M54 to M6 Link Road
TR010054
Volume 6
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Appendix 9.1: Ground Investigation Report

Regulation 5(2)(a)

Planning Act 2008

Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

January 2020

Infrastructure Planning

Planning Act 2008

The Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009

M54 to M6 Link Road
Development Consent Order 202[ ]

6.3 Environmental Statement Appendices
Appendix 9.1 Ground Investigation Report

Regulation Number	Regulation 5(2)(a)
Planning Inspectorate Scheme Reference	TR010054
Application Document Reference	6.3
Author	M54 to M6 Link Road Project Team and Highways England

Version	Date	Status of Version
P03	January 2020	S8

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1. Executive Summary

1.1.1 The M54-M6 Link Road Scheme (the Scheme) will involve the modification of the M54 J1 and M6 J11 roundabouts, the construction of seven new bridge structures, including the New M54 J1 Overbridge, Featherstone Overbridge, Hilton Lane Overbridge and two new bridges above the M6 at J11. The Scheme will also require several retaining walls and a series of substantial cutting and embankment earthworks.

1.1.2 This Ground Investigation Report builds upon the Preliminary Sources Study, assimilating and interpreting historical information together with the findings of the 2019 ground investigation, which comprised boreholes, trial pits and infiltration testing along the proposed route of the Scheme.

1.1.3 Geological plans, geotechnical and geo-environmental constraint plans, and long section drawings are presented, showing all exploratory hole locations and summarising key geological features encountered.

1.1.4 The drift geology of the Scheme comprises Made Ground deposits (which can be divided into engineered fill and colliery spoil related fills), Alluvial deposits (mainly granular) and Glacial Till deposits. The underlying solid geology is composed mainly of Sandstone (with a thick highly weathered sandstone horizon) and, to a lesser extent, of mudstone and siltstone.

1.1.5 Sections of the alignment run along the edge of drift deposits (Glacial Till) overlying the solid geology. The Glacial Tills are undifferentiated in the geological maps, but investigation has shown them to be divided into cohesive Glacial Till and Glacial Sand and Gravel, which in some cases form deep glacial channels

1.1.6 The in-situ tests, laboratory tests undertaken on samples obtained during the ground investigation, published data for known strata and the existing data from historic investigations, have been interpreted to derive soil and rock parameters ranges suitable to use in the subsequent detailed design stage.

1.1.7 Geo-environmental information has been interpreted and presented with recommended considerations regarding contamination of the ground and of both surface water and groundwater.

1.1.8 The geotechnical risk register, previously presented in the Preliminary Sources Study Report subsequent addendum (ATKINS, 2008 and 2015) has been updated to account for the new information gathered during the ground investigation.

2. Introduction

2.1 Scope and Objectives

2.1.1 This Ground Investigation Report (GIR) relates to the M54 to M6 Link Road (the Scheme).

2.1.2 This report has been prepared in accordance with the requirements of the *Design Manual for Roads and Bridges, Vol. 4, Section 1, Part 2 HD22/08, Managing Geotechnical Risk.* The new standard *CD622 Managing Geotechnical Risk* was reviewed but it was agreed that it should not be implemented for this GIR. A summary of the report objectives based on the requirements of *HD22/08* are provided below:

- Summarise and update existing geotechnical and geo-environmental desk study information and to highlight the implications of this information to the Scheme.
- Describe the results of the 2019 Ground Investigation in relation to the Scheme.
- Present a summary interpretation on the geography, geology, hydrogeology, geomorphology, man-made features and historical development of the Scheme.
- Present an interpretation of the ground conditions along the Scheme route including descriptions of materials and justification for the parameters adopted for geotechnical design.
- Present details of geotechnical and geo-environmental risks and mitigation measures in the form of a geotechnical risk register.
- Provide details of interpretation and justification for the geotechnical design criteria adopted.
- Characterise the geochemical conditions encountered along the route of the proposed Scheme;
- Interpret chemical analysis data and undertake Tier 2 (Environment Agency, 2019) risk assessments for human health, groundwater and the wider environment in the context of the proposed development.
- Input geo-environmental risks to the geotechnical risk register.
- Identify outline requirements for remedial works to mitigate significant contamination risks along the route of the proposed Scheme.

2.2 Project Description

2.2.1 A Scheme location and description can be found in Chapter 2 of the Environmental Statement [TR010054/APP/6.3].

2.2.2 In total, the Scheme will comprise the modification of the M54 J1 and M6 J11 roundabouts, the construction of seven new bridge structures, multiple retaining walls, and a series of substantial cutting and embankment earthworks. A summary of the main engineering works and their locations are shown in Table 2.2.1.

Table 2.2.1: Summary of Proposed Geotechnical Engineering Works

Engineering Works	Location
M54 J1 Overbridge	Ch. 1530
Featherstone Overbridge	Ch. 1800
Hilton Lane Overbridge	Ch. 2720
Accommodation Overbridge	Ch. 3050
Latherford Brook Structure	Ch. 3750
M6 J11 Northern Bridge	M6 J11
M6 J11 Southern Bridge	M6 J11
Retaining Walls	Predominantly Ch. 500 to Ch. 1370, Ch. 1300 to Ch. 1450 and the Dark Lane Lower Pool Ponds at Ch. 2300.
Earthwork Cuttings	Widespread across the *Scheme.* Significant cuttings from Ch. 2500 to Ch. 3600.
Earthwork Embankments	Widespread across the *Scheme.* Significant embankments at the Featherstone Overbridge and Ch. 3600 to M6 J11.
Road Gantries	Generally clustered around the M54 motorway and M6 J11 roundabout.
Culverts	There are three culverts, at Ch. 1720, Ch. 2245 and Ch. 3220.

Note: Chainages noted in Table 2.2.1 and throughout this report refer to the main scheme alignment, which is highlighted on Geological Long section drawings (HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1019 and HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1012 to 1015.)

2.3 Geotechnical Category of Project

2.3.1 The proposed Scheme is classified under HD22/08 as being within the parameters of Category 2.

3. Existing Information

3.1 Introduction

3.1.1 The basis for most of the existing information for the Scheme has been obtained from previous Preliminary Sources Study Reports (PSSR). A summary of the existing PSSRs relevant to the current route alignment are highlighted in Table 3.1.1 below.

Table 3.1.1: Summary of Preliminary Sources Study Reports

PSSR Title	Originator	Date Issued	Document Reference	Description of Report Content
Geotechnical Preliminary Sources of Study	Atkins Ltd	October 2002	HA069/001/0068211, HA GDMS No. 22495	The report reviews six potential route corridors cast wide across the M54, M6 and Featherstone area, covering a 25km square area. Two of the route corridors follow a similar path as the alignment currently proposed.
Geotechnical Preliminary Sources Study Report	Atkins Ltd	December 2008	5049906/52/021, HA GDMS No. 22372	The six potential route corridors have been reduced to one corridor containing three route alignments. The three alignments cover the general area currently proposed for development.
Preliminary Sources Study Report Addendum	Atkins Ltd	April 2015	M54M6-ATK-0000-ZZ00-RP-C-0001 P02	The 2015 PSSR Addendum expands upon the previous route alignments but with minor variations. The report notes that as the changes are minor, the PSSR would only review notable new features and risks.

3.1.2 Most of the existing information used in this GIR has been sourced from the 2008 Atkins PSSR (hereafter referred to as the PSSR). This PSSR contains the most comprehensive and relevant sources of information for the alignment assessed in this report. The information given in the above reports are still considered to be current with additional information provided below. A summary of the key findings of the PSSR are presented in the following sections.

3.2 Topographical Maps

3.2.1 The PSSR obtained the following topographical maps for the *Scheme* area:

- 1:50,000 scale OS Landranger Map Sheet 127, Stafford and Telford
- 1:25,000 scale OS Explorer Map Sheet 257 – Crewe and Nantwich, Whitchurch and Tattenhall (paper format)
- 1:10,000 Map Ref SJ90NW, SJ90SE and SJ90SW

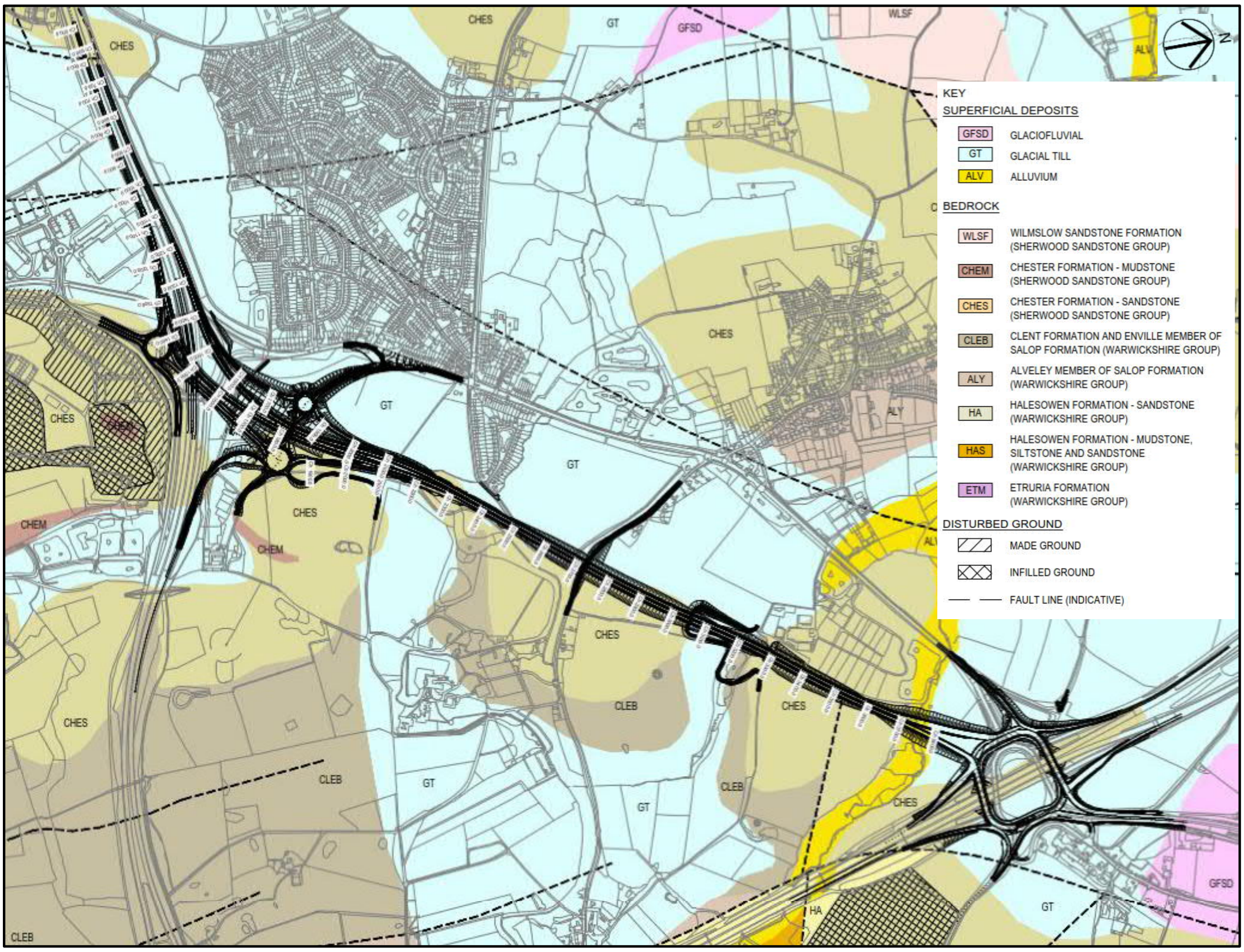
3.2.2 An interpretation of these maps, aerial photographs and site walkover information is included in the PSSR and are considered appropriate for this stage of the works.

3.3 Geological Maps

3.3.1 A detailed assessment of the geological maps of the Scheme area are included in the PSSR. A summary of the findings that are pertinent to this report are outlined below.

3.3.2 The general site geology was obtained from published British Geological Survey (BGS) Solid and Drift Geology Maps, Sheet 153, Wolverhampton, 1:50,000 scale and the BGS online Geological Map Viewer. The Geological Map Viewer has been used to form the Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013). An extract of the drawing is shown in Figure 3.3-1 and included in Appendix A to this GIR.

Figure 3.3-1: Extract of Scheme Geology Plan and Unit Key



3.3.3 As can be seen in Figure 3.3-1, sections of the alignment run a zone of discontinuous drift deposits (Glacial Deposits) overlying the solid geology with rockhead assumed to dip from south to north. In some cases, this has the potential to result in significant ground profile contrasts between abutments of the same bridge structures.

3.3.4 The distribution of rock outcrops and overlying Glacial Deposits shown on the maps suggest that glacial channels may be present.

3.4 Aerial Photographs

3.4.1 Historical aerial photographs were obtained as part of the 2002 Atkins PSSR from the English Heritage National Monuments Records Centre's Aerial Photo Collection and are dated from 1946 to 1948. Additional aerial photographs from the 1990s were obtained from the Highways Agency as part of the PSSR. No further photographs have been acquired to produce this GIR. The PSSR should be consulted for interpretation on the available aerial photographs.

3.5 Records of Mines and Mineral Deposits

3.5.1 Information relating to the historical mining within the Scheme area was purchased from the Coal Authority as part of the PSSR 2002 and was used in the PSSR.

3.5.2 The records show three mine shafts located to the south of M54 J1. Two of these shafts were confirmed by the Coal Authority Interactive Map Viewer (accessed: 27/11/19) associated with the former Hilton Main colliery. One mine shaft (Reference: 394304-001) is 569m in depth and the second (Reference: 394304-002) is 583m in depth. Both mine shafts are described as having been treated, the depths to which the shafts have been backfilled is unknown.

3.5.3 The PSSR 2002 states that coal seams worked directly underneath the site are restricted to the southern section of the site between Hilton Lane and the M54 J1. The Interactive Map Viewer also indicates areas of underground working located above Hilton Lane up to (just south of) the Brookfield farm area.

3.5.4 Mine works have not been considered a hazard for the project due to the following data:

- According to the PSSR information, the Coal Authority records indicate that manmade voids associated with historic mining are considered a possibility within the southern to central portion of the Scheme. The old mine workings are related to mining of Carboniferous Coal Measures; hence risk for coal mining related subsidence and voids should be considered wherever Coal Measures are found close to the surface. However, the mining records indicate that the Coal Measures seams are at a minimum depth of 250m below the Scheme and that the last working date was in 1969. As such, subsidence from the presence of collapsing mine workings is not considered to be a particular constraint to the Scheme.
- According to the coal Authority Interactive Map Viewer, the scheme is not within a Development High Risk Area.
- There is no identified hazard from shallow mining within the Scheme.

3.5.5 Based on the above information, further consultations with the Coal Authority is deemed not to be necessary. Further details of the historical mining in the area are included in the PSSR 2002 and PSSR.

3.6 Land Use and Soil Survey Information

3.6.1 Ordinance Survey (OS) maps assessed as part of the PSSR noted that land use within the Scheme area is predominantly agricultural with areas of residential. The M54 is located at the south-western end of the Scheme running east to west/west to east. At the other end of the Scheme, the new link road will connect to the M6 which runs north to south/south to north.

3.6.2 During the production of the PSSR, an extract of the National Soil Map from the Soil Survey and Land Research Centre was previewed as a method of assessing the soil survey information for the Scheme. After assessing the extract, the report determined that it did not provide adequate value and therefore did not purchase the full document. A summary of the historical land use at the Scheme is presented in Table 3.6.1.

Table 3.6.1: Summary of Historical Land Use

Date	Scale	Onsite	Offsite (Outside Site Boundary)
1884 – 1886	1:10,560	Most of the proposed route crosses *agricultural land* with areas of *deciduous wood* through the centre of the site. Dark Lane, Hilton Lane are present onsite along with several other unnamed roads.	The surrounding area predominantly comprises of *agricultural land*. *Hilton Hall* is present just east of the centre of the boundary (within 250m), and *Home Farm* is located about 200m below that. A small residential plot is located northwest of the site (within 500m) in Shareshill.
1902	1:10,560	No significant change.	A *Gravel Pit* is present about 250m from the Site boundary between Hilton hall and Home farm. Another is present approximately 700m from the Site boundary near the M6 junction 11. An Old Quarry is noted within 500m east of the Site boundary near Home farm.
1924	1:10,560	*Sewage Filter Beds* noted at the Site boundary in the Lower Pool area. An old *Clay Pit* is located about 250m south-west of this.	South of the M54 J1 a *Gravel Pit, Reservoir Shaft, Pumping Shaft, Tramway* and *Hilton Main Colliery* are noted. *Home Farm* now noted as *Hilton Tower House.*
1938	1:10,560	No significant change.	No significant change.
1954 – 1955	1:10,000	No significant change.	*Old Clay Pit* and *Gravel Pit* no longer noted on map. *Mineral Railway* running from the south into the Hilton Main Colliery.
1966 – 1968	1:10,000	No significant change.	*Brookfield farm (Brookfield Leisure Centre)* located north of site at the western boundary. Small *Residential Plot* on Dark Lane.
1972 – 1973	1:10,000	M6 J11 is present.	M6 Motorway has been constructed.
1980 – 1983	1:10,000	No significant change.	Residential housing in Featherstone is more populated. Hilton Main Colliery noted as 'Disused Workings'. Large *Sand & Gravel Pit* and *Clay Pit* located just south of this. Area north-west of sand pit is noted as a *Timber Yard.* Riding stables just south of Hilton Lane within 100m of site boundary. Mineral Railway marked as 'dismantled'.
1989	1:10,000	M54 Junction 1 is present.	Half of the disused workings are marked as a *Coal Depot*. *Hilton Main Industrial Estate* noted as the former Timber Yard. Unspecified *Works* noted just south of this.
1999 – 2000	1:10,000	No significant change.	No significant change.
2006	1:10,000	*Fishing Ponds* located next to (west of) Brookfield Farm.	*Fishing Ponds* located next to (east and west of) Brookfield Farm.
2017	1:10,000	No significant change.	No significant change.

Source: Envirocheck ref: 120340012_1_1

3.6.3 No additional land use information has been obtained since the production of the PSSR.

3.7 Archaeological and Historical Investigations

3.7.1 No archaeological or historical investigations were undertaken or reported on in previous PSSR's.

3.8 Existing Ground Investigations

3.8.1 Historical exploratory holes from multiple sources have been used as a preliminary source of geotechnical data, to prepare the GI proposal and support the findings of the 2019 Ground Investigation. These are predominantly located under the M54 and M6 carriageways and were drilled in the early 1970's and mid-1990's, respectively. A summary of the historic exploratory holes considered for this GIR are shown in Table 3.8.1. The locations of the historic boreholes are included on the Ground Investigation Location Plans HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1009 to 1016 in Appendix A to this GIR.

Table 3.8.1: Summary of Historic Boreholes

Exploratory Hole ID	Exploratory Hole BGS Reference	Date Drilled	Approximate Chainage (m)	Ground Level (m AOD)	Depth Drilled (m)
BH 378	SJ90SW204	23/03/1979	490	124.25	8.0
BH 180	SJ90SW68	03/04/1971	560	125.84	8.0
BH 183	SJ90SW69	11/06/1971	650	123.45	7.01
BH 184	SJ90SW71	09/06/1971	780	123.79	7.01
BH 185	SJ90SW78	05/06/1971	1050	128.42	10.1
BH 185B	SJ90SW84	17/06/1971	1210	130.91	6.09
BH 186	SJ90SW85	10/06/1971	1345	132.96	10.0
BH 188	SJ90SW90	11/06/1971	1370	134.03	10.0
BH 187	SJ90SW87	09/06/1971	1380	130.24	12.0
BH 189	SJ90SW93	09/06/1971	1480	143.12	17.22
BH 191	SJ90SW95	17/04/1971	1490	143.49	16.0
BH 190	SJ90SW94	09/06/1971	1510	138.62	10.1
BH 383	SJ90SW206	Unknown	1550	138.90	4.0
B/1000	SJ90SW238	14/01/1987	1590	135.80	15.5
B/1000A	SJ90SW239	16/01/1987	1800	147.52	20.5
B/1001	SJ90SW240	24/01/1987	1890	146.37	15.0
B/1003	SJ90NW58	28/01/1987	2110	146.11	18.0
B/3000	SJ90NE217	04/08/1987	2720	143.76	20.3
B/3001	SJ90NE218	20/08/1987	2725	145.00	20.0
B/1004	SJ90NE305	27/01/1987	2740	144.10	20.0
B/2	SJ90NE292	30/10/1986	2960	143.23	14.0
B/1005	SJ90NE306	30/01/1987	2970	141.60	15.0
B/1005A	SJ90NE307	14/01/1987	3240	127.95	7.61
3000B	SJ90SW233	25/06/1987	1850	Unknown	14.25
3000A	SJ90SW232	29/06/1987	1860	Unknown	13.9
BH 88	SJ90NE154	Unknown	4050	132.86	10.06

3.9 Consultation with Statutory Bodies and Agencies

3.9.1 The PSSR accessed the Multi-Agency Geographic Information for the Countryside (MAGIC) website to review information from the following bodies:

- Department of the Environment, Food and Rural Affairs (DEFRA)
- Countryside Agency
- English Heritage
- English Nature
- Environment Agency (EA)
- Forestry Commission
- Office of the Deputy Prime Minister
- Rural Development Service
- Coal Authority

3.9.2 The information discussed in the PSSR has been considered relevant to this Scheme.

3.10 Flood Records

3.10.1 The Flood Risk Assessment [TR010054/APP/7.1] conducted for M54-M6 Link Road was based on publicly available information including the South Staffordshire Council Level 1 Strategic Flood Risk Assessment and Environment Agency Interactive Flood Maps (online), Environment Agency Flood Maps for Planning (online) and models analysed in the Hydrologic Engineering Centre – River Analysis System (HEC-RAS).

3.10.2 According to the Flood Risk Assessment [TR010054/APP/7.1], the existing fluvial flood risk in the Scheme area is primarily ‘low’ with the area surrounding Latherford Brook in the northern portion of site as having a ‘medium’ to ‘high’ fluvial flood risk. The Scheme is not at risk of tidal flooding. There are also numerous pockets of risk along the Scheme alignment, often where there are existing ponds or troughs in the topography. Existing flood risk to site from artificial sources, such as canals, lakes and reservoirs, is considered as low. Existing flood risk from groundwater, sewers and drains is also considered to be low.

3.10.3 Regarding the flooding risk mitigation measures, the Flood Risk Assessment states that the impact on surface water flooding mechanisms due to the Scheme is low, provided all the overland surface water runoff and highway drainage generated by the Scheme is captured and attenuated in the proposed drainage network. Sustainable drainage systems such as ponds have been designed to accommodate a 1% AEP storm event with 40% allowance for climate change as per the requirements of Staffordshire County Council’s Flood Risk Management team. Discharge from the ponds would be at greenfield runoff rates to nearby watercourses. Surface water flows from areas upstream of the Scheme would be managed via interception gullies and drainage channels.

3.11 Contaminated Land

3.11.1 An assessment of the potential geo-environmental and contaminated land issues was undertaken in the PSSR, using information obtained from EnviroCheck Reports. Details of the key constraints are presented on the Geotech. And Geo-Env. Constraints Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) in Appendix A to this report and within the PSSR.

3.11.2 The areas of key concern pertinent to the Scheme are highlighted in Section 4.8 of the PSSR and include discharge consents, pollution incidents to controlled water, registered landfill sites and fuel station entries. The main concerns outlined in Chapter 9 of the Environmental Statement [TR010054/APP/6.1], are three landfills within 500m of the Scheme and the historic coal mining in the area. In addition to this, the Conceptual Site Model highlights the potential contaminative legacy of the sites previous agricultural use.

3.11.3 Historical land uses with potentially contaminative effect are addressed in Section 7.1 of the PSSR and include backfilled pits at various point on and around the Scheme, historic sewage filter beds present along the Scheme boundary and the abandoned mineral railway that served the former Hilton Main Colliery south of the M54 J1, just outside the Order limits.

3.11.4 The Geotech and Geo-Env Constraints Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) has been updated based on the findings of the 2019 Ground Investigation and is included in Appendix A of this report.

3.12 Additional Information

3.12.1 Highways England's Geotechnical Data Management System (HAGDMS) has been consulted to check the exiting slope defects, recorded in the HAGDMS reports, for the M54 and M6 roads. The Geotechnical Asset Management Plan (GeoAMP) (Ref: 29777), that outlines the geotechnical condition of the Area 9 trunk road network has also been consulted.

3.12.2 The findings of the geotechnical Inspections undertaken on the slopes in the vicinity of the M54 J1 and M6 J11 roundabouts are summarised in Table 3.12.1 below and are shown on the constraints drawing (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) in Appendix A.

Table 3.12.1: Summary of earthworks inspections (M54 and M6 Roads)

Observation	Date	Unique Ref	Characteristics	Length (m)	HD41 Class	HD41 Feature Grade	Location
580191	13/03/2017	9_M54_34333_580191	Slip Ravelling Animal burrowing Geotech.Measure: Concrete R. Wall	155	1D	3	M54 WB Whitgreaves Wood
587946	17/01/2018	9_M54_34324_587946	Slip Slope bulge	14	1D	3	M54 J1 WB
587542	16/01/2018	9_M54_35828_587542	Slip Terracing	6	1D	3	M54 J1 EB
587545	16/01/2018	9_M54_35841_587545	Terracing	73	2	2	M54 J1 EB
587507	16/01/2018	9_M54_35758_587507	Terracing	147	2	2	M54 J1 EB
587504	16/01/2018	9_M54_35756_587504	Subsidence	2	2	2	M54 EB
520920	30/01/2013	9_M6_50035_520920	Slip	10	1D	3	M6 J11

4. Field and Laboratory Studies

4.1 Walkover Survey

4.1.1 A Scheme walkover has not been undertaken specifically for the preparation of this report. A Scheme walkover was previously undertaken for the PSSR. The findings of this walkover have been verified during visits to the site as part of the 2019 Ground Investigation scoping and planning. A summary of previous Scheme walkover findings can be found in the PSSR.

4.2 Geomorphological / Geological Mapping

4.2.1 Outline geomorphological mapping at a scale of 1:10,000 was undertaken for the PSSR with the aim of identifying the general landforms and areas of marshy of otherwise poorer ground conditions. The outcome of the geomorphological mapping exercise along with comments from the site walkover are detailed on drawings 5049906/GT/PSSR/020 and 5049906/GT/PSSR/021 of the PSSR.

4.2.2 Based on the review of existing information and findings from 2019 Ground Investigation, a Geological Long Section for the main alignment has been created for the Scheme. The Geological Long sections (HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1019 and HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1012 to 1015) presented in Appendix A show a profile of all the historical and recent boreholes and an interpreted geological profile along the Scheme.

4.3 Ground Investigations

4.3.1 Ground Investigation works were carried out by BAM Ritchies on behalf of BAM Nuttall Limited between the 17th June 2019 and 20th August 2019. The works were completed under the Early Order DIP Agreement.

4.3.2 Historical boreholes from various sources have been used to support the findings of the 2019 Ground Investigation. Details of these historical boreholes are outlined in Section 3.8 and included in drawings HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1009 to HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1016, which are included in Appendix A.

Description of Field Work

4.3.3 The scope of the 2019 Ground Investigation comprised:

Table 4.3.1: Summary of the 2019 Ground Investigation Works

Type of Investigation	Quantity	Maximum Depth Achieved (m bgl)
Cable Percussive Boreholes	34	22.8
Rotary Core Boreholes	23	30.6
Trial Pits	19	4.5

4.3.4 The details and locations of all exploratory holes carried out as part of the 2019 Ground Investigation are shown in Table 4.3.2,

4.3.5 Table 4.3.3 and drawings HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1009 to HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1016 which are included in Appendix A.

Table 4.3.2: Summary of 2019 Ground Investigation Boreholes

Exploratory Hole ID	Exploratory Hole Type	Depth Drilled (m)	Date Started	Date Completed	Coordinates		Ground Level (m AOD)
					Eastings	Northings	
BH01	CP	10.0	03/07/2019	05/07/2019	393459.4	304562.1	124.80
BH02	CP	15.0	04/07/2019	07/07/2019	393651.0	304634.7	128.83
BH03	CP	8.0	17/07/2019	18/07/2019	394401.2	304838.0	140.78
BH04	CP+RC	30.0	02/07/2019	08/07/2019	394146.2	304629.5	135.79
BH05	CP+RC	15.2	01/07/2019	10/07/2019	394153.0	304583.1	134.33
BH06	CP+RC	30.0	18/06/2019	25/06/2019	394180.8	304766.7	134.51
BH07	CP+RC	30.0	26/06/2019	05/07/2019	394273.3	304730.2	137.47
BH08	CP+RC	25.0	18/06/2019	20/06/2019	394467.4	304907.0	141.98
BH08A	CP+RC+RO	28.2	16/07/2019	07/07/2019	394464.7	304909.2	142.17
BH09	CP+RC	30.5	19/06/2019	26/06/2019	394431.9	304933.3	141.36
BH10	CP+RC	15.0	28/06/2019	09/07/2019	394334.0	304992.4	136.75
BH11	CP+RC	15.0	05/07/2019	09/07/2019	394530.9	305133.4	138.28
BH12	CP+RC	15.1	24/07/2019	01/08/2019	394658.0	305236.6	139.79
BH13	CP+RC	15.0	15/07/2019	05/08/2019	394851.5	305477.6	140.16
BH14	CP+RC	30.5	09/07/2019	16/07/2019	394932.7	305628.6	144.20
BH15	CP	30.0	10/07/2019	17/07/2019	394938.3	305691.8	140.75
BH16	CP+RC	30.0	18/07/2019	20/08/2019	394983.0	305657.3	142.46
BH17	CP+RC	30.1	11/07/2019	17/07/2019	395173.3	305910.9	138.84
BH18	CP+RC	30.0	16/07/2019	18/07/2019	395129.1	305940.0	137.82
BH19	CP+RC	13.5	19/07/2019	16/08/2019	395242.1	306038.5	130.90
BH20	CP+RO	20.0	17/07/2019	19/07/2019	395371.4	306189.0	139.54
BH20A	CP	20.0	20/07/2019	22/07/2019	395369.3	306186.8	139.55
BH21	CP+RC	30.0	23/07/2019	14/08/2019	395485.4	306456.9	125.65
BH22	CP	5.5	17/07/2019	03/08/2019	395563.9	306501.7	124.62
BH22A	CP+RC	30.0	23/07/2019	07/08/2019	395563.5	306506.8	124.54
BH23	CP	15.0	15/07/2019	16/07/2019	395633.1	306654.1	130.81
BH24	CP	10.0	11/07/2019	15/07/2019	395388.1	306651.4	125.67
BH25	CP	22.8	02/07/2019	04/07/2019	395607.3	306756.5	130.79
BH26	CP+RC	30.2	07/07/2019	03/08/2019	395721.4	306664.6	137.04
BH27	WLS+RC	30.5	17/07/2019	23/07/2019	395675.61	306814.7	136.30
BH28	CP	1.2	10/07/2019	11/07/2019	395825.3	306646.3	136.98
BH28A	WLS+RC	30.3	11/07/2019	31/07/2019	395816.0	306661.3	137.03
BH29	CP+RC	30.6	09/07/2019	17/07/2019	395756.4	306839.8	136.26
BH30	CP	4.45	18/07/2019	18/07/2019	394323.1	304878.0	134.99

Note: CP – Cable Percussive
RC – Rotary Coring
RO – Rotary Open hole Coring
WLS – Windowless Sampling

Table 4.3.3: Summary of 2019 Ground Investigation Trial Pits

Exploratory Hole ID	Exploratory Hole Type	Depth Drilled (m)	Date Started	Date Completed	Coordinates		Ground Level (m AOD)
					Eastings	Northings	
TP01	TP	1.4	02/07/2019	02/07/2019	394503.4	304709.1	143.71
TP02	TP	4.5	03/07/2019	03/07/2019	394318.3	304852.0	136.92
TP03	TP	4.5	02/07/2019	02/07/2019	394390.9	304821.2	140.03
TP04	TP	4.5	11/07/2019	11/07/2019	394400.1	304723.2	138.94
TP05	TP	2.9	01/07/2019	01/07/2019	394503.1	304819.8	140.24
TP06	TP	4.5	03/07/2019	03/07/2019	394471.4	304965.2	142.09
TP07	TP	4.5	03/07/2019	03/07/2019	394332.7	305079.9	135.42
TP08	TP	4.5	05/07/2019	05/07/2019	394567	305012.4	145.12
TP09	TP	2.5	05/07/2019	08/07/2019	394395.0	305314.7	134.12
TP10	TP	2.0	05/07/2019	05/07/2019	394603.7	305263.0	137.33
TP11	TP	3.9	11/07/2019	11/07/2019	394773.3	305414.6	137.57
TP12	TP	4.5	09/07/2019	09/07/2019	394892.5	305589.4	141.59
TP13	TP	4.5	09/07/2019	09/07/2019	395082.5	305821.9	142.35
TP14	TP	2.5	10/07/2019	10/07/2019	395145.5	306085.2	130.83
TP15	TP	4.5	09/07/2019	09/07/2019	395313.2	306131.6	137.35
TP16	TP	4.5	10/07/2019	10/07/2019	395433.6	306302.2	135.65
TP17	TP	2.5	10/07/2019	10/07/2019	395545.6	306520.6	124.41
TP18	TP	2.5	11/07/2019	11/07/2019	395468.7	306600.3	124.58
TP19	TP	4.5	10/07/2019	10/07/2019	395602.1	306586.0	127.89

***Note**: TP – Trial Pit*

Ground Investigation Report

4.3.6 The Ground Investigation Report (HE514465-BAM-EGT-ZZ-RP-WM-0001) from BAM Ritchies can be provided if required.

In-Situ Test Results

4.3.7 The following in-situ tests were undertaken as part of the 2019 Ground Investigation:

- Standard Penetration Tests (SPT)
- Hand Shear Vane Tests
- Pocket Penetrometer Tests
- Falling Head Tests
- Soakaway Tests

4.3.8 The in-situ test results from the 2019 Ground Investigation discussed in Section 6 and are presented in the Ground Investigation Report (HE514465-BAM-EGT-ZZ-RP-WM-0001).

4.4 Drainage Studies

4.4.1 Preliminary drainage studies have been assessed as part of the PSSR.

4.4.2 In the Drainage Strategy provided in Appendix 13.2 of the Environmental Statement [TR010054/APP/6.3], it is stated that existing drainage information is limited; additional surveys were completed (between June and August 2019) to further understand the existing drainage infrastructure and to confirm the assumptions. CCTV, connectivity and level surveys were undertaken on all drainage features. The initial results indicate the existing drainage is in poor condition and would need replacement / upgrade. To date nothing has been identified in the surveys that would change the approach described in the drainage strategy.

4.5 Geophysical Surveys

4.5.1 Not used

4.6 Pile Tests

4.6.1 Not used

4.7 Other Fieldwork

Archaeology

4.7.1 In July 2019 ADAS undertook an archaeological watching brief during the 2019 Ground Investigation. This was undertaken during excavation of the 19No. geotechnical trial pits positioned along the Scheme alignment. ADAS noted that the proposed works for the Scheme are in an area which is known to contain archaeological material ranging from the Neolithic to the late Iron Age and Romano-British Period. Despite being in an area of rich archaeological potential, the survey found no archaeological features or artefacts during monitoring of the trial pits. The ADAS Archaeological Report is presented in Appendix 6.2 to the Environmental Statement [TR010054/APP/6.3].

Unexploded Ordnance (UXO)

4.7.2 A Detailed Unexploded Ordnance (UXO) Threat Assessment was undertaken by Alpha Associates in January 2018 and showed that a portion of the M54 carriageway is within a “very high” UXO risk area. The M54 carriageway west of Ch. 800 is classified as having a very high risk of encountering UXO’s due to its proximity to a historic Royal Ordnance Factory (ROF). Records indicate that the now abandoned ROF near Featherstone was used as a filling factory for munitions of bombs, shells and cartridges. BH01 is within this risk area and therefore was monitored by an UXO specialist during the 2019 Ground Investigation and found no traces of UXOs. The remainder of the Scheme is classified as a “low” UXO risk. The UXO Safety Sign-Off Certificate from the monitoring works is included in Appendix 10 of the Ground Investigation Factual Report (HE514465-BAM-EGT-ZZ-RP-WM-0001). The very high UXO risk area is shown in the Constraints Plan in Appendix A to this report (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014)

Post Ground Investigation Monitoring

4.7.3 As part of the 2019 Ground Investigation, ground gas and groundwater monitoring were undertaken during the fieldwork period and during three post-fieldwork return visits. The results from this monitoring are summarised in the Groundwater section of this report (Section 6.17). Additional rounds of groundwater level monitoring are scheduled on a monthly basis until June 2020 with the subsequent results being assessed at the detailed design stage. These on-going groundwater level monitoring results will be captured in the GDR.

4.8 Laboratory Investigation

4.8.1 Geotechnical and geochemical laboratory testing was carried out as part of the 2019 Ground Investigation on selected samples. Soil descriptions and laboratory testing was carried out by BAM Ritchies, Geolabs, MATtest Limited, Chemtest Ltd and i2 Analytical under instructions from AECOM and in accordance with BS EN ISO 14688-1:2018, BS EN ISO 14688-2:2018, BS5930:1999 and BS1377:1990.

Description of Tests

4.8.2 The following geotechnical laboratory tests were carried out as part of the 2019 Ground Investigation:

Table 4.8.1: Geotechnical Laboratory Testing Summary

Laboratory Test	Quantity
Moisture Content Determination	273
Atterberg Limit Determination	123
Particle Density Determination	4
Particle Size Distribution Analysis (wet sieving)	87
Sedimentation by hydrometer	50
Density by Linear Measurement	6
Organic Matter	19
BRE Special Digest 1 Soils Suite with triggers	44
One Dimensional Oedometer Consolidation Testing	10
California Bearing Ratio Testing (re compacted)	20
4.5kg Compaction Tests	13
Moisture Content Value (MCV) tests at Natural Moisture Content	12
Moisture Condition Value (MCV) Calibration Line Tests	9
Undrained Triaxial Compression Strength Tests (single stage testing techniques on 102mm diameter undisturbed samples)	8
Consolidated Undrained Triaxial Compression Strength tests with pore water pressure measurement (single stage testing techniques on 102mm diameter undisturbed samples)	18
Laboratory Hand Vane Tests	28
Rock Tests	
Uniaxial Compression Strength (UCS) Test	25
Deformability in Uniaxial Compression Strength (UCS)	9
Point Load Tests (PLT)	151
Moisture Content Tests	36

Copies of Test Results

4.8.3 Laboratory results are presented in the Ground Investigation Report (HE514465-BAM-EGT-ZZ-RP-WM-0001).

5. Ground Summary

5.1 Ground Summary Introduction

5.1.1 This section provides a summary interpretation of the PSSR and findings during the 2019 Ground Investigation.

5.2 Geography & Topography

5.2.1 The proposed Scheme starts on the M54 motorway, 900m west of the M54 J1 roundabout. It runs along the existing M54 alignment before reaching the M54 J1 roundabout where it then extends in a north-easterly direction to Junction 11 of the M6 (M6 J11). The route passes east of Featherstone and south-east of Shareshill and Brookfield Farm. The proposed route is shown on drawings HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1001 to HE514465-ACM-SGT-M54_ZZ_ZZ_Z-DR-GE-1007 in Appendix A.

5.2.2 The current topography of the proposed route begins at an elevation of 120m AOD at Ch. 500 on the M54 motorway before gradually rising to 132m AOD along the M54 J1 link roads and roundabout. The main M54 carriageway continues to rise over the M54 J1 roundabout from 138.7m AOD and 143.8m AOD. From the M54 J1 roundabout the Scheme moves in a north-easterly direction where the existing topography rises over undulating farmland to the fishing ponds, south of Dark Lane. The route continues on relatively flat ground until a peak at Hilton lane of 142m AOD. The route then drops into a dip of 130m AOD to Brookfield Farm fishing ponds then over a peak of 138m AOD to the Latherford Brook at 124m AOD. Finally, the route rises to a height of 137m AOD at M6 J11 roundabout road level in the north-east of the Scheme. The M6 motorway passes under the existing M6 bridges at an elevation of 129m AOD.

5.2.3 The road is proposed to pass through and over several cuttings and embankments. These range from a cutting of maximum height of 6.80m at Ch. 2895 to an 8.50m high embankment at Ch. 3730.

5.3 Geology & Geomorphology

5.3.1 Key geological features at the Scheme are shown on Geological Long Section drawings (Ref: HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1019 and HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1012 to 1015) in Appendix A.

5.3.2 Results from the 2019 Ground Investigation show that there is significant variation in the thickness and depths of lithologies, including many interbedded units, due to the site's glacial history. Outline ground models have been produced for three sections across the Scheme and are presented in Table 5.3.1 to Table 5.3.3 in this report.

5.3.3 These ground models should only be used to illustrate the typical ground stratification and the range of depths that lithologies are interpreted to be found. Due to the large variability in geology across the Scheme, a more detailed ground model at the location of each structure/earthwork should be created during detailed design.

5.3.4 The main geological units encountered within the Scheme are described in the sections below.

Made Ground – Engineering Fill

5.3.5 Made Ground associated with the existing M54 and M6 construction has been encountered at either end of the Scheme. The Made Ground/engineering fill recovered at the M54 J1 and M6 J11 roundabouts and motorways shared similar geotechnical properties to one another and have therefore been assessed together under the name MG/Eng Fill.

5.3.6 Made Ground possibly associated with the former Hilton Colliery has also been encountered north of the M54 J1 intersection and near the proposed Featherstone Overbridge. This Made Ground has been assessed under the name MG FOB (Featherstone Overbridge).

Alluvial Deposits

5.3.7 The Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) and the Geotech and Geo-Env Constraints Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) in Appendix A indicate that Alluvium is expected near the Latherford Brook. Granular Alluvium (ALL-G) was recorded in this area during the 2019 Ground Investigation but due to dense woodland limiting access to the area surrounding the Latherford Brook, the full extent and depth of Alluvium was not confirmed.

5.3.8 Local pockets of Alluvium are present near the various fishing ponds that are situated within the Scheme extents.

5.3.9 Additional alluvial deposits might typically comprise clays, silts, sands and gravels of varying proportions and localised thin lenses of peat.

Glacial Deposits

5.3.10 The PSSR notes that Glacial Deposits within the Scheme extents were formed in periods of glacial and periglacial conditions resulting in two main forms of superficial deposits; Glacial Till (GT) and Glacial Sands and Gravels (GSG). The two different glacial deposits in the geological maps are not differentiated as they both appear as Devensian Glacial Deposits. The 2019 Ground Investigation identified that the majority of glacial deposits beneath the Scheme comprised GSG.

5.3.11 The PSSR also explains that Glacial Deposits (also known as Boulder Clay) has typically been deposited from melting ice and is typically a poorly sorted, unstratified mixture of rock fragments, up to boulder size, in a matrix of sand to clay grade material. Laminated clays may also be present within glacial deposits.

5.3.12 GSG are characterised by well sorted material dominated by coarser fractions with lesser but significant proportions of fine-grained material. These materials are typically of low plasticity or non-plastic, medium dense to dense with generally low compressibility and medium permeability.

5.3.13 Based on the Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) in Appendix A, Glacial Deposits are expected to be widespread across the Scheme extents. The boundary between Glacial Deposit areas and those with little to no superficial deposits dissects several key bridge structures such as the M1 J11 Overbridge, Featherstone Overbridge, Hilton Lane Overbridge and Accommodation Bridge. The 2019 Ground Investigation shows that for these structures one abutment will sit on Glacial Deposits and the other on shallow solid bedrock. It should be noted that the glacial materials will mainly be derived from local bedrock; it is therefore not always clear where the boundary lies between glacial materials and residual soils from rocks.

5.3.14 As highlighted in the PSSR, a deep glacial channel was identified under a previous proposed alignment 200-300m north west of the M6 J11 roundabout. Whilst the current main alignment is not located directly over the deep glacial channel, borehole BH25 detected a possible area of deep GSG and GT which does not correspond to the surrounding stratigraphy. Borehole BH25 recorded loose GSG from 6.0m bgl (124.79m AOD) to 19.0m bgl (111.79m AOD) over firm GT to 21.0m bgl (109.79m AOD). Borehole BH27 which is located 80m east of BH25 recorded dense GSG from 8.30m bgl (128.0m AOD) and sandstone at 19.4m bgl (116.9m AOD).

5.3.15 It is therefore considered that the area around borehole BH25 may be part of a glacial channel. Risks associated with the glacial channel include, construction difficulties if piling is required and/or possible excessive differential settlements of embankments or structures. Note that low SPT-N values were recorded within the GSG of BH25, although these could be a result of poor drilling. It is therefore recommended that confirmatory ground investigation is undertaken north west of the M6 J11; this is discussed further in the Geotechnical Risk Register included in Section 7 of this report.

5.3.16 In addition to the previously identified glacial channel in the PSSR, another glacial channel has been identified during the 2019 Ground Investigation. This glacial channel is located under the western abutment of the Hilton Lane Overbridge around Ch. 2745 and reaches an unproven depth over 30.0m (>111.0m AOD). Unlike the glacial channel near the M6 J11, the superficial deposits under Hilton Lane did not present any reduced geotechnical properties with the majority of SPTs in the GSG and GT recording values greater than N = 40.

Sherwood Sandstone Group

5.3.17 The Sherwood Sandstone Group (formerly known as the Bunter Pebble Beds) underlies much of the Scheme as shown in Figure 3.3-1. Within the Sherwood Sandstone Group family is the Chester Formation which the British Geological Society Lexicon describes as a “sandstone and conglomerate, interbedded Sedimentary bedrock formed during the Triassic period”. The Sherwood Sandstone was recorded in the majority of deep boreholes and recorded a weathering profile from completely weathered sandstone (WSST) to slightly weathered sandstone (SST), whilst the conglomerate beds were recorded only in two boreholes within the SST.

Clent Formation and Enville Member

5.3.18 The Clent Formation and Enville Member is expected to cross the proposed alignment at two points, according to geological maps. Once near the Lower Pool Ponds east of Dark Lane and again under the fishing ponds south east of Brookfield Farm. The BGS lexicon describes the material as “breccia, mudstone, sandstone, locally pebbly, and lenticular beds of conglomerate. Sedimentary bedrock formed during the Carboniferous and Permian periods”. Siltstone (SLST) and Mudstone (MST) were both recorded during the 2019 Ground Investigation.

Indicative Ground Models

5.3.19 Indicative ground models are provided below, which are intended to serve as a reference point in deriving structure specific or earthwork specific ground models for design within each of the zones of the site. The geological units relating to each of the strata in the indicative ground model tables are described in Section 6.

Table 5.3.1: Indicative Ground Model – Ch. 500 to Ch. 2000

Stratum	Top of Layer Range (m bgl)	Base of Layer Range (m bgl)	Top of Layer Range (m AOD)	Base of Layer Range (m AOD)	Typical Thicknesses (m)
Made Ground – Engineered Fill (MG/Eng Fill)	0.0	0.0 – 5.5	136.0 – 124.0	133.5 – 122.5	0.0 – 5.5
Made Ground (MG FOB)	0.0	0.0 – 5.5	142.0 – 138.5	142.0 – 133.5	0.0 – 5.5
Glacial Till (GT)	0.0 – 6.0	1.0 – 10.0	140.5 – 115.0	140.5 – 112.0	0.0 – 7.0
Glacial Sands and Gravels (GSG)	0.0 – 11.0	1.0 – Not Proven	142.0 – 120.5	138.5 – Not Proven	0.1 – Not Proven
Weathered Sandstone (WSST)	4.0 – 16.0	11.0 – Not Proven	138.5 – 111.0	121.5 – Not Proven	5.0 – Not Proven
Sandstone (SST)	10.0 – 21.0	Not Proven	128.5 – 113.0	Not Proven	Not Proven

Table 5.3.2: Indicative Ground Model – Ch. 2000 to Ch. 3250

Stratum	Top of Layer Range (m bgl)	Base of Layer Range (m bgl)	Top of Layer Range (m AOD)	Base of Layer Range (m AOD)	Typical Thickness Range (m)
Glacial Sands and Gravels (GSG) (interbedded with GT)	0.0 – 2.5	1.0 – 22.5	141.5 – 131.5	138.0 – 119.0	1.0 – 13.0
Glacial Till (GT) (interbedded with GSG)	0.0 – 11.5	0.0 – 14.0	137.0 – 130.0	135.5 – 119.5	0.0 – 11.5
Weathered Sandstone (WSST)	4.0 – 6.0	6.0 – 12.5	136.0 – 132.0	132.0 – 127.0	0.0 – 8.5
Siltstone (SLST)	13.0 – 16.0	14.0 – 20.0	129.0 – 121.0	126.5 – 117.0	0.0 – Not Proven
Mudstone (MST)	6.5 – 22.5	7.0 – Not Proven	133.0 – 119.0	130.5 – Not Proven	0.0 – Not Proven
Sandstone (SST)	4.0 – Not Proven	12.0 – Not Proven	137.0 – Not Proven	129.0 – Not Proven	0.0 – Not Proven

Table 5.3.3: Indicative Ground Model – Ch. 3250 to M6 J11

Stratum	Top of Layer Range (m bgl)	Base of Layer Range (m bgl)	Top of Layer Range (m AOD)	Base of Layer Range (m AOD)	Typical Thickness Range (m)
Made Ground – Engineered Fill (MG/Eng Fill)	0.0	0.0 – 4.0	135.5 – 125.0	131.0 – 125.0	0.0 – 4.5
Granular Alluvium (ALL-G)	0.0 – 2.0	0.0 – 3.0	126.5 – 124.0	126.0 – 122.5	0.0 – 3.0
Glacial Sands and Gravels (GSG)	0.0 – 4.5	6.0 – 30.0	137.5 – 126.5	123.5 – Not Proven	3.0 – > 20.0
Glacial Till (GT)	0.0 – 24.0	0.0 – 26.0	130.0 – 112.0	129.5 – 109.5	0.0 – 5.0
Weathered Sandstone (WSST)	5.5 – Not Proven	11.5 – Not Proven	120.5 – Not Proven	115.0 – Not Proven	4.0 – 12.0
Sandstone (SST)	10.5 - Not Proven	Not Proven	123.0 – Not Proven	Not Proven	Not Proven

Structural Geology

5.3.20 The Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) in Appendix A shows the distribution of geological faulting across the site area.

5.3.21 Along the eastern boundary of the Scheme there is a large unnamed sinuous fault trending NNE-SSW down throwing west which does not intersect the proposed Scheme alignment. This fault comes within 150m south east of M6 J11. A smaller fault bifurcates from the large fault trending NW-SE which terminates <10m east of Brookfield farm at Ch.3525 and is down throwing north east.

5.3.22 The Bushbury fault bifurcates west of the A460 producing with the first fault trending N-S and downthrows east which intersects the M54 at approximately Ch.1000. The second fault trends NE-SW with a western downthrow and fault intersects the current M54 west of the Scheme.

5.3.23 A series of three smaller faults within the Moseley Old Hall area trend NE-SW with a north west downthrow between Ch.500 and Ch.600.

5.4 Hydrogeology and Hydrology

5.4.1 An assessment of the hydrogeology and hydrology of the Scheme area has been carried out in the PSSR. The PSSR indicates that the drainage pattern across area shows that the surface water is draining from the south-east to the north-west.

5.4.2 No licensed groundwater abstraction boreholes are present within the study area. Most of the site is underlain by major aquifers formed by the Triassic sandstones of the Sherwood Sandstone Group. The Glacial Deposits overlying the Sherwood Sandstone are in areas of low leaching potential as the Glacial Deposit is clay rich which attenuates pollutants or prevents significant vertical movement.

5.4.3 The Devensian Till superficial deposits are designated as Secondary (Undifferentiated) aquifers, while the Alluvium deposits are designated as secondary 'A' aquifer. The Chester Formation (Sherwood Sandstone Group) is designated a 'Principal' aquifer by the Environment Agency. The Clent and Enville Formation (Warwickshire Group), are designated as 'Secondary A' aquifers.

5.4.4 The majority of the *Scheme* does not lie within a Source Protection Zone (SPZ). However, the area from M54 Junction 2 eastwards for approximately 1.2km, heading northwards through Featherstone and towards Latherford Brook, west of the Scheme, is within SPZ 3 (Total Catchment). This can be seen in the Geotechnical and Geo-environmental Constraints Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) in Appendix A.

5.5 Man-made Features and Historical Development

5.5.1 Man-made features and historical developments are summarised in the PSSR.

5.5.2 The main findings of the PSSR show that 60m south of the M54 J1, the land now containing the present-day Hilton Main Industrial Estate, was first built upon between 1902 and 1924 when Hilton Main Colliery opened. Coal mining took place here from 1902 until 1989 and therefore the associated colliery spoil is located around M54 J1, extending 200m north of M54 J1.

5.5.3 A landfill site is located 250m west of M6 J11. This landfill accepted unknown wastes and therefore it must be assumed that this site may have taken waste that degrades and thus may be polluting the local groundwater or emitting landfill gas and may settle with time if built over.

6. Ground Conditions and Material Properties

6.1 General

6.1.1 This section describes the findings and results of the ground investigations undertaken for the Scheme. The section assesses each key geological unit found within the Scheme extents and derives suitable typical material parameters for design.

6.1.2 The following assessments and summaries are based on the 2019 Ground Investigation data and relevant information from historical ground investigations.

6.1.3 Parameters have been derived using a combination of direct test results from the ground investigation, interpretation of the test results using established engineering correlations, and in the absence of any other data, assumptions based on engineering knowledge, available literature and previous experience with similar materials.

6.1.4 Strength variations with depth have been identified for several materials across the Scheme and are based on depth below ground level. Relationships relative to elevation were not considered to be relevant due to the undulating terrain and strata and the interbedded nature of the materials encountered.

6.1.5 Indicative characteristic values for each geological unit are presented in the following sections. These are intended to serve as a reference point for establishing design values, which are defined as 'cautious estimate(s) of the value(s) affecting the occurrence of the limit state' (Section 2.4.5.2 (2), BS EN 1997-1:2004). As such, the tabulated characteristic values are provided as a cautious estimate of a parameter but would need to be reviewed and adjusted, during detailed design, according to the limit state being assessed in the specific location of the structure/earthwork; taking into account the local ground variations.

6.1.6 A graphical representation of the interpreted geological strata and their respective thicknesses, positions and depths are presented on the Geological Long Section drawings (HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1019 and HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1012 to 1015) in Appendix A. It should be noted that 2019 Ground Investigation recorded substantial lateral variations away from the geological long section centre line. This is particularly relevant for areas which straddle the superficial and solid geology boundary, and the zone near the M6 J11 roundabout which showed evidence of channels, infilled hollows, and possible kames.

6.1.7 It is therefore recommended that the Geological Long Section drawings should serve to provide an indicative background to more focused interpretations connected with specific design features.

6.1.8 Graphs showing laboratory or in-situ test results that directly support the typical parameter determination process are included in the following sections. All the graphs with geotechnical results for each geological unit are included in Appendix B of this report.

6.1.9 A summary of the typical geotechnical parameters are presented in Section 6.12.

6.2 Topsoil (TS)

6.2.1 Topsoil (TS) was encountered within exploratory holes from both the 2019 and historic ground investigations. TS was recorded widely across the Scheme area but particularly prevalent in the agricultural fields between Ch. 1620 and Ch. 3900. Up to 0.4m of TS was recorded in boreholes BH05, BH06 and BH07 associated with the M54 J1 roundabout. Boreholes BH26, BH27, BH28A and BH29 on the M6 J11 roundabout all recorded 0.2m of TS.

6.2.2 Topsoil was described as both granular and cohesive across the Scheme with a random spread across the site. The granular topsoil was typically described as compact brown sandy gravelly topsoil. The cohesive topsoil was typically described as dark brown gravelly sandy clay with occasional to frequent rootlets. Gravel is angular to rounded fine to coarse of various lithologies including sandstone, mudstone, siltstone and quartz.

6.2.3 The Scheme has been split into three sections based on typical material descriptions and locations. A summary of the descriptions and typical thicknesses are presented in Table 6.2.1. It is expected that the TS will be removed prior to any construction works so geotechnical parameters have not been determined for this report.

Table 6.2.1: Thicknesses and Descriptions of Topsoil

Section	Minimum Thickness (m)	Maximum Thickness (m)	Typical Thickness (m)	Typical description
Ch.1400 – Ch. 2000	0.1	0.4	0.2	Firm, friable, dark brown, slightly gravelly, sandy, clayey topsoil with numerous plant rootlets, and rare subrounded siltstone cobbles. Sand is fine to coarse. Gravel is subrounded to rounded fine to coarse of quartz, granite, siltstone and sandstone.
Ch. 2000 – Ch. 3250	0.1	0.5	0.3	Vegetation over dark brown, slightly gravelly, sandy, clayey topsoil with numerous rootlets. Gravel is angular to rounded, fine to coarse with various lithologies. Brown sandy gravelly topsoil.
Ch.3250 – M6 J11 Roundabout	0.1	0.6	0.3	Compact, brown, sandy, gravelly topsoil.

6.3 Made Ground (MG/Eng Fill & MG FOB)

6.3.1 Made Ground has been found in 33 No. exploratory locations on the site; due to its artificial nature and origin, it is prone to variability that cannot be analysed by the same rules of origin, deposition, sorting and history as natural soils. The following descriptions of the distribution of MG as identified in the exploratory holes, and the nature of constituent materials is based on the most commonly encountered conditions on the site.

6.3.2 Made Ground was encountered in three main areas within the Scheme area; the M54 J1 motorway and roundabout embankments (Ch. 700 to Ch. 1600), the M6 J11 roundabout embankments and the area near the proposed Featherstone Overbridge (Ch. 1600 to Ch. 1950). Reviewing the material descriptions and geotechnical test results show that the Made Ground recovered near the M54 J1 and M6 J11 motorways share similar properties and have therefore being assessed together in the following sections (named as MG/Eng Fill).

6.3.3 The Made Ground near the Featherstone Overbridge has been assessed separately due to its unique properties (named as MG FOB).

Made Ground – Engineered Fill (MG/Eng Fill)

6.3.4 MG/Eng Fill is encountered near the M54 J1 and motorway originated from the construction of the M54 in the mid-1970s. The following boreholes encountered Made Ground in the M54 J1 area: BH01, BH02, BH04, BH05, BH07, SJ90SW93, SJ90SW94, SJ90SW95.

6.3.5 Historic boreholes SJ90SW69, SJ90SW71, SJ90SW78 and SJ90SW84 are positioned along the M54 motorway and were drilled in June 1971, prior to the construction of the M54. Since the ground level for these holes are below the current carriageway level and the holes did not record any MG/Eng Fill, it can be safely assumed that the M54 carriageway has been constructed on engineered fill. This was confirmed in BH01 and BH02 from the 2019 Ground Investigation with the boreholes recording 1.5m and 4.4m of MG/Eng Fill respectively.

6.3.6 Historic boreholes SJ90SW95, SJ90SW93 and SJ90SW94 were drilled in 1971 to provide information for construction of the M54 motorway construction. The boreholes all recorded a red, brick, ash and stone, shale FILL with traces of coal and brown clay. Prior to the construction of the M54 J1 roundabout, the historical maps included in the PSSR showed that the area was largely undeveloped. Hilton Main Colliery was located 200m south of the future M54 J1 roundabout location, but its full extent is not shown. The level of the M54 motorway across the M54 J1 roundabout rises from 138.75m AOD to 143.79m AOD, which is approximately 2.0m below the recorded ground level of the historic boreholes. This indicates that the shale fill recorded in the historical boreholes is unrelated to the construction of the M54 J1 roundabout and will no longer be present, it therefore has been excluded from this assessment.

6.3.7 In borehole BH04, MG/Eng Fill was described as slightly gravelly, sandy clay with coal, brick and slag, which is indicative of colliery spoil material. BH07 was described as the same material with the addition of concrete. BH05 did not find any colliery spoil material and is assumed to be engineered fill derived from beneath the M54 carriageway.

6.3.8 The MG/Eng Fill encountered near the M6 J11 most likely originated from the construction of the M6 in the mid-1960s.

6.3.9 Historical boreholes SJ90NE153 and SJ90NE154 do not have drilled dates but are assumed to have been undertaken prior to the construction of the M6 motorway due to their position over the motorway, recorded ground level (133.07m AOD and 132.74m AOD, respectively) in relation to the current M6 road level (129.5m AOD) and use of imperial units on the borehole logs. Boreholes BH26, BH27, BH28A and BH29 from the 2019 Ground Investigation were drilled at the M6 J11 Northern and Southern bridge deck levels, with ground levels of 137.04m AOD (BH26), 136.3m AOD (BH27), 137.03m AOD (BH28A) and

136.26m AOD (BH29). Based on the difference in levels between the ground prior to the construction of the M6 and post-construction, it is expected that up to approximately 4.0m of Made Ground or engineered fill was used to create the M6 J11 roundabout and embankments. This approximate 4.0m of MG/Eng Fill was confirmed in boreholes: BH25, BH26, BH27, BH28A, BH29.

6.3.10 The MG/Eng Fill within the M54 J1 and M6 J11 areas is mostly cohesive in nature with occasional granular zones. Due to the variable nature of MG/Eng Fill, it is difficult to accurately interpret the ground profile and foresee changes in the granular and cohesive layers. This report therefore assesses the MG/Eng Fill as cohesive in nature as it is more representative of the conditions encountered at these locations and will also produce more cautious results.

6.3.11 The MG/Eng Fill was typically described as:

- Soft to firm, dark brown to black, sometimes locally mottled reddish brown, slightly gravelly, sandy, silty, clay with low cobble content.
- Sand is fine to coarse.
- Gravel is angular to sub-rounded, fine to coarse of sandstone, siltstone quartz and mudstone.
- Cobbles are angular of slag and concrete.
- Includes fragments of coal, brick and slag.

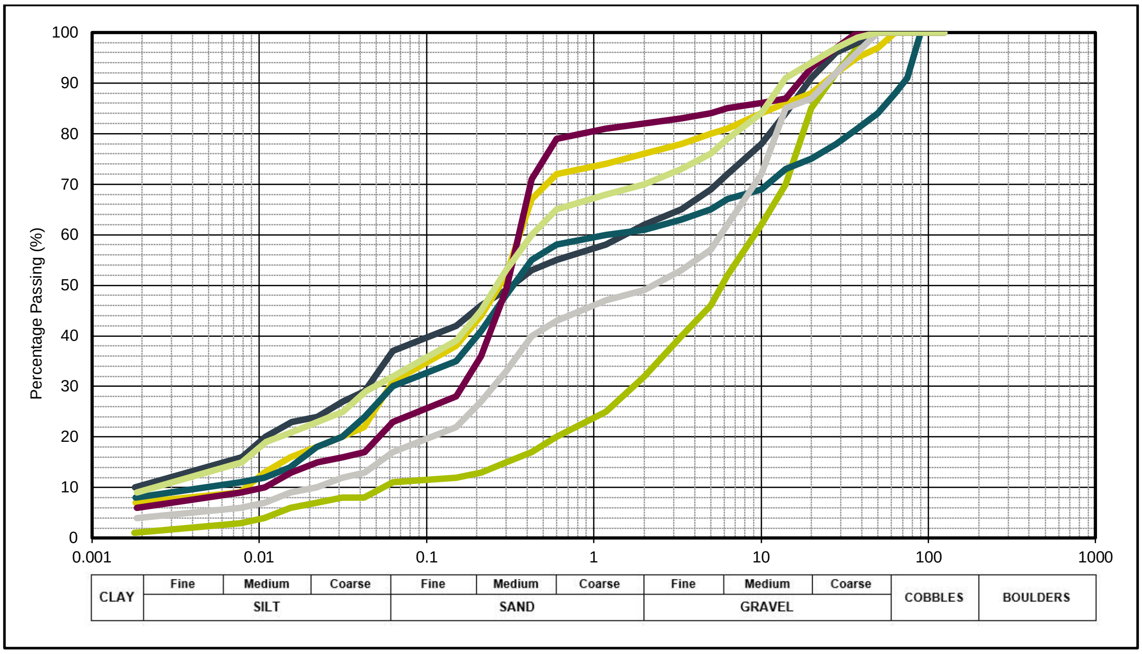
6.3.12 The thickness of the MG/Eng Fill in the M54 J1 area ranged from 0.4m (in BH05) to 5.45m (in BH02) and typically, the MG/Eng Fill near the M54 J1 area overlies the GSG and GT.

6.3.13 MG/Eng Fill associated with the M6 J11 area was recorded to be between 1.4m (in BH25) to 5.0m (in BH26) thick.

6.3.14 Laboratory tests showed that the 29No. moisture content tests for the MG/Eng Fill ranged from 6.6% to 19% with an average value of 12%. 4No. bulk density test results recorded a range between 20.2kN/m3 and 21.3kN/m3 with an average of 20.7kN/m3. Graphs of the recorded moisture content and bulk density values with depth are plotted in Appendix A.

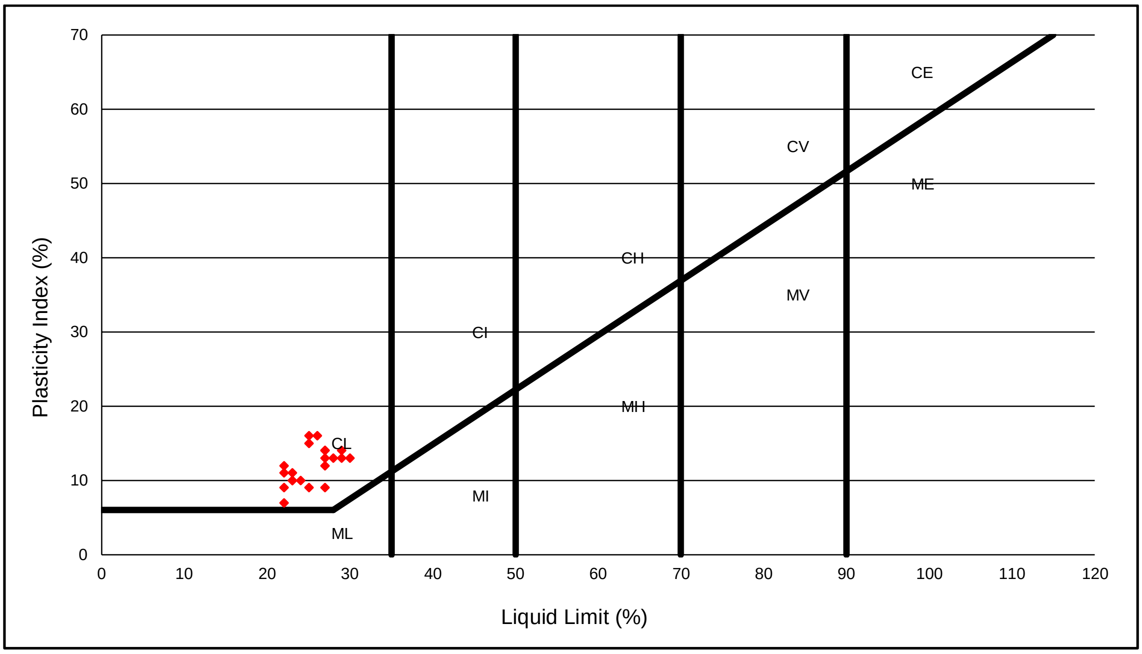
6.3.15 Figure 6.3-1 shows that the particle size distributions of the MG/Eng Fill typically indicate fines contents ranging from 10% to 37%, sand content ranging from 21% to 60% and gravel contents ranging from 18% to 68%. The average fines, sand and gravel content is calculated to be 26%, 36% and 37% respectively. The large range of particle sizes highlights the variability of the MG/Eng Fill.

Figure 6.3-1: MG/Eng Fill. Particle Size Distribution



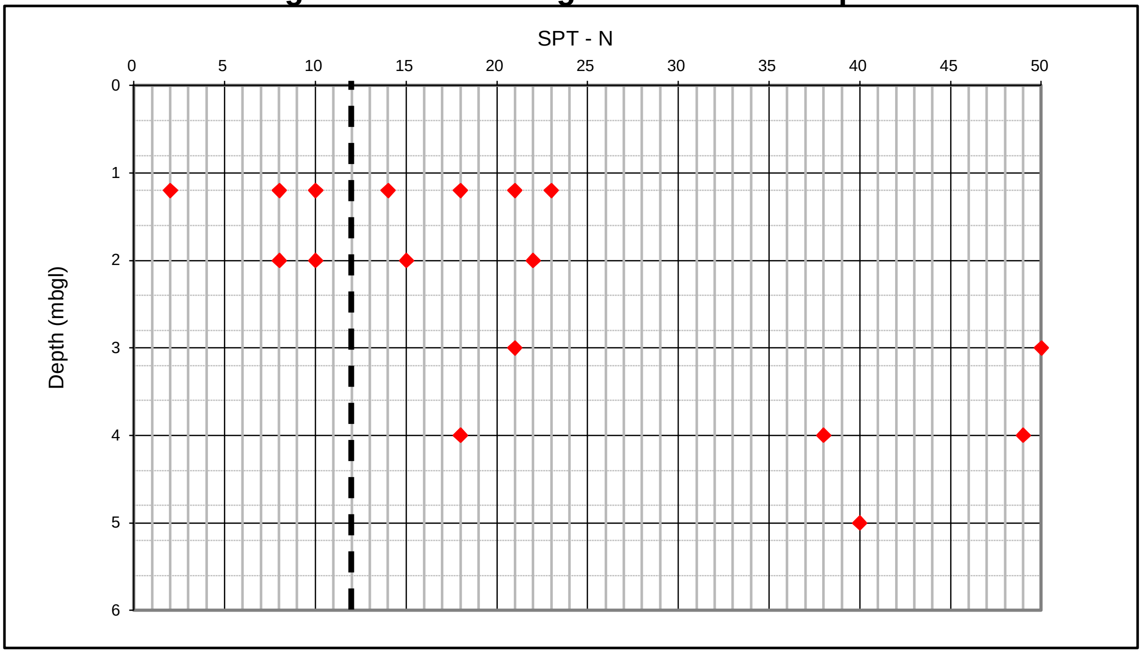
6.3.16 Figure 6.3-2 shows the plasticity index values for the MG/Eng Fill range between 7% to 16%. Results from the Atterberg Limit tests show that the material is classified as a low plasticity clay (CL). A plasticity index of 14% was used to determine the constant volume angle of friction ($\varphi'_{cv,k}$) of 28° using correlations from BS8002:2015 and Sorensen and Okkels (2013) for normally consolidated cohesive soils.

Figure 6.3-2: MG/Eng Fill. Atterberg Limit A-Line Plot



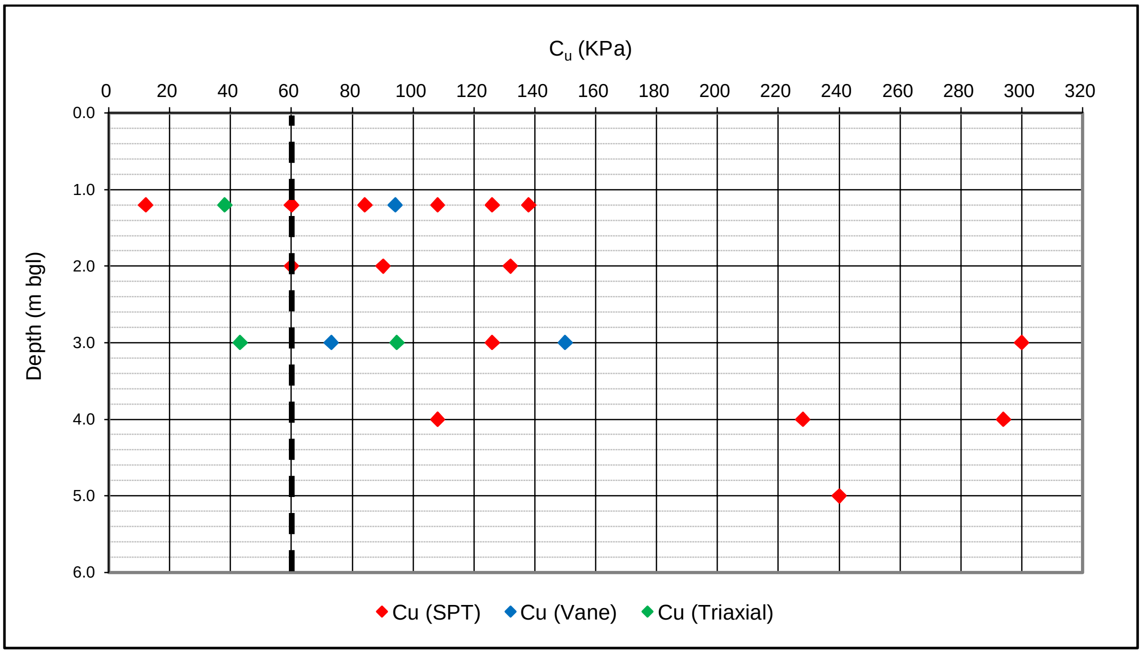
6.3.17 Figure 6.3-3 shows the distribution of SPT-N values against depth, the results range between 2 and 50 and typically lie between 8 and 23. The SPT-N results greater than N = 40 are likely due to the presence of cobbles of brick and concrete and do not reflect the actual strength of the soil matrix. A cautious SPT-N value of 12 has been selected for the MG/Eng Fill.

Figure 6.3-3: MG/Eng Fill. SPT-N vs Depth



6.3.18 Figure 6.3-4 shows the undrained strength (C_u) of the MG/Eng Fill from Undrained Triaxial and Hand Shear Vane tests and SPT-N results which have been converted to C_u using the relationship C_u = 5 x SPT-N. A cautious correlation factor of 5 has been selected based on information given in CIRIA 143 and considering the variability of the material. The C_u results shown in Figure 6.3-4 range from 10kPa to 250kPa with most results lying between 40kPa and 120kPa. A cautious C_u value of 60kPa has been proposed for the MG/Eng Fill.

Figure 6.3-4: MG/Eng Fill – Cu vs Depth



6.3.19 The drained Young's Modulus has been correlated using a relationship of E' = Cu x 250 from Stroud et al, 1975 and taking a plasticity index of 14%. Based on a Cu value of 60kPa, this produces an E' of 15MPa.

6.3.20 A single oedometer test was undertaken within BH25 at 4.0m bgl with a coefficient of volume compressibility (mv) at in-situ stress conditions of $0.32m^2/MN$. According to Tomlinson (2001) clays with mv values of $0.32m^2/MN$ are classified as 'medium to high compressibility' soils.

6.3.21 The oedometer tests recorded t50 coefficient of consolidation values (cv) at in-situ conditions of $0.14m^2/yr$.

6.3.22 It should be noted that oedometer tests are very sensitive to variations in the soil composition so the tests undertaken on ground which is not naturally deposited, such as those in this area, should be treated with caution.

6.3.23 IAN 73/6, 2009 provides a CBR estimate based on the soil type and plasticity index. For a sandy clay with a plasticity index of 14%, a CBR value between 3% and 5% can be assumed.

6.3.24 Based on the oedometer test result, the permeability of the sample can be calculated using the following formula:

$$c_v = \frac{k}{\gamma_w \, m_v}$$

6.3.25 Using the results of the oedometer test, the permeability of the MG/Fill sample is calculated to be 1.39×10^{-08} m/s, which is a very low permeability material. This however does not consider the variability and gravelly, sandy composition of the fill. Information from CIRIA C580 notes that for slightly gravelly, sandy, silty, clays like those encountered under the M54 J1 and M6 J11 motorways the permeability of the soil can vary between 10^{-4} m/s and 10^{-7} m/s.

6.3.26 A summary of the geotechnical testing for the MG/Eng Fill is summarised in Table 6.3.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the MG/Eng Fill have been selected and are presented in Table 6.3.2.

Table 6.3.1: Geotechnical Testing Summary. MG/Eng Fill

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	4	20.2 – 21.3	20.7
Moisture Content (%)	29	6.6 – 19.0	12.4
Plasticity Index, PI (%)	21	7 – 16	11
PSD – Fines Content (%)	7	10 – 37	26
PSD – Sand Content (%)	7	21 – 60	36
PSD – Gravel Content (%)	7	18 - 68	37
Undrained shear strength, C_u (kPa)	3No. Triaxial Tests 3No. Hand Vane Tests	38 - 150	82
Coefficient of volume compressibility, m_v (m^2/MN)	1No. Oedometer Test	0.32	0.32

Type of Test	Number of Tests	Range of Results	Mean
Coefficient of consolidation values, c_v (m^2/yr)	1No. Oedometer Test	0.14	0.14
SPT 'N' (No.)	17	2 – 50	22

Table 6.3.2: Indicative Values of Characteristic Parameters. MG/Eng Fill

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	20.5	Based on laboratory test results and ranges given in BS 8002 (2015) and Barnes (2000).
Plasticity Index, PI (%)	14	Cautious estimate based on laboratory test results.
SPT 'N' (No.)	12	Cautious estimate based on SPT 'N' test results.
Undrained shear strength, C_u (kPa)	60	Cautious estimate based on in-situ and laboratory test results.
Cohesion, c' (kPa)	0	Cautious estimate due to no available data.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	28	Based on correlations with plasticity index as given in BS8002:2015 and Sorensen and Okkels (2013).
Drained Young's modulus, E' (MPa)	15	Based on correlations from Stroud et al, 1975
Coefficient of volume compressibility, m_v (m^2/MN)	-	To be confirmed locally at the detailed design stage.
Coefficient of consolidation values, c_v (m^2/yr)	-	To be confirmed locally at the detailed design stage.
Permeability, k, (m/s)	10^{-4} – 10^{-7}	Based on guidance given in CIRIA C580
CBR (%)	3 – 5	Cautious estimate using IAN 73/06, 2009.

Made Ground – Near Featherstone Overbridge (MG FOB)

6.3.27 MG FOB was encountered in the area between the M54 J1 roundabout and proposed Featherstone Overbridge (Ch. 1600 to Ch. 1950) in exploratory holes TP01, TP02, TP03, TP04, TP06, BH03, BH08, BH08A, BH09 and BH30. The material was typically described as:

- Firm dark sometimes greyish brown to black, slightly gravelly, sandy, clay with aromas of tar/oil, fragments of brick, concrete, charcoal, wood, rubber, metal, plastic, railway sleepers and truck tyres.

6.3.28 The origins of this material are not known, but it is possible that previous quarries or sand and gravel pits that historical maps showed to be within 200m of the area, were historically infilled. The Geotech and Geo-Env Constraint Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) and Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) drawings in Appendix A outlined this potential area of Made Ground.

6.3.29 TP05, TP07, TP08 and BH10 described Made Ground with no descriptions of potentially contaminated material. Laboratory data for these trial pits do however show that the Made Ground has similar geotechnical properties as the potentially contaminated material noted above so it has been included in this parameter determination.

6.3.30 Due to the descriptions of oil, tar, wood, rubber, plastics and tyres the Made Ground found near the Featherstone Overbridge (MG FOB) has been tested for contamination. The results of these tests are discussed in Section 6.18.

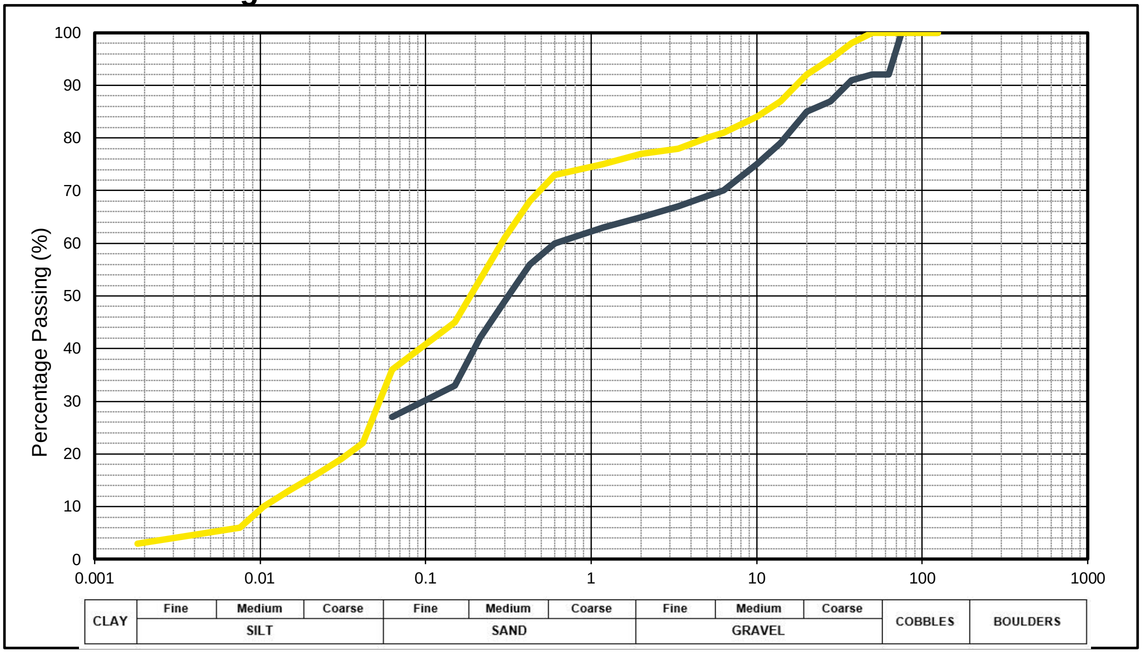
6.3.31 Both cohesive and granular MG FOB was recovered in the 2019 Ground Investigation exploratory holes, with cohesive soil being the more prevalent of the two. Due to its variable nature, it is difficult to model the boundaries between granular and cohesive MG FOB. This report will assess the MG FOB material as a cohesive material as it is more cautious, and it better represents the ground conditions encountered in the area.

6.3.32 The thickness of the MG FOB ranged from a minimum of 1.1m (in TP08) towards Ch. 1950 and a maximum of 5.0m (in BH03) near Ch. 1700. The average thickness of the MG FOB was recorded as 2.6m.

6.3.33 Laboratory tests show moisture content values ranged from 7.9% to 23%, with most results sitting between 12% and 19%. The average moisture content value from the 26No. laboratory tests is 15.6%. A graph of the recorded moisture content values with depth are plotted in Appendix A. 3No. bulk unit weight tests were carried out producing values of 20.4kN/m^3, 21.0kN/m^3 and 21.4kN/m^3.

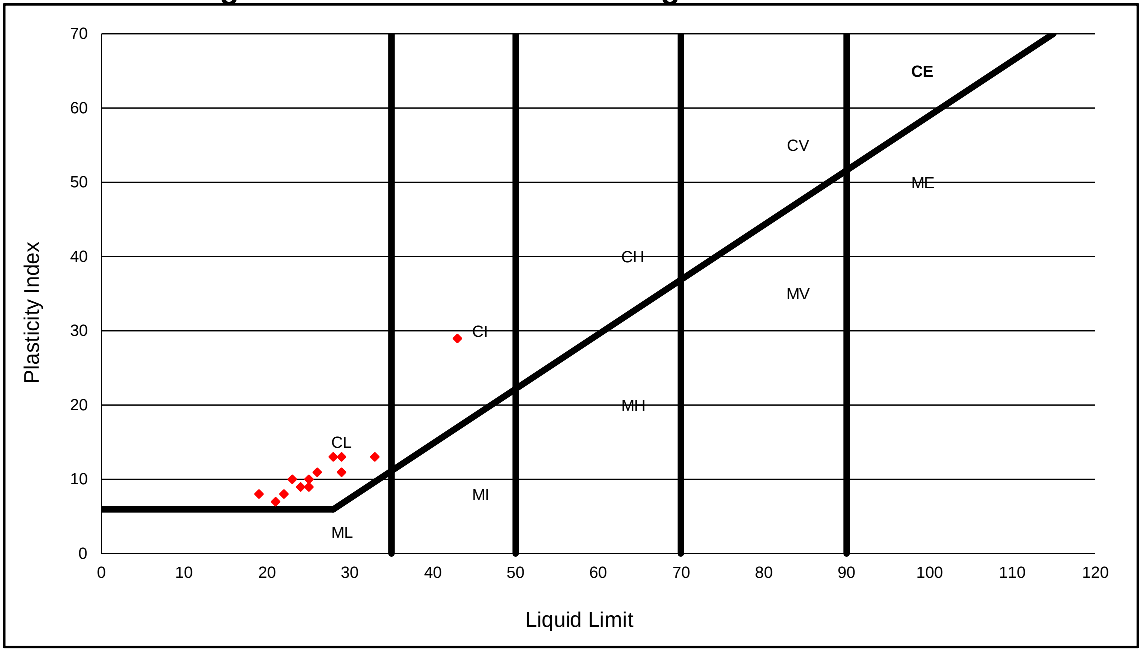
6.3.34 Figure 6.3-5 shows the 2No. particle size distribution tests undertaken on the MG FOB which indicate that the material is a typically a slightly clayey, sandy, gravelly, silt. The fines content ranges from 27% to 36%, sand content ranges from 38% to 40% and gravel content ranges from 23% to 27%. On average the MG FOB is recorded as 32% fines, 39% sands and 25% gravel.

Figure 6.3-5: MG FOB. Particle Size Distribution



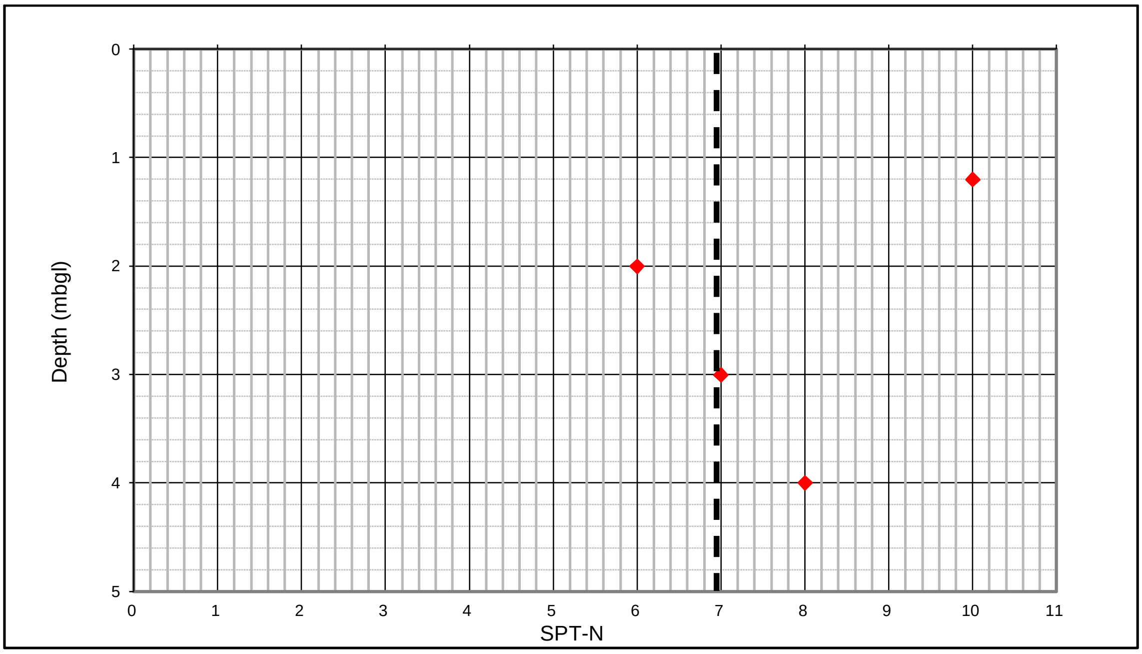
6.3.35 Figure 6.3-6 shows that the plasticity index value for the MG FOB ranges between 7% to 29%, and with the plasticity index value of 29% considered an outlier, the typical range is 7% to 13%. The average plasticity index value is 11.5%. Results from the Atterberg Limit tests show that the material is classified as a low plasticity clay (CL). A cautious plasticity index of 13% was used to determine the constant volume angle of friction of $\varphi'_{cv,k}$ = 26° using correlations from BS8002:2015 and Sorensen and Okkels (2013). However, due to the variability of the material, a cautious angle of friction of 26° has been adopted.

Figure 6.3-6: MG FOB. Atterberg Limit A-Line Plot.



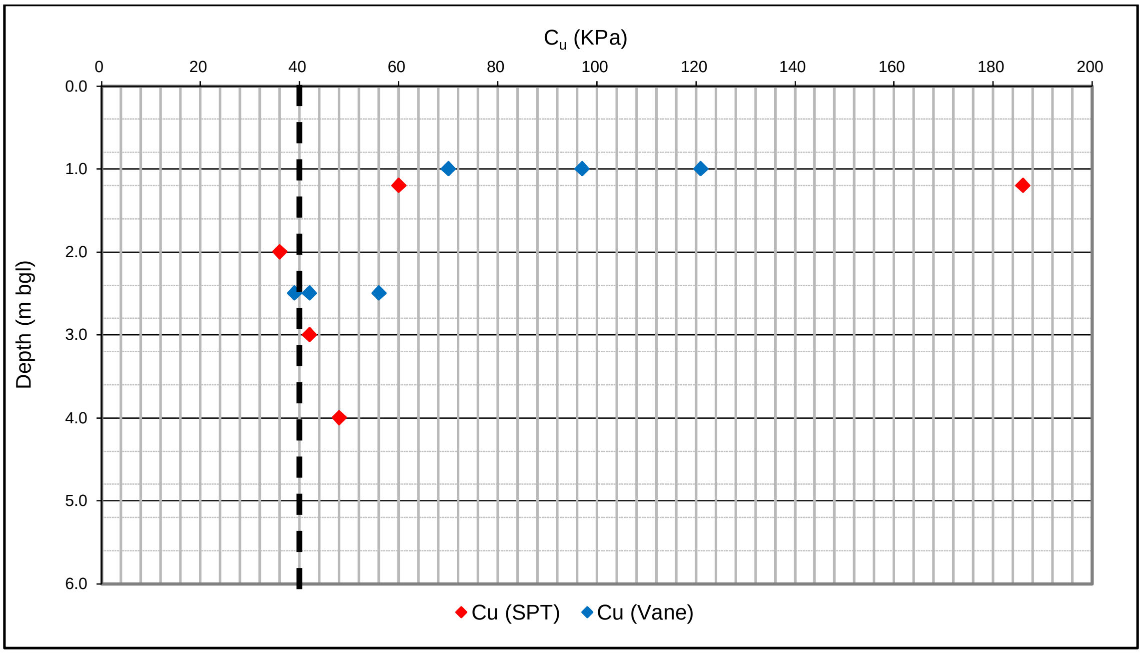
6.3.36 Figure 6.3-7 shows the 4No. SPT-N tests undertaken within the MG FOB which range between 6 and 10. A cautious estimate of the SPT has been used to give a typical SPT-N value of 7.

Figure 6.3-7: MG FOB. SPT-N vs Depth



6.3.37 Figure 6.3-8 shows the undrained strength (C_u) of the MG FOB from in-situ Hand Shear Vane tests and SPT-N results which have been converted to C_u using the relationship C_u = 5 x SPT-N. A cautious correlation factor of 5 has been selected based on information given in CIRIA 143 and considering the variability of the material. The C_u results shown in Figure 6.3-8 range from 30kPa to 135kPa and average of 60kPa. It is considered that the Hand Shear Vane tests marginally overestimate the undrained shear strength so greater consideration has been given to the undrained shear strength values determined from SPT results. Therefore, a cautious C_u value of 40kPa has been proposed for the MG FOB.

Figure 6.3-8: MG FOB. Cu vs Depth



6.3.38 The Young's modulus (E') has been correlated using the relationship of E' = C_u x 250 from Stroud et al (1975) and assuming a plasticity index value of 13%. Based on a C_u value of 40kPa, this produces a drained Young's Modulus of E' = 10.0MPa.

6.3.39 2.No oedometer tests were completed on the MG FOB near the Featherstone Overbridge. The two tests produced coefficient of volume compressibility (m_v) at in-situ stress conditions values of 0.05m^2/MN (BH08) and 0.10 m^2/MN (BH09). The oedometer tests recorded t_{50} coefficient of consolidation values (c_v) at in-situ conditions of 0.13m^2/yr (BH08) and 1.12m^2/yr (BH09).

6.3.40 It should be noted that oedometer tests can be very sensitive to variations in the soil composition so the tests undertaken on ground which is not naturally deposited, such as those in this area, should be treated with caution. It is also noted that the oedometer tests were undertaken on cohesive samples within the variable MG FOB and may not reflect the sandy and gravelly constituents of the whole soil matrix. It is likely that the compressibility and permeability of the MG FOB is higher than the oedometer tests results indicate. The results of the oedometer tests for the MG FOB should be applied locally to the locations surrounding the Featherstone Overbridge and boreholes BH08 and BH09.

6.3.41 Based on guidance given in CIRIA C580, a sandy, gravelly, clay soil such as those identified in the MG FOB, the permeability is expected to be in the range of $x10^{-4}$ m/sec and $x10^{-7}$ m/sec.

6.3.42 One CBR measurement was undertaken in TP03 at 3.0m bgl, resulting in a CBR value of 1.15%. Using this value and IAN 73/06, with the material classified as silty clay the CBR has been estimated at 1-2%. According to IAN 73/06, when the subgrade has a CBR value less than 2.5% it is considered unsuitable support for pavement foundation and will need to be permanently improved by excavation and replacement or other in-situ treatments.

6.3.43 A summary of the geotechnical testing for the MG FOB is summarised in Table 6.3.3. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the MG FOB have been selected and are presented in Table 6.3.4.

Table 6.3.3: Geotechnical Testing Summary. MG FOB

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	3	20.4 – 21.4	20.9
Moisture Content (%)	26	7.9 – 23	15.6
Plasticity Index, PI (%)	14	7 – 29	11.5
PSD – Fines Content (%)	2	27 – 36	32
PSD – Sand Content (%)	2	38 – 40	39
PSD – Gravel Content (%)	2	23 – 27	25
SPT ‘N’ (No.)	4	6 - 10	8
Undrained shear strength, C_u (kPa)	6No. laboratory vane tests	39 - 121	70
Coefficient of volume compressibility, m_v (m^2/MN)	2No. Oedometer Tests	0.05 – 0.10	0.075
Coefficient of consolidation values, c_v (m^2/yr)	2No. Oedometer Tests	0.13 – 1.12	0.625
CBR (%)	1	1.15	1.15

Table 6.3.4: Indicative Values of Characteristic Parameters. MG FOB

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	20.5	Based on laboratory test results and ranges given in BS 8002 (2015) and Barnes (2000).
Plasticity Index, PI (%)	13	Cautious estimate based on laboratory test results.
SPT ‘N’ (No.)	7	Cautious estimate based on SPT ‘N’ test results.
Undrained shear strength, C_u (kPa)	40	Based on guidance within Handbook of Geotechnical Investigation and Design Tables (Look, 2007)
Cohesion, c’ (kPa)	0	Cautious estimate due to no available data.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	26	Based on correlations with plasticity index as given in BS8002:2015 and Sorensen and Okkels (2013).

Parameter	Indicative Characteristic Value	Remarks
Drained Young's modulus, E' (MPa)	10	Based on guidance from Stroud et al., 1975
Coefficient of volume compressibility, m_v (m^2/MN)	-	To be confirmed locally at the detailed design stage.
Coefficient of consolidation values, c_v (m^2/yr)	-	To be confirmed locally at the detailed design stage.
Permeability, k, (m/s)	10^{-4} to 10^{-7}	Based on guidance given in CIRIA C580.
CBR (%)	1-2	Cautious estimate using laboratory testing and IAN 73/06, 2009.

6.4 Granular Alluvium (ALL-G)

6.4.1 The Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) and the Geotech and Geo-Env Constraints Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1014) indicate that Alluvium is expected near the Latherford Brook between chainages Ch. 3700 and Ch. 3770. The 2019 Ground Investigation recorded possible granular Alluvium (ALL-G) either side of the Latherford Brook which trends north-west to south-east across the A460 (Ch. 3660 – Ch. 3820).

6.4.2 Possible ALL-G was found in boreholes BH21, BH22, BH22A, TP17 and TP18. It is thought that the area surrounding BH24 previously contained ALL-G due to its proximity to the Latherford Brook which has since been excavated and replaced by Made Ground associated with the historic landfill.

6.4.3 The ALL-G is described as:

- Medium dense to dense, reddish to yellowish brown, mottled grey, slightly clayey, fine to coarse SAND. Gravel is subangular to rounded fine to coarse of various lithologies.

6.4.4 It should be noted that the description of ALL-G is very similar to GSG.

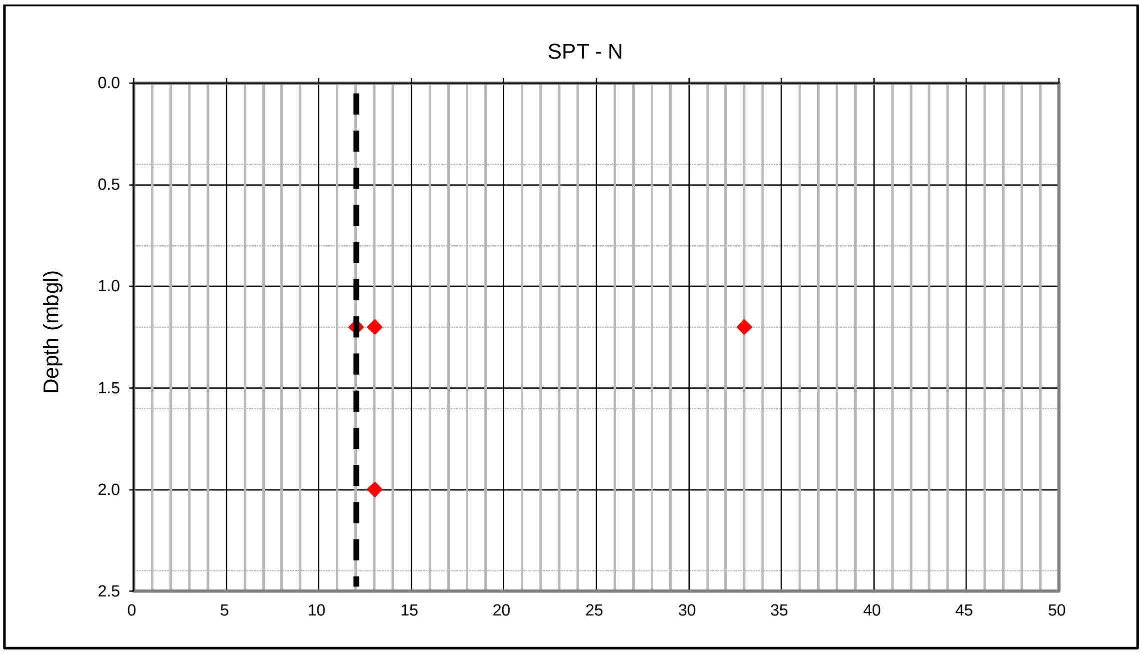
6.4.5 The ALL-G was recorded from ground level, except where Made Ground is present in BH21, to 2.0m bgl. The thickness of the ALL-G ranged from 1.6m (in TP18) to 2.5m (in BH21), BH22 and BH22A). The ALL-G was found to predominantly overly the GSG and a small lens of GT. Borehole BH21 recorded 0.5m of Made Ground overlying the ALL-G. TP17 recorded soft clay from 1.1m bgl (123.3m AOD) to 1.7m bgl (122.7m AOD) which indicates that there could be compressible cohesive layers near the Latherford Brook.

6.4.6 A bulk unit weight of 18kN/m3 has been estimated from ranges within BS 8002 (2015) and Barnes (2000).

6.4.7 6No. moisture content tests undertaken in the ALL-G recorded values between 7.9% and 16% with an average of 12.1%.

6.4.8 Figure 6.4-1 shows that the 4No. SPT-N values recorded within the ALL-G ranged from 12 and 33. When determining a derived SPT-N value, the SPT-N value of 33 was discounted and a cautious estimate of N = 12 was selected from the remaining data.

Figure 6.4-1: ALL-G. SPT-N vs Depth



6.4.9 The stiffness modulus was determined using the correlation with SPT-N from ICE Manual of Geotechnical Engineering, 2012. As the ALL-G is normally consolidated the correlation with SPT-N is assumed to be E' = 1 x SPT-N. Therefore, an E' value of E' = 12MPa has been determined.

6.4.10 The peak friction angle has been determined using the correlation with SPT-N from Peck et al. (1974); from a SPT-N = 12 a peak angle of friction of $\varphi'_{pk,k}$=30° has been determined. According to BS8002:2015, the dilatancy angle for the ALL-G for a SPT-N=12 (I_D=42%) is φ'_{dil}=2° and therefore $\varphi'_{cv,k}$= $\varphi'_{pk,k}$- φ'_{dil} = 28°.

6.4.11 The CBR was determined using guidance from IAN 73/06, which suggests well graded sand can achieve a CBR value of 20%. With limited classification tests available for the material and the possible presence of soft clay which will result in a lower CBR value, a cautious CBR between 3% and 10% has been selected.

6.4.12 The permeability of the ALL-G has been estimated based on information provided in CIRIA C760, 2017. The guidance document notes that slightly clayey sands such as the ALL-G will have permeability values between x10^{-4} and x10^{-6} m/sec.

6.4.13 Due to the dense woodland near the Latherford Brook, it was not feasible to position the exploratory holes close to the existing stream. It may be the case that the ALL-G is more extensive closer to the Latherford Brook or that the cohesive alluvium is more prevalent in the area. The risks of unexpected cohesive or soft ground conditions are discussed in the Geotechnical Risk Register in Section 7 of this report.

6.4.14 A summary of the geotechnical testing for the ALL-G is summarised in Table 6.4.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the ALL-G have been selected and are presented in Table 6.4.2.

Table 6.4.1: Geotechnical Testing Summary. ALL-G

Type of Test	Number of Tests	Range of Results	Mean
Moisture content (%)	6	7.9 – 16.0	12.1
SPT 'N' (No.)	4	12 - 33	17.7

Table 6.4.2: Indicative Values of Characteristic Parameters. ALL-G

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	18	Based on ranges given in BS 8002 (2015) and Barnes (2000).
SPT 'N' (No.)	12	Estimate based on SPT 'N' results.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	28	Based on the relationship provided by Peck et al (1974) and BS8002:2015
Effective peak friction angle, $\varphi'_{pk,k}$ (°)	30	Based on the relationship between SPT N- values and angle of shearing resistance (Peck et al, 1974.)
Drained Young's modulus, E' (MPa)	12	Profile is based on the SPT-N / E' correlation provided by ICE Manual of Geotechnical Engineering (2012).
Permeability, k (m/sec)	$10^{-4} – 10^{-6}$	Based on guidance given in CIRIA C760 (2017).
CBR (%)	3 – 10	Based on guidance given in IAN 73/06 (2009).

6.5 Glacial Sands and Gravels (GSG)

6.5.1 As noted in Section 3.3 of this report, Till is expected to be widespread across the Scheme. To differentiate between granular and cohesive Till, the superficial deposits have been divided into Glacial Sands and Gravels (GSG) and Glacial Till (GT).

6.5.2 The boundary between GSG and the underlying Weathered Sandstone (WSST) is not always clear as the WSST was often recovered as sand during the cable percussive or dynamic sampling drilling process. Typically, the GSG was distinguishable by its fine to coarse sand grain size, gravel and cobles content and inclusion of various or mixed lithologies (quartz, sandstone, mudstone, siltstone, etc.).

6.5.3 Information from the 2019 Ground Investigation and historical boreholes show that GSG are widespread across the Scheme. GSG are typically described as medium to dense, reddish brown, slightly fine to coarse sand with fine to coarse, sub-rounded gravel. The GSG is often interbedded with the GT. GSG was generally more prevalent than GT. From Ch. 720 the GSG underlies the GT as it passes under the M54 J1. The GT then recedes at Ch. 1900 and the GSG is recorded at ground level. GSG is recorded at ground level to approximate depths of 4.0 – 5.0m bgl from Ch. 1900 to Ch. 2700. A deeper channel of GSG and GT is recorded under the existing Hilton Lane between Ch. 2550 and Ch. 2995. In this section, a second layer of GSG is recorded under the GT from 15.5m bgl (125.0m

AOD) and 22.5m bgl (119.0m AOD). From Ch. 2995 to Ch. 3280, the GSG is interbedded with the GT. A thick layer of GSG was recorded in BH20 and BH20A, where the base of the GSG was not recorded in their 20m drilled depths. It is assumed that this thick layer of GSG extends from Ch. 3280 to Ch. 3660 where it reduces to a thickness of approximately 4.0m and underlies granular alluvium and the Latherford Brook. As noted in Section 5.3., the GSG is suspected to form part of glacial channel north west of the M6 J11.

6.5.4 The thickness of the GSG varies from a minimum thickness of less than 1.0m at Ch. 2200 to a maximum thickness greater than 20.0m at Ch. 3400, where the base of the GSG was not recorded.

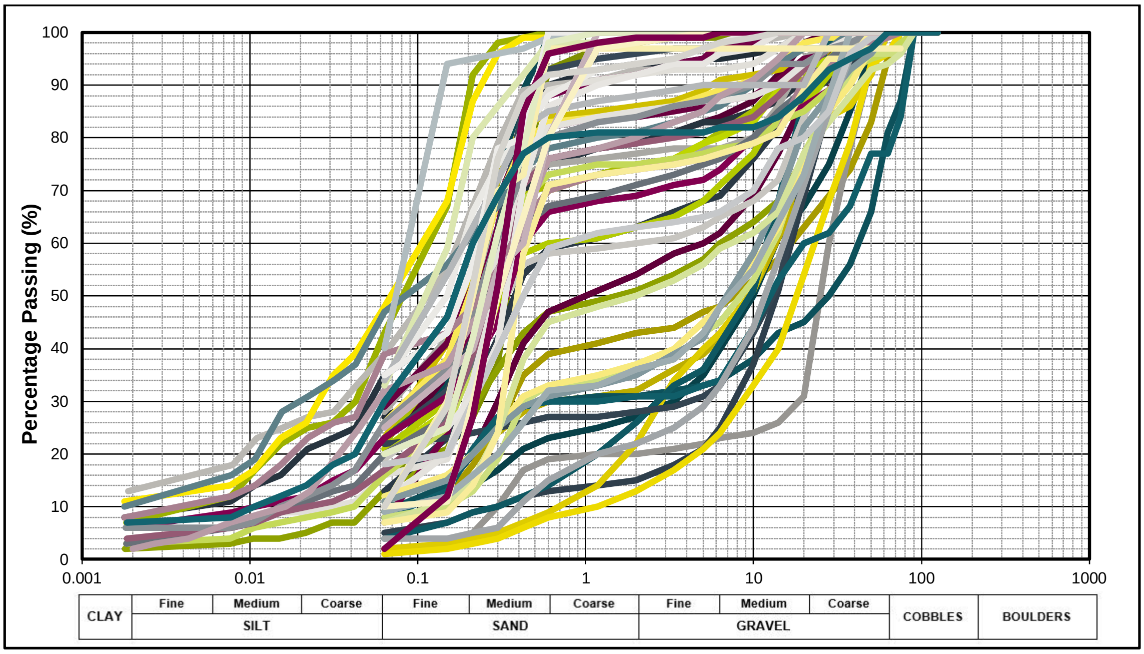
6.5.5 Details of the stratification of the GSG across the Scheme can be found on the Geological Long Section (HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1019 and HE514465-ACM-SGT-Z1_ML_PR_Z-DR-GE-1012 to 1015) shown in Appendix A.

6.5.6 Laboratory tests show that the moisture content for the GSG ranged from 1.9% to 28.6%, with most results sitting between 6% and 20%. An average moisture content value of 13.7% was recorded from the 218No. tests.

6.5.7 Results from 22No. laboratory tests undertaken in the GSG produced bulk density values (γ_{bulk}) between 19.32kN/m^3 and 22.66kN/m^3. An average value of 21.1kN/m^3 was calculated.

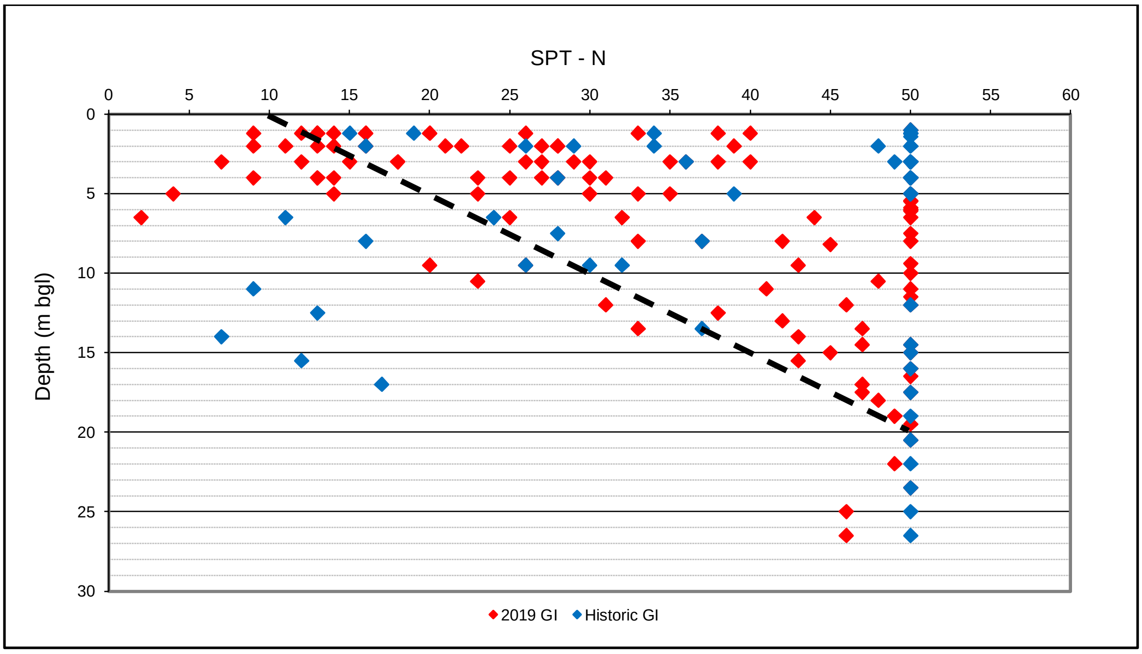
6.5.8 62 No. particle size distribution (PSD) tests were performed on the GSG and are presented on Figure 6.5-1. The PSD test results varied from slightly gravelly, very silty, fine to medium sand to sandy, fine to medium gravel. The results indicated fines contents ranging from 1% to 48%, sands contents ranging from 6% to 96% and gravel contents ranging from 0% to 85%. On average the GSG was recorded as 19% fines, 51% sands and 30% gravel content which can be described as a clayey, silty, very gravelly SAND in accordance with BS5930:2015. The samples with higher gravel content are spread evenly across the Scheme and typically located within the upper 2.0m to 3.0m of ground. PSD results with high fines content within the GSG are due to the presence of pockets or bands of clay.

Figure 6.5-1: GSG. Particle Size Distribution



6.5.9 216No. SPTs were undertaken within GSG layers, 64No. of which came from historic data. The values for SPT-N varied from 2 to 50 with approximately 52% of the tests achieving SPT-N results above 40. The average N value for the GSG is 36. Based on the test results shown in Figure 6.5-2 it is assumed that the SPT-N value increases with depth at a gradient of N = 10 + 2.0z, where z is the depth below ground level.

Figure 6.5-2: GSG. SPT-N vs Depth



6.5.10 According to BS8002-2015 the constant volume (also known as critical state) effective angle of shearing resistance can be determined by $\varphi'_{cv,k}$ = 30 ° +φ'_{ang} + φ'_{PSD}; where φ'_{ang} is contribution to $\varphi'_{cv,k}$ from the angularity of the particles; and φ'_{PSD} is contribution to $\varphi'_{cv,k}$ from the soil's particle size distribution.

6.5.11 As the average fines content is between 15% and 25% the dilatancy and the angularity has been determined by linear interpolation between zero and the value obtained for soils with ≥15% fines.

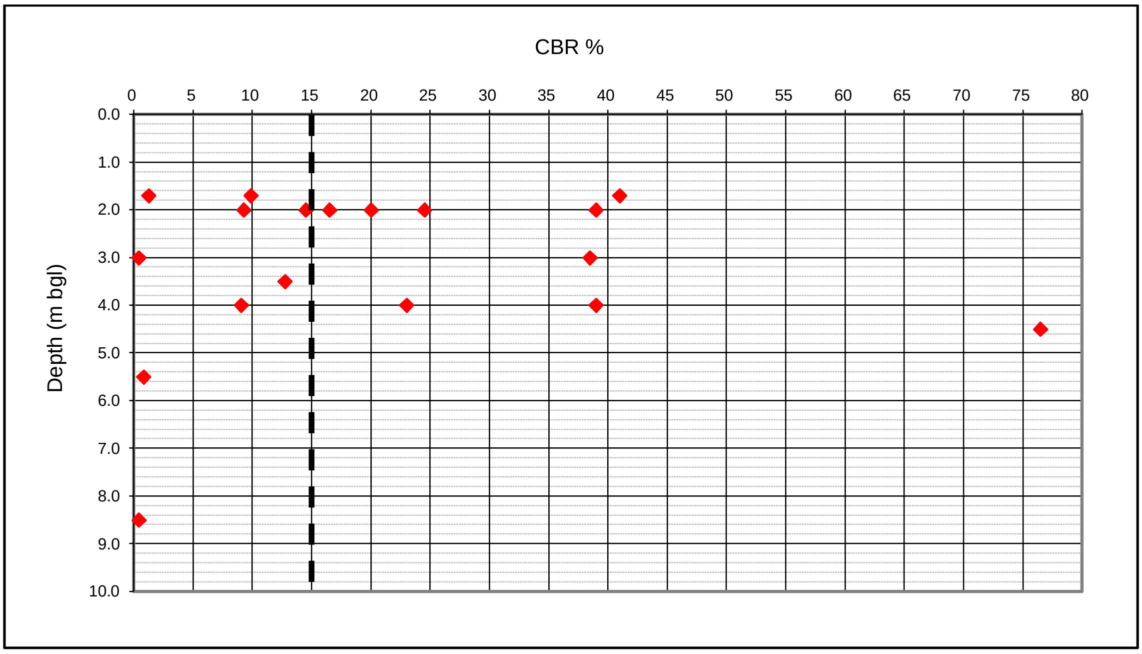
6.5.12 The uniformity coefficient for the average GSG is 40, which corresponds to a well graded granular soil ($\varphi'_{PSD} = 4°$), whilst the angularity of the particles is described as sub-angular to sub-rounded ($\varphi'_{ang} = 2°$, once interpolated due to fines content $\varphi'_{ang} = 1.2°$). Therefore, a constant volume friction angle of $\varphi'_{cv,k} = 35°$ can be determined.

6.5.13 To obtain the peak friction angle the contribution from soil dilatancy (φ'_{dil}) has been added; in this case φ'_{dil}=3.8°, once interpolated for the fines content. Hence, for GSG $\varphi'_{pk,k}$ =39°.

6.5.14 The ICE Manual of Geotechnical Engineering, 2012 recommends that the stiffness modulus for overconsolidated sands like the GSG are calculated using E'=2*SPT-N. Based on the SPT-N profile previously discussed, the typical E' profile is assumed to be E' = 20 + 4.0z MPa, where z is the depth below ground level.

6.5.15 13No. CBR laboratory tests were undertaken as part of the 2019 Ground Investigation. As shown in Figure 6.5-3 there is a large spread of results, ranging from 0.4% to 76.5% with an average CBR of 20%. The CBR tests were undertaken on remoulded samples using a 4.5kg rammer. Based on the guidance given in IAN 73/06, 2009, a well graded sand such as the GSG found at the Scheme, can achieve a CBR value of 60%. Based on the variability of the CBR test results and the information provided by IAN 73/06, 2009, a typical CBR value of 15% has been selected for the GSG.

Figure 6.5-3: GSG. CBR vs Depth



6.5.16 3No. Soakaway Tests and 3No. Falling Head Tests were undertaken in within the GSG layer. The permeability results of the infiltration tests ranged from 1.4×10^{-7} m/sec and 8.1×10^{-6} m/sec and produced an average permeability value of 2.60×10^{-6} m/sec. CIRIA C580, 2003 indicates that soils with these permeability properties are considered to have poor drainage properties.

6.5.17 A summary of the geotechnical testing for the GSG is summarised in Table 6.5.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the GSG have been selected and are presented in Table 6.5.2.

Table 6.5.1: Geotechnical Testing Summary. GSG

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	22	19.3 – 22.6	21.1
Moisture content (%)	218	1.9 – 28.6	13.7
PSD – Fines Content (%)	62	1 – 48	19
PSD – Sand Content (%)	62	6 – 96	51
PSD – Gravel Content (%)	62	0 – 85	30
SPT ‘N’ (No.)	216	2 - 50	36
Permeability, k (m/sec)	3No. Soakaway Tests 3No. Falling Head Tests	1.4×10^{-7} – 8.1×10^{-6}	2.6×10^{-6}
CBR (%)	13	0.4 – 76.5	20

Table 6.5.2: Indicative Values of Characteristic Parameters. GSG

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	21	Based on laboratory tests.
SPT ‘N’ (No.)	15 + 1.4z	Where z is the depth below ground level. Cautious estimate based on SPT tests.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	35	Based on the relationships provided by BS8002:2015
Effective peak friction angle, $\varphi'_{pk,k}$ (°)	39	Based on the relationships provided by BS8002:2015
Drained Young’s modulus, E’ (MPa)	30 + 2.8z	Where z is the depth below ground level. Profile is based on the SPT-N / E’ correlation provided by ICE Manual of Geotechnical Engineering, 2014.
Permeability, k (m/sec)	10^{-7} – 10^{-6}	Based on soakaway and falling head in-situ tests and guidance given in CIRIA C580.
CBR (%)	15	Cautious estimate based on laboratory test results and guidance given in IAN 73/06, 2009

6.6 Glacial Till (GT)

6.6.1 Glacial Till (GT) is encountered throughout the Scheme area, except for the area near the Latherford Brook (Ch. 3275 to Ch. 3630) where it was largely absent. GT is typically described as:

- Firm to stiff, reddish brown, slightly gravelly, slightly silty, slightly sandy, CLAY. The gravel is subangular to rounded fine to coarse.

6.6.2 Generally, the GT is interbedded with the GSG and overlies the bedrock material. More specifically, the GT is situated between two layers of GSG from the Ch. 500 to Ch. 720, before rising to the historic ground level prior the construction of the M54 motorway. From Ch. 720 to Ch. 1570 the GT is located under the M54 motorway fill material and over GSG. GT was not recorded from Ch. 1570 to Ch. 2040 where it then reappears under the GSG. GT is then recorded to overlie the GSG in the area of the suspected glacial channel, as discussed in Section 5 of this report. Pockets of GT are recorded within the GSG in boreholes BH23 and BH25.

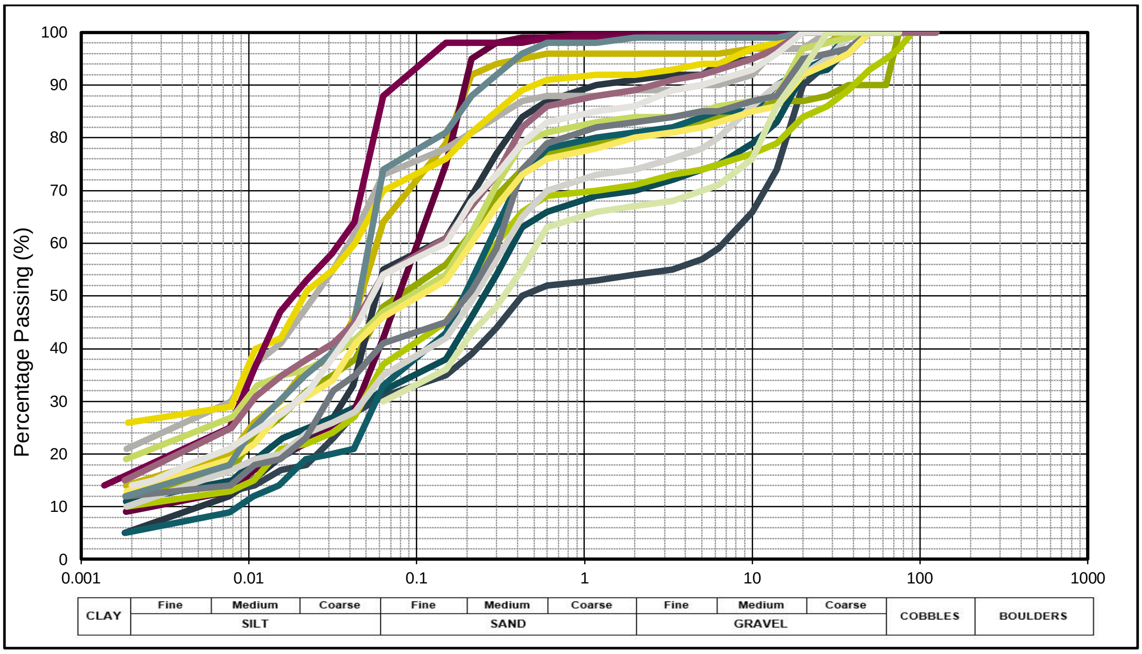
6.6.3 Across the site, different thicknesses of the GT layers are recorded. Under the M54 carriageway and M54 J1 roundabout (Ch. 500 to Ch. 1570), the thickness of the GT ranges from 1.2m (in SJ90SW85) to 7.5m (in BH01) and was recorded at a maximum depth of 11.0m bgl (117.8m AOD) in BH02. From Ch. 2040 to Ch. 2510 the GT is relatively consistent with a typical thickness of 3.0m and reaching depths of 7.0m bgl (132.8m AOD) in BH12. The depth and thickness of the GT increases under the existing Hilton Lane, reaching a maximum thickness of 11.0m and maximum depth of 17.5m bgl (125.0m AOD) at Ch. 2725. From Ch. 2725 to Ch. 3265, there are two layers of GT that are interbedded between the GSG and over the SST. The upper GT layer is approximately 2.5m thick and recorded between the depths of 6.5m bgl (131.3m AOD) and 9.5m bgl (128.3m AOD). The lower GT layer has a similar thickness and is recorded between the depths of 8.5m bgl (122.4m AOD) and 11.0m bgl (119.9m AOD). A band of approximately 4.0m thick GT was recorded under the MG/Eng Fill and M6 motorway, reaching a maximum depth of 10.4m bgl (125.86m AOD) in BH29. Borehole BH25 is in the area of the suspected glacial channel and recorded GT from 1.4m bgl (129.39m AOD) to 6.0m bgl (124.79m AOD) and a smaller GT pocket between 19.0m bgl (111.79m AOD) and 21.0m bgl (109.79m AOD).

6.6.4 139No. laboratory tests on the GT show that the moisture content for the material ranges from 2.6% to 28.1%, with most results sitting between 10% and 20%. The results produce an average moisture content value of 15%.

6.6.5 23 No. bulk density tests were undertaken as part of the laboratory testing. The tests produced values ranging from 19.4kN/m^3 to 23.9kN/m^3 and an average value of 21.2kN/m^3. Based on these results and information a typical bulk density of 21.0kN/m^3 has been selected.

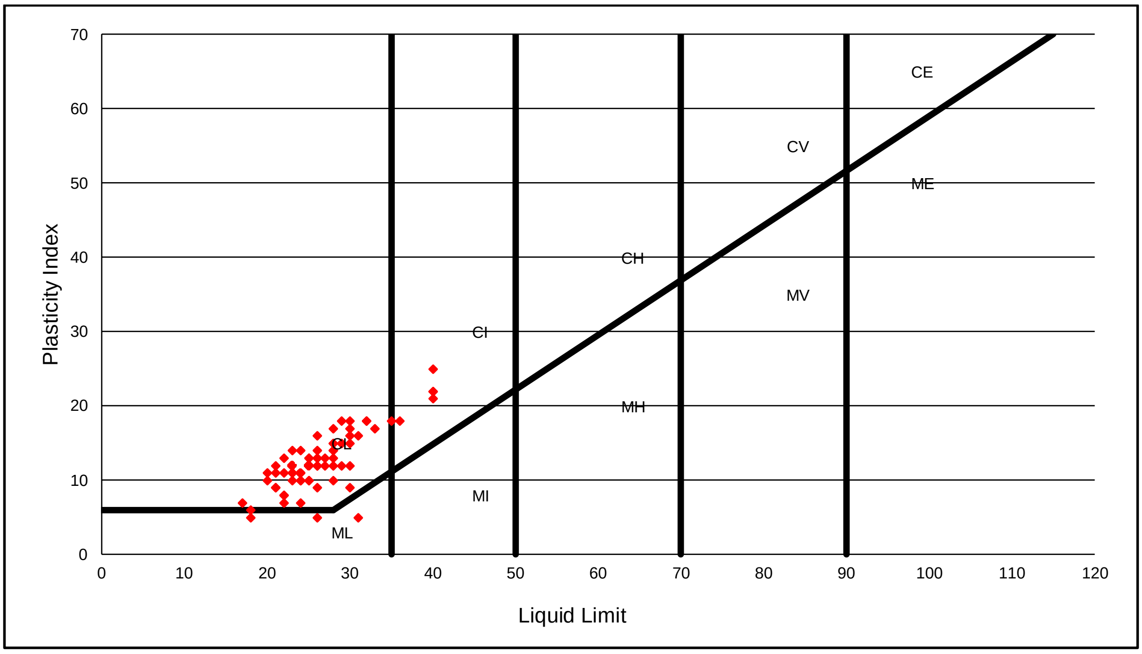
6.6.6 21No. PSD tests were performed on GT samples as part of the 2019 Ground Investigation and historical investigations and are include in Figure 6.6-1. Particle size distributions typically varied indicating fines contents ranging from 15% to 88%, sands contents ranging from 11% to 58% and gravel contents ranging from 0.3% to 46%. On average the PSD tests recorded 47% fines, 35% sands and 18% gravel content which can be generally described as slightly gravelly, silty, sandy, CLAY in accordance to BS5930:2015.

Figure 6.6-1: GT. Particle Size Distribution Test Result



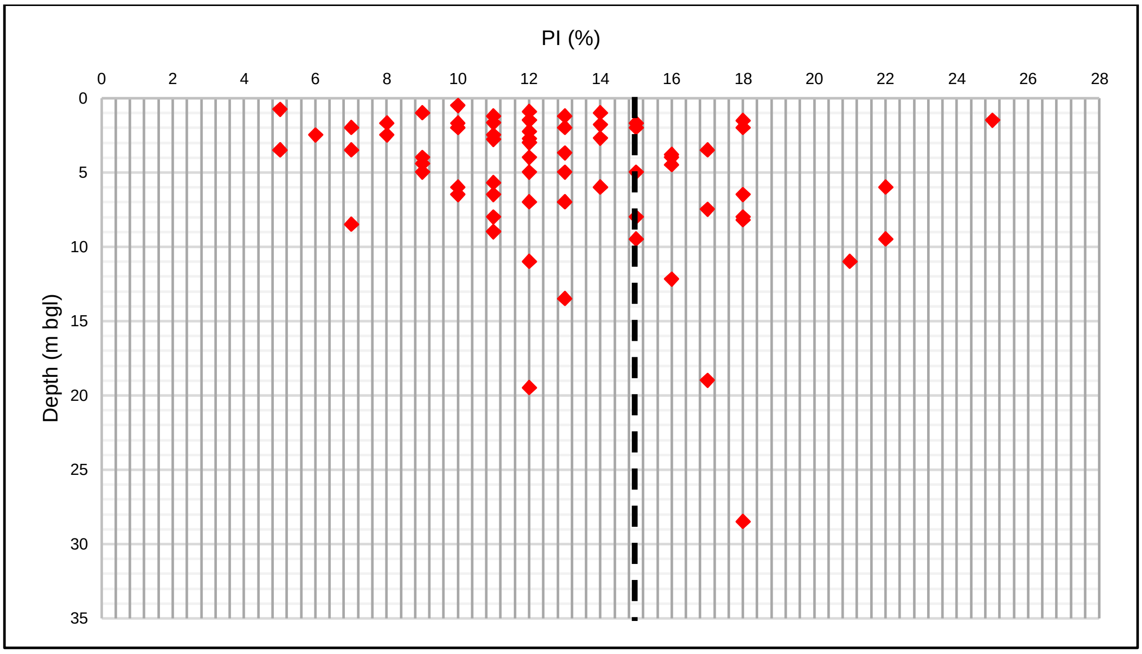
6.6.7 77No. Atterberg Limits Tests, as shown in Figure 6.6-2, indicate that the GT is typically classified as a low plasticity (CL) clay with a few samples identifying as low plasticity silts (ML) or intermediate plasticity clays (CI).

Figure 6.6-2: GT. Atterberg Limit A Line Plot



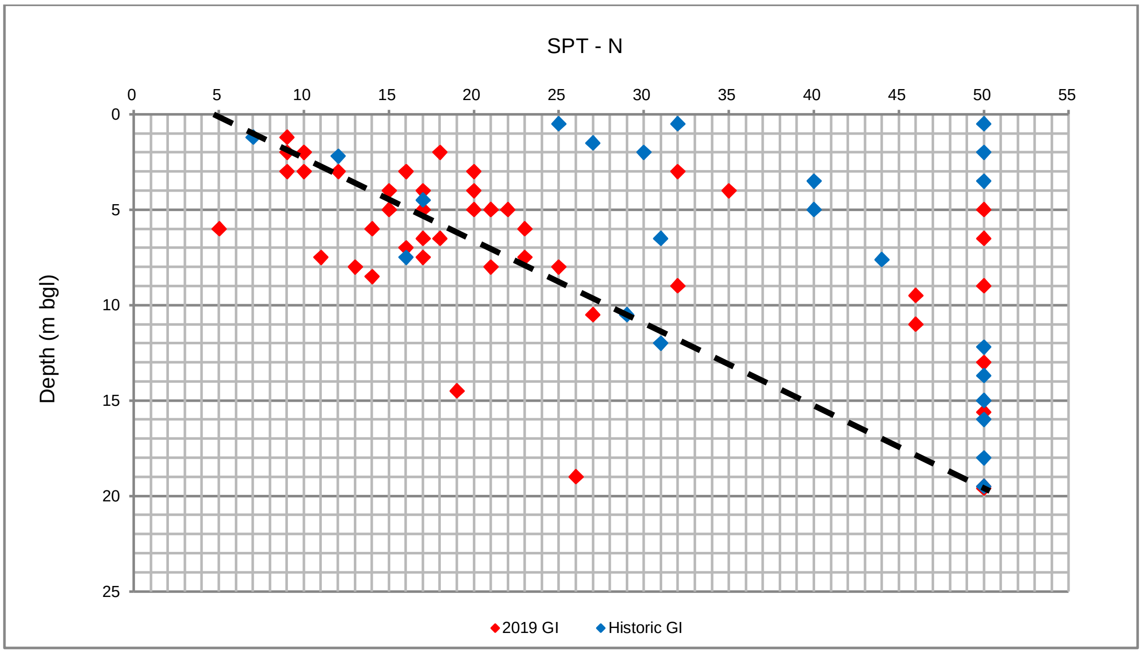
6.6.8 The plasticity index results shown in Figure 6.6-3 for the GT range from 5% to 25% with an average value of 12.7%. A cautious plasticity index of 15% was used to determine the constant volume friction angle of 28° using the correlations from BS8002:2015 and Sorensen and Okkels (2013).

Figure 6.6-3: GT. Plasticity Index vs Depth



6.6.9 73 No. SPT's were carried out in the GT with results varying between 5 to 50 with an average value of N = 28. As shown in Figure 6.6-4, there is a trend line between SPT increase and depth approximated to be N = 5 + 2.25z, where z is the depth below ground level.

Figure 6.6-4: GT. SPT-N vs Depth

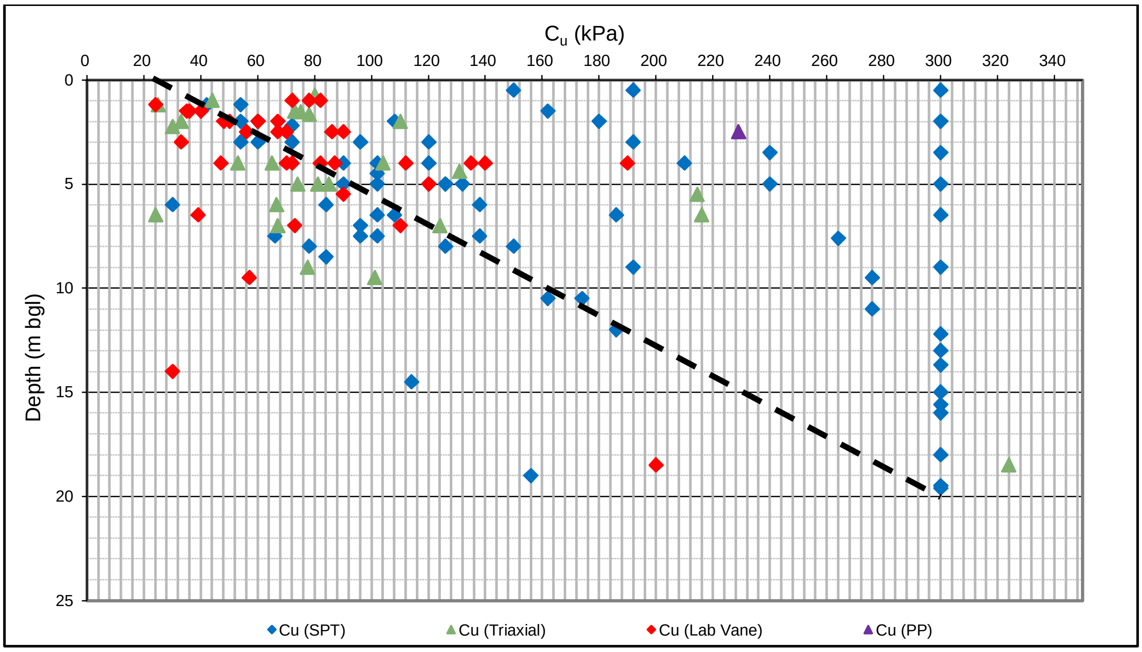


6.6.10 27No. triaxial strength tests were undertaken in the GT to determine the undrained shear strength (C_u) of the material. The SPT results presented in Figure 6.6-4 can be converted to C_u using the correlation C_u/SPT-N = 5, which is based on information provided by Stroud, 1974 for boulder clays and a derived plasticity index of 15%. 36No. Hand shear vane tests were carried out on suitable cohesive material within the hand dug pits and undisturbed samples.

6.6.11 Undrained shear strength results from in-situ and laboratory tests are presented in Figure 6.6-5. The results show a range from 24kPa to 324kPa with the majority of results lying between 30kPa and 140kPa.

6.6.12 As seen in Figure 6.6-5, there is an increase in strength profile with depth equating to a cautious C_u profile of $C_u = 25 + 11.25z$ kPa, where z is the depth below ground level.

Figure 6.6-5: GT. Undrained Shear Strength (Cu) vs Depth

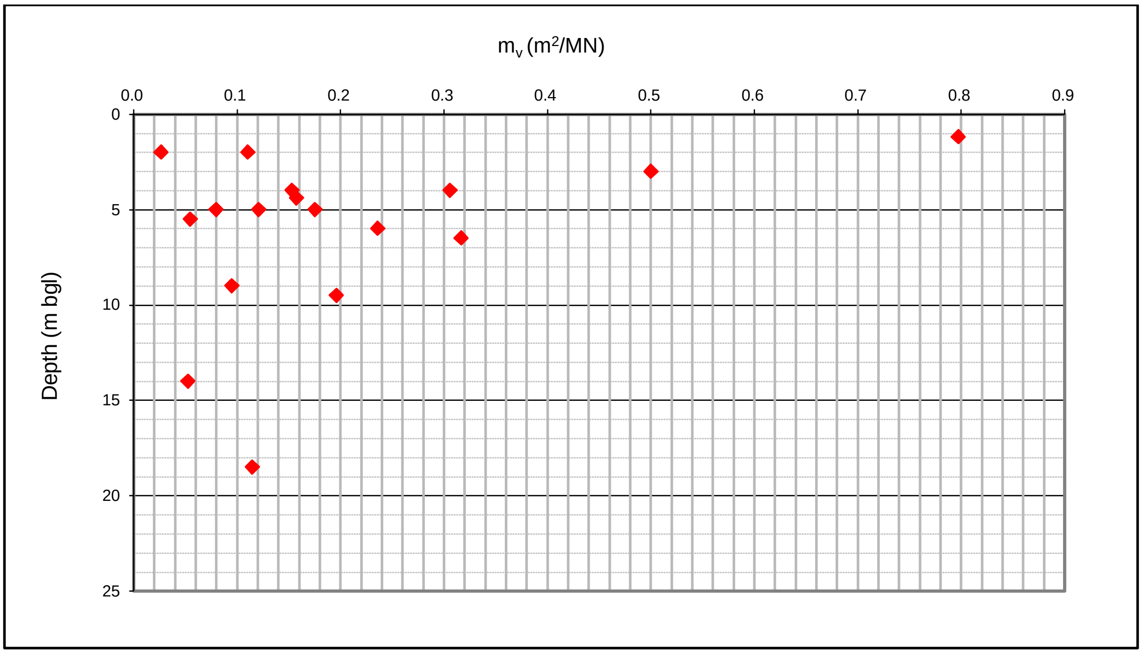


6.6.13 In order to determine the drained Young's Modulus, the relationship $E'/C_u = 250$ was used from Stroud et al (1975); assuming a plasticity index value of 15%. Based on a C_u profile of $C_u = 25 + 11.25z$ kPa, the E' of the GT is assumed to be $E' = 6.25 + 2.81z$ MPa, where z is the depth below ground level.

6.6.14 The undrained Young's Modulus stiffness (E_u) has been derived from the correlations from E'. Look (2004) notes that for silty clays such the GT encountered at the *Scheme*, the E_u is calculated using $E_u = E' / 0.7$. Using this correlation, the E_u profile for the GT is assumed to be $E_u = 8.92 + 4.0z$, where z is the depth below ground level.

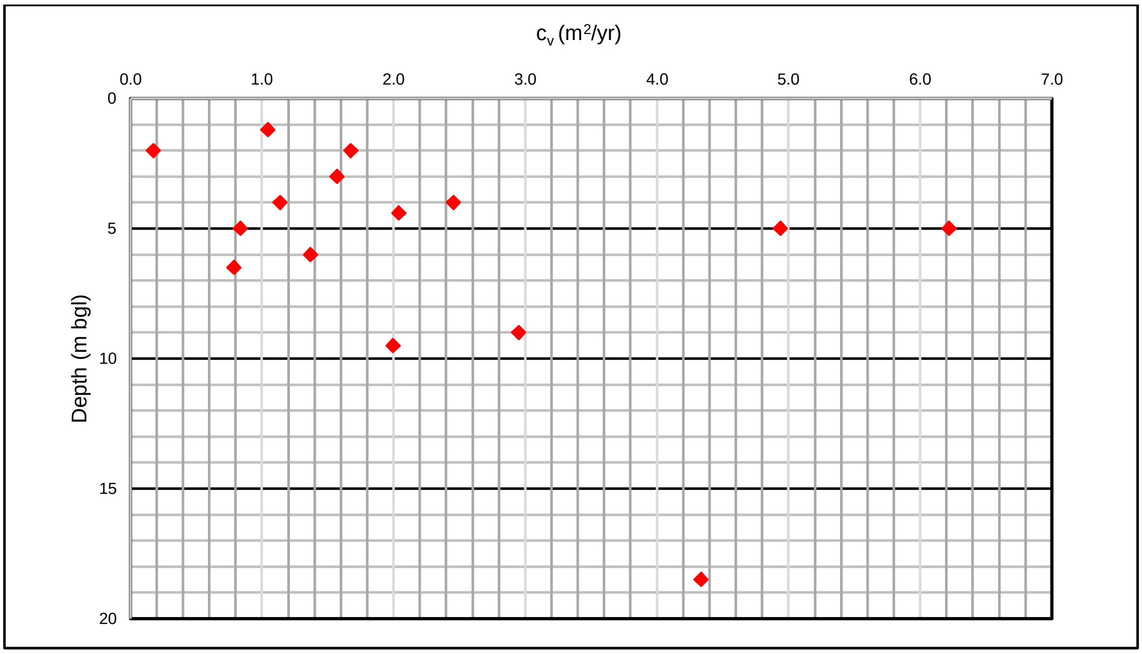
6.6.15 17No. One Dimensional Consolidation (oedometer) tests were undertaken within the GT. Calculating the coefficient of volume compressibility values (m_v) for in-situ conditions returned values between m_v= 0.03m^2/MN and 0.8m^2/MN, as shown in Figure 6.6-6. The average m_v of the tests is 0.21m^2/MN which Tomlinson (2001) describes as a medium compressibility material and is typical for weathered boulder clay. Tomlinson (2001) also notes that non-weathered boulder clays would be expected to be described as a low compressibility material with m_v values between 0.05m^2/MN and 0.10m^2/MN.

Figure 6.6-6: GT. Coefficient of Volume Compressibility (mv) vs Depth



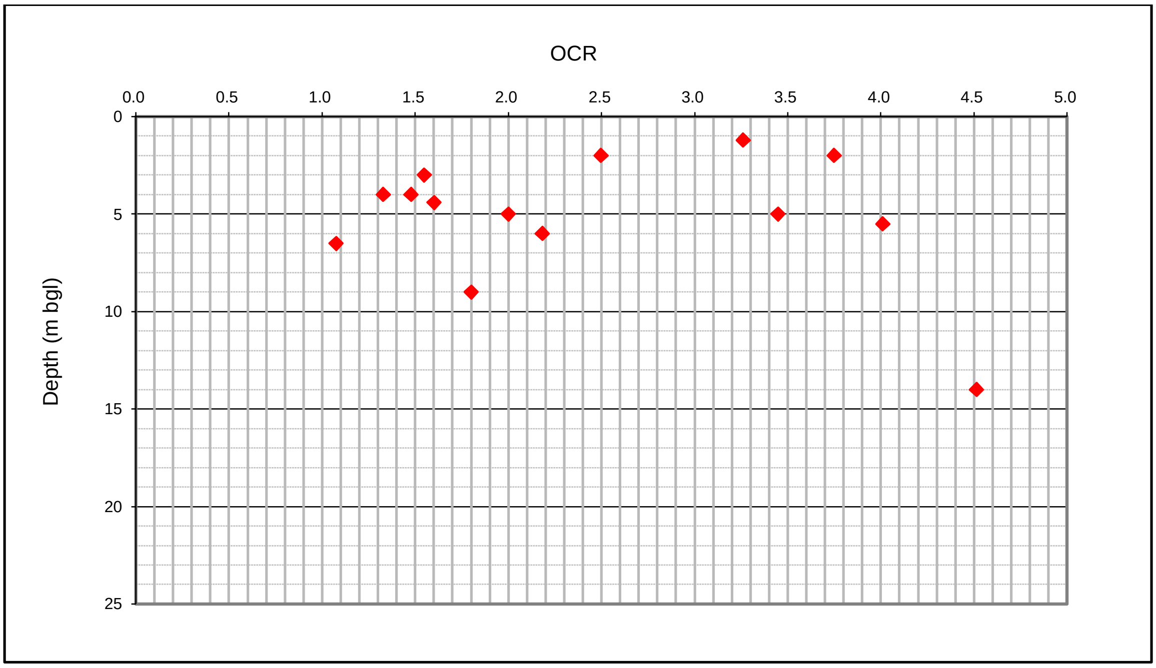
6.6.16 The coefficient of consolidation (c_v) has been calculated for in-situ conditions from the oedometer tests. As shown in Figure 6.6-7, the c_v values range from 0.17m^2/yr to 6.22m^2/yr and are typically between 0.8m^2/yr and 2.9m^2/yr. An average c_v for the results is calculated to be 2.0 m^2/yr.

Figure 6.6-7: GT. Coefficient Consolidation (cv) vs Depth



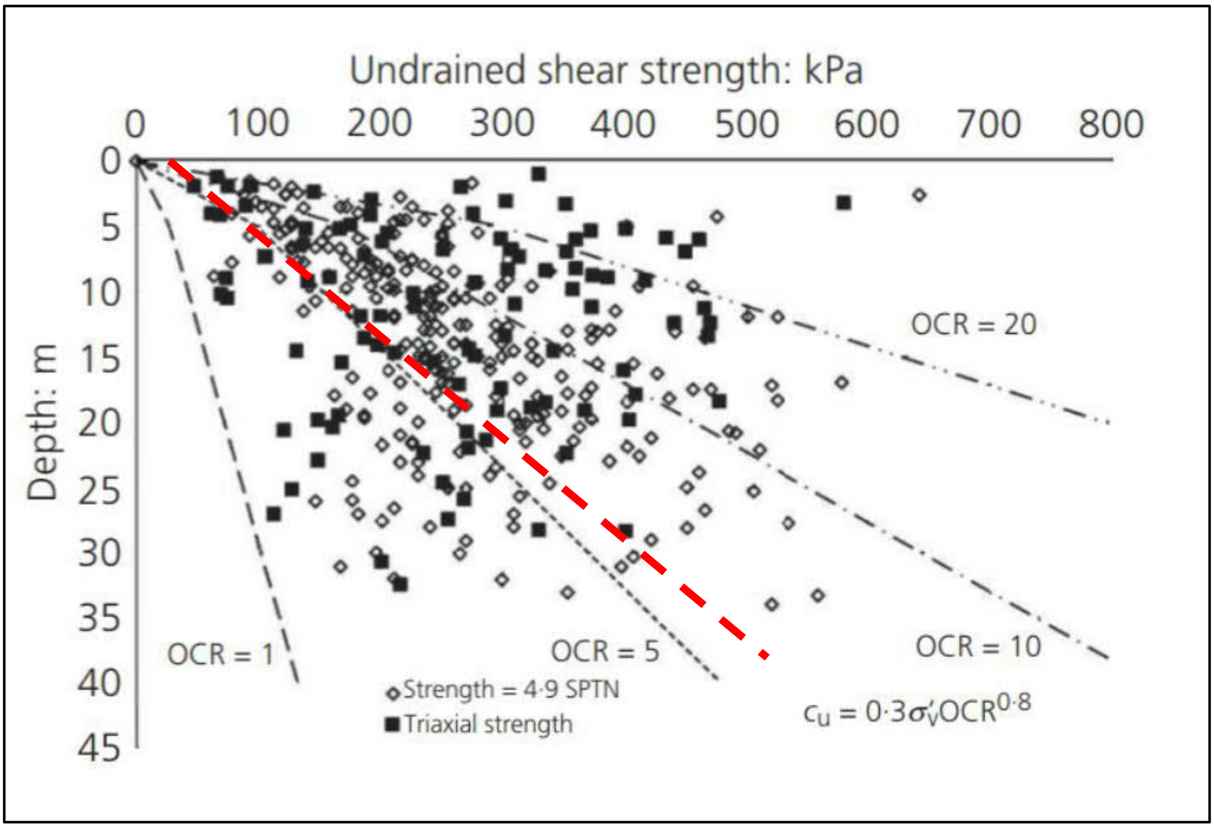
6.6.17 The Overconsolidation Ratio (OCR) of the GT has been calculated from the oedometer tests and is presented in Figure 6.6-8. The results show that the OCR results range from 0.75 to 4.5, with most results sitting between 1.0 and 4.0. Treating OCR values less than 1.0 as erroneous, the average OCR value for the GT is 2.5 which indicates that the material is overconsolidated but still lower than expected for Devensian Glacial Deposits.

Figure 6.6-8: GT. OCR vs Depth



6.6.18 Clarke (2018) produces a visualisation of the variation in C_u and OCR values with depth for Glacial Tills in Figure 6.6-9. Applying the typical C_u profile determined for the GT to Figure 6.6-9 shows that the material can be considered to have an OCR value of approximately 5.0. Based on this information and the OCR results highlighted in Figure 6.6-8, a typical OCR value of 4.0 has been selected for the GT.

Figure 6.6-9: Variation in Cu and OCR with depth for Glacial Tills (Clarke, 2018)



6.6.19 1No. Falling Head Test was undertaken within the GT material; BH15 at 3.5m bgl (137.2m AOD) to 4.5m bgl (136.2m AOD), to determine the permeability of the soil. The result returned a value of 9.07x10-8 m/sec, which is comparable to the permeabilities calculated using the formula below.

$$c_v = \frac{k}{\gamma_w \, m_v}$$

6.6.20 Calculating k using the above formula and data from the oedometer tests produces permeability (k) values ranging between $1.4\text{x}10^{-9}$ m/sec and $2.69\text{x}10^{-7}$ m/sec with an average value of $1.3\text{x}10^{-7}$ m/sec. Information from CIRIA C580, 2003 indicates that materials with permeability values of $\text{x}10^{-9}$ to $\text{x}10^{-7}$ have very poor to practically impervious properties.

6.6.21 1No. CBR test was carried out in the GT with a value of 2.75%. According to IAN 73/06 Revision 1 (2009). The CBR value for a sandy clay with a plasticity index value of 15% can be estimated to be between 3.5% and 5.5%.

6.6.22 A summary of the geotechnical testing for the GT is summarised in Table 6.6.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the GT have been selected and are presented in Table 6.6.2.

Table 6.6.1: Geotechnical Testing Summary. GT

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	23	19.4 – 23.9	21.2
Moisture content (%)	139	2.6 – 28.1	15
Plasticity Index, PI (%)	77	5 - 25	12.7
PSD – Fines content (%)	21	15 – 88	47
PSD – Sand content (%)	21	11 – 58	35
PSD – Gravel Content (%)	21	0.3 – 46	18
SPT ‘N’ (No.)	73	5 - 50	28
Undrained Shear Strength, C_u (kPa)	27No. Triaxial Tests 36No. Hand Shear Vane test	24 - 324	100
Coefficient of volume compressibility, m_v (m^2/MN)	17No Oedometer Tests	0.03 – 0.8	0.21
Coefficient of consolidation, c_v (m^2/yr)	17No Oedometer Tests	0.17 – 6.22	2.0
Permeability, k (m/sec)	1No. falling head test	9.07×10^{-8}	9.07×10^{-8}
CBR (%)	1	2.75	2.75

Table 6.6.2: Indicative Values of Characteristic Parameters. GT

Parameter	Indicative characteristic value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	21	Based on laboratory tests.
Plasticity Index, PI (%)	15	Cautious estimate based on laboratory test results
SPT 'N' (No.)	5 + 2.25z	Where z is the depth below ground level. Cautious estimate based on SPT 'N' results.
Undrained Shear Strength, C_u (kPa)	25 + 11.25z	Where z is depth below ground level. Cautious estimate based on laboratory and in-situ tests. SPT's have been correlated to C_u using information provided by Stroud, 1974.
Cohesion, c' (kPa)	0	Cautious estimate due to no available data.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	28	Based on correlations with plasticity index as given in BS8002:2015 and Sorensen and Okkels (2013).
Drained Young's Modulus, E' (MPa)	6.25 + 2.81z	Where z is the depth below ground level. Cautious estimate based on C_u results and correlations produced by Stroud et al, 1975
Undrained Young's Modulus, E_u (MPa)	8.92 + 4.0z	Where z is the depth below ground level. Cautions estimate based on E' results and correlations produced by the Look (2004)
Permeability, k (m/sec)	$10^{-8} – 10^{-9}$	Cautious estimate based on falling head and oedometer tests and typical values outlined in CIRIA C580.
Coefficient of volume compressibility, m_v (m^2/MN)	-	To be confirmed locally at the detailed design stage.
Coefficient of consolidation, c_v (m^2/yr)	-	To be confirmed locally at the detailed design stage.
OCR	4.0	Estimate based on oedometer testing and guidance given by Clarke (2018).
CBR (%)	3.5 – 5.5	Cautious estimate based on one CBR test and typical values provided in IAN 73/06.

6.7 Weathered Sandstone (WSST)

6.7.1 Weathered Sandstone (WSST) was encountered across most of the Scheme and mainly recovered in deeper exploratory holes. Thicker layers of WSST were found under the M54 J1, Featherstone Overbridge and the M6 J11.

6.7.2 As highlighted in Section 6.3, the upper and lower bounds of the WSST is not always clear. The WSST has been described as material that could either be drilled by cable percussion techniques or when drilled by rotary coring the boreholes recorded poor or no recovery. The degree of weathering for the WSST is deemed to be lie between a highly weathered (Class IV) to a completely weathered (Class V) rock. as stated in BS5930:1999. Because of the high degree of weathering, the geotechnical properties of the WSST can mainly be considered as an overconsolidated, very dense sand.

6.7.3 WSST recovered from cable percussion drilling was described as:

- Dense to very dense, reddish brown, silty sometimes slightly clayey, fine to medium, micaceous SAND.

6.7.4 WSST recovered from rotary drilling was described as:

- Extremely weak to very weak, completely to partially weathered, reddish brown, silty, fine to medium SANDSTONE. Fractures are horizontal and sub-horizontal, closely to medium spaced, undulating rough sometimes with clay infill. Occasional rare bands of mudstone and sometimes gravel beds.

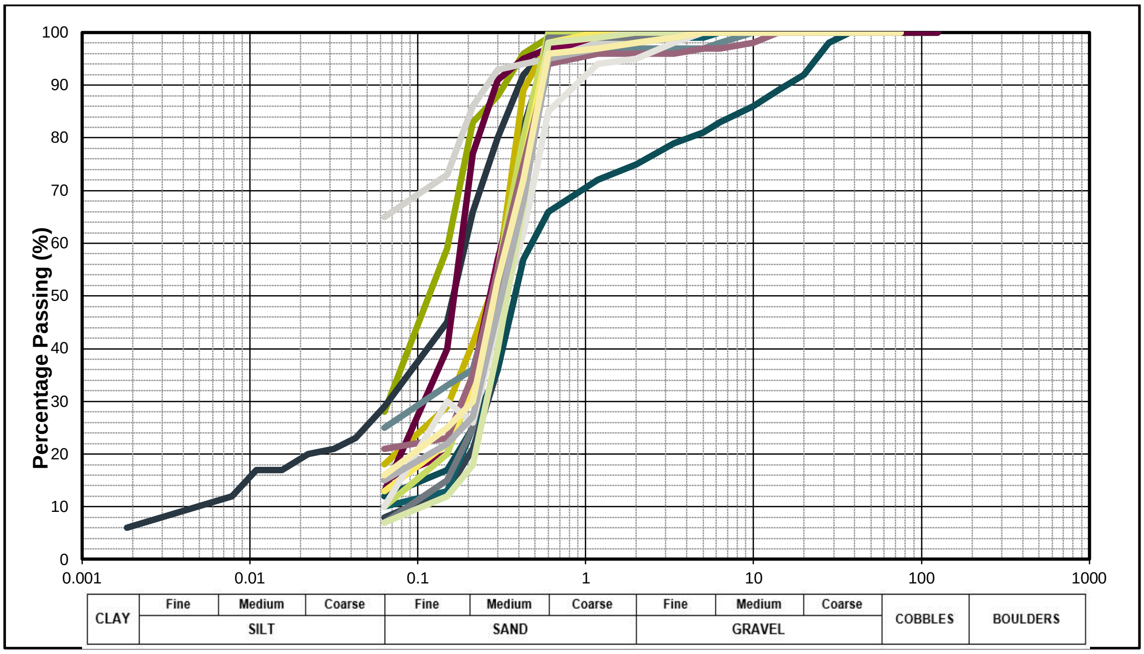
6.7.5 The top of the WSST was found between 143.0mAOD (in SJ90NW58) and 96.0mAOD (in BH22A). The thickness of WSST ranged from <1.0m (in SJ90SW239) to 23.0m (in BH06), whilst the maximum depth to the top of the WSST was recorded as 28.9m bgl (in BH22A). Typically, the GSG and GT overlay the WSST. The WSST overlies SST with a gradational weathering boundary between both units.

6.7.6 The 2019 Ground Investigation noted that the top of the WSST level sometimes differs significantly between bridge structure abutment locations. This was the case at both the Featherstone Overbridge and Accommodation Overbridge where there is a 5.3m and 9.0m difference in bedrock level between opposite abutments, respectively. This is supported by the Geology Plan (HE514465-ACM-SGT-M54_SW_PR_Z-DR-GE-1013) in Appendix A, which shows a lithological boundary between the abutments at these locations that separates the Sandstone and GT. The varying levels of bedrock across the abutments poses a differential settlement risk which has been added to the Geotechnical Risk Register in Section 7 of this report.

6.7.7 Moisture content results for the WSST ranged from 1.1% to 47% with values typically lying between 10% to 25%. An average moisture content value of 16.2% has been calculated for the WSST.

6.7.8 Figure 6.7-1 shows that the 19No. PSD tests undertaken within the WSST are very similar across the site with fines contents typically ranging from 7% to 35%, sand content between 63% and 95% and gravel content less than 5%. The average PSD tests classified the WSST as 16% mostly silty fines, 82% sands and 2% gravel content with the sands content being almost exclusively fine to medium.

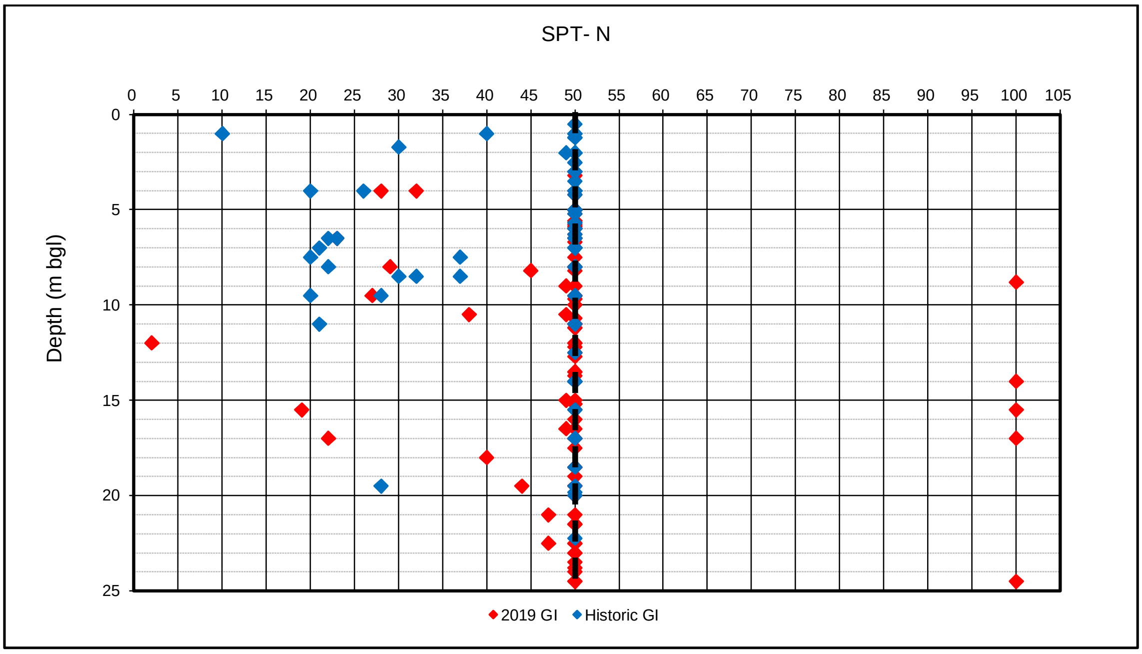
Figure 6.7-1: WSST. Particle Size Distribution



6.7.9 2No. bulk density tests were undertaken with values of 18.9kN/m^3 and 20.0kN/m^3. Using these laboratory tests and ranges given in BS8002:2015 and Barnes (2000) a typical value of 20.0kN/m^3 has been determined for the WSST.

6.7.10 Figure 6.7-2 shows that 154No. of the 191No. SPT's undertaken within the WSST gave results of N = 50 or higher. Up to 20% of the tests recorded values less than N = 40 with the SPT-N value of 2 expected to be a result of erroneous drilling. Based on the results shown in Figure 6.7-2, a typical cautious SPT-N value of 50 has been considered for the WSST.

Figure 6.7-2: WSST. SPT vs Depth



6.7.11 As the WSST was found to have a degree of weathering between Class IV and Class V and therefore may behave more like a very dense sand than a rock. The WSST has been assessed as a very dense, overconsolidated sand when determining the stiffness modulus. The ICE Manual of Geotechnical Engineering, 2012 recommends that the stiffness modulus for overconsolidated sands is calculated with E'=2*SPT-N. For the WSST, this produces an E' = 100MPa based on a SPT-N of 50.

6.7.12 According to BS8002-2015, as explained in Section 6.5, the determined constant volume and peak effective angle of shearing resistance are $\varphi'_{cv,k}$ = 36° and $\varphi'_{pk,k}$ =43°.

6.7.13 A cohesion value of c' = 0kPa has been assumed for the WSST as it is assumed to behave more like a sand than a rock.

6.7.14 The permeability of the WSST is expected to behave in a similar fashion to the GSG which has permeability values between 10^{-7} m/sec and 10^{-6} m/sec. This is supported by information in CIRIA C580 which describes fine sands and silts, comparable to the WSST, as possessing permeability properties between 10^{-5} m/sec and 10^{-7} m/sec.

6.7.15 IAN 73/06, 2009, notes that a poorly graded sand such as the WSST found at the Scheme, the material is likely to achieve a CBR value of 20%.

6.7.16 A summary of the geotechnical testing for the WSST is summarised in Table 6.7.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the WSST have been selected and are presented in Table 6.7.2.

Table 6.7.1: Geotechnical Testing Summary. WSST

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	2	18.9 – 20.0	19.4
Moisture content (%)	100	1.1 - 47	16
SPT 'N' (No.)	191	2 – 100	50

Table 6.7.2: Indicative Values of Characteristic Parameters. WSST

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	20	Based on laboratory test results and ranges given in BS 8002 (2015) and Barnes (2000).
SPT ‘N’ (No.)	50	Estimate based on SPT ‘N’ test results.
Constant volume effective friction angle, $\varphi'_{cv,k}$ (°)	36	Based on the relationships provided by BS8002:2015
Effective peak friction angle, $\varphi'_{pk,k}$ (°)	43	Based on the relationships provided by BS8002:2015
Cohesion, c’ (kPa)	0	Cautious estimate due to no available data.
Drained Young’s modulus, E’ (MPa)	100	Based on SPT results and correlations provided by ICE Manual of Geotechnical Engineering (2014).
Permeability, k (m/sec)	10^{-7} – 10^{-5}	Based on guidance given by CIRIA C580 derived values for GSG.
CBR (%)	20	Cautious estimate using IAN 73/06, 2009

6.8 Sandstone (SST)

6.8.1 Sandstone (SST), principally from the Chester Formation, was recovered in deeper exploratory holes, underlying the WSST. In the section between M54 J1 and Featherstone Overbridge and surrounding the M6 J11. As previously noted, the boundary between WSST and SST is not always clear. The SST is typically described as:

- Very weak to weak, thinly laminated to very thickly bedded, sometimes cross bedded, reddish brown to pinkish brown, fine to medium grained, micaceous, sometimes argillaceous SANDSTONE. Slightly to moderately weathered.

6.8.2 The SST is often interbedded with thin to thick interlaminations of very weak siltstone, mudstone and conglomerate. Bedding was recorded at 0-15° with fractures following closely with bedding. Fractures are undulating, rough to smooth, open sometimes with sand infill.

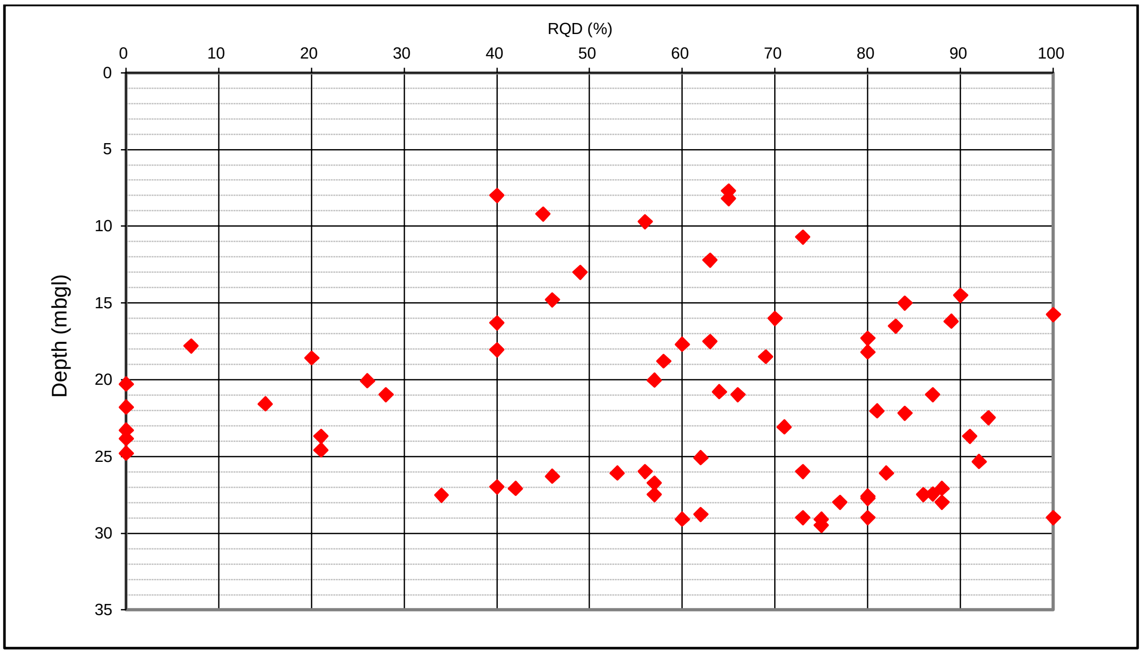
6.8.3 The top of the SST is recorded between 7.30m bgl (132.86m AOD) in BH13 and 27.7m bgl (108.6m AOD) in BH27. The base of the SST was not recorded in the exploratory holes. The WSST overlays the SST with a gradational boundary between units.

6.8.4 46No. laboratory test results show that the moisture content for the SST ranged from 0.4% to 18% and typically between 2% to 10%, with an average moisture content of 7.4%.

6.8.5 A total of 29No. bulk density tests were undertaken within the SST which give a range of 20.8kN/m^3 to 24.8kN/m^3 with an average value of 22.6kN/m^3. Based on these results a typical bulk density of 22.0kN/m^3 has been selected for the SST.

6.8.6 The quality of the recovered SST samples was highly variable with RQD values shown in Figure 6.8-1 typically ranging between 20% and 90%.

Figure 6.8-1: SST. RQD vs Depth



6.8.7 17No. SPTs were undertaken within the SST. All the SPT-N values reached the refusal criteria and were either terminated at N = 50 or when requested by the site investigation supervisor at N = 100. CIRIA181 notes that SPT-N values from terminated tests can be extrapolated based on their recorded penetration depths. It does however note that this only be used as a classification guide. If the extrapolated SPT-N values of the refusals are calculated, then SPT-N values vary between 300 and 3000. After discarding values above 600, the average N value is 540. According to Stroud, 1989 the extrapolated SPT N of 540 will correspond to a UCS of 18.0MPa.

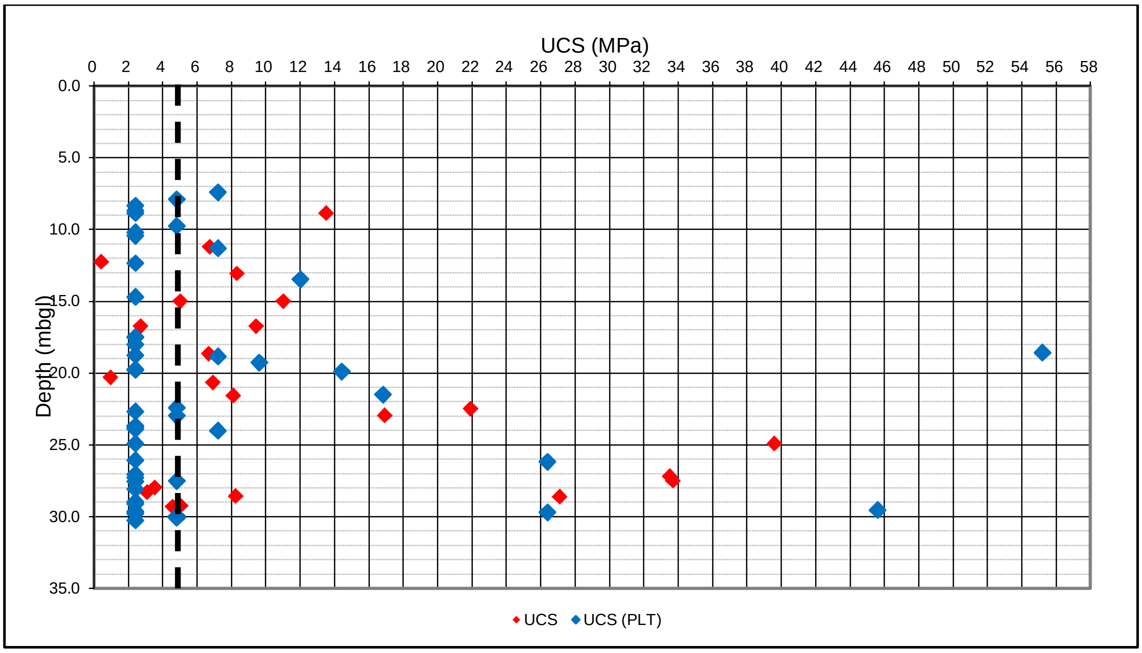
6.8.8 BS EN 14689:2018 estimates that for rocks typically described as very weak to weak, as the SST, the UCS ranges between 1.0MPa to 12.5MPa.

6.8.9 23No. UCS tests were completed on the SST, tests which produced a UCS range between 0.40MPa and 39.6MPa. Considering UCS values less than 1.0MPa and greater than 12.5MPa as outliers, the average UCS from UCS testing is 5.8MPa.

6.8.10 49.No Point Load Tests (PLT) were performed on SST samples with $I_{s(50)}$ values ranging from 0.1MPa and 2.3MPa with an average of 0.3MPa. 22No. PLT recorded $I_{s(50)}$ values of 0.0MPa that were deemed to have failed during the preparation stage and have therefore been discounted from this analysis. The high number of failed samples give an indication of the friable nature of the SST. With a relatively small data set and a large scatter of results, there was no clear correlation between PLT $I_{s(50)}$ and UCS. Broch and Franklin, 1972 proposed a universal $I_{s(50)}$ to UCS conversion factor of UCS = 24 x $I_{s(50)}$, which is based on eleven rock types across the UK. As can be seen in Figure 6.8-2, the UCS/PLT correlation of 24 has a well-defined fit and therefore has been considered appropriate for the SST. The UCS values converted from PLTs range from 2.4MPa and a 55.2MPa and are typically grouped between 2.4MPa and 12MPa. Discounting values above 12.5MPa, the average UCS from PLTs is 3.96MPa. The concentration of USC from PLT values at 2.4MPa is due to the format of the $I_{s(50)}$ results, which are presented in 0.1MPa increments.

6.8.11 Based on all the information detailed above and the results shown in Figure 6.8-2, a typical UCS value of 5.0MPa has been proposed for the SST.

Figure 6.8-2: SST. UCS vs Depth



6.8.12 The Young's Stiffness modulus, E', for the intact SST was determined in 8No. samples which were able to withstand the coring and preparation processes. The UCS deformability testing measures the stiffness of more intact rock and is not considered representative of the properties of the rock mass. The values recorded from Deformability in UCS tests ranged from 0.31GPa to 15.48GPa and produced an average value of 4.0GPa (4000MPa), which is higher than the rock mass stiffness calculated below using guidance from Tomlinson (2007). This method has been applied by electing a typical RQD value of 60% and a mass factor (j) of 0.35. Assuming the SST possesses both well cemented and poorly cemented bands, a Mr value of 250 has been selected. Using the formula below, a drained stiffness modulus for the SST is estimated to be 394MPa, which has been rounded up to 400MPa.

$$E' = 0.35\ (Mass\ factor) \times 250\ (Mr) \times\ 5\ (UCS) = 394MPa$$

6.8.13 The Guide to Permeability Indices, 2006 document notes that the permeability of rocks are dependent on the degree of fracturing, weathering, cementation, lithological variation and induration. The report notes that typical hydraulic conductivity (permeability) values for sandstone can range between $3x10^{-5}$ m/sec and $8x10^{-10}$ m/sec. It is thought that as the first tens of metres of the intact SST will have a more extensive network of fractures and will be more permeable. A cautious permeability value of 10^{-4} to 10^{-5} m/sec is considered for appropriate the SST.

6.8.14 A summary of the geotechnical testing for the SST is summarised in Table 6.8.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the SST have been selected and are presented in Table 6.8.2.

Table 6.8.1: Geotechnical Testing Summary. SST

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	29	20.8 – 24.8	22.6
Moisture content (%)	46	0.4 - 18	7.4
SPT 'N' (No.)	17	50 - 100	55
$I_{s(50)}$ from Point Load Tests (MPa)	49	0.1 – 2.3	0.3
UCS (MPa)	23	0.4 – 39.6	12 (including outliers)

Table 6.8.2: Indicative Values of Characteristic Parameters. SST

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	22.0	Based on laboratory test results.
SPT 'N' (No.)	> 100	Estimate based on SPT-N test results and extrapolated terminated SPT-N tests.
Drained Young's modulus, E' (MPa)	400	Based on correlations provided by Tomlinson (2007)
Permeability, k (m/sec)	10^{-4} – 10^{-5}	Based on guidance given in the Guide to Permeability Indices, 2006 and engineering judgement.
UCS (MPa)	5.0	Cautious estimate based on UCS and PLT results.

6.9 Siltstone (SLST)

6.9.1 Siltstone (SLST) was recovered sporadically in deeper exploratory holes located between Ch. 1400m – Ch. 3800m.

6.9.2 Siltstone was typically described as:

- Very weak to medium strong, very thinly to medium bedded, dark reddish brown, locally light grey, SILTSTONE with occasional mudstone laminations.
- Partially to slightly weathered. Bedding was recorded at 0-15° undulating smooth with occasional clay infill. Fractures typically follow bedding, medium to widely spaced, undulating to stepped smooth with localised clay infill <5mm.

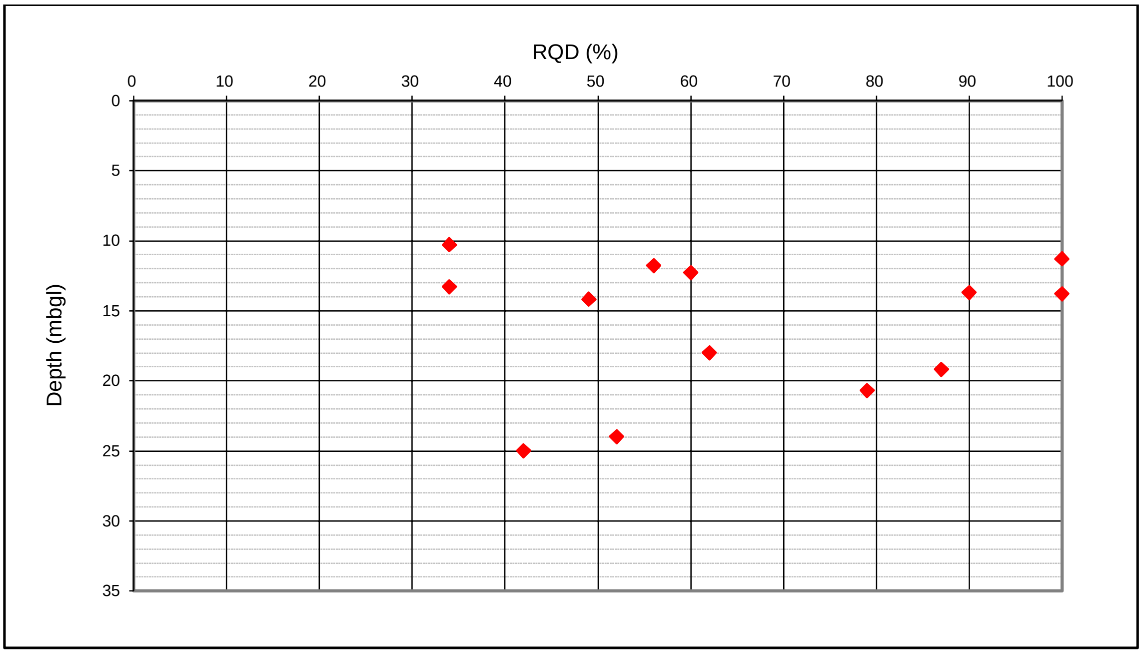
6.9.3 The SLST was recorded between 10.3m bgl (114.24m AOD) in BH22A and 29.5m bgl (107.97m AOD) in BH07. When observed, thickness of SLST ranged from <1.0m (in BH07) to 4.5m thick (in BH22A) and the maximum depth was not proven.

6.9.4 5No. bulk density tests were undertaken on SLST samples, recording values from 24.0kN/m^3 to 26.6kN/m^3 and an average of 25.0kN/m^3.

6.9.5 10No. moisture content tests were undertaken on SLST samples with the results ranging from 4.6% to 12% with an average of 8.9%.

6.9.6 Figure 6.9-1 shows there is a wide spread in RQD values ranging from 34% and 100%. Unlike WSST and SST which is more extensive across the Scheme, SLST is relatively limited and has therefore not been divided into weathered and less-weathered units.

Figure 6.9-1: SLST. RQD vs Depth



6.9.7 4No. SPTs were undertaken within the SLST bedrock, all reaching refusal with SPT N values of 50, 50, 50 and 100. When extrapolating the terminated SPT-N values the results vary between 250 and 850. Due to the low number of tests and wide spread of results, an average SPT-N value cannot be considered representative. Using the minimum value obtained and correlations from Stroud, 1989 however, shows that UCS value for SLST is approximately 4.0MPa.

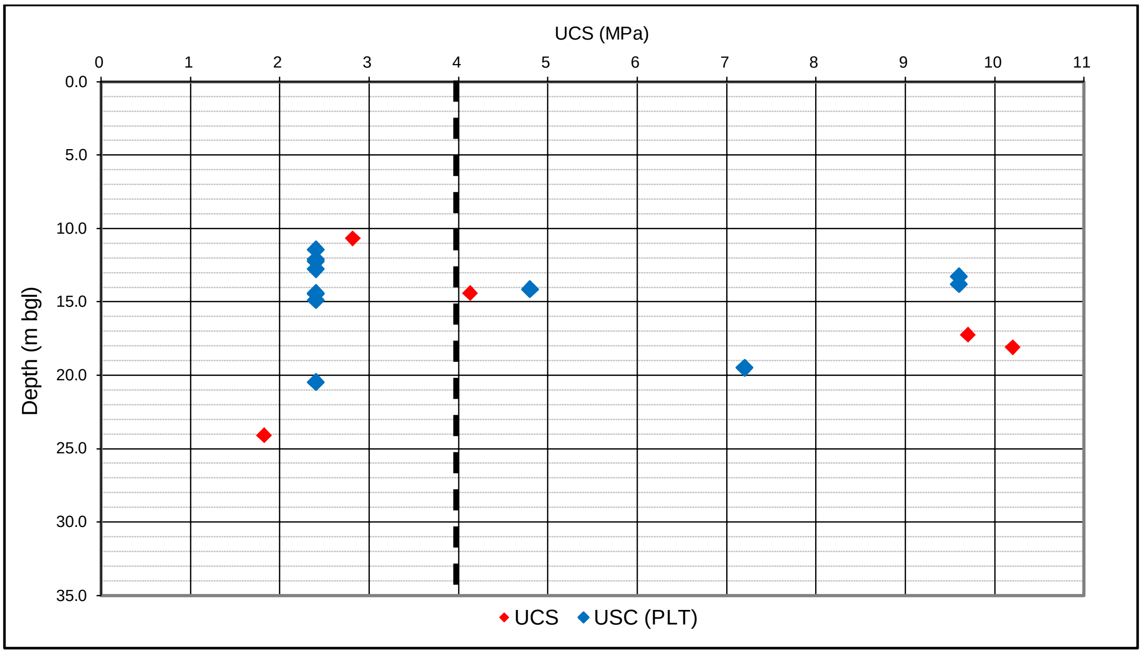
6.9.8 BS EN 14689:2018 estimates that for rocks typically described as very weak to medium strong, such as the SLST, the UCS ranges between 1.0MPa to 50.0MPa.

6.9.9 5No. USC tests were completed on the SLST samples which produced an UCS range between 1.82MPa and 10.2MPa with an average of 5.7MPa.

6.9.10 12No. PLTs were also performed on SLST samples and recorded $I_{s(50)}$ values between 0.1MPa and 0.4MPa with an average of 0.175MPa. Due to the limited number of samples, it is not possible to accurately determine a PLT $I_{s(50)}$ to UCS relationship. Without a site-specific correlation available, a universal correlation factor of UCS = 24 x $I_{s(50)}$ from Broch and Franklin, 1972 has been used for the SLST. 5No. PLTs recorded $I_{s(50)}$ values of 0.0MPa which is an indication of its friable nature when weathered/fractured. The UCS results from PLT ranged between 2.4MPa and 9.6MPa with an average of 4.2MPa. As with the SST, the concentration of USC from PLT values at 2.4MPa is due to the format of the $I_{s(50)}$ results, which are presented in 0.1MPa increments.

6.9.11 Based on all the information detailed above and the results shown in Figure 6.9-2, a typical UCS value of 4.0MPa has been proposed for the SLST.

Figure 6.9-2: SLST. UCS vs Depth



6.9.12 A single Young's Stiffness modulus was determined from UCS deformability testing with a value of 1.93GPa. UCS deformability testing measures the stiffness of intact rock is not considered representative of the properties of the rock mass. The stiffness of the in-situ rock can be derived from the equation below from Tomlinson (2008) which considers the UCS, mass factor (determined from RQD and fracture frequency) and Mr (ratio of elastic modulus of the intact rock to its unconfined compression) with values for Mr given in BS8004:2015. Using the typical UCS value of 4.0MPa the stiffness modulus of the SLST is MPa.

$$E = 0.4\ (Mass\ factor) \times 250\ (Mr) \times\ 4.0(UCS) = 400MPa$$

6.9.13 The Guide to Permeability Indices, 2006 document notes that Siltstones generally possess moderate to low permeability properties and are predominantly influenced by their fracture state.

6.9.14 A summary of the geotechnical testing for the SLST is summarised in Table 6.9.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the SLST have been selected and are presented in Table 6.9.2.

Table 6.9.1: Geotechnical Testing Summary. SLST

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	5	24.0 – 26.6	25.0
Moisture content (%)	10	4.6 – 12.0	8.9
SPT 'N' (No.)	4	50 - 100	62.5
$I_{s(50)}$ from Point Load Tests (MPa)	12	0.1 – 0.4	0.175
UCS (MPa)	5	1.82 – 10.2	5.7

Table 6.9.2: Indicative Values of Characteristic Parameters. SLST

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	22	Based on laboratory test results and ranges given in BS 8002 (2015) and Barnes (2000).
SPT ‘N’ (No.)	> 100	Cautious estimate based on SPT ‘N’ test results and extrapolated terminated SPT-N tests.
Drained Young’s modulus, E’ (MPa)	400	Based on correlations provided by Tomlinson (2008).
Permeability, k (m/sec)	$10^{-5} – 10^{-7}$	Based on guidance given in the Guide to Permeability Indices, 2006 and engineering judgement.
UCS (MPa)	4.0	Cautious estimate based on UCS and PLT results.

6.10 Mudstone (MST)

6.10.1 Mudstone (MST) was recovered from greater depths in exploratory holes located between Ch. 1400m – Ch. 3100m and predominantly between Lower Pool and Brookfield Farm ponds Ch. 2600 and Ch. 2800. The MST comprises the Clent Formation and Enville Member.

6.10.2 MST was typically described as:

- Extremely to very weak, moderately to highly weathered, indistinctly laminated to thickly bedded, reddish brown, locally mottled bluish grey, slightly silty, micaceous MUDSTONE. Sometimes interbedded with weak very thin beds of brown siltstone. Bedding varies between 0-15°. Discontinuities, horizontal, medium to widely spaced, stepped, striated, partly open, clean sometimes clay infill.

6.10.3 MST is found between 30.0m bgl (105.79m AOD) in BH04 and 5.0m bgl in SJ90NW50. The thickness of MST ranged from <1.0m (in BH05) to 6.7m thick (in BH17). MST was typically encountered as interbeds (<1m thick) throughout the SST. Geotechnical testing generally classified the MST as very weak to weak, clayey mudstone and is moderately to highly weathered.

6.10.4 Boreholes BH13 and BH16 at Ch. 2510 and Ch. 2735, respectively, recorded Weathered Mudstone (WMST). The WMST was recovered between the depths of 5.4m bgl (134.72m AOD) and 6.0m bgl (134.12m AOD) in BH13 and 18.5m bgl (123.96m AOD) and 21.0m bgl (124.46m AOD) in BH16. The WMST was described as:

- Stiff to very stiff, reddish brown, gravelly, silty CLAY. Gravel is subangular fine to medium of mudstone lithorelicts.

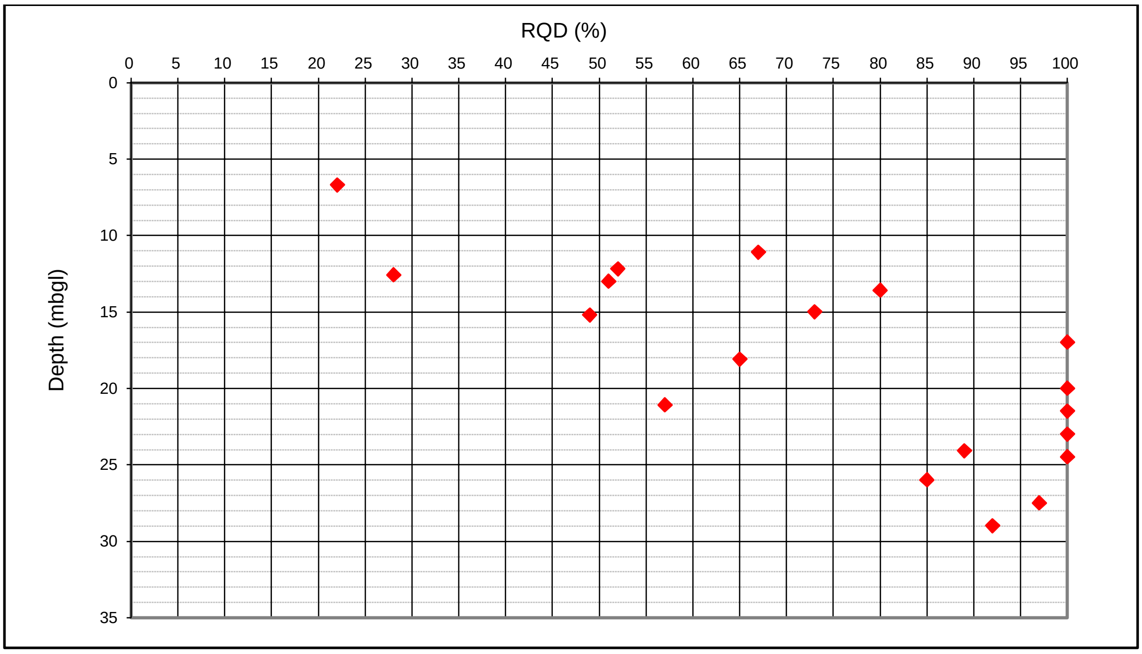
6.10.5 A total of 4No. bulk density tests were undertaken within the MST which give a range of 24.0kN/m^3 to 25.0kN/m^3 with an average value of 24.5kN/m^3. Based on these results a typical bulk density of 24.0kN/m^3 has been selected for the MST.

6.10.6 11No. moisture content tests were undertaken within the MST with results ranging from 4.7% to 25% and typically achieving values between 5% and 15%. An average moisture content value of 12.6% was recorded.

6.10.7 An effective cohesion value of 5kPa has been assumed for the MST, which is based on Look, 2004, CIRIA 181 and previous project experience.

6.10.8 Figure 6.10-1 shows that recorded RQD values range from 24% to 100% with an average RQD of 74% and an increase in RQD with depth.

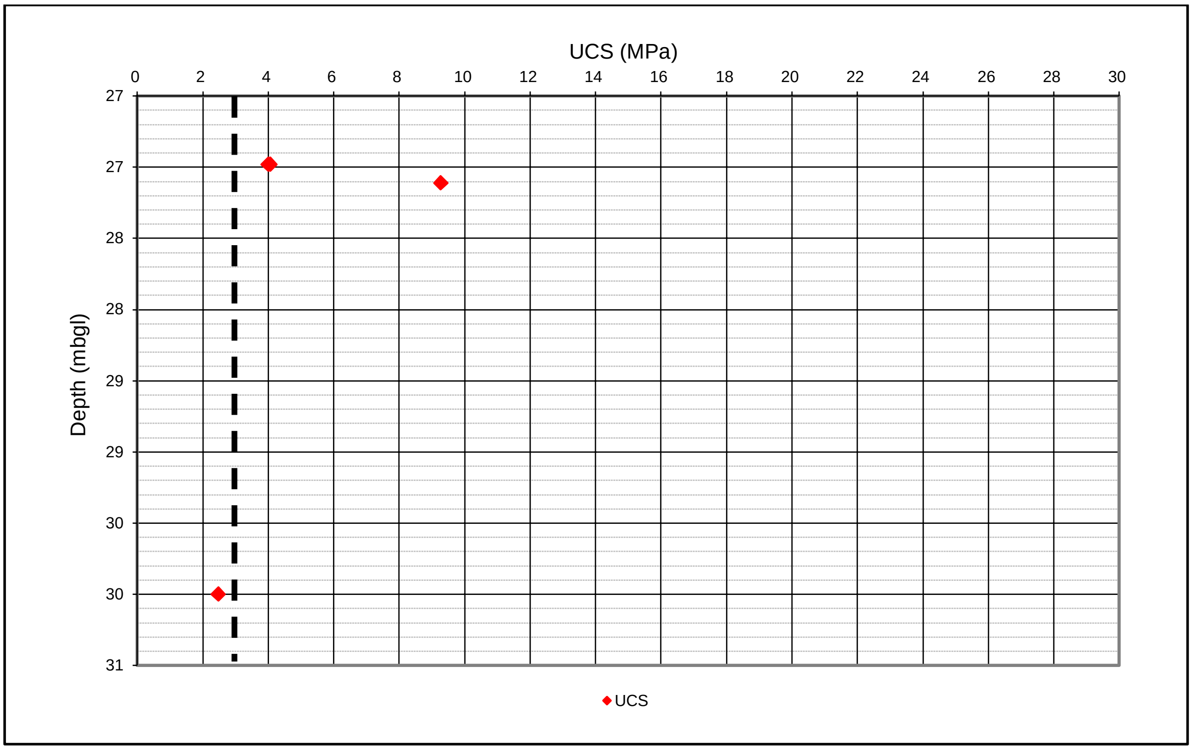
Figure 6.10-1: MST. RQD vs Depth



6.10.9 11No. SPT-N tests were conducted within the MST all recording SPT-N values of 50 before terminating. When extrapolating the terminated SPT-N values the results vary between 71 and 196, not including three results which were discarded as they were far greater than 200. The average of the considered results is 120 which according to Stroud, 1989 corresponds to a UCS of 2.0MPa.

6.10.10 As shown in Figure 6.10-2, 4No. UCS results were undertaken on MST samples and produced values between 2.4MPa and 9.2MPa, with an average of 4.9MPa. Based on these results a cautious UCS value of 3.0MPa has been derived for the MST.

Figure 6.10-2: MST. UCS vs Depth



6.10.11 To obtain the stiffness of the MST the equation below from Tomlinson (2008) was used to determine E' which considers the UCS, mass factor (determined from RQD and fracture frequency) and Mr (ratio of elastic modulus of the intact rock to its unconfined compression) with values for Mr given in BS 8004. Using the UCS of 3.0MPa the stiffness modulus is E' = 450MPa, which is at the lower end of the 490MPa to 615MPa range outlined by Seedhouse and Sanders, 1993 for grade III weathered mudstones.

$$E' = 0.5\,(Mass\ factor) \times 150\,(Mr) \times\ 3.0\,(UCS) = 225MPa$$

6.10.12 The Guide to Permeability Indices, 2006 document notes that mudstones generally possess low to very low permeability properties and are predominantly influenced by their fracture state.

6.10.13 A summary of the geotechnical testing for the MST is summarised in Table 6.10.1. Based on these results, established engineering correlations, engineering knowledge and available literature, indicative characteristic geotechnical properties for the MST have been selected and are presented in Table 6.10.2.

Table 6.10.1: Geotechnical Testing Summary. MST

Type of Test	Number of Tests	Range of Results	Mean
Bulk unit weight, γ_{bulk} (kN/m^3)	4	24.0 – 25.0	24.5
Moisture content (%)	11	4.7 - 25	12.6
SPT 'N' (No.)	11	50	50
UCS (MPa)	4	2.4 – 9.2	4.9

Table 6.10.2: Indicative Values of Characteristic Parameters. MST

Parameter	Indicative Characteristic Value	Remarks
Bulk unit weight, γ_{bulk} (kN/m^3)	22	Based on laboratory test results and ranges given in BS 8002 (2015) and Barnes (2000).
SPT 'N' (No.)	> 50	Cautious estimate based on SPT 'N' test results and extrapolated results.
Drained Young's modulus, E' (MPa)	225	Based on correlations provided by Tomlinson (2008) and guidance from CIRIA 181 (1999).
Permeability, k (m/sec)	$10^{-9} – 10^{-10}$	Based on guidance given in the Guide to Permeability Indices, 2006 and engineering judgement.
UCS (MPa)	3.0	Cautious estimate based on UCS and PLT results.

6.11 Conglomerate (CONG)

6.11.1 Conglomerate beds (CONG) were encountered in four boreholes, which were located near the Featherstone Overbridge, Hilton Lane Overbridge, Accommodation Bridge and Latherford Brook Bridge areas. The CONG does not indicate lateral continuity and appears in pockets of variable importance (thickness generally ≤1.0m) along the scheme. It does not have a widespread distribution and therefore could be considered a minor unit within the MST and WSST/SST.

6.11.2 CONG recovered as soil, mainly due to poor recovery, was typically described as:

- Dense to very dense slightly sandy, slightly clayey subangular to rounded medium to coarse gravel with cobbles of quartzite.

6.11.3 CONG recovered as rock was typically described as:

- Weak to medium strong pinkish/reddish brown/grey conglomerate comprising subangular to rounded fine to coarse gravel of quartzite, and occasionally sandstone and mudstone, within a fine to medium grained sandstone or mudstone matrix (depending on the surrounding material).

6.11.4 The top of the CONG was found between 128.13m AOD (10.71m bgl in BH17) and 111.30m AOD (14.35m bgl in BH21) with an average of 119.5m AOD. The maximum depth to the top of the CONG was recorded at 24.1m bgl (in BH08A). The thickness of CONG ranged from ≤1.0m (in BH16, BH17 and BH21) to >4.0m (in BH08A where the base of CONG was not found).

6.11.5 There is one moisture content result for the CONG of 6.6%.

6.11.6 This material was always been drilled by rotary techniques; only one SPT was undertaken on this material (BH16) in an area where recovery was poor. The result was an SPT-N of 43. This concurs with the description of the material in areas of core loss where it is described as dense gravel; this could be due to its poorly cemented nature.

6.11.7 8No. PLT were undertaken in CONG. Two of the $I_{s(50)}$ values were recorded as 0.0MPa which may be due to the poorly cemented core or drilling/handling induced fracturing. Using an $I_{s(50)}$ to UCS conversion factor of UCS = 24 x $I_{s(50)}$, UCS values determined for the remaining tests vary between 2.4MPa and 7.2MPa. It is considered highly likely that these results underestimate the strength of the CONG due to drilling disturbance caused by the contrasting strength of the matrix and clasts.

6.11.8 Due to its scarce distribution and generally low thickness, there are not many in-situ or laboratory tests available for the CONG; therefore, similar properties of the surrounding MST or WSST/SST can be cautiously adopted for this material.

6.11.9 However, it must be considered that the CONG formed of quartzite gravel may be an issue for the piling works and may require early piling contractor involvement during the detailed design stage. The piling contractor should be consulted to ensure that adequate information is available for the correct piling method to be chosen.

6.12 Geotechnical Parameters Summary

6.12.1 A summary of the indicative geotechnical characteristic parameters derived for the geological units are summarised in Table 6.12.1 below.

Table 6.12.1: Indicative Geotechnical Characteristic Parameters Summary

Geotechnical Parameter	Made Ground (MG/Eng)	Made Ground (MG FOB)	Alluvium Granular (ALL-G)	Glacial Sands and Gravels (GSG)	Glacial Till (GT)	Weathered Sandstone (WSST)	Sandstone (SST)	Siltstone (SLST)	Mudstone (MST)
PI (%)	14	13	I/D	N/A	15	N/A	N/A	N/A	N/A
γ_{bulk} (kN/m^3)	20.5	20.5	18	21	21	20	22	22	22
SPT-N	12	7	12	15 + 1.4z[1]	5 + 2.25z[1]	50	> 100	> 100	> 50
C_u (kPa)	60	40	N/A	N/A	25 + 11.25z[1]	N/A	N/A	N/A	N/A
$\varphi'_{cv,k}$ (°)	28	26	28	35	28	36	N/A	N/A	N/A
$\varphi'_{pk,k}$ (°)	-	-	30	39	-	43	N/A	N/A	N/A
c' (kPa)	0	0	N/A	N/A	0	0	N/A	N/A	N/A
E' (MPa)	15	10	12	30 + 2.8z[1]	6.25 + 2.81z[1]	100	400	400	225
E_u (MPa)	-	-	N/A	N/A	8.92 + 4.0z[1]	N/A	N/A	N/A	N/A
UCS (MPa)	N/A	N/A	N/A	N/A	N/A	N/A	5	4	3
m_v (m^2/MN)	I/D	I/D	N/A	N/A	DD	N/A	N/A	N/A	N/A
c_v (m^2/yr)	I/D	I/D	N/A	N/A	DD	N/A	N/A	N/A	N/A
k (m/sec)	10^{-4} - 10^{-7}	10^{-4} to 10^{-7}	10^{-4} - 10^{-6}	10^{-7} – 10^{-6}	10^{-8} – 10^{-9}	10^{-7} – 10^{-5}	10^{-4} – 10^{-5}	10^{-5} – 10^{-7}	10^{-9} – 10^{-10}
OCR	N/A	N/A	N/A	4	4	N/A	N/A	N/A	N/A
CBR (%)	3 – 5	1-2	3 - 10	15	3.5 – 5.5	20	I/D	I/D	I/D

Note: 1) Where z is depth below ground level
DD – To be determined at Detailed Design Stage.
N/A – Not applicable for this soil/rock type.
ID – Insufficient data to provide derived parameter.

6.13 Concrete Aggressivity

6.13.1 The concrete aggressivity testing, undertaken in accordance with BRE special Digest 1:2005 Concrete in Aggressive Ground, targeted the superficial deposits underlying the Scheme. The results for each stratum have been analysed to produce derived values for pH, water soluble sulphate as SO4 (mg/l) and total potential sulphate as SO4. In total 45No. samples for testing were taken.

6.13.2 The derived value for each stratum was then used to derive corresponding Design Sulphate (DS) and Aggressive Chemical Environment for Concrete (ACEC), according to limit set by BRE SD1.

6.13.3 These are shown in the Table 6.13.1 below.

Table 6.13.1: Concrete Aggressivity Class Summary

Geological Unit	Design Sulfate Class	ACEC Class	Number of Tests
Made Ground/Eng Fill	DS-2	AC-2	14
Made Ground FOB	DS-1	AC-1	5
GT	DS-1	AC-1	10
GSG	DS-1	AC-1	16

6.14 Soakaway Testing

6.14.1 Four soakaway testing trial pits were performed during the 2019 Ground Investigation as part of the attenuation ponds design. TP09, TP14 and TP18 were constructed to their specified depths of 2.50m bgl; whilst TP01 was terminated at 1.40m bgl due to local risk of ground instability. Prior to testing, groundwater was observed in all four trial pits, at depths ranging between 1.4m bgl and 2.5m bgl.

6.14.2 Details of the soakaway tests are included in the Ground Investigation Report (HE514465-BAM-EGT-ZZ-RP-WM-0001) and summarised in Table 6.14.1 below.

Table 6.14.1: Summary of Soakaway Tests

Exploratory Hole	Stratum	Soil Infiltration Rate (m/s)
TP01	MG FOB	$4.0\ e^{-5}$
TP09	GSG over GT	$1.0\ e^{-6}$
TP14	GSG over GT	$9.7\ e^{-7}$
TP18	GSG over GT	$6.6\ e^{-7}$

6.15 Permeability Testing

6.15.1 Falling Head tests were undertaken in boreholes BH15, BH16, BH22 and BH23 to determine the permeability of the targeted soil. Details of the falling head tests are included in the Ground Investigation Report (HE514465-BAM-EGT-ZZ-RP-WM-0001) and summarised in Table 6.15.1 below.

Table 6.15.1: Summary of Falling Head Tests

Borehole ID	Response Zone (m bgl)		Target Stratum	Permeability, k (m/s)
	Top	Base		
BH15	3.5	4.5	Cohesive GT	9.07 e^{-8}
BH16	2.2	3.8	GSG	4.72 e^{-6}
BH22	1.5	2.5	GSG	8.10 e^{-6}
BH23	3.0	4.0	GSG	1.40 e^{-7}

6.16 Compaction Testing

6.16.1 A total of 13No. compaction tests with a 4.5kg rammer have been undertaken on samples expected to be within cuttings. Compaction tests were completed on GSG and GSG and GT samples mixed together. A summary of the Maximum Dry Densities (MDD) and Optimum Moisture Contents (OCM) are presented in Table 6.16.1.

Table 6.16.1: Compaction Testing Summary

Geological Unit	MDD Range (Mg/m^3)	MDD Average (Mg/m^3)	OMC Range (%)	OMC Average (%)	Number of Tests
GSG	1.96 – 2.19	2.09	5.3 – 9.0	7.8	9
GSG and GT	2.01 – 2.1	2.07	7.4 – 9.1	8.0	4

6.16.2 12No. Moisture Condition Value (MCV) testing was undertaken on samples of GSG. A summary of the MCV results are presented in Table 6.16.2.

Table 6.16.2: MCV Testing Summary

Geological Unit	MCV Range (%)	MCV Average (%)	Number of Tests
GSG	0.1 – 12.6	4.6	12

6.17 Groundwater Level

6.17.1 Groundwater was encountered in many trial pits and boreholes. Out of a total of 33 boreholes drilled during the 2019 Ground Investigation, 16 recorded groundwater strikes between 1.5m bgl and 24.3m bgl. Water levels typically rose following the initial strike. Isolated strikes at greater depth were recorded near Featherstone Junction Overbridge and near Brookfield Farm Accommodation Overbridge. The groundwater strikes and the levels after their standing period and highest recorded levels are presented in Table 6.17.1. Exploratory holes not included in Table 6.17.1 did not encounter groundwater.

Table 6.17.1: Summary of Groundwater Levels

Borehole	Groundwater Strike (m bgl)	Depth after Standing Period (m bgl)	Highest monitored groundwater level/ strike (m bgl)	Geological Formation
BH01	9	4.1	4.1	GT
BH02	4.6	4.1	4.1	MG
BH03	-	-	0.6	MG
BH04	-	-	5.0	GSG
BH06	4	3.6	3.0	GSG
BH07	-	-	5.0	GSG
BH08	9.7	4.8	4.8	CWST
BH08A	-	-	2.0	GT
BH09	24.3	10.1	8.2	GSG
BH10	10.5	6.9	4.9	CWST
BH11	11.8	4	4.0	CWST
BH12	1.5	1.3	0.3	GSG
BH13	4.2	3.1	3.1	GSG
BH16	-	-	6.8	GSG
BH17	7.8	4	4.0	GSG
BH18	8.7	4.1	1.5	GSG
BH19	3	0.6	0.6	GT
BH20	14.5	14	10.7	GSG
BH21	2.5	2	1.0	ALL-G
BH22	2	1.5	1.5	GT
BH22A	-	-	+0.0	ALL-G
BH24	-	-	3.0	GT
BH25	-	-	7.0	GSG
BH26	-	-	4.7	MG
BH27	-	-	12.4	GSG
TP01	1.4	1	1.0	MG
TP05	3	2.9	2.9	GSG
TP09	2	1.8	1.8	GT
TP10	2	-	2.0	GSG
TP11	2.4	-	2.4	GSG
TP12	4.3	-	4.3	GT
TP14	1.3	-	1.3	GSG
TP17	3	1.7	1.7	GSG
TP18	2.5	-	2.5	GT
TP19	4.5	-	4.5	GSG

*Note: * The groundwater strike in BH19 originates in the GSG at 11.0m bgl and rises to 0.6m bgl, which is the approximate level of the nearby Brookfield Ponds. This is close to artesian water conditions and is considered a potential risk for construction. Further details of this are discussed in Section 7 of this report.*

6.17.2 Up to ten rounds of groundwater monitoring were undertaken between the 11th July 2019 and 25th November 2019 as part of the 2019 Ground Investigation. The results of the groundwater monitoring are summarised in Table 6.17.2 and a plot showing the ground water levels fluctuation with time, including boreholes and trial pits the water strikes and response after 20 minutes, is shown in Figure 6.17-1.