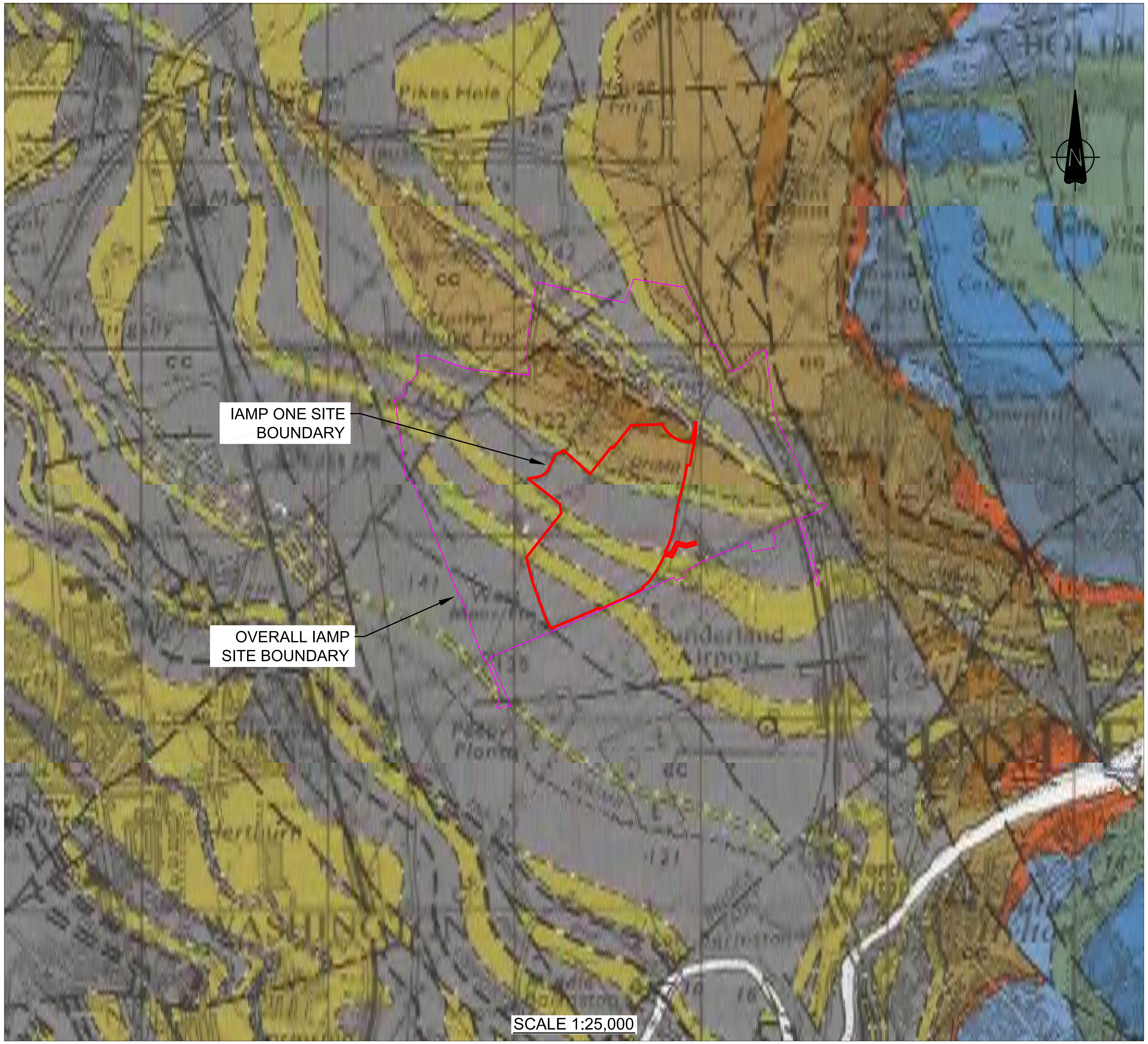
AECOM

Sunderland City Council

South Tyneside Council

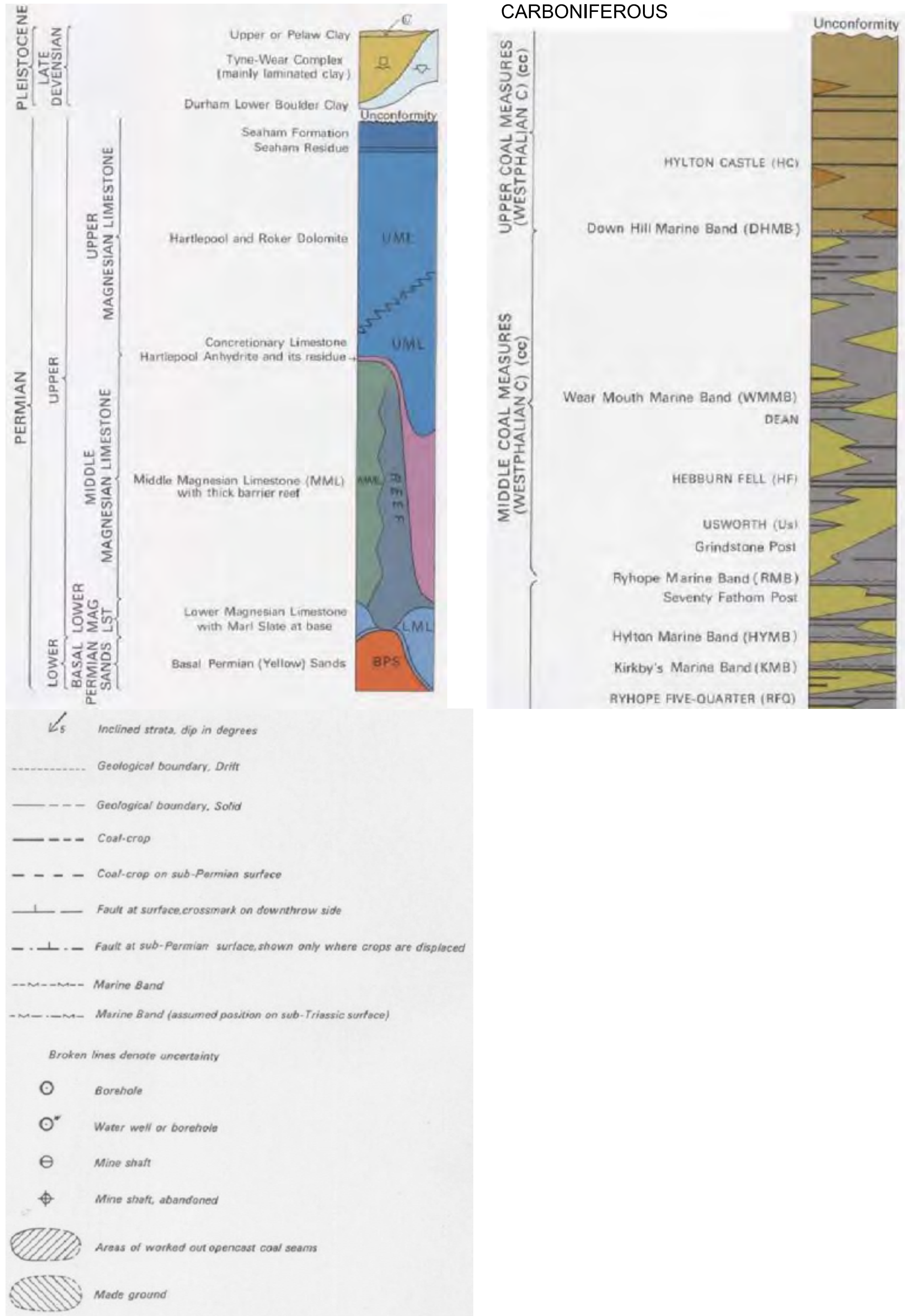
Drawing Number:
60283414_M015_GEO_DR_005_P001

Project No: 60283414
Date: April 2018



BASED UPON 1:50,000 MAP SHEET 21 - SUNDERLAND (SOLID WITH DRIFT, 1978), WITH THE PERMISSION OF THE BRITISH GEOLOGICAL SURVEY

GEOLOGICAL MAP KEY



KEY

IAMP ONE SITE BOUNDARY

OVERALL IAMP SITE BOUNDARY

INTERNATIONAL ADVANCED MANUFACTURING PARK

Geological Map - Solid

AECOM

Sunderland City Council

South Tyneside Council

Drawing Number:
60283414_M015_GEO_DR_006_P001

Project No: 60283414
Date: April 2018