Welsh Government

A40 Llanddewi Velfrey to Penblewin Improvement

Geotechnical Design Report

A40LVP-ARP-HGT-SWI-RP-C-0002

P03 | S4

20/09/18

Contents

0 The Project

0.1 Context

0.1.1 In February 2017 the Welsh Government appointed Carillion, with Arup and RML (the 'Carillion Team') as their technical and environmental advisors, to develop the design of the proposed A40 Llanddewi Velfrey to Penblewin Improvements up to publication of draft Orders.

0.1.2 Carillion entered liquidation in January 2018. The Welsh Government subsequently appointed Arup, supported by RML, to continue the development of the design up to publication of draft Orders and to support the Welsh Government through the Statutory process.

0.1.3 In December 2004 the Welsh Government announced the outcome of the A40 West of St Clears study into the consideration of both single carriageway and dual carriageway improvements to the A40 between St Clears and Haverfordwest. This study came about as a result of a number of previous reports that all concluded that the A40 needed improvement.

0.2 Project history

0.2.1 In December 2004 the Minister announced the publication of his Addendum to the 2002 Trunk Road Forward Programme (TRFP) and this included two major single carriageway improvement schemes for the A40 west of St Clears. The improvements would use the 2+1 configuration allowing overtaking on the two lane direction, with overtaking prohibited in the one lane direction and would be delivered in the following phases:

1. A40 Penblewin - Slebech Park
2. A40 Llanddewi Velfrey - Penblewin.

0.2.2 The first of these projects, Penblewin - Slebech Park, was completed in March 2011.

0.2.3 In July 2013, Edwina Hart AM CStJ MBE, Minister for Economy, Science and Transport, published a written statement outlining her priorities for Transport. The statement included the following

"Improving the A40 has been identified as a priority by the Haven Waterway Enterprise Zone Board and I intend to undertake further development of previously proposed improvements."

0.2.4 On 12 November 2014, in providing an update on the closure of the Murco Refinery in Milford Haven, the Minister made an oral Statement in Plenary:

"In terms of transport links, I have instructed my officials to accelerate to the fullest extent possible the programme for delivering improvements at Llanddewi Velfrey."

0.2.5 In June 2015, in a written statement on the A40 Improvement Study the Minister noted *"It is my intention to progress delivery of the A40 Llanddewi Velfrey to Penblewin scheme as soon as possible…"*

0.3 The problems

0.3.1 Consultation with key stakeholders, including the Local Authority, Welsh Government Departments and the Regional Transport Planner has identified the following problems :

1. The road is substandard and where overtaking provision does exist it is currently not spread along the length of the A40 such as there are long lengths in each direction with no safe overtaking opportunities
2. Limited overtaking opportunities lead to poor journey time reliability and driver frustration.
3. Occasional convoys of heavy goods vehicles from the ferry ports and slow moving agricultural vehicles contribute to periods of platooning and journey time unreliability, which is exacerbated with limited overtaking opportunities.
4. Seasonal spikes in traffic volumes along the A40 especially during the summer months leads to slow moving traffic causing journey time unreliability, which is exacerbated with limited overtaking opportunities.
5. The community of Llanddewi Velfrey is severed by the A40, which reduces accessibility, increases risks of non-motorised user accidents and results in noise and air pollution.

6. There are many side road junctions and direct accesses to properties and agricultural fields off the A40, which contributes to operational problems along the road.
7. A mix of traffic types using the road, contributing to journey time unreliability and driver frustration, risky manoeuvres and collision incidents.
8. A lack of strategic public transport connectivity in Pembrokeshire generally means there is a dependence on the private car for inter-urban connections.

0.4 Scheme objectives

0.4.1 A number of transport planning objectives have been developed iteratively during previous development work and engagement on the A40 project, aiming to address one or more of the identified problems. During the early stages of Key Stage 3 the problems and objectives were refreshed during a focused workshop event with key stakeholders to take into account the WelTAG 2017 guidance and Wellbeing of Future Generations (Wales) Act wellbeing goals. The scheme objectives are:

O1 To enhance network resilience and improve accessibility along the east-west transport corridor to key employment, community and tourism destinations.

O2 To improve prosperity and provide better access to the county town of Haverfordwest, the Haven Enterprise Zone and the West Wales ports at Fishguard, Milford Haven and Pembroke Dock.

O3 To reduce community severance and provide health and amenity benefits.

O4 To reduce the number and severity of collisions.

O5 To promote active travel by cycling, horse riding and walking to provide opportunities for healthy lifestyles.

O6 To deliver a scheme that promotes social inclusion and integrates with the local transport network to better connect local communities to key transport hubs.

O7 Deliver a project that is sustainable in a globally responsible Wales, taking steps to reduce or offset waste and carbon.

O8 Give due consideration to the impact of transport on the environment and provide enhancement when practicable.

0.5 Purpose of this report

0.5.1 This report is the Preliminary Geotechnical Design Report (GDR) for the A40 Llanddewi Velfrey to Penblewin Improvements.

0.5.2 This report is in line with the procedures and certification process set out in with DMRB Standard HD22/08, Managing Geotechnical Risk [1] and HD22/08: Managing Geotechnical Risk Implementation Guidance – Wales [2] to manage geotechnical risks. Specific issues relating to ground modelling and geotechnical analyses both for the proposed earthworks and structures are considered within this report.

0.5.3 This Preliminary Geotechnical Design Report has been prepared to document the proposed approach and key principles for the geotechnical design of the scheme, and present the outcomes of the preliminary geotechnical design completed at Key Stage 3. A separate and more comprehensive Geotechnical Design Report will be prepared once detailed design has been completed at Key Stage 6.

0.5.4 This Preliminary Geotechnical Design Report has been written on the basis of the highway model and scheme proposals as of July 2017. Any subsequent change will be captured as part of the Key Stage 6 Geotechnical Design Report.

0.5.5 This report should be read in conjunction with the following:

a) Approval in Principles (AIP) which are currently prepared and will be submitted subsequent to this report

0.5.6 Ground Investigation Report (GIR), A40 Llanddewi Velfrey to Penblewin Improvements, issued for Welsh Government and OOGA review on May 2017. Report Ref.: A40LVP-ARP-VGT-SWI-RP-C-0001.

1 Earthworks

1.1 Introduction

1.1.1 The scheme has been divided into a number of different earthworks sections based on the presence of cuttings and embankments and the geographical locations of these features. The locations of these earthworks sections are shown on the Features and Constraints Plans (A40LVP-ARP-VGT-SWI-DR-C-0006 to 0008, P03).

1.1.2 Table 1 summarises the earthworks sections in terms of chainage extent, earthworks feature and approximate maximum height of each earthwork.

Table 1. Summary of earthworks sections

Earthworks section	**Chainage (m)**		**Earthwork feature**	**Approximate maximum height* (m)**
	From	**To**		
Penblewin Roundabout	Includes 0+000	0+040	Cutting (localised embankment in western end)	5m cutting 5m embankment
Embankment 1	0+040	0+370	Embankment	2.5m
Embankment 2	0+370	1+610	Embankment (localised cutting in western end)	4m (<2m localised cutting)
Approach cutting to Underpass 1	1+300	1+300	Cutting	6m
Embankment 3	1+610	2+030	Embankment	9m
Cutting 1	2+030	2+460	Cutting	6m
Embankment 4	2+460	2+720	Embankment	11m
Cutting 2	2+720	2+950	Cutting	14m
Embankment 5	2+950	3+480	Embankment	24m
Cutting 3	3+480	3+850	Cutting	21m
Llanddewi Velfrey Roundabout	Beyond 3+850		Cutting / Embankment	15m cutting 12m embankment
Note: *Height based on maximum height from toe to crest of earthwork				

1.2 Cutting stability

General

1.2.1 There are five cuttings along the proposed Scheme as summarised in Table 1. For the preliminary design analysis, all cutting slopes in the less weathered rock were assumed at 1(V):2(H).

1.2.2 The major cutting sections, with depth greater than 10m, are Cutting 2 between chainage 2+720m and 2+950m, Cutting 3 between 3+480m and 3+850m and the proposed Llanddewi Velfrey Roundabout.

1.2.3 The cuttings are anticipated to be formed predominantly within rock with an upper depth of fully weathered rock overlying less weathered rock.

1.2.4 Where the existing groundwater table is anticipated to be within the cutting depth, toe drains are to be provided to maintain groundwater levels below the base of the cutting. Crest drain are to be constructed where surface water may run towards the crest of the cutting slopes to prevent slope erosion.

1.2.5 Allowance for local seepages in the cuttings should be made.Proposed methodology for cutting stability assessment

1.2.6 The following section presents the proposed design methodology to be adopted for cutting stability design.

Surcharge loading assumed in stability assessment

1.2.7 It is proposed that external temporary loads are included in the analyses where they are considered to represent unfavourable conditions in terms of stability (e.g. where they occur at the crest of a slope). Where such loads are considered to be favourable, they are proposed to be omitted.

1.2.8 The following surcharge loads are proposed for cutting design in accordance with BS6031:2009 Code of Practice for Earthworks:

a) A permanent UDL of 10kN/m^2 to be applied to the surface at the top of the cutting to satisfy the minimum surcharge requirement set out in BS6031:2009.

A live UDL of 10kN/m^2 to be applied to areas on the slope that may be accessed by construction plant and maintenance plant.**Stability assessments**

1.2.9 Characterisation of the ground conditions specific to each cutting is proposed based on the ground investigation and other information presented in the Ground Investigation Report (GIR) Report Ref. A40LVP-ARP-VGT-SWI-RP-C-0001.

1.2.10 Characteristic strength parameters are to be attributed to the individual strata defined for each individual cutting. These are to be developed using the site wide derived values presented in the GIR for each material type. It is proposed that the factual data specific to each cutting is reviewed against the site wide parameters provided in the GIR; should there be significant difference between the site specific parameters and the site wide parameters, revised characteristic values specific to the cutting are to be adopted.

1.2.11 Both short term and long term stability of the cuttings are to be considered in the design. The undrained case is to be considered for the short term stability. The drained case is to be considered for the long term stability to reflect dissipation of pore water pressures after construction.

1.2.12 Once the parameters for the cutting are characterised, cutting stability is to be undertaken in general accordance with HD44/91 [3] and BS 6031 [4]. Stability calculations are to be undertaken with partial factors in accordance with BS 1997-1:2004 [5] and its associated UK National Annex [6].

1.2.13 The design is to adopt Design Approach 1 Combination 2 (DA1-C2) as this is considered to be the worst case design scenario for the cuttings. This is based on guidance presented in BS EN 1997-1:2004 [5] (Clause 2.4.7.3.4.2(3)) and BS 6031:2009 [4] (Clause 7.3.3). Sensitivity checks have been carried out to verify that DA1-C1 is not the least favourable case. The effect of considering DA1-C2 results in an equivalent global factor of safety of approximately 1.25 where an effective stress analysis is used.

1.2.14 The design is proposed to consider minimum weights of slip mass to avoid very shallow slip surfaces which are not considered critical for the performance of the cutting. The slope analyses will consider slip masses corresponding to a minimum depth of around 1m to the failure surface.

1.2.15 The cuttings are to be analysed using a combination of limit equilibrium and kinematic methods using proprietary software. The limit equilibrium method is to be used where it is appropriate to attribute mass strength parameters to the ground, whether it be fully weathered or partially weathered rock. The kinematic method is for situations where discrete planes of weakness may be present in the rock mass, associated with large scale discontinuities.

1.2.16 For limit equilibrium methods, circular failure mechanism are proposed to be analysed. The rock mass strength parameters derived as equivalent soil parameters using RocLab as presented in the GIR are proposed to be used.

1.2.17 Kinematic stability analyses are to be carried out for the cuttings where unfavourable discontinuity sets have been identified. Planar, wedge and toppling failure mechanisms are proposed to be considered as appropriate.

Summary of specific assessments for preliminary design

1.2.18 The sections below describe the preliminary cutting stability assessment undertaken based on the methodology above for a number of ground condition scenarios likely to be encountered on site.

1.2.19 The stability of the upper depth of weathered material is considered to be critical for the cuttings on the Scheme. Oasys Slope was used to carry out the circular failure mechanism assessment.

Scenario 1: Cuttings in granular weathered mudstone over mudstone (Drained, DA1-2)

1.2.20 Table 2 below summarises the design ground parameters which were assumed for the analyses.

Table 2. Cuttings in granular weathered mudstone over mudstone design parameters

Stratum	phi' (deg)	c' (kPa)
Granular weathered mudstone	34	0
Mudstone	28	60
Groundwater 1m below toe of cutting and varying depths below the cutting crest and slope		

1.2.21 The analyses considered varying thicknesses of granular weathered material and depths of groundwater below the crest of a 1(V): 2(H) cutting. It is possible that even in the deeper cuttings, Granular Weathered Mudstone could extend below the base of the cutting and so the analyses considered scenarios up to a maximum of 20m of granular material.

1.2.22 The groundwater table was defined as a straight line between 1m below the toe of cutting and a varying depth below crest of cutting.

1.2.23 A summary of the results in terms of the Over Design Factor is presented in Table 3 below.

Table 3. Cuttings in granular weathered mudstone over mudstone results summary

Depth of cutting (m)	Depth of groundwater below crest (m)	ODF
10	2.5	0.97
10	5.0	1.10
10	7.5	1.10
15	2.5	0.84
15	5.0	1.02
15	7.5	1.09

1.2.24 The results indicate that the cutting is stable for depths of up to 10m for depths of groundwater of approximately 2.5m below the crest, and for cutting depths of up to 15m where groundwater depths are at least 5m below crest.

Scenario 2: Cuttings in cohesive weathered mudstone over mudstone (Drained, DA1-2)

1.2.25 Significant thicknesses of Cohesive Weathered Mudstone daylighting in the cuttings would impact on stability.

1.2.26 In terms of shallow cuttings, the results of the analyses undertaken for the embankments slopes formed from Cohesive Weathered Mudstone are relevant, as presented in Section 1.3.

1.2.27 Table 4 below summarises the design ground parameters which were assumed for the analyses when considering shallow slips through the Cohesive Weathered Mudstone.

Table 4. Cuttings in cohesive weathered mudstone over mudstone design parameters

Stratum	phi' (degrees)	c' (kPa)
Cohesive weathered mudstone	29	2 (revised from GIR)
Mudstone	28	60
ru = 0.2 (revised from GIR)		

1.2.28 The linear relationship used in the Mohr-Coulomb equation to calculate the shear strength of the soil is conservative at small vertical stresses. As the failure slips considered in this scenario are shallow, and therefore under low stress situations, a small c' value was introduced for the Cohesive Weathered Mudstone, as a reasonable response to this conservatism. In addition, published guidance on the selection of shear strength parameters mentions that the use of a zero or very small effective cohesion intercept would be quite inappropriate for shale or other materials where diagenetic bonding are likely to lead to significant cohesion values (Trenter 2001).

1.2.29 A conservative c' value of 2kPa has therefore been selected for preliminary cut slope design.

1.2.30 An r_u value was introduced instead of a groundwater table to allow for potential higher pore water pressures at shallow depth in cohesive deposits.

1.2.31 The conclusion of the preliminary design for such shallow failures is that thicknesses of Cohesive Weathered Mudstone greater than around 4m would be unstable at slope angles of 1(V): 2(H).

1.2.32 A check is also needed for deeper failures in cuttings as there may be a permanent water table in cuttings unlike in embankments. This consideration is discussed below.

1.2.33 Analyses considered varying thicknesses of cohesive weathered material and depths of groundwater below the crest of the cutting. The analyses considered depths of up to 20m of cohesive material.

1.2.34 The groundwater table was defined as a straight line between 1m below the toe of cutting and varying depths below crest of cutting.

1.2.35 A summary of the results in terms of the resulting Over Design Factor for these deep seated slips is presented in Table 5 below.

Table 5. Cuttings in cohesive weathered mudstone over mudstone results summary

Depth of cutting (m)	Depth of groundwater below crest (m)	ODF
10	2.5	0.87
10	5.0	1.05
10	7.5	1.08
15	2.5	0.74
15	5.0	0.89
15	7.5	1.00

1.2.36 The results show that when the cutting is entirely within cohesive material, the groundwater table needs to be 5m below the crest for it to be stable for 10m deep cuttings, and 7.5m below the crest for 15m deep cuttings.

Analyses summary

1.2.37 The summary of the preliminary design assessment for 1 in 2 cuttings is as follows:

a) The groundwater level is critical in the assessment of the stability of the cuttings.
b) Cuttings in Granular Weathered Mudstone are stable up to 10m for depths of groundwater below crest of about 2.5m or lower, and up to 15m for depths of groundwater below crest of 5m or lower.
c) When the cutting is entirely within cohesive material, the groundwater table needs to be 5m below ground with 10m cuttings,

and 7.5m below ground for 15m cuttings to ensure stability against deep seated failure. However, thicknesses of Cohesive Weathered Mudstone greater than around 4m would result in shallow failures.

Cutting 1

General

1.2.38 Cutting 1 is approximately 370m in length and up to 6m in depth (from crest to toe) at approximate chainage 2+360m.

1.2.39 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A.

1.2.40 BH101 indicates that the ground conditions in this area comprise Granular Weathered Mudstone over Mudstone with a groundwater table at 3.2mbgl. Based on "Scenario 1: Cuttings in granular weathered mudstone over mudstone (Drained, DA1-2)", the cutting is assessed to be stable with slopes of 1(V):2(H).

1.2.41 As Cutting 1 is shallower than the other cuttings on the Scheme and is directly underlain by unweathered bedrock from shallow depth, Cutting 1 is not proposed to be discussed in further details for the preliminary design stage.

Cutting 2

General

1.2.42 Cutting 2 is approximately 230m in length and up to 14m in depth (from crest to toe) at approximate chainage 2+840m; see cross section on Figure 1.

1.2.43 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.2.44 BH10 recovered an extensive depth (approximately 10m) of weathered material described as 'cohesive'. Although described as cohesive, the upper 2.2m of this material is shown to contain only 8%

fine material based on PSD testing, and hence this upper layer along with the underlying stratum to 4.5m depth is considered granular for the design. Based on this, the design for Cutting 2 considers 4.5m of Weathered Granular Mudstone over 7.5m of Cohesive Weathered Mudstone underlain by Mudstone.

1.2.45 It should be noted that Weathered Conglomerate has been recovered overlying Conglomerate in the southern end of the cutting, however, this stratigraphy is considered to be localised and not representative of the majority of the cutting.

1.2.46 Groundwater monitoring in BH11 recorded the equilibrium groundwater level between 3.6m and 4.2m depth. It is therefore prudent to assume a groundwater table at 3.5m depth or higher along the cutting. The Weathered Conglomerate in the south part of the cutting has been identified to correspond to a series of springs further east in the Scheme (see Drawings A40LVP-ARP-VGT-SWI-DR-C-0007 and 0008). There is therefore a potential that this stratum may yield some water seepages during excavation. It is recommended to undertake further groundwater monitoring in BH10 and BH11 for detailed design. Based on the above, local seepages during excavation can be anticipated in the weathered zone.

Limit equilibrium analysis

1.2.47 Paragraph 1.2.34 summarises the results of limit equilibrium analyses for various scenarios, including comparisons between granular and cohesive weathered mudstone, and various groundwater levels.

1.2.48 Based on the evidence from the relevant boreholes, at Cutting 2 there could be an extensive thicknesses of cohesive material (up to around 7.5m) and relatively shallow groundwater (at around 3.5m).

1.2.49 The preliminary design calculations as set out in Scenario 2, indicate that for these ground and groundwater conditions, the 1(V):2(H) slope might not be viable in Cutting 2. Slope angles of 1(V):2.5(H) are likely to be needed for stability. Additional ground investigation is however proposed to support this conclusion at detailed design.

Kinematic analysis

1.2.50 Based on dip angles and orientations indicated on the geological maps, the cutting centreline appears to be located along the line of axis of an anticline with bedding planes falling into the proposed side slopes of the cutting; see Figure 2 in the GIR (Report Ref.: A40LVP-ARP-VGT-SWI-RP-C-0001). Based on this, for the purpose of preliminary design, the cutting is assumed to be stable in terms of planar, wedge and toppling failure.

1.2.51 BH09 recorded a 45 degrees bedding angle. The orientation of the bedding recorded in the borehole is not known but based on the geological information, this should be towards the cut.

Cutting 3

General

1.2.52 Cutting 3 is approximately 270m in length and up to 21m in height at approximate chainage 3+640m; see cross section on Figure 2.

1.2.53 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.2.54 The design considers 2.0m of Granular Weathered Mudstone overlying Mudstone.

1.2.55 Groundwater was struck in the lower part of the weathered mudstone at 1.8m depth however the subsequent monitoring did not record any groundwater. The water strike may be perched within a local permeable zone in the bedrock but is not anticipated to be associated with the main groundwater body. Based on this and the cutting located on a 'high point' in the topography, the groundwater level is interpreted to be below the base of the cutting. Further groundwater monitoring in BH04 is recommended to verify this assumption ahead of detailed design. Based on the above, local seepages during excavation can be anticipated in the weathered zone.

Limit equilibrium analysis

1.2.56 Paragraph 1.2.34 summarises the results of limit equilibrium analyses for various scenarios, including comparison between granular and cohesive weathered mudstone, and varying groundwater table.

1.2.57 Based on evidence from the relevant boreholes, 2.0m of Granular Weathered Mudstone overlying Mudstone with a groundwater table below the base of cutting is considered.

1.2.58 The preliminary design calculations as set out for Scenario 1 indicate that the cutting would be stable with slopes at 1(V):2(H).

Kinematic analysis

1.2.59 Based on dip angles and orientations indicated on the geological maps, the cutting centreline appears to be located along the line of axis of an anticline with bedding planes falling towards the side slopes of the cutting; see Figure 3 in the GIR (Report Ref.: A40LVP-ARP-VGT-SWI-RP-C-0001). Based on this, the cutting is assumed to be stable in terms of planar, wedge and toppling failure.

1.2.60 BH04, BH05 and BH06 recorded 10 to 30 degrees, 10 degrees and 10 to 20 degrees dip angles respectively. The dip orientations recorded in the boreholes are not known but they are anticipated to dip into the face of the cuttings. Based on this, for the purpose of preliminary design, the cutting is assumed to be stable in terms of planar, wedge and toppling failure.

Llanddewi Velfrey Roundabout

General

1.2.61 Llanddewi Velfrey Roundabout includes two cuttings; one for the tie-in to the southwest and one to link to the existing A40 immediately to the east. The stability assessment below focuses on the deeper cutting at the tie-in which is up to 15m high.

1.2.62 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.2.63 The design considers 2.5m of Weathered Granular Mudstone over Mudstone. It should be noted that part of the tie-in to the southwest is anticipated to be formed in Conglomerate with around 3.5m of overlying Cohesive Weathered Mudstone as recorded in BH03.

1.2.64 Groundwater was struck at 1.8m and at 3.5m in BH03. The water strikes may be associated with perched water contained within a local permeable zone in the bedrock but are not anticipated to be associated with the main groundwater body. Groundwater monitoring of BH04 about 150m to the east did not record water with the installation which contains a response zone that spans between 124.6mAOD and 109.6mAOD. Based on this, the design considers the groundwater level to be below the base of the cutting. The Weathered Conglomerate present in the southwest of the cutting has been identified to potentially be a water-bearing stratum which is the cause of groundwater springs to the west (see Drawing A40LVP-ARP-VGT-SWI-DR-C-0008). There is therefore a potential that this strata will yield some water seepages during excavation.

Limit equilibrium analysis

1.2.65 Paragraph 1.2.34 summarises the results of limit equilibrium analyses for various scenarios, including comparison between granular and cohesive weathered mudstone, and varying groundwater table.

1.2.66 Based on the relevant boreholes, 2.5m of Weathered Granular Mudstone over Mudstone with a groundwater table below the base of the cutting is considered relevant to this location.

1.2.67 The preliminary design calculations as set out for Scenario 1 indicate that the cutting would be stable with slopes at 1(V):2(H).

Kinematic analysis

1.2.68 A dip angle of 60° with a southwest direction is indicated in the north side of the main cutting; see Drawing A40LVP-ARP-VGT-SWI-DR-C-0008. Discontinuity planes may therefore daylight in the north side of the cutting with slopes at 1(V):2(H). Planar, wedge and toppling failure checks will need to be considered at detailed design, and there is potential for shallower slopes or stabilisation measures to be required.

1.3 Embankment stability

General

1.3.1 As summarised in Table 1, there are six embankments along the proposed route. For the preliminary design analysis, all embankment slopes were assumed as 1(V):2(H).

1.3.2 Two embankments, of less than 5m height, are proposed between Penblewin Roundabout at chainage 0+000m and chainage 1+610m.

1.3.3 The major embankments are Embankment 3 between 1+610 and 2+030m, Embankment 4 between 2+460m and 2+720m, Embankment 5 between 2+950m and 3+840m and at Llanddewi Velfrey Roundabout beyond 3+850m.

1.3.4 The embankments are to be formed predominantly over an upper depth of fully weathered bedrock underlain by less weathered bedrock. The exceptions to this are an area of Glaciofluvial Deposits between chainage 0+430m and 0+510m and an area of Diamicton Till between 3+010m and 3+090m.

1.3.5 It is assumed at this stage that topsoil will be stripped out prior to commencement of embankment construction. For the higher embankments on sidelong ground, topsoil replacement with a granular basal starter layer is likely to be needed to manage any potential seepages from the existing ground.

1.3.6 For the main construction of the embankments, it has been assumed that the embankment will be constructed of acceptable, well compacted general fill (Class 1 or 2) derived from the material excavated from the cuttings along the Scheme.

Proposed methodology for embankment stability assessment

1.3.7 The following section presents the proposed design methodology to be adopted for embankment stability design.

Surcharge loading assumed in stability assessments

1.3.8 It is proposed that external temporary loads are included in the analyses where they are considered to represent an unfavourable condition in terms of stability (e.g. where they occur at the crest of a slope). Where such loads are considered to be favourable, they are proposed to be omitted.

1.3.9 The following surcharge loads at the top of the slopes are proposed for embankment design:

a) Main line carriageway at top of slopes – 20kN/m^2.

b) Slip roads, local roads and verges on top of slopes – 10kN/m^2.

A UDL of 10kN/m^2 to be applied to areas on the slope that may be accessed by construction plant and maintenance plant**Stability assessments**

1.3.10 Embankment stability assessment is to be generally undertaken in accordance with HA44/91 [3] and BS 6031 [4]. Stability calculations are to be undertaken with partial factors in accordance with BS 1997-1:2004 [5] and its associated UK National Annex [6].

1.3.11 Effective stress analyses shall be carried out for all embankments. Total stress analyses based on undrained strength parameters are to be undertaken where cohesive materials have been identified to form the embankment and/or the embankment foundation.The design approach is to adopt Design Approach 1 Combination 2 (DA1-C2) as this is considered to be the worst case design scenario for the embankments. This is based on guidance presented in BS EN 1997-1:2004 [5] (Clause 2.4.7.3.4.2(3)) and BS 6031:2009 [4] (Clause 7.3.3). Sensitivity checks have been carried out to verify that DA1-C1 is not the least favourable case. The effect of considering DA1-C2 results in an equivalent global factor of safety of approximately 1.25 where an effective stress analysis is used.

1.3.12 The proposed design approach is to consider a minimum weight of slip mass that does not take account of very shallow slips not considered critical in the design. The slope analyses will consider minimum depths of around 1m to the failure surface. In most cases, such shallow slips are likely to be stable with appropriate modification of shear strength parameters to allow for low stress conditions. This will be considered at detailed design stage.

1.3.13 The analyses are to be undertaken using proprietary slope stability software.

1.3.14 The design will consider the aspects presented in BS1997-1:2004 [5], in particular the following:

a) The 'short term' undrained stability of embankments with cohesive foundation soil.
b) The bearing stratum is adequate or if not, the use of stabilisation measures (such as dig and replace) will be considered.
c) The influence of groundwater on the embankment stability over the design life of the embankment.

Summary of specific assessments for preliminary design

1.3.15 The sections below describe the preliminary embankment stability assessments undertaken based on the methodology above for a number of ground condition scenarios likely to be encountered on site.

1.3.16 The analyses have been carried out based on the range of materials likely to be available from the proposed cuttings. This comprises cohesive embankment fill obtained from locations where the bedrock has been weathered to a clay, and granular embankment fill, sourced from locations where the bedrock is less weathered and may require some crushing and sorting. For the purposes of these analyses, materials have been characterised in accordance with engineering descriptions rather than earthworks classification. Cohesive material comprises >35% fines content (as opposed to Class 2 fill which contains >15% fines content), and granular fill comprises >65% granular material (as opposed to Class 1 fill which contains >85% granular material).

1.3.17 For the analyses, the embankment and the founding material interfaces were assumed to be horizontal.

1.3.18 For preliminary design it is assumed that any soft spots (less than 40kPa) underlying the footprint of the embankments will be removed during construction.

Scenario 1: Large embankments constructed from granular embankment fill (Weathered Mudstone) over Cohesive Weathered Mudstone

1.3.19 A range of slope stability analyses have been carried out to assess the stability of the larger embankments along the scheme. These analyses have considered the greatest height of embankment currently proposed, comprising 24m from toe to crest, as proposed for Embankment 5. Table 6 below summarises the design ground parameters which were assumed for the analyses.

Table 6. Granular embankment fill over Cohesive Weathered Mudstone design parameters

Stratum	phi' (degrees)	c' (kPa)	c_u (kPa)
Granular embankment fill (weathered mudstone fill)	34	0	-
Cohesive weathered mudstone	29	0	75
Mudstone	28	60	-
Groundwater level at base of embankment			

1.3.20 A summary of the results in terms of the Over Design Factors for 1(V):2(H) slopes in the undrained and drained cases are presented in Tables 7 and 8 below.

Table 7. Granular embankment fill over Cohesive Weathered Mudstone undrained analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	1.10
5	0.95
7.5	0.86
10	0.78

Table 8. Granular embankment fill over Cohesive Weathered Mudstone drained analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	1.11
5	1.11
7.5	1.11
10	1.11

1.3.21 The results indicate that a 24m high granular embankment would not achieve the desired Over Design Factor if underlain by more than approximately 4m of cohesive foundation materials of c_u = 75kPa.

1.3.22 An additional undrained analysis with underlying cohesive material of c_u = 40kPa, representative of an area where alluvium might be present. This analysis showed that the embankment would achieve the desired Over Design Factor if underlain by no more than 1.5m thickness of such material.

Scenario 2: Large embankments constructed from cohesive embankment fill (Weathered Mudstone) over Cohesive Weathered Mudstone

1.3.23 A range of slope stability analyses have been carried out to assess the stability of the larger embankments along the scheme. These analyses have considered the greatest height of embankment currently proposed, comprising 24m from toe to crest, as proposed for Embankment 5. Table 9 below summarises the design ground parameters which were assumed for the analyses.

Table 9. Cohesive embankment fill over Cohesive Weathered Mudstone design parameters

Stratum	phi' (deg)	c' (kPa)	c_u (kPa)
Cohesive fill (weathered mudstone fill)	29	0	75
Cohesive weathered mudstone	29	0	75
Mudstone	28	60	-
Groundwater level at base of embankment			

1.3.24 Three design cases were considered as follows:

a) Undrained cohesive embankment fill over undrained cohesive founding material

b) Undrained cohesive embankment fill over drained cohesive founding material

c) Drained cohesive embankment fill over undrained cohesive founding material

d) Drained cohesive embankment fill over drained cohesive founding material

1.3.25 A summary of the results in terms of Over Design Factor for these cases are presented in Tables 10, 11, 12 and 13 below.

Table 10. Cohesive embankment fill over Cohesive Weathered Mudstone (all undrained) analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	0.83

Table 11. Cohesive embankment fill over Cohesive Weathered Mudstone (undrained fill over drained founding material) analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	0.90

Table 12. Cohesive embankment fill over Cohesive Weathered Mudstone (drained fill over undrained founding material) analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	0.92

Table 13. Cohesive embankment fill over Cohesive Weathered Mudstone (all drained) analysis summary

Cohesive Weathered Mudstone thickness (over Mudstone) (m)	ODF
2.5	1.00

1.3.26 The results of the assessment show that a 24m cohesive embankment would not achieve the desired Over Design Factor with underlying cohesive founding material of 2.5m thickness with 1(V):2(H) slopes.

1.3.27 Further analyses have shown that 1(V):2.5(H) slopes would achieve suitable stability of the above.

Scenario 3: Shallow embankments constructed from cohesive embankment fill (Weathered Mudstone) over Cohesive Weathered Mudstone

1.3.28 Table 14 below summarises the design ground parameters that were assumed for assessment of instability of shallow cohesive embankments. This allows for modified material properties associated with low stress conditions.

Table 14. Shallow cohesive embankment fill (Weathered Mudstone) over Cohesive Weathered Mudstone design parameters

Stratum	**phi' (deg)**	**c' (kPa)**	**c_u (kPa)**	**r_u**
Cohesive weathered mudstone fill	29	2 (revised from GIR)	75	0.2 (introduced from GIR)
Cohesive weathered mudstone	29	2 (revised from GIR)	75	0.2 (introduced from GIR)

1.3.29 In embankments formed from cohesive deposits, internal pore pressures may develop. $r_u = 0.2$ is proposed for preliminary design to account for this. This is line with published guidance, for example CIRIA 123 [7].

1.3.30 The drained case was considered to reflect the long term stability of the embankment. A summary of the results in terms of Over Design Factors is presented in Table 15 below.

Table 15. Shallow cohesive embankment fill (Weathered Mudstone) over Cohesive Weathered Mudstone drained analysis summary

Embankment height (m)	**ODF**
2.5	1.143
3.0	1.113
4.0	1.000
5	0.986
7.5	0.893
10	0.851

The results of the assessment show that cohesive embankments are stable up to 4m height at 1(H):2(V) slopes.Analyses summary

1.3.31 The summary of the preliminary design assessment is as follows:

a) For high embankments, the foundation soil type and thickness have an impact on the critical deep seated slips.
b) For 24m high granular embankments, thicknesses of underlying cohesive material ($c_u = 75$kPa) greater than 4.0m are problematic. For underlying cohesive material of $c_u = 40$kPa, thicknesses in excess of 1.5m are problematic.

c) Cohesive embankments did not achieve the desired Over Design Factor for 1(V):2(H) slopes at heights of 4.0m. Above this height, 1(V):2.5(H) slopes would be required for stability.

1.3.32 Based on the above, three options are considered viable for the high embankments to be constructed at 1(V):2(H) slope:

a) Construct embankments entirely out of granular fill.
b) Zoning of the embankments with for example a cohesive fill ‘core’ and a granular fill shoulders. An assessment has not been undertaken at this stage for this option. Restrictions on where cohesive material could be zoned and associated material volumes would need to be considered.
c) Construct embankments from either cohesive fill or granular fill with slacker slopes of 1(V):2.5(H). If this approach is adopted, further analysis would be needed to assess deep seated stability, with consideration of the range of foundation conditions that are likely to be present.

Large embankments design considerations

1.3.33 Embankment 1 and Embankment 2 are not proposed to be discussed in detail within this preliminary design report due to their relatively low height of 2.5 and 4m respectively (see Table 1).

Embankment 3

General

1.3.34 The height of Embankment 3 varies up to a maximum of 9m. The embankment is on-line or partly on-line between chainage 1+610m and 1+850m and is then off-line from 1+850m to 2+030m; see Drawing A40LVP-ARP-VGT-SWI-DR-C-0007.

1.3.35 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.3.36 The design considers 2m of made ground associated with the existing embankment over 4m of Granular Weathered Mudstone underlain by Sandstone.

1.3.37 Based on the various nearby watercourses and localised risk of flooding (see Drawing A40LVP-ARP-VGT-SWI-DR-C-0007), groundwater is assumed to be at the base of the embankment.

Interface with existing embankment

1.3.38 The embankment would be partly built over the existing embankment supporting the current A40 as shown on Figures 3 and 4.

1.3.39 In the context of pavement design, differential settlements will need be considered longitudinally along the road alignment, at the transitions between the on-line section and the offline section, as well as transversally where the infill heights vary.

1.3.40 The maximum infill thickness on top of the crest of the existing embankment will be approximately 1.5m. For preliminary design it is assumed that all settlements in the new fill and existing embankment will have occurred before the road pavement is constructed.

1.3.41 Limit equilibrium slope stability analyses considering both the old and the new embankment will be considered at detailed design.

1.3.42 Benching in line with SHW Clause 608 Construction of Fills Paragraph 12 [8] will be required at the interface between the side slopes of the existing embankment and the new fill to avoid shallow sliding type failures.

1.3.43 An existing culvert crosses at depth beneath the existing embankment between chainage 1+700m and 1+740m. It is not currently proposed to be extended and is not within the footprint of the new embankment and therefore is not considered further as part of the preliminary design.

Embankment 4

1.3.44 Embankment 4 varies in height up to a maximum of 11m and extends between chainages 2+460m and 2+720m; see Drawing A40LVP-ARP-VGT-SWI-DR-C-0007.

1.3.45 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.3.46 The design considers 6m of Granular Weathered Mudstone over 2m of Cohesive Granular Weathered Mudstone underlain by Granular Weathered Mudstone to depth.

1.3.47 For design purposes, the groundwater is assumed to be at the base of the embankment.

1.3.48 A culvert will need to accommodate the existing watercourse at approximate chainage 2+640m. An underpass is proposed at chainage 2+570m (see Figure 7).

1.3.49 Softer ground is expected locally in the area of the existing watercourse which will need to be excavated and replaced with a granular material.

Embankment 5

1.3.50 Embankment 5 varies in height between 0m and 24m and extends between chainages 2+950m and 3+480m; see Drawing A40LVP-ARP-VGT-SWI-DR-C-0008.

1.3.51 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.3.52 The design considers 4m of Cohesive Weathered Mudstone over Mudstone. It should be noted that an area of Diamicton Till is indicated within the footprint of the embankment between chainage 3+000m and 3+110m; see Drawing A40LVP-ARP-VGT-SWI-DR-C-0008. No ground investigation information about the Diamicton Till is currently available. It is recommended that this material is further investigated ahead of the detailed design.

1.3.53 Based on the nearby watercourses and the zone of flood risk (see Drawing A40LVP-ARP-VGT-SWI-DR-C-0008), the groundwater table is assumed to be at the base of embankment for design.

1.3.54 Two culverts are proposed to accommodate the existing watercourses that cross the alignment at approximate chainages 3+110m and

3+270m. An underpass is proposed at chainage 3+200m.

1.3.55 Softer ground is expected locally in the area of the existing watercourses which will need to be excavated out and replaced with a granular material.

Llanddewi Velfrey Roundabout

1.3.56 The proposed Llanddewi Velfrey Roundabout would include two embankment sections; one for the side road tie-in (up to 12m high) and one for the eastern approach (up to 7m high) as shown on Drawing A40LVP-ARP-VGT-SWI-DR-C-0008.

1.3.57 A summary of the groundwater conditions, stratigraphy and the associated site-wide geotechnical parameters derived in the GIR are presented in the relevant Form C in Appendix A. The ground model proposed for the design is discussed below.

1.3.58 The design considers 2.5m of Weathered Granular Mudstone over Mudstone.

1.3.59 For preliminary design the groundwater is assumed at the base of the embankments.

Embankment assessment summary

1.3.60 Paragraph 1.3.30 summarises the results of the embankment stability assessment.

1.3.61 The results show that the taller embankments (all greater than 4m height) will need to be granular to be stable with 1(V):2(H) slopes. Slopes of 1(V):2.5(H) would be required if these were to be formed entirely from cohesive fill.

1.3.62 Cohesive embankments could be formed up to heights of 4.0m with 1(V);2(H) slopes as generally proposed for the embankments on the western end of the scheme.

1.3.63 The founding material type and thickness will need to be considered, as this presents the potential for deep seated failures. It is therefore recommended that further investigation of foundation conditions is carried out ahead of detailed design.

1.4 Embankment settlement

1.4.1 For cohesive fills, which take longer to drain, the timescales for internal settlement can be greater, and for larger embankments the total magnitude of settlement can become more significant.

1.4.2 To assess the embankment self-settlement, the relationship proposed by Trenter [9] can be used as follows:

$$\boldsymbol{s = 0.5\,\frac{\gamma H^2}{D^*}\,(m)}$$

Where:

$\boldsymbol{s}$ is the embankment self-settlement

$\boldsymbol{\gamma}$ is the unit weight of the fill

$\boldsymbol{H}$ is the embankment height

$\boldsymbol{D}$*is the constant equivalent constrained modulus

1.4.3 Trenter Table 4.1 [9] presents typical values of D* for a variety of fill types for different embankment heights. These values can be used in detailed design for each of the materials to be used to form the embankments.

1.4.4 The assessment of the fill types to be won within the Scheme and where within the Scheme the material is to be placed has not be determined at this stage. However, a preliminary assessment of the likely self-settlement for varying embankment heights is presented in Table 16. The assessment assumes Class 2 fill material with D* of 6 to 9MN/m^2 and Class 1 fill material with D* of 30 to 40 MN/m^2 based on the recommendations of Trenter [9]. The fill unit weight has been assumed as 20kN/m^3.

Table 16. Anticipated internal embankment settlement

Fill height (m)	Class 2 fill anticipated self-settlement (mm)	Class 1 fill anticipated self-settlement (mm)
2.5	10	2
5	40	8
10	165	35
12	240	50
15	280	70
20	500	115
25	700	225

1.4.5 A large proportion of the internal settlement in the granular fill will occur during construction.

1.4.6 The anticipated internal embankment settlement is to be reviewed at detailed design. For the high embankments this may involve selection of fill types to control internal settlements. Should zoning be used in embankments, internal settlement assessment between the different zones will need to be carried out.

1.4.7 Monitoring may be recommended for the larger embankments to verify that no significant continuing self-settlement occurs, if cohesive materials are used. This will need to be reviewed at detailed design.

1.5 Re-use of materials

General

1.5.1 The construction work will be carried out under the Manual of Contract Documents for Highway Works (MCHW), Volume 1 Specification for Highway Works, Series 600 – Earthworks [8], and the associated Scheme specific appendices that shall be prepared at detailed design stage. The Series 600 specification provides a system for classifying material as acceptable or unacceptable for a particular use and also gives methods of compaction for specific materials to provide an acceptable degree of compaction for the particular classes of material.

1.5.2 If a material is to be accepted for re-use as fill it generally needs to satisfy the following criteria:

a) Stability,

b) Settlement, and

c) Trafficability.

1.5.3 Appropriate geotechnical parameters shall be selected for the material classes, to ensure that the above criteria are met for the Scheme specific proposals. For site control and verification, testing will be carried at regular frequencies, with the results compared against pre-defined criteria to determine whether the fill is acceptable.

1.5.4 The construction planning will need to take account of the sensitivity of excavated and exposed cohesive weathered mudstone material to changes in moisture content.

1.5.5 For preliminary design, it has been assumed that there will be no import of general fill material and that any surplus will stay on site for landscape areas or grading out of slopes. This will be confirmed at detailed design.

Types of earthworks material

1.5.6 Cuttings will be formed predominantly in the mudstone of the Slade and Redhill Formation. The only exception to this is Cutting 2 between chainages 2+740m and 2+850m where conglomerate from the Portfield and Haverford Formation encroaches into the cutting footprint.

1.5.7 The material likely to be encountered within the cuttings generally includes a layer of fully weathered mudstone over less weathered mudstone which becomes stronger and less fractured with depth. Much of the granular element of the weathered mudstone is expected to be weak, making it prone to degradation during excavation transportation, placement and compaction. Sandstones and conglomerates will probably yield more durable granular material but layers are comparatively thin and widely spaced (see long sections on Figures 4 to 6 in the GIR) so selection of material from these layers will probably not be practical.

1.5.8 The materials excavated from the cuttings is expected to be predominantly granular in nature, which is likely to easily break down under compaction to a Class 1 material in grading. However, a mantle of mudstone rock along some lengths of proposed cutting is likely to have weathered down to a clay, and will comprise a Class 2 material in grading.

1.5.9 A summary of the anticipated materials to be won from the cuttings is presented in Table 17 below.

Table 17. Summary of cutting material class

Cutting Name	Cutting Chainage	Material Description	Anticipated General Fill Class
Penblewin Roundabout	Includes 0+000m to 0+040m	Slade and Redhill Formation (granular weathered mudstone over mudstone)	Typically Class 1
Cutting 1	2+030m to 2+460m	Slade and Redhill Formation (granular weathered mudstone over mudstone)	Typically Class 1
Cutting 2	2+720m to 2+950m	Slade and Redhill Formation (granular and cohesive weathered mudstone over mudstone) between 2+740m and 2+850m, Portfield and Haverford Formation (granular conglomerate over conglomerate). Significant depths of fully weathered rock, that will behave as a clay.	Class 1 and Class 2C
Cutting 3	3+480m to 3+850m	Slade and Redhill Formation (granular weathered mudstone over mudstone).	Typically Class 1
Llanddewi Velfrey Roundabout	3+850m	Slade and Redhill Formation, Portfield and Haverford Formation, and Haverford Mudstone Formation (typically granular weathered mudstone with some conglomerates possible).	Typically Class 1

Testing and acceptability

1.5.10 Testing of acceptability will be in accordance with the Series 600 specification, including the relevant project specific appendices that will be prepared at detailed design stage.

1.6 Proposed excavatability assessment methodology

1.6.1 For preliminary design, an excavatability assessment has been completed for the different formations across the alignment of the Scheme based on Pettifer and Fookes 1994 [10]. The discontinuity (fracture) spacing index (If) and the point load strength values reported in the Factual Report [11] were used to evaluate the anticipated methods of excavation. Point load strength testing was only available for Cutting 2 and Cutting 3.

1.6.2 The results of this excavatability assessment are included in Appendix B. Table 18 below summarises the findings. The below conclusions should be treated with caution, as the assessment is sensitive to any drilling induced fracturing of the borehole cores that may have occurred.

Table 18. Summary of anticipated excavation methods for cut materials

Location and Geological Bedrock Formation	**Point Load (MPa)**	**Discontinuity spacing, If (mm)**	**Typical excavation methods**
Cutting 2 – Conglomerate from the Portfield and Haverford Formation	0.02 to 0.04 (diametrical)	20 to 60 based on very closely spaced	'Easy Digging'
Cutting 3 – Mudstone from the Slade and Redhill Formation	0.10 to 2.56 (axial) 0.04 to 1.70 (diametrical)	20 to 60 on very closely spaced	'Easy Digging' to 'Easy Ripping (D6, D7)'

2 Highway structures

2.1 Introduction

2.1.1 A list of structures proposed along the scheme and their respective chainages is presented in Table 19 below. The list includes pedestrian underpasses that require an AIP.

2.1.2 A number of culverts and mammal crossings are proposed which do not require a AIPs. These structures are presented in Table 20 below.

2.1.3 Draft Form Cs are presented in Appendix C for each structure. The forms include a summary of the investigation undertaken, the ground and groundwater conditions, and details of the stratigraphy. The draft Form Cs are taken from the GIR and have been updated as part of this report.

Geological sections at each structure location accompany each of the draft Form Cs; see Figures 6 to 10.Table 19. Proposed structures requiring an AIP

Approximate chainage	Structure type
1+300m	Underpass 1
2+560m	Underpass 2
2+840m	Overbridge
3+200m	Underpass 3
3+550m	Footbridge (TBC – omitted from this report)

Table 20. Proposed and existing culverts

Approximate chainage	Culvert type
0+290m	Existing culvert to be extended.
0+305m	Proposed bat crossing 1.8m diameter.
1+800m	Existing culvert to remain.
2+640m	Proposed culvert 1.8m diameter.
2+990m	Proposed mammal crossing 1.8m diameter.
3+150m	Diversion of watercourse via proposed culvert 1.8m diameter.
3+270m	Proposed culvert 1.8m diameter.

2.2 Underpasses

Structure and foundation type

2.2.1 The purpose of the underpasses is to provide pedestrian and/or bridleway access beneath the scheme alignment.

2.2.2 The underpasses would consist of single span reinforced concrete box segments comprising four elements:

a) Upper Section: precast upside-down “U” shaped roof

b) Middle Section: two precast “L” shaped walls

c) Lower Section: cast-in-situ base

2.2.3 The upper and middle sections would be separated by longitudinal ball-socket joints. The middle and lower sections would be made monolithic by the cast-in-situ concrete required for the base of the boxes. Joints between the sections are assumed to be “shear lock” type connections.

2.2.4 The wing walls are proposed to be ‘L’ shaped units together with a cast in-situ stitch to form an inverted ‘U’ shape section. The units would be made monolithic by the cast-in-situ concrete to form the base. Joints between the wing walls and the underpass will be provided and so the wing wall and the underpass will act independently.

2.2.5 The base of the box would act as the foundation, bearing directly on the ground beneath.

Geometry

2.2.6 Transverse movement joints would be provided at regular spacings along the length of the cast-in-situ slab to eliminate early thermal cracking.

2.2.7 The wing walls provided at each end of the underpass would have a varying crest level, shaped according to the adjacent earthworks profile. The wings walls would run straight and square to the underpass.

2.2.8 The underpasses would be square to the scheme alignment and be of different lengths.

Design actions

2.2.9 The design actions are to be in accordance with the AIP as summarised in the following sections.

2.2.10 Dead loads:

a) Load of fill and road pavement overlying underpass.

b) Self-weight of underpass.

c) Earth pressures on side of underpass.

2.2.11 Live loads:

a) Vertical traffic loading.

b) Longitudinal traffic loading due to braking and acceleration.

c) Transversal traffic loading due to skidding.

All load combinations and factors used are to be in accordance with BS EN 1997-1:2004 and NA to BS EN 1990:2002+A1:2005.

Characteristic soil parameters

2.2.12 The characteristic soil parameters derived in the GIR (A40LVP-ARP-VGT-SWI-RP-C-0001) shall be used. These are included with the Form Cs in Appendix C.

Design approach

Design objectives

2.2.13 The main objectives of the preliminary design are to be as follows:

a) Ensure stability of the underpass.

b) Control vertical deflection of the underpass.

c) Control differential settlement along the length of the underpass, and differential settlement of the highway that will run over the underpasses.

Design standards

2.2.14 The design is to be in accordance with the following standards:

a) BS EN 1997-1:2004.

b) PD6694-1:2011.

Serviceability Limit State (SLS)

2.2.15 The SLS case is to be modelled in accordance with clause 2.4.8 of BS EN 1997-1:2004 and PD6694-1:2011 using the characteristic soil parameters.

Ultimate Limit State (ULS)

2.2.16 The ULS case is to be modelled in accordance with clause 2.4.7 of BS EN 1997:1-2004 and PD6694-1:2011 to assess the following:

a) Internal failure or excessive deformation of the structure (STR).

b) Failure or excessive deformation of the ground (GEO).

c) Loss of equilibrium of the structure or the ground due to uplift by water pressure (UPL).

2.2.17 Partial factors as presented in Annex A of BS EN 1997-1:2004 and the accompanying National Annex, and their combinations as given for Design Approach 1 are to be adopted for the analyses.

2.2.18 For Load Combination 1, the partial factors to actions are to be applied directly to the effects of actions derived from representative values of the actions, whereas for Load Combination 2 partial factors are to be applied to soil resistance and variable unfavourable actions in accordance with Annex A of BS EN 1997-1:2004.

Design methodology

2.2.19 The following stability checks shall be considered for both the permanent and temporary case:

a) Bearing capacity.

b) Uplift bearing capacity.

c) Sliding.

d) Overturning.

e) Uplift by water pressure.

f) Total settlement.

g) Differential settlement.

2.2.20 The stability checks for the underpasses listed above shall also be considered for the wing walls. These shall include consideration of potential for rotational failure of the wing walls for both the permanent and temporary case.

Site supervision

2.2.21 No specific monitoring requirements are considered to be required for the underpasses at this stage.

Underpass 1 (SBR-0130) – Ch 1+300m

Structure and foundation type

2.2.22 The underpass is proposed to be founded at around 5m below existing ground level; see cross section on Figure 6.

Construction sequence

2.2.23 The construction sequence assumed for the preliminary design is as follows:

a) Remove existing road pavement.

b) Excavate to temporary profile.

c) Construct box culvert.

d) Pour cast in-situ base.

e) Construct wing walls either side of the underpass.

f) Place 6N/6P backfill behind box and wing walls.

g) Construct road pavement.

Design assumptions

Geometry

2.2.24 The thickness of the road pavement immediately on top of the underpass is considered to be 600mm for preliminary design.

Design ground conditions

2.2.25 The following exploratory holes from the 2016 ground investigation have been used to inform the ground conditions at the location of the underpass: TP20, TP21, BH14 and BH15. A summary of the ground conditions encountered in the relevant exploratory holes is presented in the relevant Form C in Appendix C.

2.2.26 Made ground was present in BH14 to a depth of 0.85m. Below the made ground, weathered bands of mudstone and sandstone were encountered up to the base of the borehole at 14.5m depth. In BH15, topsoil was encountered overlying weathered siltstone extending up to 6.2m depth underlain by siltstone up to the base of the borehole at 14.5m depth. It should be noted that the boundary between the Haverford Mudstone Formation and the underlying Portfield and Haverford Mudstone Formation is indicated to the north of the underpass.

2.2.27 Based on the above and the proposed founding levels, the underpass is anticipated to be founded in rock, varying between weathered mudstone and weathered siltstone. For design, 2m of Cohesive

Weathered Mudstone over Granular Weathered Mudstone to depth has been considered.

Design groundwater

2.2.28 A summary of the groundwater encountered in the relevant exploratory holes is presented in Appendix C.

2.2.29 No groundwater monitoring has been undertaken in this area. Groundwater was struck at 4.5mbgl in BH14 within the Weathered Mudstone. An area of boggy ground has been identified during the site walkover at the south end of the proposed underpass.

2.2.30 For preliminary design calculations, the groundwater level has been assumed to be at the base of the underpass.

2.2.31 Toe drains will be installed at the bottom of the cutting slopes to capture surface runoff and potentially groundwater inflow. Back of wall drainage is proposed to capture water build up in the backfill material.

Design outcome

2.2.32 The stability checks considered for both the permanent and temporary case and the outcomes are presented below:

a) Bearing capacity – the temporary case is considered to be the most critical as a large overburden would be provided by the fill on either side in permanent case. Deemed to be acceptable in the drained and undrained case based on the formulae provided in Annex D of BS EN 1997-1:2004.

b) Uplift bearing capacity – the permanent case when traffic loading is acting on the edge of the culvert is considered most critical. Assessed to be acceptable.

c) Sliding – it has been assumed that the underpass will be loaded evenly on either side of the structure by the placed fill during construction. Therefore, sliding due to out of balance earth pressures forces have been assumed to be acceptable.

d) Overturning – as above.

e) Uplift by water pressure – as joints between segments and seepage holes will let groundwater seep through, the maximum water pressure beneath the underpass will not be sufficient to uplift the structure.

f) Total settlement – based on elastic theory [12], total settlement has been assessed to be of the order of 25mm for the underpass.

g) Differential settlement – based on the total settlement, differential settlements between the centre and each end of the underpass is anticipated to be less than 25mm.

h) Differential settlement – based on the total settlement, differential settlement 25mm between the underpass and the adjacent highway earthworks will be less than 25mm. Much of this settlement will however occur during construction, prior to installation of the highway drainage and construction of the road surface.

2.2.33 The wing walls are currently proposed to be constructed independently from the underpass. The differential movements between the less loaded wing walls and the underpass need to be considered, including the potential for heave effects in the cutting. This will be assessed at detailed design.

Underpass 2 (SBR-0257) – Ch 2+570m

Structure and foundation type

2.2.34 The underpass is proposed to be founded at 6.5m below proposed embankment surface level, with the northern side on top of about 1.5m of fill and the southern side in a localised cut through the hillside; see cross section on Figure 7.

Construction sequence

2.2.35 The construction sequence assumed for the preliminary design is as follows:

a) Place fill on northern end of embankment and form cut through hill side on southern end of embankment to form underpass founding level.

b) Construct box underpass.

c) Pour cast in-situ base of underpass.

d) Construct wing walls either side of the underpass.

e) Place 6N/6P backfill behind box and wing walls.

f) Fill over the underpass.

g) Construct road pavement.

Design assumptions

Geometry

2.2.36 A combined thickness of 2m for the road pavement and soil cover immediately on top of the underpass has been considered for preliminary design.

Design ground conditions

2.2.37 The following exploratory holes from the 2016 ground investigation have been used to inform the design ground conditions at the location of the underpass: BH11 located approximately 150m to the east. A summary of the ground conditions encountered in the exploratory hole is presented in Appendix C.

2.2.38 Based on the above and the proposed founding levels, the southern section of the underpass is anticipated to be founded in granular weathered mudstone and the northern section on approximately 1.5m of granular weathered mudstone fill.

Design groundwater

2.2.39 A summary of the groundwater encountered in the relevant exploratory holes is presented in Appendix C.

2.2.40 BH11 approximately 150m to the east recorded groundwater seepage at 3.0mbgl (94.0mAOD) and groundwater monitoring recorded levels of between 3.6-4.5mbgl (92.8 to 93.4mAOD). A tertiary river has been identified 70m to the east of the underpass (see Drawing

A40LVP-ARP-VGT-SWI-DR-C-0007).

2.2.41 For preliminary calculations, the groundwater level is assumed at the base of the embankment where the underpass is to be built on fill and at the base of the underpass where the underpass is in cutting.

Design outcome

2.2.42 The same stability checks as for Underpass 1 were considered for the box and the wing walls (see Section 2.1). On account of the less onerous underlying ground conditions all the checks considered in Section 2.1 were considered to be acceptable. There is an additional surcharge due to the additional soil cover on top of this underpass compared to Underpass 1, however the impact is assumed to be negligible. This will need to be checked at detailed design.

Underpass 3 (SBR-0320) – Ch 3+200m

Structure and foundation type

2.2.43 The underpass is proposed to be founded approximately 7m below the surface of the proposed embankment within the embankment fill; see cross section on Figure 8.

Construction sequence

2.2.44 As Underpass 2; see paragraph 2.2.35.

Design assumptions

Geometry

2.2.45 A combined thickness of 2m for the road pavement and soil cover immediately on top of the underpass has been considered for preliminary design.

Design ground conditions

2.2.46 The following exploratory holes from the 2016 ground investigation have been used to inform the design ground conditions at the location of the underpass: BH07. A summary of the ground conditions encountered in the exploratory hole is presented in Appendix C.

2.2.47 Based on the above and the proposed founding levels, the underpass is anticipated to be entirely founded on granular mudstone fill.

Design groundwater

2.2.48 A summary of the groundwater encountered in the relevant exploratory holes is presented in Appendix C.

2.2.49 No groundwater strikes were recorded and no groundwater monitoring was undertaken in BH07. The proposed location of the underpass is close to two watercourses; one to the east and one to the west, both approximately 50m away. The groundwater level is assumed to be at the base of embankment for preliminary design.

Design outcome

2.2.50 Same as Underpass 2; see paragraph 2.2.42.

Overbridge 1 (SBR-0285) – Ch 2+850m

Structure and foundation type

2.2.51 At the time of writing this report, Overbridge 1 was proposed as either a single span or a three span bridge over the new alignment of the A40 mainline. The single span option would be of steel composite form and the three span option would consist of prestressed beams; both options would be constructed integral with the abutments; see cross sections for each option on Figures 9 and 10.The abutments are to be founded on bank seat foundations within Cutting 3. For the three span bridge option, two piers on either side of the carriageway would be provided, founded at the base of the cutting.

2.2.52 Both options are considered in the sections below.

Construction sequence

2.2.53 The construction sequence assumed for preliminary design for the single span steel-composite option is as follows:

a) Local excavation to abutment founding level and beam soffit level.

b) Abutments cast up to beam soffit level.

c) Backfill placed up to beam soffit level (the contractor needs access to the superstructure during the works).
d) Steel landed on temporary supports.
e) Deck cast and support to steelwork removed once deck had achieved 28 day strength (deck acts in a simply supported manner)
f) Diaphragms cast and integral connection made.
g) Remaining backfill placed.
h) Full excavation made in front of abutments to construct the mainline.

2.2.54 The construction sequence for the three span prestressed beams option has not yet been developed at the time of writing this report.

Design assumptions

Geometry

2.2.55 See indicative outlines for both options on Figures 9 and 10.

Design ground conditions

2.2.56 A description of the ground conditions is presented in the relevant Form Cs in Appendix C.

2.2.57 Table 21 and Table 22 below summarise the assumed ground model at each foundation for the three span and single span option.

Table 21. Three span option ground model

Foundation location	Approximate founding level (mAOD)	Relevant borehole	Assumed ground conditions at founding level
North abutment	94.4	BH10	8m of cohesive weathered mudstone underlain by mudstone
North pier	90.1	BH10	3m of cohesive weathered mudstone underlain by mudstone
South pier	93.4	BH09	7m of cohesive weathered mudstone underlain by mudstone
South abutment	99.3	BH09	4m of conglomerate over 8m cohesive weathered mudstone underlain by mudstone

Table 22. Single span option ground model

Foundation location	Approximate founding level (mAOD)	Relevant borehole	Assumed ground conditions at founding level
North abutment	93.1	BH10	6m of cohesive weathered mudstone over mudstone
South abutment	98.1	BH09	2.5m of conglomerate over 8m cohesive weathered mudstone underlain by mudstone

Design groundwater

2.2.58 A description of the groundwater conditions is presented in the relevant Form C in Appendix C.

2.2.59 The groundwater table is assumed to be at 3.5m depth below existing ground level along the length of cutting; see discussion for Cutting 2 in the previous section (paragraph 1.2.39) for more details.

Characteristic soil parameters

2.2.60 The characteristic soil parameters derived in the GIR (A40LVP-ARP-VGT-SWI-RP-C-0001) are proposed to be used and are included in the relevant Form C in Appendix C.

Design actions

2.2.61 The design actions for the Ultimate Limit State (ULS) case are to be in accordance with BS EN 1997-1:2004 [5] using Design Approach 1 Load Combination 1 and Load Combination 2. For the serviceability limit state case (SLS) a factor of unity is to be applied to all actions.

2.2.62 The following are to be considered as actions for the ULS case in accordance with BS EN 1997-1:2004:

a) The earth pressures that result from the mass of the retained soil.

b) Super Dead Load and Dead Load of the bridge deck.

c) Highway loading on the bridge deck.

d) Highway loading on the retained ground.

e) Highway braking on the bridge deck, both towards and away from the abutment.

f) Thermal expansion and contraction of the bridge deck.

2.2.63 The characteristic highway loading applied to the bridge deck is to be in accordance with Clause NA.2.34.2 of the National Annex to BS EN 1991-2 [13].

2.2.64 The characteristic vehicle loading on the retained ground is proposed to be 15kN/m^2.

Design approach

Design objectives

2.2.65 The objectives of the design are to:

a) Ensure stability of the foundations.
b) Control vertical and lateral deflections of the abutments, and differential movement between structural elements and adjacent earthworks to within acceptable limits.
c) Ensure that the bending moments and shear forces that result from abutment deflections and reactions can be accommodated by the structure.

2.2.66 The design standards are proposed to be BS EN 1997-1:2004.

2.2.67 BS EN 1997-1:2004 does not provide specific guidance on the design of integral bridges. Published guidance in the form of Chapter 9 of PD6694-1:2011 has been prepared to address this, and is proposed to be used for the geotechnical design.

Serviceability Limit State (SLS)

2.2.68 The SLS case shall be modelled in accordance with clause 2.4.8 of BS EN 1997-1:2004 and PD6694-1:2011 using the characteristic soil parameters.

Ultimate Limit State (ULS)

2.2.69 The ULS case is to be modelled in accordance with clause 2.4.7 of BS EN 1997-1:2004 and PD6694-1:2011 to assess the following:

a) Internal failure or excessive deformation of the structure (STR).
b) Failure or excessive deformation of the ground (GEO).

c) Loss of equilibrium of the structure or the ground due to uplift by water pressure (UPL).

These shall be considered for the various intermediate stages of construction, and for the completed structure.

2.2.70 Partial factors as presented in Annex A of BS EN 1997-1:2004 and their combinations as given for Design Approach 1 shall be adopted in the analyses.

2.2.71 For Load Combination 1, the partial factors to actions shall be applied directly to the effects of actions derived from representative values of the actions, whereas for Load Combination 2 partial factors shall be applied to soil resistance and variable unfavourable actions in accordance with Annex A of BS EN 1997-1:2004.

Design methodology

2.2.72 Integral bridge structures rely on the soil behind the abutments to carry longitudinal forces generated by braking and thermal expansion. Under repeated cycles of loading laboratory tests have shown that granular soil resistance increases; bridges and bridge abutments must therefore be designed to carry these soil pressures.

2.2.73 The analysis of the bridge structure is proposed to be undertaken using Oasys GSA. The abutments would be modelled in GSA with applied earth pressures and soil springs to model the soil reaction.

2.2.74 Limit equilibrium methods are to be adopted to determine the appropriate earth pressures to apply behind the abutments. This is proposed to be carried out in accordance with the procedures presented in Section 9 of PD6694-1:2011. The abutment foundations are to be designed to ensure stability against overturning, sliding, bearing capacity and uplift for critical stages of construction and the permanent case.

2.2.75 The bearing capacity of each formation is to be calculated for all pad foundations in accordance with BS EN 1997-1:2004 and should account for the cutting slopes. The applied bearing pressures, together with the results of the settlement and lateral displacement assessments, are to be used to calculate soil spring stiffnesses for input into the GSA analysis.

2.2.76 Slope stability of the cutting slopes is to be checked using Oasys Slope. Sliding, overturning and settlement of the pad foundations shall be carried out in accordance with BS EN 1997-1:2004.

2.2.77 The earth pressure reaction due to thermal loading is to be assessed in accordance with PD6694-1:2011 Clause 9.4.5.

2.2.78 The moments and shear forces generated in the structural model are to be checked against the abutment stem wall capacities.

2.2.79 There may be unfavourable discontinuity planes that will need to be considered. The horizontal forces applied by the pad foundations on these discontinuity planes will need to be considered. This will need to be checked at detailed design.

Design outcome

2.2.80 The preferred overbridge option has not been confirmed by the client at the time of writing this report and therefore it is not proposed to undertaken a specific assessment of the bridge foundations at this stage.

2.3 Footbridge 1 (SBR-0355) – Ch 3+550m

2.3.1 Not considered as the structure has not been confirmed at the time of writing this report. Should the structure be included as part of the Scheme, an addendum to this report will be submitted to include the structure or alternatively the structure may be included in the Key Stage 6 Geotechnical Design Report.

3 Strengthened earthworks

3.1.1 Not used.

4 Drainage

4.1 Cuttings

4.1.1 General recommendations for drainage for cuttings are discussed in Section 1.2 of this report.

4.1.2 Details of drainage measures required are to be considered during the detailed design stage.

4.2 Embankments

4.2.1 General recommendations for drainage for cuttings are discussed in Section 1.3 of this report.

4.2.2 Details of drainage measures required are to be considered during the detailed design stage.

4.3 Structures

4.3.1 For each structure, drainage details are to be considered during the detailed design stage.

5 Pavement design, subgrade and capping

5.1.1 The following design approaches are proposed to assess the CBR values along the Scheme length.

5.2 Embankments

5.2.1 The CBR values for the new embankments will be from available information for the material expected to be obtained from the cuttings on the Scheme.

5.2.2 The material parameters currently available are mainly classification data in the form of Plasticity Index and Particle Size Distribution. These results have been used in conjunction with IAN 73/06 Rev 1 Design Guidance for Road Pavement, in particular Table 5.1, to determine the preliminary Equilibrium Subgrade CBR for embankment fill.

5.2.3 It is assumed that the new embankments will be formed largely from Mudstone fill obtained from the cuttings along the scheme.

5.2.4 The material from the cuttings is also expected to include Weathered Conglomerate from Cutting 2 which will also be suitable as a general fill material.

5.2.5 The typical plasticity indices and corresponding equilibrium CBR for the various fill sources are presented in Table 23 below:

Table 23. Summary of PI% for fill sources

Location and Material Description	**PI range (average) (%)**	**Equivalent Equilibrium CBR (average) (%)**
Weathered mudstone – Haverford Mudstone Formation	11 – 30 (16)	3 to 5 (5.5)
Weathered mudstone – Slade and Redhill Formation	15 – 46 (18)	3 to 5 (5)
Weathered conglomerate – Portfield and Haverford Mudstone Formation	13 – 21 (16.5)	3 to 5 (5.5)

5.2.6 Based on the above table a preliminary design CBR value of 5% is considered for the new embankments.

5.3 Cuttings

5.3.1 The CBR values in cuttings have been evaluated based on the classification tests for materials anticipated at formation level. As for the embankments, the CBR values has been based on the guidance given in IAN 73/06 Rev 1 Table 5.1.

5.3.2 The design CBR in the cuttings will be dependent on the material exposed at formation level. It will also need to consider the groundwater table depth and conditions during construction. Where a high groundwater table is identified in cuttings within cohesive material, consideration in detailed design will need to be given to a conservative groundwater table condition as the reliance on cut off drains may not be practicable.

5.3.3 The cuttings required at Penblewin Roundabout, Cutting 1, Cutting 2 and Cutting 3 are predominantly within mudstone from the Slade and Redhill Formation. Based on the values in Table 23 above, the preliminary design CBR is selected as 5%.

5.4 At grade or on shallow embankment

5.4.1 Nine in-situ CBR tests were undertaken in areas at grade or on shallow embankments; see Drawings A40LVP-ARP-VGT-SWI-DR-C-0006 to 0008.

5.4.2 The results of the CBR tests are reproduced from the GIR in Table 24 below.

Table 24. Summary of in situ CBR test results (reproduced from the GIR)

Location	Test Depth (m)	Soil Description	Moisture Content (%)	CBR Value (%)
CBR1	0.5	Firm light brown sandy gravelly clay	35.7	1.9
CBR2	0.6	Light brown very clayey sandy fine to coarse gravel	11.4	5.7
CBR3	0.65	Light brown slightly clayey sandy fine to coarse gravel	16.6	8.9
CBR4	0.65	Light brown very clayey sandy fine to coarse gravel	17.8	4.5
CBR5	0.8	Firm light brown sandy gravelly clay	22.2	4.2
CBR6	0.6	Firm light brown slightly sandy slightly gravelly clay	19.5	4.5
CBR7	0.7	Firm light brown slightly sandy slightly gravelly clay	14.7	3.4
CBR8	0.6	Firm light brown slightly sandy slightly gravelly clay	14.7	3.6
CBR9	0.9	Red brown highly weathered shale	21.3	15.3

5.4.3 Based on Table 24 above a reasonably conservative estimate for the preliminary CBR value for areas at grade is 3.5%.

6 Assessment of potential contamination

6.1 Summary of the extent of the contamination testing that has been undertaken

6.1.1 The potential for contamination has been assessed based on the logs and laboratory chemical tests result as reported for the 2016 WSP ground investigation. Testing was completed on five made ground samples. No groundwater sampling or groundwater testing was carried out.

6.1.2 Ground investigations undertaken along the proposed route encountered Made Ground at discrete locations (seven holes out of 37 No). Typically, made ground was recorded to 1.5m depth, with deeper made ground deposits encountered in areas of historic infilled quarries, to between 3.1m and 4.1m. The details on the encountered areas of made ground are presented in the GIR (Report Ref.: A40LVP-ARP-VGT-SWI-RP-C-0001).

6.1.3 Manmade inclusions such as fragments of brick, metal, plastic, tile and concrete were noted in two of the investigated locations. No evidence of hydrocarbon or asbestos contamination was recorded. However, as a result of the existing road operation e.g. accidental fuel spillages and leakages, localised areas of hydrocarbon contamination may have previously occurred, and therefore could be encountered during construction.

6.1.4 The soil samples were tested for the presence of metals and petroleum hydrocarbons, and were also screened for asbestos.

6.2 Summary of the findings and conclusions of the risk assessments including site remediation

Methodology

6.2.1 The risk assessments for the scheme have been undertaken in line with current industry best practice, including CLR11 [14]. In summary, the process comprises a tiered approach which starts with a simple and conservative Tier 1 assessment of potential risks from possible Pollutant Linkages (Source-Pathway-Receptor). Any potential risks identified at Tier 1 have been studied in more detail through a Tier 2: Generic Quantitative Risk Assessment (GQRA). This concluded that a Tier 3: Detailed Quantitative Risk Assessment (DQRA) was not needed.

6.2.2 The risk assessment process is underpinned throughout by the development of the Conceptual Site Model (CSM) which provides a schematic representation of the identified contamination linkages.

6.2.3 The assessment of risk to human health from direct exposure to soils has been undertaken by screening the available soil chemical test results against published generic assessment criteria for a suitable land use scenario, such as DEFRA Category 4 Screening Levels (C4SLs)

[15], and where these are not available, the LQM/CIEH Suitable 4 Use Levels (S4ULs) [16]. Risk assessment for construction and maintenance workers has been undertaken applying a residential end use scenario due to the direct exposure to soils during any intrusive works. This is a conservative approach, however, it is considered suitable for identification of potential risks requiring consideration as part of the health and safety assessment of the works. The same scenario has been applied to the scheme neighbours including residents of the farms and residential properties. The risk to site end users has been undertaken for a commercial end use scenario.

6.2.4 A quantitative assessment of risk to controlled waters, such as groundwater and surface water, cannot be undertaken as no groundwater or soil leachate testing has been carried out as part of the recent ground investigations. However, a qualitative assessment has been carried out.

Risk Assessment

6.2.5 A conceptual site model was derived for both the construction and operation phases, which identified a number of potential pollution linkages. These linkages, together with the conclusions of the assessments, have been summarised in Table 25 below.

Table 25. Conceptual Site Model

Sources	**Pathways**	**Receptors**	**Comments**
Construction			
Sources: Made ground associated with existing road network, agricultural activities, historical infilled quarries. Accidental fuel spillages along the existing road network Locally potentially impacted groundwater by off-line sources (sewage discharge via soakaways, burial ground, sewage treatment works). Unexpected contamination. Imported and site won construction materials. Dust created during construction. Groundwater removed from ground during dewatering.	Direct exposure to soil and/or soil dust, and groundwater via ingestion, dermal contact and inhalation . Inhalation of gas and volatile contamination.	Human health: A40 maintenance workers and users. Scheme neighbours (residents of the farms and properties agricultural land users). Construction workers.	Construction workers, and also the existing A40 maintenance workers and highway users may be directly exposed to soil and/or groundwater and potentially contaminated dust generated from made ground during construction. There is a potential risk of ground gas migration into and accumulation in confined spaces.
	Leaching and migration .	Controlled waters: surface water courses and groundwater (secondary aquifers).	Potential contaminants within the identified sources may leach to groundwater and via lateral migration have potential to impact the surface water quality. Where construction activities are undertaken in a proximity to surface water course, water quality may be impacted by a direct surface run-off.
Operation			
Sources: Made ground associated with agricultural activities, historical infilled quarries. Accidental fuel spillages along the new road network. Locally potentially impacted groundwater by off-line sources (sewage discharge via soakaway, burial ground, sewage treatment works). Imported and site won construction materials.	Direct exposure to soil and/or soil dust via ingestion, dermal contact and inhalation. Inhalation of gas and volatile contamination. Ground gas migration and accumulation in confined spaces.	Human health. Scheme users. Maintenance workers. Scheme neighbours (residents of the farms and properties, agricultural land users).	Maintenance workers and highway users may be directly exposed to soil and potentially contaminated dust generated from made ground during maintenance works. There is a potential risk of ground gas migration into and accumulation in confined spaces.
	Leaching and migration	Controlled waters: surface water courses and groundwater (secondary aquifers).	Potential contaminants within the identified sources may leach to groundwater where rainwater infiltration will occur. These have a potential to impact the surface water quality via lateral migration.

Note: For locations of the identified on-line and off-site sources refer to Drawings A40LVP-ARP-VGT-SW1-DR-C-0006 to 0008.

6.2.6 Construction and operation of the proposed scheme may pose a risk to construction workers, scheme neighbours and end users through a direct exposure to potentially impacted soils or soil dust. However, the assessment of the laboratory testing results did not identify any exceedances of the applied assessment criteria. In addition, no asbestos was identified within the analysed samples.

6.2.7 No testing for polycyclic aromatic hydrocarbons (PAHs) was included in the scope of testing. However, considering that no evidence of contamination with these compounds has been recorded, these are unlikely to pose a significant risk to human health.

6.2.8 Therefore, the analysed soils are unlikely to pose a risk to human health during construction and operation and consequently remediation works are not considered to be required. This may be with an exception of areas of unexpected contamination, which may require further assessments and possibly remediation.

6.2.9 Previous assessments identified a number of sources of contamination, as listed in the table above, that are located away from the proposed scheme alignment. Contamination originating from these sources may be migrating via groundwater into the scheme area. Considering that the groundwater flow direction is determined by the area topography and groundwater is likely to discharge to the surface water courses, the proposed scheme is located hydraulically up-gradient of both the sewage discharge into the ground and the sewage works. Therefore, these sources are unlikely to have a detrimental impact on the groundwater quality beneath the scheme. The scheme is however located approximately 200m hydraulically down-gradient in relation to the burial ground (approximate scheme chainage 1+100m to 1+200m) and therefore there is a potential for contaminated groundwater presence beneath the scheme area. The contamination may primarily include elevated levels of metals or organic carbon. Although these are unlikely to pose a significant risk to human health during construction, should groundwater be encountered during excavation works in that area of the scheme, appropriate health and safety measures will be required.

6.2.10 Ground gas monitoring undertaken as part of the recent ground investigations indicated relatively low levels of ground gas. However, no ground gas monitoring was completed within the infilled quarries. The encountered fill materials comprised clayey gravels. No organic matter, which is a potential source of ground gas, has been identified and therefore the potential for significant ground gas generation is considered to be low. However, where excavation works or other engineering works are undertaken in the areas of the infilled quarries, it is recommended that the fill materials are further characterised with respect to the potential for gas generation particularly the content of organic carbon to inform a ground gas and subsequently health and safety risk assessments.

6.2.11 No data on soil leachate or groundwater quality is available therefore a potential impact on the controlled water quality from construction works or site won soils reused within the scheme is unknown. Considering the relatively low levels of subsurface contamination, the encountered made ground materials are unlikely to pose a significant risk to controlled waters during construction. Any discharge into controlled waters (surface water and/or groundwater) will require appropriate controls and monitoring during construction subject to regulatory approvals.

6.2.12 In addition, the Specification for Highway Works (SHW) sets out requirements for made ground materials designated for reuse to be sampled, tested for potential contamination, and screened against threshold values in relation to human health and the water environment. This would provide verification safeguards against potential adverse effects. This is to be developed at the detailed design stage.

6.2.13 In summary, the construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.

6.3 Details of contaminated material to be removed from site

6.3.1 No specific areas of contaminated material have been identified at this stage. However, there are isolated areas of made ground on the scheme, as presented in sections above. Acceptability criteria and a suitable material sampling and testing regime will be established for the construction phase, with the requirements to be incorporated into the earthworks specification for the scheme. Any material encountered that proves unacceptable for reuse during the works on account of it being in exceedance of the agreed acceptability criteria, will be required to be disposed of at a suitably licenced facility.

7 Ground treatment including treatment of any underground voids etc.

7.1.1 Softer ground is expected locally in the area of existing watercourses which will need to be excavated and replaced with a acceptable fill material.

8 Specification appendices (not included at preliminary design stage)

8.1.1 The specification appendices shall be prepared as part of the Key Stage 6 detailed design.

9 Instrumentation and monitoring

9.1 Full details of purpose, installation requirements, restrictions and frequency of readings

9.1.1 None anticipated to be required at present.

9.2 Use of Observational Methods/Controls. Predicted and critical readings and restrictions on work

9.2.1 To be confirmed at detailed design stage.

9.3 Pile testing requirements

9.3.1 Piles are not anticipated to be used. To be confirmed at detailed design.

10 References

[1] DMRB Standard HD22/08, Volume 4, Section 1, Part 2, Managing Geotechnical Risk. August 2008.

[2] DRMB Standard HD22/08: Managing Geotechnics Risk Implementation Guidance – Wales. December 2009.

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[6] NA+A1:2014 to BS EN 1997-1:2004+A1:2013, UK National Annex to Eurocode 7: Geotechnical design – Part 1: General rules, BSI Standards Publications.

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[11] A40 Llanddewi Velfrey – Penblewin, Ground Investigation Report, WYG . Environment Planning Transport Ltd June 2016.

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[15] SP1010 Development of Category 4 Screening Levels for Assessment of Land Affected by Contamination – Policy Companion Document. Department for Environment Food and Rural Affairs (Defra). 2014.

[16] The LQM/CIEH S4ULs for Human Health Risk Assessment (Land Quality Management (LQM) / Chartered Institute of Environmental Health (CIEH). 2015.

Figures

Figure 1 – Cross section at Ch. 2+840m at Cutting 2

Figure 2 - Cross section at Ch. 3+640m at Cutting 3

Figure 3 – Cross section at Ch. 1+700m at Embankment 3

Figure 4 – Cross section at Ch. 1+760m at Embankment 3

Figure 5 - Cross section at Ch. 3+120m at Embankment 5

Figure 6 – Cross section at Ch. 1+300m at Underpass 1

Figure 7 – Cross section at Ch. 2+560m at Underpass 2

Figure 8 – Cross section at Ch. 3+200m at Underpass 3

Figure 9 - Cross section at Ch. 2+840m (overbridge three span option)

Figure 10 - Cross section at Ch. 2+840m (overbridge single span option)

Appendix A – Earthworks Draft Form Cs

Cutting 1 – Ch 2+030m to 2+460m

Cutting 2 – Ch 2+720m to 2+950m

Cutting 3 – Ch 3+480m to 3+850m

Embankment 1 – Ch 0+040m to 0+370m

Embankment 2 – Ch 0+370m to 1+610m

Embankment 3 – Ch 1+610m to 2+030m

Embankment 4 – Ch 2+460m to 2+720m

Embankment 5 – Ch 2+950m to 3+480m

Llanddewi Velfrey Roundabout

Penblewin Roundabout

Cutting 1 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Cutting 1</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>2+030m to 2+460m</td><td>TYPE:</td><td>Cutting</td></tr>
<tr><td colspan="5">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">BH101</td><td></td></tr>
<tr><td colspan="5">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The current ground level at Ch2+030m is approximately 90mOD which then rises up in an easterly direction to approximately 100mOD at Ch2+460m. The road alignment in this zone roughly follows the route of a ridge feature with the land sloping down away from the road to the north and south.</td><td></td></tr>
<tr><td colspan="5">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Weathered Mudstone</td><td>1.8</td><td colspan="2">Grey clayey sandy gravel of mudstone.</td><td>Dip angles indicate a 25° dip of the Slade and Redhill formation to the south.

Conglomerate of Portfield and Haverford Mudstone formation were encountered in BH101. The boundary of the Portfield and</td></tr>
</table>

Mudstone	>1.8	Moderately strong grey **mudstone** with very closely spaced planar smooth bedding discontinuities. Conglomerate and sandstone of the Portfield and Haverford Mudstone formation may also be encountered, particularly in the south face of the cutting	Haverford Mudstone formation may lie closer to the proposed cutting (further northwest) than shown on geological plans. In the absence of any further GI in this area, the ground profile presented is based on a typical profile for the Slade & Redhill formation which is anticipated to be more commonly present in this area. Published geology indicates an inferred fault line immediately to the west of the cutting, with a vertical displacement of 40m.
PREVIOUS GROUND HISTORY	The proposed highway alignment lies entirely within existing farmland.		
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.		
GROUNDWATER	Groundwater seepage was recorded at 3.2m bgl at BH101. A well has been recorded approximately 50m to the north of the alignment. Monitoring has been undertaken in BH11 approximately 250m to the east, where the existing ground is at a similar elevation. Monitoring results indicated a groundwater level at approximately 4mbgl. Monitoring undertaken in BH102 located approximately 80m to the west of the cutting further down slope was dry.		

PRELIMINARY EARTHWORKS DESIGN		
SLOPE ANGLE	1V:2H	
MAX. EARTHWORKS HEIGHT	6m	
DESIGN STRATIGRAPHY	2m Granular Weathered Mudstone over Mudstone	
DESIGN GROUNDWATER	3mbgl	

MATERIAL	DENSITY γ'	ANGLE OF FRICTION φ'	COHESION c'	UNDRAINED SHEAR STRENGTH / UCS c_u / σ_c	YOUNGS MODULUS E'	
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	N/A	
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	N/A	
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	N/A	
Sandstone	22 kN/m^3	28°	60 kPa	10 MPa	N/A	
Conglomerate	24 kN/m^3	65°	350 kPa	50 MPa	N/A	

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.
3. The parameters derived for the Conglomerate have been derived on a location specific basis as detailed in Appendix C1 of the GIR.

SETTLEMENT

TOTAL (mm)	N/A	
DIFFERENTIAL (mm)	N/A	

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

None.

NOTES

1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as 'TBC' will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Cutting 2 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="5">Cutting 2</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td colspan="2">2+720m to 2+950m</td><td colspan="2">TYPE:</td><td>Cutting</td></tr>
<tr><td colspan="7">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="6">BH09, BH10, BH11</td><td></td></tr>
<tr><td colspan="7">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="6">The ground level at Ch 2+720 is 97mOD which rises to approximately 105mOD by Ch 2+860. The ground level then falls back to approximately 95mOD at Ch2+950. The proposed road alignment will cut through a ridge feature which generally slopes to the north.</td><td></td></tr>
<tr><td colspan="7">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td colspan="2">STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="3">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td colspan="2">Weathered Mudstone</td><td>18</td><td colspan="3">Dark grey, occasional red brown, sandy clay and sub-angular to angular mudstone gravel with occasional mudstone cobbles.</td><td rowspan="3">Cutting likely to be generally within the Slade and Redhill formation with the Portfield and Haverford Mudstone formation in part of the southern face of the cutting. Dip angles indicate the axis of an anticline lies along the alignment with a 15º northerly dip to the north and a southernly dip to the south, with no dip angles provided.</td></tr>
<tr><td colspan="2">Mudstone</td><td>>20</td><td colspan="3">Drillers description indicates mudstone between 17.6 – 20.0 mbgl at BH10.</td></tr>
<tr><td colspan="2">Weathered Conglomerate

(BH09 only)</td><td>3.5</td><td colspan="3">Brown occasionally grey slightly clayey sand with occasional rounded to sub-angular gravel of mixed lithologies and rare sandstone cobbles.</td></tr>
<tr><td colspan="2">Conglomerate (BH09 only)</td><td>9.5</td><td colspan="3">Very weak brown fine grained conglomerate with red brown staining and very closely spaced 45º bedding with rough planar discontinuities of variable angle.

Underlain by Weathered Mudstone as described above.</td><td>Weathered conglomerate (Portfield and Haverford Mudstone formation) was encountered overlying weathered mudstone (Slade and Redhill formation) in BH09</td></tr>
</table>

PREVIOUS GROUND HISTORY	Proposed alignment through existing farmland. The Llanfallteg road crosses the alignment at Ch2+840. Historic gravel pit has been identified ~10m to the south of the proposed cutting at Ch2+860.	
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.	
GROUNDWATER	The base of the cutting varies from approximately 95mOD to 90mOD from west to east. Groundwater seepages were recorded at 4.5m bgl (100mOD), 6.0m bgl (94mOD) and 3.0m bgl (94mOD) in BH09, BH10 and BH11 respectively. Groundwater monitoring in BH11 recorded a groundwater levels of between 3.6 and 4.2m bgl (93.4 and 92.8mAOD) which is equivalent to the approximate base of the cutting at this location. Groundwater monitoring in BH10 was dry, with a response zone from 3.0 to 12mbgl (96.4 to 87.4mOD). It is unclear why the results of monitoring from BH10 do not correspond with the monitoring from BH11 and the groundwater strikes.	

PRELIMINARY EARTHWORKS DESIGN

SLOPE ANGLE	1V:2.5H
MAX. EARTHWORKS HEIGHT	14m (Ch2+830)
DESIGN STRATIGRAPHY	4.5m Granular Weathered Mudstone over 7.5m Cohesive Weathered Mudstone over Mudstone
DESIGN GROUNDWATER	3.0mbgl or higher

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'
Weathered Conglomerate	18 kN/m^3	41°	0	-	N/A
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	N/A
Weathered Mudstone - Cohesive	20 kN/m^3	29°	2	75 kPa	N/A
Conglomerate	24 kN/m^3	50°	34 kPa	1 MPa	N/A
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	N/A

<table>
<tr><td colspan="2">Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.
3. The parameters derived for the Conglomerate have been derived on a location specific basis as detailed in Appendix C1 of the GIR.
4. Cohesive Weathered Mudstone c’ value of 2kPa introduced in GDR.</td><td></td></tr>
<tr><td colspan="3">SETTLEMENT</td></tr>
<tr><td>TOTAL (mm)</td><td>N/A</td><td rowspan="2"></td></tr>
<tr><td>DIFFERENTIAL (mm)</td><td>N/A</td></tr>
<tr><td colspan="3">OTHER DESIGN FEATURES (e.g. foundation treatment/hazards)</td></tr>
<tr><td colspan="2">Earthworks design may need to take account of the following:

▪ Proposed bridge over the proposed route at Ch 2+840m for the existing Llanfallteg road that runs approximately perpendicular to the proposed highway alignment.
▪ There is a nearby historic quarry. It may be prudent to further characterise any infill material with respect to the potential for gas generation particularly the content of organic carbon and inform a ground gas and subsequently health and safety risk assessments.</td><td></td></tr>
<tr><td colspan="3">NOTES</td></tr>
<tr><td colspan="3">1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 Preliminary GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.</td></tr>
</table>

Cutting 3 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="4">Cutting 3</td><td rowspan="2">REFERENCES / COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>3+480m to 3+850m</td><td>TYPE:</td><td colspan="2">Cutting</td></tr>
<tr><td colspan="6">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="5">TP04, TP05, BH04, BH05, BH06</td><td></td></tr>
<tr><td colspan="6">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="5">Existing ground level increases from 107mOD at western end of cutting (Ch3+480) to 131mOD at Ch3+680. Road alignment is approximately perpendicular to slope of existing ground and therefore southern side of cutting is greater in height.</td><td></td></tr>
<tr><td colspan="6">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="3">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Weathered Mudstone</td><td>1.4 to 2.4</td><td colspan="3">Typically brown and/or grey sandy gravel of mudstone. Also described locally as sand and clay.</td><td rowspan="2">Dip angles indicate cutting may be located near a anticline in the Slade and Redhill bedding, with the bedrock dipping towards the north to the north of the cutting and to the south to the south of the cutting.</td></tr>
<tr><td>Mudstone</td><td>>18</td><td colspan="3">Moderately weak becoming moderately strong with depth dark grey mudstone with very closely spaced smooth planar discontinuities at variable angle.</td></tr>
<tr><td>PREVIOUS GROUND HISTORY</td><td colspan="4">Proposed highway alignment through existing farmland.</td><td></td></tr>
<tr><td>CONTAMINATION RISK ASSESSMENT</td><td colspan="4">The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.</td><td></td></tr>
<tr><td>GROUNDWATER</td><td colspan="4">Base of cutting varies from 108mOD to 112mOD from west to east. Groundwater strike at 1.8mbgl (125.8mAOD) recorded in BH04 within Mudstone of the Slade and Redhill formation. No groundwater was detected during the subsequent ground water monitoring at BH04 with a response zone 3 to 18mbgl (124.6 to 109.6mOD)</td><td></td></tr>
<tr><td colspan="5">PRELIMINARY EARTHWORKS DESIGN</td><td rowspan="4"></td></tr>
<tr><td>SLOPE ANGLE</td><td colspan="4">1V:2H</td></tr>
<tr><td>MAX. EARTHWORKS HEIGHT</td><td colspan="4">21m (Ch3+670)</td></tr>
<tr><td>DESIGN STRATIGRAPHY</td><td colspan="4">2.5m Granular Weathered Mudstone over Mudstone</td></tr>
</table>

DESIGN GROUNDWATER	Below base of cutting				
MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	N/A
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	N/A
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	N/A

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.

SETTLEMENT

TOTAL (mm)	N/A
DIFFERENTIAL (mm)	N/A

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Earthworks design may need to account for the following:

- 1 No. Proposed footbridge at Ch 3+550m

NOTES

1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Embankment 1 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Embankment 1</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>0+040m to 0+370m</td><td>TYPE:</td><td>Embankment</td></tr>
<tr><td colspan="5">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">TP27, TP28, BH16, BH17</td><td>CBR01</td></tr>
<tr><td colspan="5">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The ground level is at 88mOD at Ch0+040m and falls to a minimum level of 77mOD at Ch 0+300m before rising back to 80mOD at Ch0+370. The topography generally slopes gently down to the south approximately perpendicular to the highway alignment.</td><td></td></tr>
<tr><td colspan="5">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Weathered Mudstone</td><td>>7.8</td><td colspan="2">Orange brown to dark grey sand, silt and clay with frequent sub-angular to sub-rounded mudstone and sandstone gravel and occasional cobbles .</td><td rowspan="2">The boundary between the Slade and Redhill formation and Portfield and Haverford Mudstone formation is shown to lie directly to the north of the proposed highway alignment. Dip angles indicate a variable 15º to 77º northerly bedrock dip angle.

Conglomerate of Portfield and Haverford Mudstone formation was encountered in BH16 and therefore the boundary may be further to the south than shown on geological plans in this location.</td></tr>
<tr><td>Weathered Conglomerate

(encountered from ground level in BH16 only)</td><td>GL to >8.0</td><td colspan="2">Firm orange brown to dark grey brown sandy silt and clay with numerous fine to coarse sub-angular gravels of mudstone and sandstone.</td></tr>
<tr><td>PREVIOUS GROUND HISTORY</td><td colspan="3">The proposed highway alignment traverses existing farmland and runs parallel to existing A40 alignment.</td><td></td></tr>
<tr><td>CONTAMINATION RISK ASSESSMENT</td><td colspan="3">The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.</td><td></td></tr>
</table>

GROUNDWATER	Groundwater strikes of 1.2m bgl (Conglomerate) and 5.2m bgl (Weathered Mudstone) were recorded at BH16 and BH17 respectively. Subsequent groundwater monitoring at BH17 recorded groundwater levels between 4.1m and 4.4m bgl. Groundwater strikes and monitoring indicate a ground water level of approximately 75mAOD.	

PRELIMINARY EARTHWORKS DESIGN

SLOPE ANGLE	1V:2H					c' of Cohesive Weathered Mudstone updated from GIR to reflect low stress conditions associated with shallow embankment., see Section 1.3.29.
MAX. EARTHWORKS HEIGHT	2.5m (Ch0+260) (Note: incomplete topographical data between Ch 0+000m and Ch 0+180m)					
DESIGN STRATIGRAPHY	10m Cohesive Weathered Mudstone over Mudstone					
DESIGN GROUNDWATER	Base of embankment					
MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNG'S MODULUS** E'	
Weathered Mudstone - Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2	
Weathered Mudstone - Cohesive	20 kN/m^3	29°	2	75 kPa	19,000 kN/m^2	
Weathered Conglomerate	18 kN/m^3	28°	0	40 kPa	10,000 kN/m^2	

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.
3. The parameters derived for the Weathered Conglomerate have been derived on a location specific basis as detailed in Appendix C1 of the GIR.
4. For Cohesive Weathered Mudstone, a c' value of 2kPa has been introduced in the GDR.

SETTLEMENT

TOTAL (mm)	<25mm (internal settlement – cohesive fill) <5mm (internal settlement – granular fill) Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.	
DIFFERENTIAL (mm)	TBC	

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Earthworks design may be impacted by the following:

- Existing culvert at Ch 0+290m to be retained and extended.
- 1 No. proposed 1.8m diameter bat crossing at Ch 0+305m.

NOTES

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2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as 'TBC' will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Embankment 2 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Embankment 2</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>0+370m to 1+610m</td><td>TYPE:</td><td>Embankment (localised cutting in western end)</td></tr>
<tr><td colspan="5">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">TP18, TP19, TP20, TP21, TP22, TP23, TP24, TP25, TP26, TP27, BH14, BH15</td><td>CBR01, CBR02, CBR03, CBR04, CBR05, CBR06, CBR07, CBR08</td></tr>
<tr><td colspan="5">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The ground level at Ch 0+370m is at 80mOD which then rises up to maximum level of 96mOD at Ch 1+280m. The ground level then drops down to approximately 85mOD at Ch 1+610m. Between Ch 0+420m to Ch 1+000m the slope perpendicular to the alignment falls to the south. However, after Ch 1+000m the ground level rises to both the north and south of the proposed highway alignment.</td><td></td></tr>
<tr><td colspan="5">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Made Ground (BH14 only)</td><td>0.85</td><td colspan="2">Dark grey to brown sandy silty clay and gravel. Inclusions of metal nails and clay pipe observed.</td><td rowspan="2">The published geology indicates the location of a Glaciofluvial superficial deposit towards the westernmost end of the embankment which was not identified in any of the exploratory holes.

The boundary between Haverford formation and underlying Portfield and Haverford Mudstone formation is shown to lie immediately to the north of the proposed</td></tr>
<tr><td>Weathered bands of mudstone and sandstone</td><td>>14.5</td><td colspan="2">Predominantly orange brown to occasionally grey gravelly sand, silt and clay. Gravel is rounded to angular mudstone and sandstone with occasional inclusions of quartz and igneous rock. Occasional to frequent mudstone and sandstone cobbles and boulders also present.</td></tr>
</table>

Weathered siltstone overlying siltstone bedrock (BH15 only)	GL to >14.5	Brown sandy **clay** and **silt** with frequent fine to coarse subangular grey siltstone gravel and cobbles to 6.2m depth. Grey with red brown black staining moderately strong **siltstone** with very closely spaced sub horizontal planar smooth bedding discontinuities. Occasionally undulating bedding.	highway alignment. Geological maps indicate an anticline to the north of the alignment and a syncline to the south. From review of the available logs it is anticipated that BH15 encountered the Portfield and Haverford Mudstone formation rather than the overlying Haverford Formation which was encountered at all other ground investigation locations.
PREVIOUS GROUND HISTORY	The proposed highway alignment runs through existing farmland parallel to the current A40 between Ch 0+420 and Ch 1+200m. Beyond Ch 1+200m the proposed alignment comprises widening of the existing highway. The existing Trefangor Cottage is to be demolished. A historic gravel pit has been identified to the south of the proposed alignment at Ch 0+450m. An unnamed road crosses the alignment at Ch 1+250m. A spring is located near the eastern end of the embankment.		
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed. This length is located approximately 200m hydraulically down-gradient in relation to the burial ground (approximate chainage. 1+100 to 1+200) and therefore there is potential for contaminated groundwater to be present beneath the scheme area.		
GROUNDWATER	A groundwater strike was recorded at 4.5m bgl in BH14. The strike at BH14 was within the Weathered Mudstone. No groundwater monitoring installations are present within this length of the scheme.		

PRELIMINARY EARTHWORKS DESIGN		
SLOPE ANGLE	1V:2H	

MAX. EARTHWORKS HEIGHT	4m (Ch 0+880) Localised shallow cutting <2m in height in western end	c' of Cohesive Weathered Mudstone updated from GIR to reflect low stress conditions associated with shallow embankment., see Section 1.3.29.
DESIGN STRATIGRAPHY	15m Cohesive Weathered Mudstone over Mudstone	
DESIGN GROUNDWATER	Base of embankment	

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'
Weathered Mudstone - Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2
Weathered Mudstone - Cohesive	20 kN/m^3	29°	2	75 kPa	19,000 kN/m^2
Weathered Sandstone	20 kN/m^3	34°	0	-	30,000 kN/m^2
Weathered Siltstone	20 kN/m^3	34°	0	-	NA
Siltstone	22 kN/m^3	55°	400 kPa	75 MPa	NA

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.

SETTLEMENT

TOTAL (mm)	<40mm (internal settlement – cohesive fill) <8mm (internal settlement – granular fill) Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.
DIFFERENTIAL (mm)	TBC

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

The earthworks design will need to take account of the following:

- 1 No. proposed bridleway underpass and associated access tracks at Ch1+300 with localised cuttings.

NOTES

1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as 'TBC' will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Embankment 3 – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Embankment 3</td><td colspan="2" rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>1+610m to 2+030m</td><td>TYPE:</td><td>Embankment</td></tr>
<tr><td colspan="6">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">TP17, BH102, BH12</td><td colspan="2">CBR11</td></tr>
<tr><td colspan="6">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The existing ground level at Ch 1+610m is approximately at 85mOD which initially falls and then rises up with increasing chainage, with minor undulations to approximately 90mOD at Ch 2+030m. The topography along this length generally slopes down to a stream fed valley formation to the north and rises up to the south approximately perpendicular to the highway alignment.</td><td colspan="2"></td></tr>
<tr><td colspan="6">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td colspan="2"></td></tr>
<tr><td>Made Ground</td><td>0.8 to 4.1</td><td colspan="2">Dark grey to brown gravelly clay and sandy gravel. The gravel is coarse angular to subangular mudstone and sandstone. (BH12 log description states ‘Probable Made Ground’).</td><td colspan="2" rowspan="4">Published geology indicates that the axis of an anticline passes through the alignment with dip angles indicating a 40^{o} to 75^{o} dip either side of the axis. The embankment lies above the Portfield and Haverford Mudstone formation and Haverford Mudstone Formation. The inferred fault line located at the easternmost extent of the embankment is shown to have a vertical displacement of 40m.

Weathered Mudstone underlain by Sandstone bedrock was encountered in BH12 only. This is interpreted to be of the Portfield and Haverford Mudstone Formation which underlies the Haverford Mudstone Formation.</td></tr>
<tr><td>Weathered Mudstone</td><td>3.2 to 6.9</td><td colspan="2">Orange brown to grey slightly sandy clay and gravel. Gravels are fine to coarse angular to subrounded flat and friable mudstone.</td></tr>
<tr><td>Mudstone</td><td>>8.5</td><td colspan="2">Grey mudstone with red brown staining. Very closely spaced 45^{o} planar smooth bedding discontinuities. Indication of increasing strength with depth.</td></tr>
<tr><td>Weathered Mudstone underlain by Sandstone (BH12 only)</td><td>>8.0</td><td colspan="2">Weathered mudstone underlain by strong dark grey medium grained sandstone.</td></tr>
</table>

PREVIOUS GROUND HISTORY	The proposed highway comprises widening of the existing alignment between Ch 1+610m and Ch 1+700m before it then deviates away from the existing highway, through existing farmland, at greater chainages. A historica quarry is located directly to the north of proposed highway alignment at Ch 1+800m.	
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed. There is a nearby historic quarry. It may be prudent to further characterise any infill material with respect to the potential for gas generation particularly the content of organic carbon and inform a ground gas and subsequently health and safety risk assessments.	
GROUNDWATER	A groundwater strike was recorded at 1.2mbgl in BH102. Monitoring of an installation in BH102, with a response zone at 2.5m to 8mbgl (82.9 to 77.4mOD), showed the installation to be dry. Monitoring of the installation in BH102, with a response zone at 2m to 8mbgl (73.7 to 67.7mbgl), showed the installation to be dry. A small watercourse passes through the alignment at the low point in the topography. Springs are located 100m to the north and 40m to the south of the alignment.	

PRELIMINARY EARTHWORKS DESIGN

SLOPE ANGLE	1V:2H
MAX. EARTHWORKS HEIGHT	8.7m (Ch 1+700m)
DESIGN STRATIGRAPHY	5m Made Ground associated with existing embankment (no ground investigation available) over 4m Granular Weathered Mudstone over Mudstone
DESIGN GROUNDWATER	Base of embankment

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** **c'**	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS E'**
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	19,000 kN/m^2
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2
Sandstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2

From Ch 1+610m to Ch 1+800m, the earthworks comprises widening of the existing embankment

<table>
<tr><td colspan="2">Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.</td><td></td></tr>
<tr><td colspan="3">SETTLEMENT</td></tr>
<tr><td>TOTAL (mm)</td><td><50mm (internal settlement – cohesive fill)

<10mm (internal settlement – granular fill)

Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.</td><td rowspan="2"></td></tr>
<tr><td>DIFFERENTIAL (mm)</td><td>TBC</td></tr>
<tr><td colspan="3">OTHER DESIGN FEATURES (e.g. foundation treatment/hazards)</td></tr>
<tr><td colspan="2">The earthworks design should take account of the following:

▪ The existing culvert at Ch 1+780m to remain. No extension required.</td><td></td></tr>
<tr><td colspan="3">NOTES</td></tr>
<tr><td colspan="3">1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.</td></tr>
</table>

Embankment 4 – Geotechnical Summary Form

EARTHWORKS ZONE	Embankment 4			REFERENCES / COMMENTS
CHAINAGE:	2+460m to 2+720m	TYPE:	Embankment (Fill)	
RELEVANT EXPLORATORY HOLES				
BH11				
EXISTING TOPOGRAPHY				
The existing ground level is approximately 100mOD at Ch 2+460m. The ground level then falls to approximately 90mOD at Ch 2+650m before rising back to 97mOD at Ch 2+720m. The topography of the land slopes gently down to the north and rises up to the existing route of the A40 in the south.				
LOCATION-SPECIFIC GROUND PROFILE				
STRATA	DEPTH OF BASE (mbgl)	TYPICAL DESCRIPTION		
Weathered Mudstone	>11.0	Brown mottled grey to dark grey very clayey sandy subangular to angular mudstone **gravel** and lithorelicts. Occasional mudstone cobbles.		Dip angles indicate a 25º southerly dip to the south and a 15º northerly dip to the north, possibly indicating the location of an anticline in the Slade and Redhill formation. Limited ground investigation available for Embankment 4.
PREVIOUS GROUND HISTORY	The proposed highway alignment runs through existing farmland.			
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.			
GROUNDWATER	A groundwater strike of 3.0m bgl (94mOD) was recorded at BH11 and subsequent groundwater monitoring recorded a ground water level between 4.2 and 3.6m bgl (92.8 and 93.4mOD).			
PRELIMINARY EARTHWORKS DESIGN				
SLOPE ANGLE	1V:2H The stability will be dependent on the strength of the subformation, and localised xcavation and replacement may be required. Further investigation is recommended.			

MAX. EARTHWORKS HEIGHT	11m (Ch 2+640m)
DESIGN STRATIGRAPHY	6m of Granular Weathered Mudstone over 2m of Cohesive Granular Weathered Mudstone underlain by Granular Weathered Mudstone
DESIGN GROUNDWATER	Base of embankment

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'
Weathered Mudstone - Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.

SETTLEMENT

TOTAL (mm)	<240mm (internal settlement – cohesive fill) <50mm (internal settlement – granular fill) Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.
DIFFERENTIAL (mm)	TBC

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Earthworks design may may need to take account of the following:

- 1 No. Proposed underpass at Ch 2+570m.
- 1 No. Proposed 1.8m diameter culvert at Ch 2+640m.
- There remains considerable uncertainty of the ground conditions beneath the proposed embankment, due to little available ground investigation information.

NOTES

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2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as 'TBC' will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Embankment 5 – Geotechnical Summary Form

EARTHWORKS ZONE	Embankment 5			REFERENCES / COMMENTS
CHAINAGE:	2+950m to 3+480m	TYPE:	Embankment (Fill)	
RELEVANT EXPLORATORY HOLES				
BH07				
EXISTING TOPOGRAPHY				
The existing ground surface undulates considerably between Ch 2+950m and Ch 3+480m. The ground level at Ch 2+950m is approximately 100mOD before then falling sharply to 80mOD within a wooded stream fed valley between Ch 3+000m to Ch 3+100m. The ground level then rises back up to 110mOD by Ch 3+480m.				
LOCATION-SPECIFIC GROUND PROFILE				
STRATA	DEPTH OF BASE (mbgl)	TYPICAL DESCRIPTION		
Weathered Mudstone	4.4	Dark brown to grey clayey sandy fine to coarse sub-angular to angular **gravel** and **cobbles** ofmudstone. Red brown staining observed.		Dip angles indicate a 30º northwesterly dip in the Slade and Redhill Formation to the northeast of the embankment. The published geology indicates an area of natural superficial Till deposits between Ch 3+020m and Ch 3+090m, however no ground investigation has been undertaken in this area.
Mudstone	>10.5	Grey moderately strong **mudstone** with very closely spaced planar smooth sub-horizontal to 45º bedding discontinuities and sub-vertical discontinuities. Occasional bands of soft grey sandy clay.		
PREVIOUS GROUND HISTORY	The proposed highway alignment runs through existing farmland.			
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.			
GROUNDWATER	No groundwater strikes recorded or groundwater monitoring undertaken in BH7. Two small watercourses pass through the alignment.			
PRELIMINARY EARTHWORKS DESIGN				

SLOPE ANGLE	1V:2H The stability will be dependent on the strength of the subformation, and localised xcavation and replacement may be required. Further investigation is recommended.					
MAX. EARTHWORKS HEIGHT	24m (Ch 3+130m)					
DESIGN STRATIGRAPHY	4m of Cohesive Weathered Mudstone over Mudstone					
DESIGN GROUNDWATER	Base of embankment					
MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'	
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2	
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2	

Notes

1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.

SETTLEMENT

TOTAL (mm)	<700mm (internal settlement – cohesive fill) <225mm (internal settlement – granular fill) Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.	
DIFFERENTIAL (mm)	TBC	

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Earthworks design may be locally impacted by the following:

- 1 No. proposed 1.8m diameter mammal crossing tunnel at Ch 2+990m.
- 2 No. proposed 1.8m diameter culverts at Ch 3+150m and Ch 3+270 m.
- 1 No. proposed underpass for existing pathway at Ch 3+200m.
- There remains considerable uncertainty of the ground conditions beneath the proposed embankment, due to little available ground investigation information.

NOTES

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2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.

Llanddewi Velfrey Roundabout – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Llanddewi Velfrey Roundabout (see Earthworks Zones figure)</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>Beyond 3+850m</td><td>TYPE:</td><td>Majority cut with some fill</td></tr>
<tr><td colspan="5">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">TP01, TP02, TP03, BH01, BH02, BH03</td><td></td></tr>
<tr><td colspan="5">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The ground level in the location of the proposed Llanddewi Velfrey Roundabout is at approximately 125mOD with the ground level rising to the west and falling steeply in a north-easterly direction. The ground level beyond the Bethel Chapel, at the most easterly extent of the scheme, is approximately 98mOD.</td><td></td></tr>
<tr><td colspan="5">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Made Ground</td><td>0.5 to 3.1</td><td colspan="2">Dark grey to brown sandy clay and fine to coarse mudstone gravel with mudstone cobble and boulder inclusions and pieces of metal, plastic, tile and concrete.</td><td rowspan="4">The majority of the earthworks zone is underlain by the Slade and Redhill Formation with the south-western spur of the proposed roundabout shown as the overlying Portfield and Haverford Mudstone Formation and Haverford Formations. Dip angles are typically towards the south, varying from 60° to 30°.

A band of Conglomerate bedrock was identified in BH03 only. The conglomerate is interpreted to be from the Portfield and Haverford Mudstone Formation that is overlying the mudstones within the Slade and Redhil Formation.</td></tr>
<tr><td>Weathered Mudstone</td><td>2.5 to 4.4</td><td colspan="2">Orange brown to light grey soft silty sandy clayey fine to coarse sub-angular predominantly mudstone gravel. Occasional sub-angular to sub-rounded sandstone and mudstone cobbles</td></tr>
<tr><td>Mudstone</td><td>>15.0</td><td colspan="2">Moderately weak to strong grey red brown stained mudstone with closely spaced planar smooth bedding discontinuities of variable angle with an indication of close random fractures. Some indication that strength increase with depth.</td></tr>
<tr><td>Conglomerate (BH03 only)</td><td>4.5 (1.0m in thickness)</td><td colspan="2">Moderately strong fine grained conglomerate of Portfield Formation and Haverford Mudstone Formation.</td></tr>
</table>

PREVIOUS GROUND HISTORY	The majority of proposed alignment involves adaptation of the existing highway. There are localised areas where proposed alignment is through existing farmland. A historic quarry has been identified to the south of the current alignment, within close proximity of the proposed Llanddewi Velfrey exit to the roundabout.	
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed. There is a historic quarry within the footprint of the cutting for the south tie-in. It would be prudent to further characterise the infill material with respect to the potential for gas generation, particularly the content of organic carbon, to inform a ground gas and subsequentl health and safety risk assessment.	
GROUNDWATER	Two strikes were recorded in BH03, at 1.0m bgl and 3.5m bgl within Weathered Mudstone. Groundwater monitoring was undertaken in BH01, which recorded levels from 2.7 to 2.6mbgl (95.8 to 95.9mOD)	

PRELIMINARY EARTHWORKS DESIGN					
SLOPE ANGLE	1V:2H				
MAX. EARTHWORKS HEIGHT	15m cutting (Llanddewi Velfrey exit of proposed roundabout) 12m embankment (Bethel Chapel exit of proposed roundabout)				
DESIGN STRATIGRAPHY	2.5m of Weathered Granular Mudstone over Mudstone				
DESIGN GROUNDWATER	Base of embankment				
MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'
Weathered Mudstone - Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2
Conglomerate	24 kN/m^3	55°	200 kPa	25 MPa	N/A (cutting)
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2

Notes

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2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.
3. The parameters derived for the Conglomerate have been derived on a location specific basis as detailed in Appendix C1 of the GIR.

<table>
<tr><td colspan="3">SETTLEMENT</td></tr>
<tr><td>TOTAL (mm)</td><td>Cutting: N/A

Embankment:

<240mm (internal settlement – cohesive fill)

<50mm (internal settlement – granular fill)

Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.</td><td rowspan="2"></td></tr>
<tr><td>DIFFERENTIAL (mm)</td><td>TBC</td></tr>
<tr><td colspan="3">OTHER DESIGN FEATURES (e.g. foundation treatment/hazards)</td></tr>
<tr><td colspan="2">The earthworks design should take due account of the following:

▪ A cutting is predominantly required, however, localised areas of considerable fill are required at the Bethel Chapel exit to the Llanddewi Velfrey roundabout and the easternmost extent of the scheme.</td><td></td></tr>
<tr><td colspan="3">NOTES</td></tr>
<tr><td colspan="3">1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.</td></tr>
</table>

Penblewin Roundabout – Geotechnical Summary Form

<table>
<tr><td>EARTHWORKS ZONE</td><td colspan="3">Penblewin Roundabout</td><td rowspan="2">REFERENCES
/ COMMENTS</td></tr>
<tr><td>CHAINAGE:</td><td>Includes 0+000m to 0+040m</td><td>TYPE:</td><td>Cut with localised fill</td></tr>
<tr><td colspan="5">RELEVANT EXPLORATORY HOLES</td></tr>
<tr><td colspan="4">TP28, TP29, TP30</td><td></td></tr>
<tr><td colspan="5">EXISTING TOPOGRAPHY</td></tr>
<tr><td colspan="4">The ground level around the existing roundabout near Penblewin is at 93mOD which then drops down to 88mOD at Ch0+040. The ground in the Penblewin Roundabout generally slopes in an easterly to south-easterly direction.</td><td></td></tr>
<tr><td colspan="5">LOCATION-SPECIFIC GROUND PROFILE</td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (mbgl)</td><td colspan="2">TYPICAL DESCRIPTION</td><td></td></tr>
<tr><td>Weathered Mudstone</td><td>>3.2</td><td colspan="2">Orange brown to grey slightly clayey gravelly sand or sandy gravel. Gravel is subrounded to subangular sandstone and mudstone with mudstone cobbles present. Gravel described as flat and friable. Locally described as brown clay.</td><td>Dip angles indicate a general 15° dip of the Slade and Redhill formation to the north. A fault line is shown to the southwest of the roundabout, with the vertical displacement unknown. The northern exit of the proposed roundabout is anticipated to lie within the Portfield and Haverford Mudstone (no existing ground investigation in this location).</td></tr>
<tr><td>PREVIOUS GROUND HISTORY</td><td colspan="3">The majority of proposed highway alignment is situated over the existing highway. There are some localised areas where the proposed alignment is through existing farmland.</td><td></td></tr>
<tr><td>CONTAMINATION RISK ASSESSMENT</td><td colspan="3">The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.</td><td></td></tr>
<tr><td>GROUNDWATER</td><td colspan="3">No groundwater strikes were recorded in the trial pits and no groundwater monitoring is available in the area.

Groundwater monitoring in BH17 located 200m to the east recorded groundwater levels between 4.1 and 4.4mbgl (76.0 to 75.7mOD).</td><td></td></tr>
</table>

<table>
<tr><td colspan="6">PRELIMINARY EARTHWORKS DESIGN</td><td rowspan="7">c’ of Cohesive Weathered Mudstone updated from GIR to reflect low stress conditions associated with shallow embankment., see Section 1.3.29.</td></tr>
<tr><td colspan="2">SLOPE ANGLE</td><td colspan="4">1V:2H</td></tr>
<tr><td colspan="2">MAX. EARTHWORKS HEIGHT</td><td colspan="4">5m cutting
5m embankment</td></tr>
<tr><td colspan="2">DESIGN STRATIGRAPHY</td><td colspan="4">10m Cohesive Weathered Mudstone over Mudstone</td></tr>
<tr><td colspan="2">DESIGN GROUNDWATER</td><td colspan="4">Base of embankment</td></tr>
<tr><td>MATERIAL</td><td>DENSITY
γ’</td><td>ANGLE OF FRICTION
φ’</td><td>COHESION
c’</td><td>UNDRAINED SHEAR STRENGTH / UCS
c_u / σ_c</td><td>YOUNGS MODULUS
E’</td></tr>
<tr><td>Weathered Mudstone - Cohesive</td><td>20 kN/m^3</td><td>29°</td><td>2</td><td>75 kPa</td><td>19,000 kN/m^2</td></tr>
<tr><td>Mudstone</td><td>22 kN/m^3</td><td>28°</td><td>60 kPa</td><td>10 MPa</td><td>150,000 kN/m^2</td><td></td></tr>
<tr><td colspan="6">Notes
1. The above are characteristic parameters for preliminary design in accordance with Eurocode 7.
2. Unless stated below, the parameters presented in this table are site wide parameters as described in Section 6 of the GIR.
3. Cohesive Weathered Mudstone c’ value of 2 introduced in GDR.</td><td></td></tr>
<tr><td colspan="7">SETTLEMENT</td></tr>
<tr><td colspan="2">TOTAL (mm)</td><td colspan="4">Cutting: N/A
Embankment:
<50mm (internal settlement – cohesive fill)
<10mm (internal settlement – granular fill)
Limited GI available for the sub-formation and therefore total settlement has not been calculated at this stage. Total settlement will be calculated at detailed design.</td><td rowspan="2"></td></tr>
<tr><td colspan="2">DIFFERENTIAL (mm)</td><td colspan="4">TBC</td></tr>
<tr><td colspan="7">OTHER DESIGN FEATURES (e.g. foundation treatment/hazards)</td></tr>
<tr><td colspan="6"></td><td></td></tr>
<tr><td colspan="7">NOTES</td></tr>
<tr><td colspan="7">1. This summary sheet has been prepared as part of the Key Stage 3 GIR and updated as part of the Key Stage 3 GDR. Any subsequent change will be presented in an update of the summary sheet in the Key Stage 6 GDR.
2. The information presented within this summary sheet is on the basis of the available ground investigation information. Should further ground investigation be undertaken as part of Key Stage 6, the summary sheet will be updated accordingly.
3. Fields marked as ‘TBC’ will be confirmed following completion of the geotechnical detailed design and will be presented in an updated version of the summary sheet in the Key Stage 6 GDR.</td></tr>
</table>

Appendix B – Excavatability assessment

Appendix C – Structures Draft Form Cs

Underpass 1 (SBR-0130) – Ch 1+300m

Underpass 2 (SBR-0257) – Ch 2+570m

Overbridge 1 (SBR-0285) – Ch 2+850m

Underpass 3 (SBR-0320) – Ch 3+200m

Footbridge 1 (SBR-0355) – Ch 3+550m

Underpass 1 – Geotechnical Summary Form

CHAINAGE INTERVAL:	Ch. 1+300m	TYPE:	Underpass 1 (SBR-0130)	REFERENCES / COMMENTS
AIP REF No:	A40LVP-ARP-SBR-0130-RP-C-0001	DESIGN LIFE:	120 years	
RELEVANT EXPLORATORY HOLES				
TP20, TP21, BH14, BH15				
LOCATION-SPECIFIC GROUND PROFILE				
STRATA	DEPTH OF BASE (m bgl)	STRATA DESCRIPTION		
Made Ground (BH14 only)	0.85	Dark grey to brown sandy silty **clay** and **gravel**. Inclusions of metal nails and clay pipe observed.		The boundary between Haverford Formation and underlying Portfield and Haverford Mudstone Formation is indicated to the north of the proposed structure.
Weathered bands of mudstone and sandstone	>14.5	Predominantly orange brown to occasionally grey gravelly **sand**, **silt** and **clay**. Gravel is rounded to angular mudstone and sandstone with occasional inclusions of quartz and igneous rock (log description). Occasional to frequent mudstone and sandstone cobbles and boulders also present.		

Weathered Siltstone overlying Siltstone (BH15 only)	GL to >14.5	Brown sandy **clay** and **silt** with frequent fine to coarse subangular grey siltstone gravel and cobbles to 6.2m depth. Grey with red brown black staining moderately strong **siltstone** with very closely spaced sub horizontal planar smooth bedding discontinuities. Occasionally undulating bedding.	Geological maps indicate an anticline to the north of the structure and a syncline to the south. Weathered bands of mudstone and sandstone (Haverford Formation) underlying minimal made ground deposits anticipated in the southern section of the underpass. Weathered siltstone overlying siltstone bedrock (Portfield and Haverford Mudstone Formation) anticipated in the northern section of the underpass.
PREVIOUS GROUND HISTORY	Proposed underpass will pass beneath the existing alignment of the A40. A historic quarry is located to the north of the proposed underpass, see features and constraints plan.		
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed. This section of the scheme is located approximately 200m hydraulically down-gradient in relation to the burial ground (scheme approximate ch. 1+100m to 1+200m) and therefore there is a potential for contaminated groundwater presence in the underpass cutting depths.		
GROUNDWATER	A groundwater strike was recorded in BH14 at 4.5m bgl. The strike in BH14 was within Weathered Mudstone. No groundwater monitoring installation at BH14.		
RECOMMENDED PARAMETERS			

MATERIAL	DENSITY γ'	ANGLE OF FRICTION φ'	COHESION c'	UNDRAINED SHEAR STRENGTH c_u	YOUNGS MODULUS E'	
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2	
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	19,000 kN/m^2	
Weathered Sandstone	20 kN/m^3	34°	0	-	30,000 kN/m^2	
Weathered Siltstone	20 kN/m^3	34°	0	-	NA	
Siltstone	22 kN/m^3	55°	400 kPa	75 MPa	NA	

SPREAD FOUNDATION DESIGN

STRUCTURE ELEMENT	BASE (mOD)	DESIGN FOUNDING STRATUM	FOOTING SIZE (m)	'ALLOWABLE' BEARING PRESSURE (kN/m^2)	
Underpass	Approx. 90.0 to 90.5	Cohesive Weathered Mudstone over Granular Weathered Mudstone	20m x 3.6m (2.5m x 3.6m units)	TBC – detailed design to consider effects of backfilling to minimise differential settlements	
Wing walls	Approx. 90.0 to 90.5	Cohesive Weathered Mudstone over Granular Weathered Mudstone	2.0m x 3.6m units	TBC – detailed design to consider effects of backfilling to minimise differential settlements	
DESIGN GROUNDWATER		Base of underpass			

PRELIMINARY PILE DESIGN

PILE TYPE:	NA
CRITERIA FOR TOE:	NA
NEG. SKIN FRICTION:	NA

STRUCTURE ELEMENT	TOE LEVEL (mOD)	FOUNDING STRATUM	LENGTH (m)	DIAMETER (m)	ULS DESIGN LOAD (kN)
NA	NA	NA	NA	NA	NA

SETTLEMENT

STRUCTURE ELEMENT	BASE (mOD)	IMMEDIATE (mm)	TOTAL (mm)	90% (Months)	REMAINING (mm)	
						To be confirmed

<table>
<tr><td>Underpass</td><td>Approx. 90.0 to 90.5</td><td>TBC</td><td><25mm</td><td>TBC .</td><td>TBC</td><td rowspan="3">at detailed design stage</td></tr>
<tr><td>Wingwalls</td><td>Approx. 90.0 to 90.5</td><td>TBC</td><td><25mm</td><td>TBC</td><td>TBC</td></tr>
<tr><td colspan="4">DIFFERENTIAL (mm):</td><td colspan="2">Anticipated to be <25mm for underpass</td></tr>
</table>

CHEMICAL ANALYSIS

Material	**ACEC Class**	**DS Class**	
All	AC-1	DS-1	Further details in Section 6 of the GIR

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Associated earthworks:

Cuttings are proposed at either side of the underpass for access. The proposed maximum height of the cutting is 6m at an angle of 1V:2H.

NOTES

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Underpass 2 – Geotechnical Summary Form

<table>
<tr><td>CHAINAGE INTERVAL:</td><td>Ch. 2+570m</td><td>TYPE:</td><td>Underpass 2 (SBR-0257)</td><td>REFERENCES
/
COMMENTS</td></tr>
<tr><td>AIP REF No:</td><td>A40LVP-ARP-SBR-0257-RP-C-0001</td><td>DESIGN LIFE:</td><td>120 years</td><td></td></tr>
<tr><td colspan="4">RELEVANT EXPLORATORY HOLES</td><td></td></tr>
<tr><td colspan="4">BH11 (located approximately 150m to the east)</td><td></td></tr>
<tr><td colspan="4">LOCATION-SPECIFIC GROUND PROFILE</td><td></td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (m bgl)</td><td colspan="2">STRATA DESCRIPTION</td><td></td></tr>
<tr><td>Weathered Mudstone</td><td>>11.0</td><td colspan="2">Brown mottled grey to dark grey very clayey sandy sub-angular to angular mudstone gravel and lithorelicts. Occasional mudstone cobbles.</td><td>No GI within 150m of proposed underpass.

Geology indicates Slade and Redhill Formation with dip of 25º in a southerly direction to the south and a 15º northerly dip to the north, possibly indicating the location of an anticline in this area</td></tr>
<tr><td>PREVIOUS GROUND HISTORY</td><td colspan="3">The proposed underpass in area of existing farmland.</td><td></td></tr>
<tr><td>CONTAMINATION RISK ASSESSMENT</td><td colspan="3">The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.</td><td></td></tr>
<tr><td>GROUNDWATER</td><td colspan="3">BH11 (Ch 2+720m) encountered a groundwater seepage at 3.0m bgl and subsequent groundwater monitoring recorded groundwater levels between 3.6m and 4.2m bgl (93.4m to 92.8mOD).</td><td></td></tr>
</table>

RECOMMENDED PARAMETERS

MATERIAL	DENSITY γ'	ANGLE OF FRICTION φ'	COHESION c'	UNDRAINED SHEAR STRENGTH / UCS c_u / σ_c	YOUNGS MODULUS E'
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2

SPREAD FOUNDATION DESIGN

STRUCTURE ELEMENT	BASE (mOD)	DESIGN FOUNDING STRATUM	FOOTING SIZE (m)	'ALLOWABLE' BEARING PRESSURE (kN/m^2)
Underpass	Approx. 93.0	Granular Weathered Mudstone	24m x 3.6m (2.5m x 3.6m units)	TBC
Wing walls	Approx. 93.0	Granular Weathered Mudstone	2.0m x 3.6m units	TBC

DESIGN GROUNDWATER	Base of underpass (southern section) Base of embankment (northern section)

PRELIMINARY PILE DESIGN

PILE TYPE:	NA
CRITERIA FOR TOE:	NA
NEG. SKIN FRICTION:	NA

STRUCTURE ELEMENT	TOE LEVEL (mOD)	FOUNDING STRATUM	LENGTH (m)	DIAMETER (m)	ULS DESIGN LOAD (kN)
NA	NA	NA	NA	NA	NA

SETTLEMENT

STRUCTURE ELEMENT	BASE (mOD)	IMMEDIATE (mm)	TOTAL (mm)	90% (Months)	REMAINING (mm)	
Underpass	Approx. 93.0	TBC	<25mm	TBC .	TBC	To be confirmed at detailed design stage
Wingwalls	Approx. 93.0	TBC	<25mm	TBC	TBC	
DIFFERENTIAL (mm):				Anticipated to be <25mm		

CHEMICAL ANALYSIS

Material	ACEC Class	DS Class

All	AC-1	DS-1	Further details in Section 6 of the GIR
OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*			
Associated earthworks: Embankment 4			
NOTES			

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Overbridge 1 – Geotechnical Summary Form

<table>
<tr><td>CHAINAGE INTERVAL:</td><td>Ch. 2+850m</td><td>TYPE:</td><td>Overbridge 1 (SBR-0285)</td><td>REFERENCES / COMMENTS</td></tr>
<tr><td>AIP REF No:</td><td>A40LVP-ARP-SBR-0285-RP-C-0001</td><td>DESIGN LIFE:</td><td>120 years</td><td></td></tr>
<tr><td colspan="4">RELEVANT EXPLORATORY HOLES</td><td></td></tr>
<tr><td colspan="4">BH09, BH10</td><td></td></tr>
<tr><td colspan="4">LOCATION-SPECIFIC GROUND PROFILE</td><td></td></tr>
<tr><td>STRATA</td><td>DEPTH OF BASE (m bgl)</td><td colspan="2">STRATA DESCRIPTION</td><td></td></tr>
<tr><td colspan="4">Northern Abutment (BH10)</td><td rowspan="8">Northern abutment anticipated to be within the Slade and Redhill Formation. Southern abutment interpreted to be within Portfield and Haverford Mudstone Formation – based on borehole descriptions.

Dip angles suggest axis of anticline lies within earthworks zone with a 15° northerly dip to the north and a southerly dip to the south, with no dip angle provided.</td></tr>
<tr><td>Weathered Mudstone</td><td>17.6</td><td colspan="2">Dark grey, occasional red brown, sandy clay and sub-angular to angular mudstone gravel. Occasional mudstone cobbles.</td></tr>
<tr><td>Mudstone</td><td>>20.0</td><td colspan="2">Driller’s description indicates mudstone between 17.6m and 20.0 mbgl at BH10.</td></tr>
<tr><td colspan="4">Southern abutment (BH09)</td></tr>
<tr><td>Weathered Conglomerate</td><td>3.5</td><td colspan="2">Brown occasionally grey slightly clayey sand with occasional rounded to sub-angular gravel of mixed lithologies and rare sandstone cobbles.</td></tr>
<tr><td>Conglomerate</td><td>9.5</td><td colspan="2">Very weak brown fine grained conglomerate with red brown staining and very closely spaced 45° bedding with rough planar discontinuities of variable angle.</td></tr>
<tr><td>Weathered mudstone</td><td>>13</td><td colspan="2">Dark grey very sandy clay</td></tr>
<tr><td>PREVIOUS GROUND HISTORY</td><td colspan="3">The proposed overbridge lies in an area of existing farmland. A historic gravel pit has been identified approximately 10m to the south of the proposed cutting at Ch 2+860.</td></tr>
</table>

CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.	
GROUNDWATER	Groundwater strikes of 4.5m bgl (100.7mOD), 6.0m bgl (93.4mOD) and 3.0m bgl (94.0mAOD) were recorded in BH09 and BH10 and BH11 respectively. Groundwater monitoring in BH10 was dry, for the response zone that extended from 3.0 to 12mbgl (96.4m to 87.4mOD). Groundwater monitoring in BH11, approximately 100m to the west, recorded a groundwater level between 3.6m and 4.2m bgl (93.4m and 92.8mAOD). It is unclear why the results of monitoring from BH10 do not correspond with the monitoring from BH11 and the groundwater strikes.	

RECOMMENDED PARAMETERS

MATERIAL	DENSITY γ'	ANGLE OF FRICTION φ'	COHESION c'	UNDRAINED SHEAR STRENGTH c_u	YOUNGS MODULUS E'
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	19,000 kN/m^2
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2
Weathered Conglomerate	18 kN/m^3	41°	0	-	NA
Conglomerate	24 kN/m^3	50°	34 kPa	1 MPa	NA

SHALLOW FOUNDATION DESIGN

STRUCTURE ELEMENT	BASE (mOD)	DESIGN FOUNDING STRATUM	FOOTING SIZE (m)	'ALLOWABLE' BEARING PRESSURE (kN/m^2)
Single span option				
Northern abutment	93.1	6m of cohesive weathered mudstone over mudstone	TBC	TBC

Southern abutment	98.1	2.5m of conglomerate over 8m cohesive weathered mudstone underlain by mudstone	TBC	TBC	
Three span option					
Northern abutment	94.4	8m of cohesive weathered mudstone underlain by mudstone	4m x 8.5m x 1m deep	TBC	
Northern pier	90.1	3m of cohesive weathered mudstone underlain by mudstone	4.5m x 4m x 1m deep	TBC	
Southern abutment	99.3	7m of cohesive weathered mudstone underlain by mudstone	4.5m x 4m x 1m deep	TBC	
Southern pier	93.4	4m of conglomerate over 8m cohesive weathered mudstone underlain by mudstone	4m x 8.5m x 1m deep	TBC	

PRELIMINARY PILE DESIGN

PILE TYPE:	NA
CRITERIA FOR TOE:	NA
NEG. SKIN FRICTION:	NA

STRUCTURE ELEMENT	**TOE LEVEL (mOD)**	**FOUNDING STRATUM**	**LENGTH (m)**	**DIAMETER (m)**	**ULS DESIGN LOAD (kN)**
NA	NA	NA	NA	NA	NA

SETTLEMENT

STRUCTURE ELEMENT	**BASE (mOD)**	**IMMEDIATE (mm)**	**TOTAL (mm)**	**90% (Months)**	**REMAINING (mm)**
Northern abutment	TBC	TBC	TBC	TBC	TBC
Southern abutment	TBC	TBC	TBC	TBC	TBC
DIFFERENTIAL (mm):				TBC	

CHEMICAL ANALYSIS			
Material	**ACEC Class**	**DS Class**	Further details in Section 6 of the GIR
All	AC-1	DS-1	
OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*			
The depth of the foundations is influenced by the proposed slope angles of cutting 2, which remain uncertain at this stage. Further investigation is proposed in the area in order to further develop the design.			
NOTES			

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Underpass 3 – Geotechnical Summary Form

CHAINAGE INTERVAL:	Ch. 3+200m	TYPE:	Underpass 3 (SBR-0320)	REFERENCES / COMMENTS
AIP REF No:	A40LVP-ARP-SBR-0320-RP-C-0001	DESIGN LIFE:	120 years	
RELEVANT EXPLORATORY HOLES				
BH07 (approx. 70m to the northeast)				
LOCATION-SPECIFIC GROUND PROFILE				

STRATA	DEPTH OF BASE (m bgl)	STRATA DESCRIPTION	
Weathered Mudstone	4.4	Dark brown to grey clayey sandy fine to coarse sub-angular to angular **gravel** and **cobbles** of mudstone. Red brown staining observed.	Dip angles indicate a 30^{o} northerly dip in the Slade and Redhill Formation to the northwest.
Mudstone	>10.5	Grey moderately strong **mudstone** with very closely spaced planar smooth sub horizontal to 45^{o} bedding discontinuities and sub vertical discontinuities. Occasional bands of soft grey sandy clay.	
PREVIOUS GROUND HISTORY	Proposed underpass in area of existing farmland.		
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.		
GROUNDWATER	No groundwater strikes or monitoring have been recorded for BH07. The proposed location of the underpass is approximately 50m to the east of the watercourse, which lies at the base of a steep slope to the west.		

RECOMMENDED PARAMETERS

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** c'	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** E'	
Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2	

Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2	

SPREAD FOUNDATION DESIGN

STRUCTURE ELEMENT	**BASE (mOD)**	**DESIGN FOUNDING STRATUM**	**FOOTING SIZE (m)**	**'ALLOWABLE' BEARING PRESSURE (kN/m^2)**	
Underpass	Approx. 91.0 to 92.0	Granular Weathered Mudstone (fill)	24m x 3.6m (2.5m x 3.6m units)	TBC	
Wing walls	Approx. 91.0 to 92.0	Granular Weathered Mudstone (fill)	2.0m x 3.6m units	TBC	
DESIGN GROUNDWATER		Base of embankment			

PRELIMINARY PILE DESIGN

PILE TYPE:	NA	
CRITERIA FOR TOE:	NA	
NEG. SKIN FRICTION:	NA	

STRUCTURE ELEMENT	**TOE LEVEL (mOD)**	**FOUNDING STRATUM**	**LENGTH (m)**	**DIAMETER (m)**	**ULS DESIGN LOAD (kN)**	
NA	NA	NA	NA	NA	NA	

SETTLEMENT

STRUCTURE ELEMENT	**BASE (mOD)**	**IMMEDIATE (mm)**	**TOTAL (mm)**	**90% (Months)**	**REMAINING (mm)**	
Underpass	Approx 91.0 to 92.0	TBC	TBC	TBC	TBC	To be confirmed at detailed design stage
Wing walls	Approx 91.0 to 92.0	TBC	TBC	TBC	TBC	
DIFFERENTIAL (mm):				TBC		

CHEMICAL ANALYSIS

Material	**ACEC Class**	**DS Class**	
All	AC-1	DS-1	Further details in Section 6 of the GIR

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

The only available ground investigation is situated approximately 70m to the northeast, therefore, some uncertainty remains regarding the founding conditions for the underpass. **Associated earthworks:** Embankment 5	
NOTES	

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Footbridge 1 – Geotechnical Summary Form

CHAINAGE INTERVAL:	Ch. 3+550m	TYPE:	Footbridge 1 (SBR-0355)	REFERENCES /COMMENTS
AIP REF No:	A40LVP-ARP-SBR-0355-RP-C-0001	DESIGN LIFE:	120 years	

RELEVANT EXPLORATORY HOLES	
TP04, TP05, BH04, BH05, BH06	
LOCATION-SPECIFIC GROUND PROFILE	

STRATA	DEPTH OF BASE (m bgl)	STRATA DESCRIPTION	
Weathered Mudstone	1.4 to 2.4	Typically brown and/or grey sandy **gravel** of mudstone. Also described locally as **sand** and **clay**.	Dip angles indicate cutting may be located near a anticline in the Slade and Redhill Formation bedding, with the bedrock dipping towards the north to the north of the cutting and to the south to the south of the cutting.
Mudstone	>18	Moderately weak becoming moderately strong with depth dark grey **mudstone** with very closely spaced smooth planar discontinuities at variable angle.	

PREVIOUS GROUND HISTORY	The proposed footbridge lies in an area of existing farmland.	
CONTAMINATION RISK ASSESSMENT	The construction and operation works are unlikely to pose a significant risk to the identified human and environmental receptors. No site remediation requirements have been identified at this stage. However, there is a risk of encountering unexpected contamination and therefore it is recommended that an action plan is developed.	
GROUNDWATER	A groundwater seepage was recorded at 1.8mbgl (125.8mAOD) in BH04, within mudstone of the Slade and Redhill Formation. No groundwater was detected during the subsequent groundwater monitoring at BH04 with the response zone situated between 3m and 18mbgl (124.6m to 109.6mOD).	

RECOMMENDED PARAMETERS

MATERIAL	**DENSITY** γ'	**ANGLE OF FRICTION** φ'	**COHESION** **c'**	**UNDRAINED SHEAR STRENGTH / UCS** c_u / σ_c	**YOUNGS MODULUS** **E'**	

Weathered Mudstone – Granular	20 kN/m^3	34°	0	-	30,000 kN/m^2	
Weathered Mudstone - Cohesive	20 kN/m^3	29°	0	75 kPa	19,000 kN/m^2	
Mudstone	22 kN/m^3	28°	60 kPa	10 MPa	150,000 kN/m^2	

SPREAD FOUNDATION DESIGN

STRUCTURE ELEMENT	**BASE (mOD)**	**FOUNDING STRATUM**	**FOOTING SIZE (m)**	**'ALLOWABLE' BEARING PRESSURE (kN/m^2)**
Northern abutment	TBC	TBC	TBC	TBC
Southern abutment	TBC	TBC	TBC	TBC

PRELIMINARY PILE DESIGN

PILE TYPE:	NA
CRITERIA FOR TOE:	NA
NEG. SKIN FRICTION:	NA

STRUCTURE ELEMENT	**TOE LEVEL (mOD)**	**FOUNDING STRATUM**	**LENGTH (m)**	**DIAMETER (m)**	**ULS DESIGN LOAD (kN)**
NA	NA	NA	NA	NA	NA

SETTLEMENT

STRUCTURE ELEMENT	**BASE (mOD)**	**IMMED'TE (mm)**	**TOTAL (mm)**	**90% (Months)**	**REMAINING (mm)**
Northern abutment	TBC	TBC	TBC	TBC	TBC
Southern abutment	TBC	TBC	TBC	TBC	TBC
DIFFERENTIAL (mm):				TBC	

CHEMICAL ANALYSIS

Material	**ACEC Class**	**DS Class**	
All	AC-1	DS-1	Further details in Section 6 of the GIR

OTHER DESIGN FEATURES *(e.g. foundation treatment/hazards)*

Details of any associated earthworks or retaining walls are yet to be determined.

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