High Speed 2 - 1MC06 - Stage One C2 - MWCC – North Portal of Chiltern Tunnels to Brackley

Aylesbury Embankment - GDR

1MC06-CEK-GT-REP-CS04_CL18-000037

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☐ LUL	☐ No Objection
☐ NRL	☐ Consent
☐ TFL	
☐ Utilities Company	
☐ Other (please specify)	

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Glossary of Terms

Acronym	Description
AA	Aylesbury Area
ACEC	Aggressive Chemical Environment for Concrete
AGS	Association of Geotechnical and Geoenvironmental Specialists
AIP	Approval In Principle
ASC	Arcadis, Setec and COWI
BGS	British Geological Survey
BRE SD	Building Research Establishment – Special Digest
CAT	Category (of check)
CBM / CBGM	Cement Bound (Granular) Material
CDM	Construction Design and Management
CEMP	Construction Environmental Management Plan
C2	Central Area 2
C3	Central Area 3
C23	Central Areas 2 and 3
CBR	California Bearing Ratio
Ch.	Chainage
CP	Cable Percussion borehole
CR	Cable Percussion borehole with Rotary Cored follow on
CSW	Continuous Surface Wave
CT / CPT	Cone Penetration Test
c'_{peak}	Peak Effective Cohesion
c'_{cv}	Critical Value Effective Cohesion
c_u	Undrained Shear Strength
c_v	Coefficient of Consolidation
DoU	Degree of Utilisation
DoWCoP	Development Industry Code of Practice
DS (Class)	Design Sulphate Class
DWS	Department of Water & Sanitation
DA1 C1	Eurocode 7 Design Approach 1, Combination 1
DA1 C2	Eurocode 7 Design Approach 1, Combination 2
DMP	Designers Monitoring Plan
DTM	Digital Terrain Model
EC7-1	Eurocode 7 – Part 1
EKFB	Eiffage, Kier, Ferrovial Construction, and BAM Nuttall
ENV	Environmental

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Acronym	Description
E, N	Easting and Northing
E_u	Elastic Modulus (undrained)
E_{v2}	Modulus of Deformation
EW	Earthworks
E&R	Excavate and Replace
GC	Geotechnical Category
GDR	Geotechnical Design Report
GI	Ground Investigation
GIR	Ground Investigation Report
GM	Ground Model
GRR	Geotechnical Risk Register
HS2	High Speed 2
IDR	Interdisciplinary Design Review
I&M	Instrumentation and Monitoring
KC	Kimmeridge Clay
LIDAR	Light Imaging Detection and Ranging
LL	Liquid Limit
LMA	Land Made Available
LOD	Limit of Land Deviation
LQ	Land Quality
L1	Level 1 (asset)
L2	Level 2 (asset)
m AOD	metres Above Ordnance Datum
MASW	Multiple Channel Analysis of Surface Wave
mbFL	Metres Below Formation Level
mbgl	Metres Below Ground Level
MMP	Materials Management Plan
m_v	Coefficient of Volume Compressibility
MWCC	Main Works Civils Contract
N	SPT 'N' blow count
ODF	Over Design Factor
OSGB	Ordnance Survey Great Britain
OXC	Oxford Clay Formation
PC	Pavement Core
PI	Plasticity Index
POST/POSA	Portland Stone/ Sand Formation

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Acronym	Description
PRoW	Public Right of Way
RC	Rotary Coring
RWAT	Rayleigh Wave Analysis Tool
Ru	Pore Pressure Ratio
SCEW	HS2 Specification for Civil Engineering Works
SLS	Serviceability Limit State
SMUGC	Strategy for Management of Uncertainty in Ground Conditions
SPT	Standard Penetration Test
SRR	Stakeholder Review Required
TFL	Transport for London
TP	Trial Pit (mechanically dug)
TPS	Total Potential Sulphate
UAID	Unique Asset Identification
UCS	Unconfined Compressive Strength
ULS	Ultimate Limit State
UTX	Under Track Crossing
UEF*	Enhanced Upper Embankment Fill
V_R	Rayleigh Wave Velocity
V_s	Shear Wave Velocity
v_u	Undrained Poisson's Ratio
v'	Drained Poisson's Ratio
WAVe	weighted average velocity
WI	Works Information
WS / WLS	Windowless Sampling
ZoI	Zone of Influence
γ_b	Bulk Density
φ'_{cv}	Critical Value Effective Angle of Friction
φ'_{peak}	Peak Effective Angle of Friction

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Executive Summary

This revision (C05) of the document 1MC06-CEK-GT-REP-CS04_CL18-000037 Aylesbury Embankment (AE) Geotechnical Design Report (GDR) is issued for the following Level 2 assets:

- IDR 207.1 - updated to incorporate SNC-8291 and SNC-8293:
  - HS2-00002C5SN - Aylesbury Embankment *(significant changes to GDR text have been made in the Executive Summary, Sections 2.1, 2.4.7.3, 2.4.8.2, 2.4.10, 2.4.11.3 [see Table 2-11] 2.4.12 and Appendix B [see Table B-2])*
- IDR 207.1 - updated to include:
  - HS2-00002N7VR - Aylesbury Embankment - Ancillary Culvert 061-AC3
- IDR 212.5
  - HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1
  - HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1

Sections 1, 9, 10, 11 and 12 present the common details of all Level 2 assets within the Aylesbury Embankment Level 1 Asset Group.

Sections 2, 6, 7, & 8 and Appendices B, C & I present the design associated with Aylesbury Embankment Level 2 mainline asset. As part of the update to incorporate SNC-8291 and SNC-8293, changes to the text have been made within the Sections listed above in italics. SNC-8293 requests the option to use Class 3A3 or Class 9M1 in place of Class 1A1 in Embankment Fill whereas, SNC-8291 requests the option to use Class 3A1 (subject to agreement by HS2 – GC-5602 sent by EKFB on 2 October 2024) or Class 3A3 in place of Class 1A1 in the Excavate & Replace (E&R). The SNCs have been raised to provide flexibility regarding the material option to be adopted based on the quality and quantity of suitable material available on site at the time of construction. The proposed changes have generally been accepted; no further calculations were deemed necessary due to the proposed materials possessing improved material strength properties for the Class 3A3 or Class 9M1 option in place of Class 1A1 for the Embankment Fill, as well as the Class 3A1 option in place of Class 1A1 for the E&R. However, restriction on the change has been provided at Sedrup North Culvert as to not impact the separate assurance process; a sloped gradient of 1V:2H has been provided to smoothly transition existing structural backfill and earthworks transition zone materials surrounding the culvert with Embankment Fill and E&R.

Section 4.9 presents the approach for the design of Ancillary Culvert 061-AC3.

Sections 4.10 and 4.11 present the foundation design related to all the ditch slab assets within Aylesbury Embankment Level 1 asset group.

Appendix A contains the GI plan and geological long section drawings for the mainline and landscaping earthworks.

Appendix D contains the Geotechnical Risk Register.

Appendix M contains the geotechnical calculations for Ancillary Culvert 061-AC3.

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This Geotechnical Design Report (GDR) presents the technical output of the Stage 2 Detailed Design for the Aylesbury Embankment Level 1 Asset Group, following the completion of Phase 1 Scheme Design.

Aylesbury Embankment is classified as a medium embankment (+6 to +12 m high) of 1V:2H slope gradients with a maximum height of 6.4m at Ch. 61+420. The embankment will be located between Ch. 60+500 and Ch. 61+763 between the adjacent HS2 earthworks assets Aylesbury North Cutting to the north and Aylesbury South Cutting to the south. Landscape bunds typically range from ground level to a maximum of 13.0m high at Ch. 61+320. Landscape bunds are proposed along the full length of the mainline embankment on the upline track. On the downline track, bunds exist except for locations between Ch. 61+190 and Ch. 61+580 due to the proposed attenuation ponds and other drainage infrastructure.

Due to the relocation of SBH-19 Overbridge, landscaping has been slightly adjusted and subsequently assured in separate packages with the creation of an additional two landscaping Level 2 asset IDs. However, landscape bund height and side slope angles are in line with original proposal and as such no further calculations have been undertaken.

Mainline and Landscaping Earthworks Design

The mainline and landscaping earthworks were designated as Class 2, which is defined as a Low Risk of potential change to design assumptions upon receipt of additional GI and/or field test data, in accordance with the Strategy for Management of Uncertainty in Ground Conditions (SMUGC), Ref. [1]. This designation was undertaken at the onset of Detailed Design, as recorded in the previous P01 version of this report. Subsequently, all outstanding GI data has been received for the design of the mainline and landscaping earthworks, and this data is included in this GDR to validate the design assumptions, which proceeded with the adoption of a reasonably cautious design approach.

The available Ground investigation (GI) data shows that the geology varies across the asset; therefore, five ground models have been used for the design calculations for the mainline embankment:

- Ground Model 1 (Ch 60+490 to Ch 61+000) consists of Head Deposits to 3.80 mbgl, Gault Clay Formation to 9.05 mbgl and Portland Stone/Sand Formation to 20+ mbgl.
- Ground Model 2 (Ch 61+000 to Ch 61+200) consists of Alluvium deposits to 1.60 mbgl, Gault Clay Formation to 5.30 mbgl, Whitechurch Formation to 6.30 mbgl, Purbeck Formation to 8.10 mbgl, Portland Stone/Sand Formation to 21.50 mbgl and Kimmeridge Clay Formation to 25+ mbgl.
- Ground Model 3 (Ch 61+200 to Ch 61+350) consists of Alluvium deposits to 2.0 mbgl, Purbeck Group to 4.50 mbgl and Portland Stone/Sand Formation to 20+ mbgl.
- Ground Model 4 (Ch 61+350 to Ch 61+500) consists of Alluvium Deposits to 1.20 mbgl, Soft Portland Stone/Sand Formation to 2.0 mbgl, Portland Stone/Sand Formation to 7.84 mbgl and Kimmeridge Clay Formation to 20+ mbgl.
- Ground Model 5 (Ch 61+500 to Ch 61+750) consists of Head deposits to 1.0 mbgl, Soft Portland Stone/Sand Formation to 2.0 mbgl, Portland Stone/Sand Formation to 7.40 mbgl and Kimmeridge Clay Formation to 20+ mbgl.

In addition to the above GI data, during site excavation activity carried out by EKFB from Ch. 61+200 to Ch. 61+763, soft/compressible materials at greater depths were identified, irrespective of the stratigraphy, due to the presence of existing watercourse. The activity at the site involved excavation underneath the footprint of the landscape bunds up to 3.5mbgl. The in situ shear strength ranged from 25 to 40 kPa at depths between 0.5 and 3.5 mbgl within this chainage extent. A conservative design parameter has been adopted to incorporate this low-strength material in the design of earthworks, which impacts Ground Models 3 to 5, as mentioned above.

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The following geohazards can potentially affect Aylesbury Embankment Level 2 asset:

Soft/Compressible Ground, High/Shallow Groundwater, Chemically Aggressive Ground, Faulting, Relic Shear Surfaces, Hard Ground/Flint Bands, Rayleigh Waves, Shrink/Swell Potential and Loose/Unstable Ground. After the assessment presented in this GDR and the proposed mitigations, the risks associated with these geohazards are mitigated to moderate or low.

Polished surfaces were identified in two locations within the Gault Clay Formation, recorded at depths between 8.10-11.00 mbgl and 14.23-16.65 mbgl. Polished surfaces were also encountered within the Kimmeridge Clay Formation between depths 19.40-20.00 mbgl. It is considered that surface activities (such as agricultural ploughing) have potentially masked further evidence of periglacial and relict landslip activities. Embankment filling and rail operations may activate or reactivate relict shear surfaces, leading to potentially significant ground displacements. It has not been possible to design out the risk of relict shear surfaces, which will require further management during the construction phase. Mitigation measures, including typical foundation treatment (excavate and replace) to provide a stabilising blanket, can be adopted where appropriate (observational method).

The Detailed Design of the earthworks, with respect to slope instability, settlement analysis, transition to culverts, subgrade requirements and dynamic performance mitigation (including Alluvium treatment), are described in detail in the proceeding sections of this report. The Mainline Embankment Fill will compromise of Class 1A1 (or Class 3A3 or Class 9M1) and the Excavate and Replace mitigation will comprise of Class 1A1 (or Class 3A1 or Class 3A3 Fill). From Ch. 60+500 to 61+763, the internal makeup of the landscape bund zone facing the track and the core (Zone 1 and Zone 2, respectively) will constitute of Class 2A3 and Class 2A4 material, respectively, whereas Class 4A1 material is suitable for the outer bund slope (Zone 3). The Excavate and Replace mitigation within the Alluvium extent (Ch. 61+200 to Ch. 61+763) below the landscape bund requires a formation treatment to a depth where a minimum undrained shear strength of 50 kPa has been attained on-site and is representative of the underlying material. The Replacement is done using Class 2A1 Fill material upon which bunds are to be constructed.

A non-contestable high-pressure Scotia Gas Networks (SGN) gas main (CSO-02-2202) crosses beneath the mainline at approximate Ch. 61+650. The loadings, geometry and the predicted settlement as a result of the proposed HS2 works were provided to SGN for assessment in terms of the potential impact to the gas main. Consequently, SGN have confirmed that the bending tolerances and settlements are within the acceptance criteria for the gas main and so the construction works can proceed. Further details are provided in Section 2.4.3.

The Series 600 earthwork specification appendices, together with the drawing references, are provided in the IDR Package 207.1 Specification, referenced in Section 10 of this GDR.

Maintenance Access Road to Pond (South Arm off Calley Farm – South) Design

The offline highway earthworks covered in this revision of the GDR within Section 3 include the following:

- Maintenance Access Road to Pond (South Arm off Calley Farm – South), a maintenance access road which will provide access to the drainage ponds within Aylesbury Embankment. The embankments are a maximum of 2.5 high at a maximum gradient of 1V:3H. Strengthened side slopes are required above 1.5m in height from Ch.0+000 to 0+500 and above 0.5 from Ch. 0+500 to 0+600 and will be formed using geogrid reinforcement.

Based on Data Drop 25, the available Ground Investigation (GI) data indicated the presence of changing ground conditions along the alignment. For this reason, two ground models have been set up. The first model indicates Portland Group overlaying Kimmeridge Clay. The second model indicates a thin layer of Alluvium and then Purbeck Group overlaying Portland Group. Groundwater level was assumed at 0.1m below existing ground level for the Detailed Design of this earthwork for the access track.

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The key Geotechnical Risks associated with these assets are:

- Soft Ground
- High Groundwater
- Reuse of recovered material

The management and mitigation of these risks are described in Section 3 of this GDR.

Oat Close SBH/19 Realignment

The non-mainline highway earthworks covered in Section 3 of the GDR include:

- Oat Close SBH/19 Realignment, which forms the approach embankments to the Oat Close SBH/19 Overbridge at Ch 60+920. The embankments are a maximum of 10.35m high and formed of geogrid reinforced side slopes at a maximum gradient of 1V:2.5H.

The available Ground Investigation (GI) data indicated the presence of Head overlying the Gault Clay Formation, Whitchurch Sand Formation, Purbeck Group, Portland Stone/Portland Sand Formations, and the Kimmeridge Clay Formation encountered at depth over the footprint of the asset.

Groundwater level was assumed at the existing ground level for the detailed design of these earthworks.

The key geohazards associated with these assets are the presence of soft ground, high groundwater, made ground / topsoil material, contaminated ground, chemically aggressive ground, shrink / swell potential, relict periglacial features, variable weathering profile, and geological faulting.

Bridleway SBH/19 Overbridge

The proposed Bridleway SBH/19 Overbridge (UAID HS2-000001293) is a Level 2 asset contained within the Level 1 asset Aylesbury Embankment (HS2-00002C5SN – Snake Grid chainage 60+500 to 61+763) within Section C2 (Aylesbury Area).

The proposed Bridleway SBH/19 will be located on the outskirts of Aylesbury in Buckinghamshire, carrying Public Rights of Way (PRoW): Bridleway SBH/19/7. The bridge provides a route over the HS2 mainline, with new approach embankments (Aylesbury Embankment) on both the east and west sides of the crossing. The terrain is a flat section of valley bottom surrounded by open fields, with the hamlet of Bishopstone to the south and the town of Aylesbury to the north. The proposed Bridleway SBH/19 Overbridge structure is a simply supported through bridge. The bridge's single span measures 42m between bearing centrelines. The deck is supported by warren trusses along its two longitudinal edges. The superstructure is perpendicular to its abutments. The bankseat abutments are constructed from reinforced concrete.

Bridleway SBH/19 Overbridge was originally located at main Snake Grid chainage 060+544 within the Aylesbury Embankment Ground Level 1 asset. The location of Bridleway SBH/19 Overbridge was later relocated 386 metres north of its original position. The new position for Bridleway SBH/19 Overbridge is Snake Grid chainage 060+930. However, the Bridleway SBH/19 Overbridge remains within the Aylesbury Embankment Ground Level 1 asset. Due to the late relocation of this overbridge, no Stage II Ground Investigation (GI) at Bridleway SBH/19 Overbridge was available at the time of carrying out the design of the overbridge. Therefore, GI from the HS2 Main GI 2016 and Stage II GI (at the vicinity of the asset) was used for the detailed design. Stage II GI has been scheduled but the GI data is not available at this stage. Section 4.6 of the GDR presents the design and outputs, taking into account the available information which includes supporting geotechnical data and ground investigation (GI) data. This section of the GDR focuses on the key performance considerations for the design of Bridleway SBH/19 Overbridge and defines the geometry and the construction measures together with the risks and opportunities associated with the earthworks and geotechnical design.

<u>Ditch slab foundation design (For IDR 212.5)</u>

Aylesbury Embankment Pedestrian Ditch Slab No.1

Aylesbury Embankment Pedestrian Ditch Slab No.1 (UAID: HS2-00002YD79) is a ditch slab with a deck width of 2.0m, clear span of 6.0m and a foundation of 1m deep and 1m wide. The ditch slab is part of drainage

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catchment covered by IDR 207.2 and is anticipated to be founded on the Superficial Deposits (comprised of Cohesive Head/Alluvium Deposits) with a minimum undrained shear strength of 40kPa.

The BRE SD1 assessment results in a design sulphate and ACEC classification of DS-4 and AC-4.

The proposed Aylesbury Embankment Vehicular Ditch Slab No.1

The proposed Aylesbury Embankment Vehicular Ditch Slab No.1 (UAID: HS2-00002YD7F) is a ditch slab with a deck width of 4.0m, clear span of 5.4m and a foundation of 1m deep and 1m wide and 0.5m thick Class 6N6 E&R beneath the foundations. The ditch slab is part of drainage catchment covered by IDR 207.1 and is anticipated to be founded on the Superficial Deposits (comprised of Cohesive Head/Alluvium Deposits) with a minimum undrained shear strength of 40kPa. The BRE SD1 assessment results in a design sulphate and ACEC classification of DS-4 and AC-4.

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1 Introduction

1.1 Background

In January 2012, the UK Government announced its intention to proceed with a new high-speed railway line (HS2) and announced the preferred line of the route from London to the West Midlands as the first phase (Phase 1) of a national high-speed rail network which is illustrated in Figure 1-1. Phase 1 will involve the construction of a new railway line of approximately 230 kilometres (km) between London and the West Midlands, with stations at Euston, Old Oak Common, Birmingham Interchange and Birmingham Curzon Street.

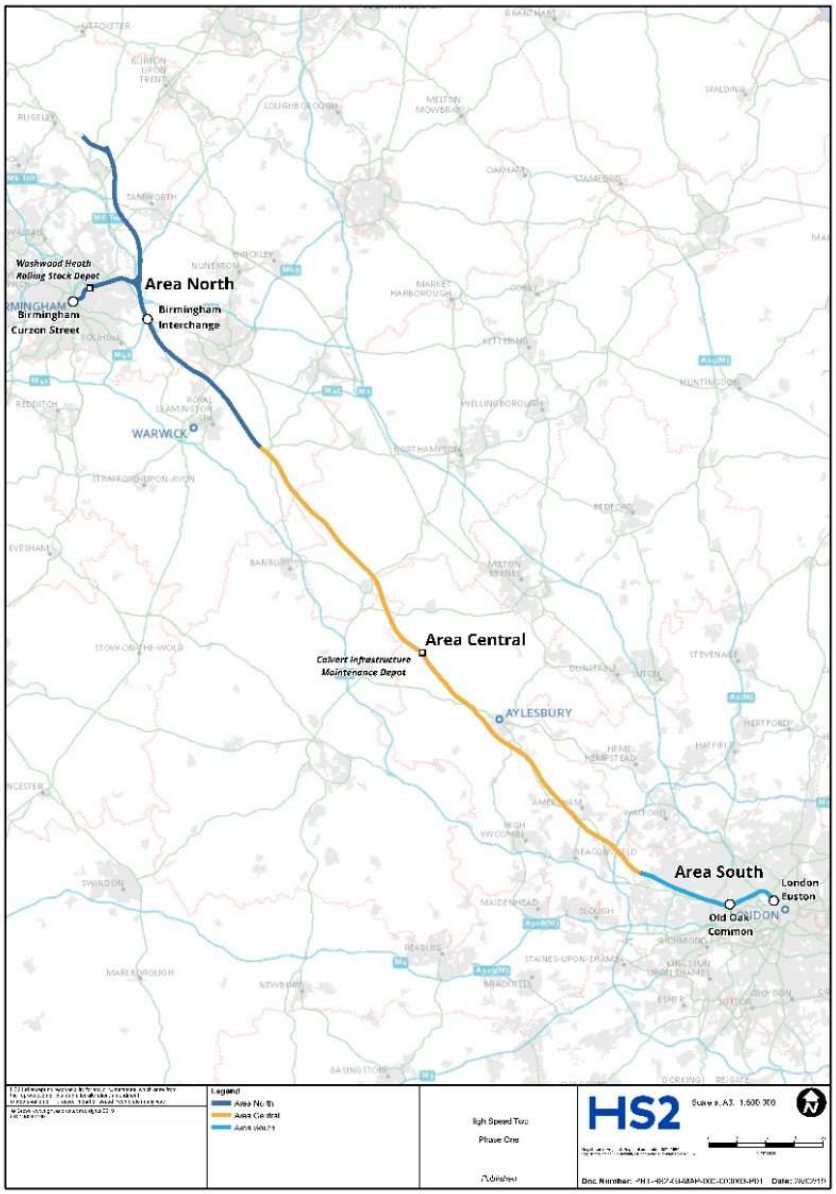


Figure 1-1: HS2 Phase 1 Route

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For design and construction purposes, the overall route has been divided into three Delivery Areas as illustrated in Figure 1-1 and Table 1-1.

Table 1-1: HS2 Delivery Areas

Area	Comment
Southern	The former London Metropolitan and London Euston Station
Central	The former Country South, with part of Country North
Northern	The remainder of Country North and Birmingham Metropolitan

The delivery areas have been further subdivided into Contracts, each of which represents a Main Works Civils Contract (MWCC). The Arcadis, Setec, COWI Design Joint Venture (ASC) has been appointed by Eiffage, Kier, Ferrovial, and BAM Nutall Joint Venture (EKFB) to provide detailed design services for the civil engineering assets within HS2 Contract C23 (formally Central Area 2 and Central Area 3), which have been further subdivided into the five delivery packages with the approximate extents provided in Table 1-2.

Table 1-2: Approximate Extents of HS2 Contract C23 Delivery Packages

Delivery Package Title	Chainage Extents (Snake Grid)
North Chilterns Area (NCA)	47+750 to 57+850
Aylesbury Area (AA)	57+850 to 71+850
Calvert Area (CA)	71+850 to 81+450
Twyford to Greatworth (T2G)	81+450 to 101+620
Greatworth to Southam (G2S)	101+620 to 126+540

1.2 Scope and Objective of the Report

This Geotechnical Design Report (GDR), as part of the geotechnical design process, is prepared to fulfil the requirements stipulated in the HS2 MWCC Works Information WI 300 Contractor's, Ref. [2], and the HS2 Earthworks Technical Standard, Ref. [3].

This report meets the requirements of a GDR as set out in BS EN 1997-1 (referred to in this report as EC7-1), Ref.[4]. The content of this GDR has been prepared to follow the general guidance as set out in CD622, Ref. [5], for Managing Geotechnical Risk (formerly HD22/08). In addition to the typical GDR requirements, this document also details how the HS2 standards are implemented for the assets covered by this report.

Where relevant, cross-references the Geo-environmental and Remediation Strategy Reports as required by HS2 Technical Standard – Land Quality, Ref. [6].

This GDR presents the geotechnical design output of the Stage 2 detailed design for the permanent works for Aylesbury Embankment Level 1 (L1) Asset Group and its associated civil engineering structures, following the Stage 1 Scheme Design as presented in the Stage 1 GDR Ref. [69]. It includes the Detailed Design of the mainline earthworks, landscaping bunds, ancillary structures, transition to the Sedrup North Culvert and earthworks associated with the Sedrup North Watercourse Diversion. The transition details between the structure and the mainline earthworks are also covered in this report, as are the carriageway/access roads and other various Level 2 (L2) assets that require geotechnical design input. The extent of the Level 2 (L2) assets is illustrated in Figure 1-2.

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A summary of the existing information, findings of the field studies, interpretation of the factual data from the Desk Study, the available Ground Investigation (GI) factual reports and associated AGS files are provided in the Supplementary GIR, Ref [7]. The Supplementary GIR includes GI information received up to a data-freeze date of March 2023 (Data Drop 30). Level 2 asset-specific sections of this report should be reviewed for a commentary on the incorporation of the 'Outstanding GI' information in the Detailed Design.

1.3 Description of the Works

1.3.1 Site Description

The Aylesbury Embankment is located between Ch. 60+500 and Ch. 61+763 (Snake Grid) within the Aylesbury Area (AA) of Section C2, centred at an approximate National Grid Reference of E 480863, N 211565 (OSGB). The assets to the north and south will be Aylesbury North Cutting (HS2-00002C5SP) and Aylesbury South Cutting (HS2-00002C5SM), respectively.

The embankment asset lies approximately 500m to the west of Aylesbury. The topography of the asset area generally rises towards the southeast. The existing ground at the asset increases in height at both ends of the asset where there are transitions with Aylesbury South Cutting at the southern end and Aylesbury North Cutting at the northern end. An existing watercourse crosses the proposed alignment at approximately Ch. 61+350 to Ch. 61+500. The existing watercourse will be diverted and culverted below the mainline embankment and landscape bunds at Ch. 61+330 via the proposed Sedrup North Culvert. A site location plan is presented in Figure 1-2.

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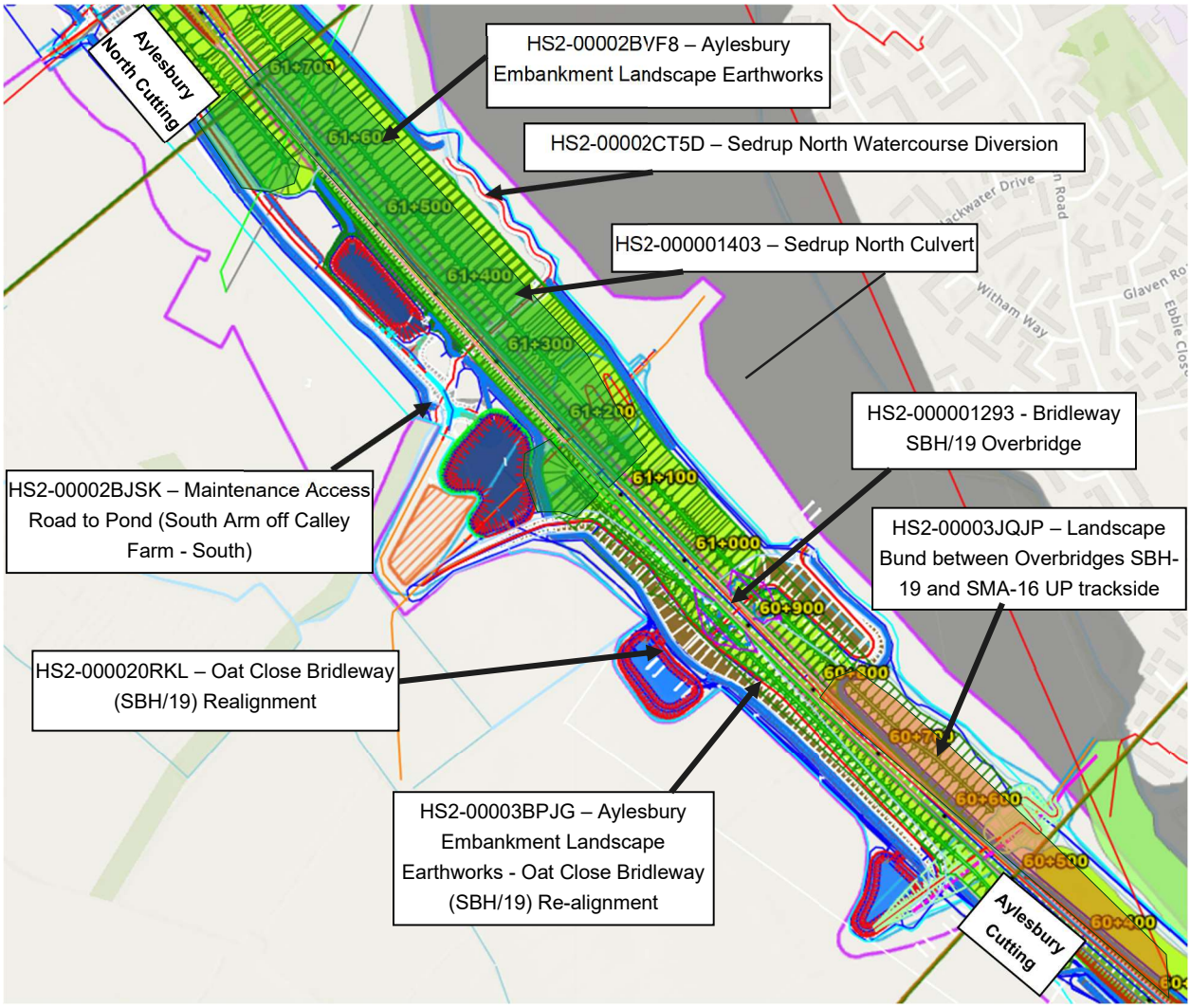


Figure 1-2: Aylesbury Embankment Location Plan

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1.3.2 Asset Description

The HS2 track alignment within Aylesbury Embankment is set above the existing ground level via the mainline embankment with an upline (to London) and a downline (to Birmingham). The vertical alignment rises at a uniform gentle grade from south to north until approximately Ch.61+200, then falls from south to north. There are no switches and crossings within the asset.

Landscape bunds are proposed on both the upline and downline sides of the mainline. On the downline side there is a gap in the landscape earthworks between Ch. 61+190 and Ch. 61+580 to accommodate the proposed attenuation ponds and other drainage infrastructure.

The location and details of the Aylesbury Embankment asset to which this GDR relates are summarised in Table 1-3.

Table 1-3: Mainline Asset Details

Asset Description	Detail
Contract/section/lot	C2/CS04/CL18
Name of design element	060-L1 Aylesbury Embankment
Mainline Embankment HS2 Asset ID	HS2-000001027
Type	Embankment
Start - End chainage (Approximate)	Ch. 60+500 – Ch. 61+763 (Snake grid)
Length	1,263m
Maximum Height of Mainline Embankment	6.4m (at Ch. 61+420)
Mainline Embankment Slope Angle (Detailed Design)	1V:2H
Mainline Embankment Slope Angle (Scheme Design)	1V:2H
Landscape bunds extents	Upline chainages: across the full length of the Asset, *from* Ch. 60+500 to 61+763 Downline chainages: between approx. Ch. 60+500 and 61+190 and Ch. 61+580 to 61+763 (the 390m gap is to accommodate attenuation ponds and other drainage infrastructure).
Maximum height of Landscape Bunds	13m (Upline landscaping at Ch. 61+320)
Maximum Landscape Earthworks Slope Angle	1V:4H
HS2 Earthwork Category	Geotechnical Category (GC) 3, Height <12m - CAT II [(1)] (Technical Standard – Earthworks, Ref. [3])

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Asset Description	Detail
Discontinuity in the mainline embankment	Sedrup North Culvert at Ch. 61+330
Long / Short Earthwork	Ch. 60+500 to 61+330: Long earthwork Ch. 61+330 to 61+763: Short earthwork

Note:
(1) The earthworks transitions to the Sedrup North culvert are reviewed by the CAT III checkers.

The Level 2 assets located adjacent to the mainline assets are referred to as non-mainline assets. The non-mainline assets that are covered by this GDR and require a geotechnical input are identified in Table 1-4.

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Table 1-4: Level 2 non-mainline earthworks assets

Level 2 Non-mainline Asset ID	Level 2 Non-mainline Asset Name	Asset Type	Approximate Chainage	Comments
HS2-00002BVF8	Aylesbury Embankment Landscape Earthworks	Mitigation Earthworks	Ch. 61+100 to Ch. 61+763 – UP and DOWN	Asset is part of IDR 207.1. Assessment is included in Section 2 of this GDR.
HS2-00003JQJP	Landscape Bund between Overbridges SBH-19 and SMA-16 UP trackside	Mitigation Earthworks	Ch. 60+500 to Ch. 60+780 - UP	Asset is part of IDR 208.4. Assessment is included in Section 2 of this GDR.
S2-00003BPJG	Aylesbury Embankment Landscape Earthworks - Oat Close Bridleway (SBH/19) Re-alignment	Landscaping	Ch. 60+780 to Ch. 61+100 - UP Ch. 60+500 to Ch. 61+100 - DOWN	Asset is part of IDR 207.2. Assessment is included in Section 2 of this GDR.
HS2-00002CT5D	Sedrup North Watercourse Diversion	Main River	Ch. 61+300 to Ch. 61+500	Asset is part of IDR 207.1. Geotechnical assessment is included in Section 3.6 of this GDR.
HS2- 00002CT57	C2.T.23 Catchment	Piped Drain	Ch. 60+975 to Ch. 61+625	Asset is part of IDR 207.1. Details are provided in Section 6 of this GDR.
HS2-00002N7VR	Ancillary Culvert 061-AC3	Culvert	Ch. 61+300	Asset is part of IDR212.5. Details are provided in Section 4.9 of this GDR.
HS2-000001403	Sedrup North Culvert	Culvert	Ch. 61+330	Asset is part of IDR 207.1. Details are provided in Section 4 of this GDR.
HS2-00002BVF6	C2.L.26 Catchment	Open Drain	Ch. 60+400 to Ch. 62+100	Asset is part of IDR 207.1. Details are provided in Section 6 of this GDR.
HS2-00002BJSK	Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Carriageway	Ch. 61+300 to Ch. 61+875	Asset is part of IDR 208.4. Details are provided in Section 3 of this GDR.

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Level 2 Non-mainline Asset ID	Level 2 Non-mainline Asset Name	Asset Type	Approximate Chainage	Comments
HS2-00002N7VP	Ancillary Culvert 060-AC4	Culvert	Ch. 60+510 to Ch. 60+580	Assessment is included in Section 4.7 of this GDR.
HS2-00002N7VQ	Aylesbury Embankment - Ancillary Culvert 061-AC1	Culvert	Ch. 61+170 to Ch. 61+190	Assessment is included in Section 4.8 of this GDR.
HS2-000001293	Bridleway SBH/19 Overbridge	Footbridge	Ch. 60+920	Assessment is included in Section 4.6 of this GDR.
HS2-00002CW1B	Oat Close Bridleway (SBH/19) Drainage System	Open Drain	Ch. 60+720 to Ch. 61+100	Assessment is included in Section 4.6 of this GDR.
HS2-000020RKL	Oat Close Bridleway (SBH/19) Realignment	Public Right of Way (PRoW)	Ch. 60+720 to Ch. 61+100	Assessment is included in Section 3 of this GDR.
HS2-000032LLR	Stoke Brook Enhancements	Watercourse	-	Asset is located within Aylesbury South Cutting L1 Group. Explicit geotechnical design is not required as set out in the Watercourse Standard (Doc No. HS2-HS2-EV-STD-000-000014); the asset is offset
HS2-00002YD79	Aylesbury Embankment Pedestrian Ditch Slab No.1	Ditch Slab Crossing	Ch. 61+325	Asset is part of IDR 212.5. Assessment is included in Section 4.10 of this GDR.
HS2-00002YD7F	Aylesbury Embankment Vehicular Ditch Slab No.1	Ditch Slab Crossing	Ch. 61+528	Asset is part of IDR 212.5. Assessment is included in Section 1.1 of this GDR.

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1.4 Geotechnical Category of the Works

The Geotechnical Category (GC) for HS2 earthwork cases (EW1 to EW9), and the category of check required (CAT I to III) are presented in Table 2.2/2 of the Technical Standard – Earthworks, Ref.[3]. The categories of checking associated with the various L2 assets are shown in Table 1-5.

Table 1-5: Geotechnical Category of Level 2 Earthwork Assets

Level 2 Asset ID	Level 2 Asset Name	Asset Type	Location	HS2 Earthworks Case (1)	Geotechnical Category (2)
HS2-000001027	Aylesbury Embankment	Mainline Embankment	HS2 Mainline	EW1	GC3, CAT II
		Mainline transition to Sedrup North Culvert (4)			GC3, CAT III
HS2-00002BVF8	Aylesbury Embankment Landscape Earthworks	Landscape Earthworks	Offline	EW8 (5)	GC2, CAT II
HS2-00003JQJP	Landscape Bund between Overbridges SBH-19 and SMA-16 UP trackside	Landscape Earthworks	Offline	EW8 (5)	GC2, CAT II
HS2-00003BPJG	Aylesbury Embankment Landscape Earthworks - Oat Close Bridleway (SBH/19) Re-alignment	Landscape Earthworks	Offline	EW8 (5)	GC2, CAT II
HS2-00002CT5D	Sedrup North Watercourse Diversion	Main River	Offline	EW6	GC2, CAT II
HS2-00002BJSK	Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Carriageway	Offline	EW6	GC2, CAT I
HS2-000020RKL	Oat Close Bridleway (SBH/19) Realignment	Public Right of Way (PRoW)	Offline	EW6	GC2, CAT II
HS2-00002YD79	Aylesbury Embankment Pedestrian Ditch Slab No.1	Ditch Slab	Offline	-	GC1, CAT I
HS2-00002YD7F	Aylesbury Embankment Vehicular Ditch Slab No.1	Ditch Slab	Offline	-	GC1, CAT I

Notes:

(1) The HS2 Earthworks Case has been assessed using the definitions provided in the HS2 Technical Standard - Earthworks Ref. [3].
(2) The Geotechnical Category has been assessed in accordance with BS EN 1997 Ref. [4] but no lower than the minimum category defined by Table 2.2/2 from the HS2 Technical Standard – Earthworks Ref. [3]

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(3) The assets highlighted in grey are part of a later IDR submission.
(4) The earthworks transitions to the Sedrup North Culvert are subject to CAT III as per checking requirements for the structures.
(5) The Earthworks Case has been assigned such that the geotechnical category of check is in line with the mainline embankment requirements.

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1.5 Uncertainty in Ground Conditions Classification

The availability of supplementary GI information for the Detailed Design has been limited due to a number of constraints (access, permits, etc.), and the COVID 19 pandemic exacerbated this problem. Much of the additional GI information requested for the Detailed Design was delayed. This was the case for the Aylesbury Embankment mainline and landscaping earthworks, which proceeded to Detailed Design Review in December 2021 with outstanding GI data. This outstanding GI data has now been received (March 2023, Data Drop 30), and this report therefore presents the Detailed Design based on all the available applicable data for the mainline earthworks and landscaping. Please see individual sections for details relating to Data Drops for each Level 2 asset submission.

To minimise the impact on the project programme, a Strategy for Management of Uncertainty in Ground Conditions (SMUGC), Ref. [1] has been prepared by ASC and EKFB for implementation across the C23 contract areas, which details the risk management process.

The fundamental steps in the design process that support the management of uncertainty in ground conditions are set out below:

- Initial assessment of uncertainty in ground condition classification (SMUGC Class) for each L1 and L2 asset;
- Meeting between ASC and EKFB to clearly communicate ground risks and to agree on an asset-specific mitigation strategy, which may be one of the following:
    - Obtain additional GI data and/or undertake field trials.
    - Design to reduce risk to an acceptable level.
    - Validate/manage the risk through construction observations and inspections.
- Implementation of the mitigation strategy;
- Carry out updates to the ground model and GRR on receipt of new data;
- Update the SMUGC Class, if required;
- Carry out Detailed Design and mitigation of the identified hazards, including reporting and approval.

The ground conditions uncertainty for this asset was designated as Class 2 – Low Risk of potential change to design assumptions upon receipt of additional GI data and/or field trials test data (categorised at the start of Detailed Design)

The uncertainty classification was based on the following:

- The presence of soft/compressible soils has been reported in an excavation beneath the footprint of the Landscape bunds on the downline of the HS2 alignment;
- The presence of faulting has been identified near Sedrup North Culvert;
- Presence of unsuitable materials at the formation level with respect to dynamic performance;
- Insufficient GI coverage along parts of the asset to support the Detailed Design.

Detailed Design was undertaken on the adoption of reasonably cautious design assumptions (Option 2). Subsequently, all Stage 2 GI information has been received and is included in this revision of the GDR. There has been no change in the ‘reasonably cautious design’ approach.

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1.6 Limitations

Detailed walkover surveys, geomorphological mapping and inspections are necessary during construction to confirm that the observations made associated with previous physical and virtual walkovers are valid and therefore still relevant to the design. The lack of on-site observations significantly increases the risks associated with the identification and mitigation of geomorphological hazards of the type discussed in this report. Geomorphological risk assessments have been undertaken based on a review of the existing data available at the Scheme Design stage and as presented in the Supplementary GIR, Ref.[7].

The GRR was updated during the Stage 2 Detailed Design to reflect the full or partial mitigation of the geotechnical risks. Details can be found in Section 9 and Appendix D.

A review of historical land uses and published geological, hydrological, and hydrogeological information has been undertaken by third parties, including the records of regulatory and statutory bodies. ASC does not warrant, nor does it accept any responsibility or liability, for the accuracy or completeness of the content or for any loss that may arise from reliance on information provided in any third-party information on which this report is based.

Ground investigation by nature only reveals a small percentage of the ground present beneath the site, and the possibility of significant variation in ground conditions existing between sampling locations cannot be discounted. Additionally, groundwater conditions are based on observations made at the time of the investigation and during subsequent monitoring visits and may be subject to significant variation due to atmospheric, seasonal, or other effects.

The information contained in this report is intended for the use of EKFB, HS2 and their consultant partners. ASC can take no responsibility for the use of this information by any third-party for uses other than described in this report.

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2 Mainline Earthworks

2.1 Introduction

This section of the report describes the approach adopted for the design of Mainline Earthworks, including asset-specific information, and provides a summary of the design solution for the mainline assets. Given the interrelated nature of the landscape and mainline embankments, slope stability and settlement calculations for landscape and mainline earthworks elements will be considered together in the mainline earthworks section of this report.

SNC-8293 requests the option to use Class 3A3 or Class 9M1 in place of Class 1A1 in the Embankment Fill, whereas, SNC-8291 requests the option to use Class 3A1 (subject to agreement by HS2 – GC-5602 sent by EKFB on 2 October 2024) or Class 3A3 in place of Class 1A1 for the Excavate and Replace (E&R). The SNCs have been raised to provide flexibility regarding the material option to be adopted based on the quality and quantity of suitable material available on site at the time of construction. The proposed changes have generally been accepted; no further calculations were deemed necessary due to the proposed materials possessing improved material strength properties for the Class 3A3 or Class 9M1 option in place of Class 1A1 in the Embankment Fill, as well as the Class 3A1 option in place of Class 1A1 for the E&R. However, restriction on the change has been provided at Sedrup North Culvert as to not impact the separate assurance process; a sloped gradient of 1V:2H has been provided to smoothly transition existing structural backfill and earthworks transition zone materials surrounding the culvert with Embankment Fill and E&R; please see Section 2.4.10 for further details. SNC-8291 and SNC-8293 have been accepted between the following chainages:

- Ch. 60+500 (start of mainline asset) to Ch. 61+301 (south of Sedrup North Culvert)
- Ch. 61+359 (north of Sedrup North Culvert) to Ch. 61+763 (end of mainline asset)

2.2 General Definitions

The terms ‘long’ and ‘short’ earthworks are referred to in this report. Respectively, these refer to an earthworks structure that is greater than and less than 800 m in length. The separation of these earthwork structures is defined by discontinuities. A discontinuity refers to a hard point in an earthwork, such as an underbridge, shallow culvert, switches and crossings, a significant fault, a cut-fill transition, or another feature identified by the designer.

In the context of this asset, the transition into the neighbouring asset groups constitutes a discontinuity, as well as the Sedrup North Culvert.

The landscape bunds are generally adjacent to the mainline earthworks across the asset. The landscape bunds transition from just above ground level to a maximum height of 13.0m.

2.3 General Design Approach

The design undertaken in this report satisfies the requirements of the Technical Standard for Earthworks [3] and the Design Basis Statements listed below:

1. Ground Engineering Design Basis Statement: Earthworks Ref. [8]
2. Ground Engineering Design Basis Statement: Seismic Design Ref. [9]
3. Ground Engineering Technical Note: Guidance on Surcharge Loading Ref. [10]
4. Ground Engineering Technical Note: Guidance on the Assessment of Settlement Ref. [11]
5. Ground Engineering Technical Note: Guidance on the Assessment of the Dynamic Performance of Earthwork Structures Ref. [12]

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2.4 Aylesbury Embankment

2.4.1 Asset Description

The proposed layout of the Aylesbury Embankment is detailed in Section 1.3 and shown in Figure 1-2.

Following Clause 2.3.11 of the Technical Standard, Ref.[3], the Sedrup North Culvert (Ch. 61+330) shall be considered as a discontinuity below the mainline embankment. Therefore, the section of the mainline embankment north to the culvert is a 'short' earthwork (less than 800 m long), and south to the culvert is a 'long' earthwork (greater than 800 m long).

A non-contestable high-pressure SGN gas main has been diverted deeper below the mainline embankment through a UTX at approximately Ch. 61+650.

2.4.2 Construction Method and Sequence

Construction of other Level 2 assets, such as the culverts and overbridges, will have significant implications on the earthworks design. The design of the embankment and landscape bunds has been undertaken based on the general construction programme provided by EKFB.

The design has been undertaken based on the following construction sequence assumptions:

- Carry out ground mitigation works (e.g., Excavate, Replace)
- Excavation for culverts, culvert construction and placement of earthwork minor transition zone material will begin, followed by placing Landscape Bund fill and then Mainline Embankment fill. These activities will be carried out simultaneously.
- Localised excavation to enable the construction of geo-structures and utilities (bridges, UTX); and
- Holding periods begin after placement of fill up to formation level and full height of landscape bunds, and track slabs are not to be placed during holding periods.

The embankment surface will be surcharged with fill to impose a vertical stress of 15kPa (equivalent to approximately 1m of soil surcharge) to represent long-term dead loads from the track slab and will be present for the duration of the Compliance period. The surcharge material will be designed by the Contractor and may consist of Class 4 or Class U1 materials. Consequently, it may be problematic for EKFB to undertake works on the surface, such as drainage, due to the potential for contamination from fines associated with the surcharge. The Contractor will note that the wait period is at least two months, but delaying further work on the embankment surface may be prudent until the surcharge is removed.

2.4.3 3rd Party Asset Constraints on Design

A non-contestable utility diversion (CSO-02-2202) is located at approximate Ch. 61+650. The stainless-steel high-pressure gas main crosses the mainline approximately 8.5 m below the proposed ground level (4.8m below the existing ground level). The pipeline is approximately 550m long and has a nominal diameter of 600mm Ref. [13].

Figure 2-1 shows a long section of the pipeline, which is described in the report HS2 Aylesbury Pipeline Diversion Design Report Ref.[14], written by Saith Ltd, and commissioned by the asset owner, Scotia Gas Networks (SGN).

The geometry of the section provided to the utility designer by ASC (Figure 2-1) is based on a scheme design geometry Ref. [15], which is very similar to the sections available at Ch. 61+640 in the detailed design geometry, shown in Table 2-1.

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Table 2-1: Earthworks geometry at the UTX Crossing in Aylesbury Embankment

Chainage	Height (m)		
	Landscape Bund[1]	Mainline Embankment	Landscape Bund[1]
Ch. 61+640 (detailed design section)	8.4	3.8	11.0
Ch. 61+640 (Saith Ltd. Drawing No. 0920-AYLE-ME-P-001, Rev 05)	8.6	3.8	10.6

Notes:

1) Landscape bunds are referred to from left to right as shown on Figure 2-1.

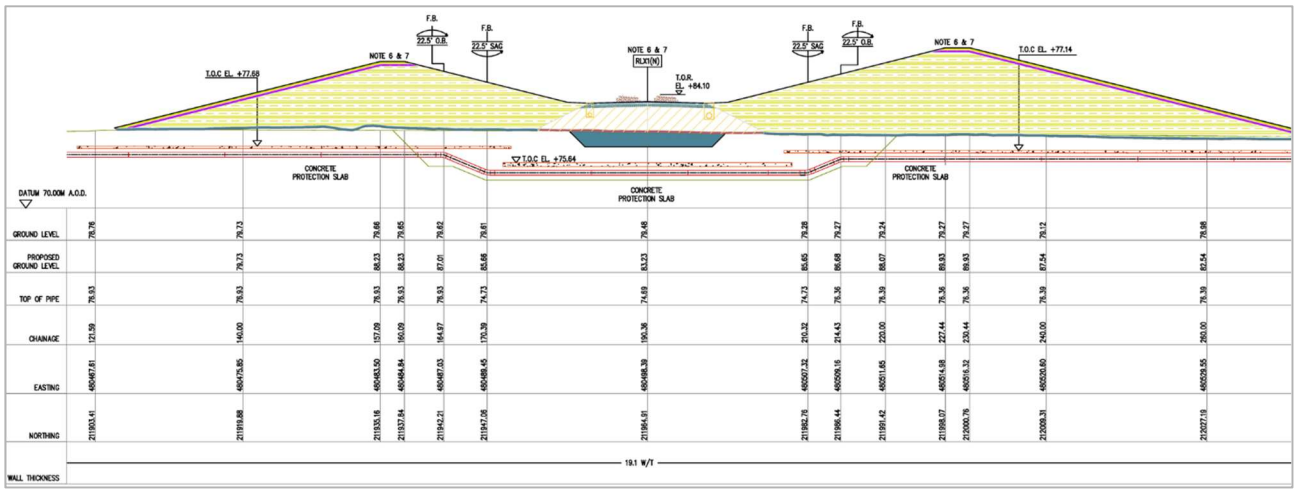


Figure 2-1: Non-Contestable Utilities Diversion Section

Design and installation of the diversion and decommissioning of the existing pipeline were undertaken by SGN in 2021, and the redundant pipes were removed. After reviewing the as-built information, ASC has provided loading and settlement estimates along the pipeline to SGN and their Principal Designer.

SGN and their Principal Designer confirmed that the loads from the proposed HS2 works, including the landscaping bunds have been considered in the design of the utility diversion and that any constraints specified by them would not influence the Aylesbury Embankment landscape design. TQ-3989 was raised in this regard.

The mainline design, undertaken by ASC estimates that the long-term settlement at the gas main is up to 125mm.

SGN has confirmed that the bending tolerances and settlements induced by the proposed HS2 works are within the acceptance criteria for the gas main. EKFB shall manage construction and ensure all works are undertaken in accordance with SGN procedures.

Monitoring shall be undertaken to ensure settlement of the proposed works is in accordance with the design of the SGN pipeline. Stockpiling of material shall not be permitted within the vicinity of the gas main,

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between Ch.61+575 and 61+725. EKFB shall undertake all construction works in line with SGN requirements and all works shall be monitored, in line with the DMP.

2.4.4 Ground Investigation and Field Trials

All phases of ground investigation for the mainline and landscaping earthworks have been completed upon receipt of Data Drop 30 (March 2023). Reference should also be made to the Supplementary GIR, Ref.[7], which presents the factual data up to the GIR data-freeze date of January 2021 (Data Drop 06).

A GI location plan and geological long section are provided in 11. All outstanding GI data, post preparation of the geological long sections (and GIR), have been incorporated into the ground modelling, parameter derivation, and the design of the mainline and landscaping earthworks in this revision of the GDR. The exploratory holes are summarised in Table 2-2.

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Table 2-2: Summary of Ground Investigation Locations

Exploratory Hole Type	Number of Locations	Exploratory Hole ID	
Cable Percussion Borehole (CP)	4	ML060-CP002 ML061-CP001 ML061-CP004 ML061-CP012	
Windowless Sample (WLS)	20	ML060-WS005 ML060-WS006 ML060-WS007 ML060-WS010 ML061-WS001 ML061-WS002 ML061-WS010 ML061-WS015 ML061-WS016 ML061-WS017	ML061-WS018 ML061-WS019 ML061-WS020 ML061-WS021 ML061-WS021a ML061-WS022 ML061-WS023 ML061-WS024 ML061-WS025 ML061-WS026
Open Hole Rotary Core Drilling (RO+RC)	13	ML060-CR004 ML060-CR008 ML060-CR009 ML060-CR011 ML060-CR012 ML060-CR424	ML061-CR008 ML061-CR009 ML061-CR012 ML061-CR013 ML061-CR014 ML061-CR442 ML061-CR462
Rotary Coring (RC)	4	ML060-RC432, ML060-RC433, ML061-CR462, ML061-RC017	
Trial Pit (TP)	24	ML060-TP427 ML060-TT429 ML061-TP400 ML061-TP422 ML061-TP435 ML061-TP436 ML061-TP437 ML061-TP437A ML061-TP445	ML061-TP449 ML061-TP450 ML061-TP451 ML061-TP452 ML061-TP453 ML061-TP454 ML061-TP455 ML061-TP456 ML061-TP459

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Exploratory Hole Type	Number of Locations	Exploratory Hole ID	
		ML061-TP446	ML061-TP460
		ML061-TP447	ML061-TP461
		ML061-TP448	ML061-TP402
Cone Penetrometer Testing (CPT)	10	ML060-CT001	ML061-CT002
		ML060-CT002	ML061-CT400
		ML060-CT425	ML061-CT433
		ML061-CT001	ML061-CT440
		ML061-CT001a	ML061-CT440A
Seismic Cone Penetrometer Testing (SCPT)	4	ML060-CT422 ML060-CT425 ML061-CT400 ML061-CT440A	
Multi-Channel Analysis of Surface Waves (MASW)	15	ML060-MA400A	ML060-MA417
		ML060-MA400B	ML060-MA419
		ML060-MA400C	ML060-MA420
		ML060-MA400D	ML060-MA426
		ML060-MA400E	ML061-MA427
		ML060-MA400F	ML061-MW434
		ML060-MA400G	ML061-MW439
			ML061-MW441

In addition to the above GI data, following email correspondence from EKFB, the designers were informed about the presence of soft/compressible materials at greater depths within the chainage extents from Ch. 61+200 to Ch. 61+763. The site activity involved excavation underneath the footprint of the landscape bunds up to 3.5mbgl. The Hand Shear Vane readings undertaken at the site ranged between 25 to 40 kPa at depths between 0.5 and 3.5 mbgl, respectively as shown in Figure 2-2.

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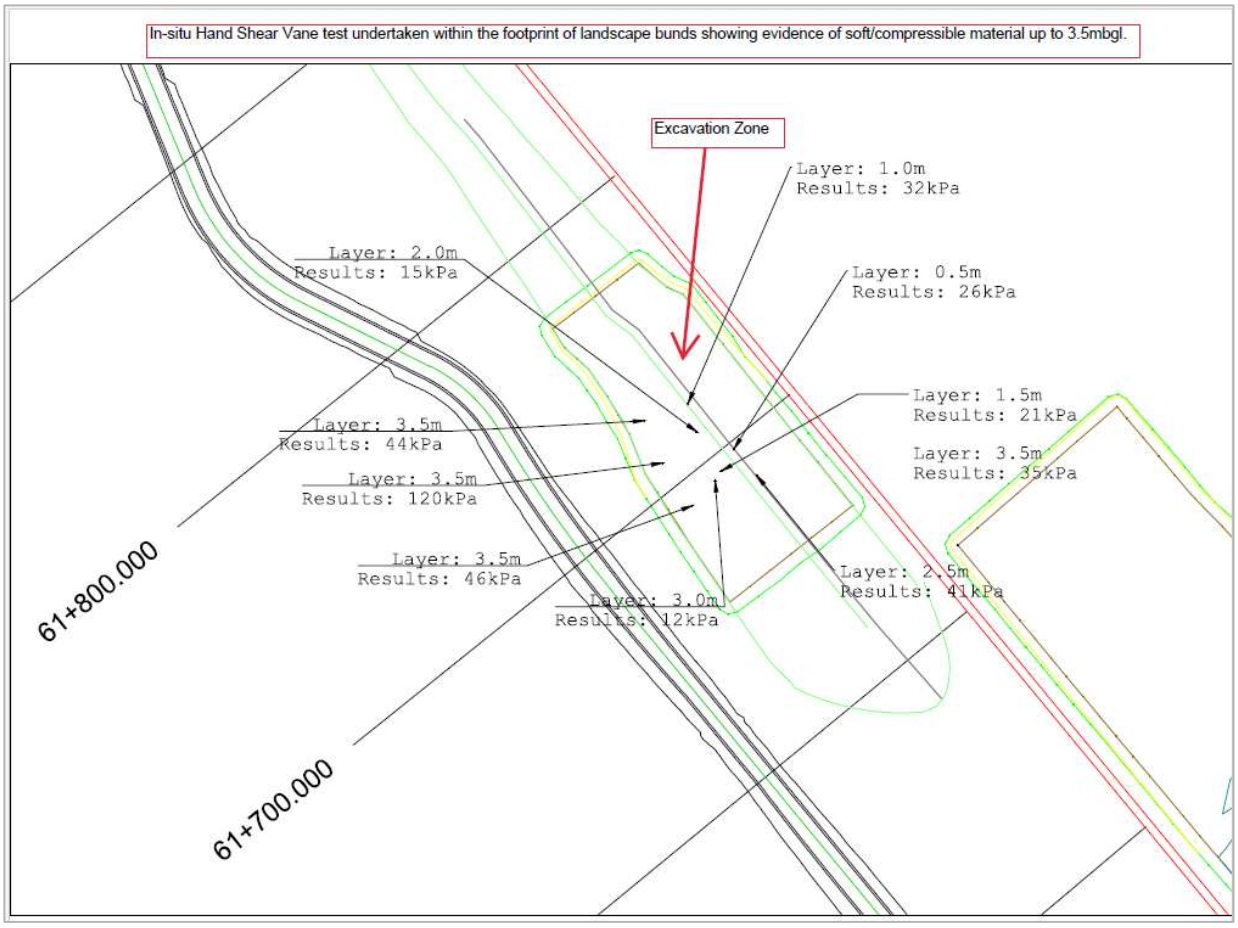


Figure 2-2: Excavation zone showing evidence of soft/compressible material within the chainage extents Ch. 61+200 to Ch. 61+763

2.4.5 Ground Conditions

Based on all the available GI data and the longitudinal stratigraphy profile, it is certain that the geology varies along the asset. The chainage has been split into five different ground models, summarised in Table 2-3 and shown in 11. Detailed descriptions of the asset lithologies are included in Appendix B and summarised in Table 2-3.

Table 2-3: Summary of Ground Conditions at Different Chainages.

Ground Model (GM)	Strata	Depth Range
Ground Model 1 (Ch. 60+490 – Ch. 61+000)	Head	0.00 – 3.80
	Gault Clay Formation	3.80 – 9.05
	Portland Stone/Sand Formation	9.05 – 20.00+
Ground Model 2 (Ch. 61+000 – Ch. 61+200)	Alluvium	0.00 – 1.60
	Gault Clay Formation	1.60 – 5.30
	Whitchurch Formation	5.30 – 6.30
	Purbeck Formation	6.30 – 8.10-

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Ground Model (GM)	Strata	Depth Range
	Portland Stone/Sand Formation	8.10 – 21.50
	Kimmeridge Clay Formation	21.50 – 25.00+
Ground Model 3 (*) (Ch. 61+200 – Ch. 61+350)	Alluvium	0.00 – 2.00
	Purbeck Group	2.00 – 4.50
	Portland Stone/Sand Formation	4.50 – 20.00+
Ground Model 4 (*) (Ch. 61+350 – Ch. 61+500)	Alluvium	0.00 – 1.20
	Soft Portland Stone/Sand Formation	1.20 – 2.00
	Portland Stone/Sand Formation	2.00 – 7.84
	Kimmeridge Clay Formation	7.84 – 20.00+
Ground Model 5 (*) (Ch. 61+500 – Ch. 61+750)	Head Deposits	0.00 – 1.00
	Soft Portland Stone/Sand Formation	1.00 – 2.00
	Portland Stone/Sand Formation	2.00 – 7.40
	Kimmeridge Clay Formation	7.40 – 20.00+

Note:
(*) The undrained shear strength (Cu) design parameter adopted for Ground Models 3, 4 and 5 are impacted by the presence of soft/compressible material as shown in Figure 2-2.

A summary of the critical sections selected for the mainline calculations is summarised in Table 2-4.

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Table 2-4: Ground Model Overview

Earthwork Analysis (1)	Ch.60+820	Ch.60+920	Ch.61+000	Ch.61+180	Ch.61+220	Ch.61+360	Ch.61+420	Ch.61+600
Slope Stability			✓				✓	
Settlement	✓	✓		✓	✓	✓		✓
Subgrade Stiffness	Asset-wide							
Rayleigh Wave	Asset-wide							
Embankment Height (m)	2.5	3.1	3.5	4.8	5.7	6.2	6.4	4.1
Landscape Bund Max Height at Chainage (m)	7.6	8.0	8.3	9.26	13.08	13.0	12.9	11.1
Comments	Check longitudinal distortion of Gault Clay due to the presence of a fault	Location of SBH/19 Overbridge	Critical geometry for GM1 & GM2	South side of fault line at 61+200	North side of fault line at 61+200	Culvert	Highest Embankment and Highest Landscape Bund.	Location near UTX (SGN Gas Main)

Notes:
(1) A 'tick' represents where analysis has been carried out.

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2.4.6 Groundwater Conditions

Groundwater strikes were encountered in several investigation locations across the asset. The groundwater monitoring program was also undertaken, which lasted approximately 23 months (April 2016 to March 2018). Some of the strikes were sub-artesian, where the water level rose after 20 minutes. Table 2-5 and Figure 2-3 summarise the water strike and monitoring data across the asset.

Table 2-5: Summary of Groundwater Readings

Location ID	Groundwater Depth Range (m bgl)	Chainage (Offset, m)	Number of Readings
ML060-CP002	1.98 – 4.13	Ch. 60+975 (-99)	13
ML060-CR004	1.67 – 2.71	Ch. 60+750 (22)	12
ML060-CR011	0.60 – 2.32	Ch. 60+990 (-35)	13
ML060-CR012	0.26 – 1.95	Ch. 60+510 (101)	16
ML060-CR424	1.02 – 2.44	Ch. 60+864 (-3)	60
ML061-CP001	0.10 – 1.30	Ch. 61+255 (-102)	13
ML061-CP004	0.71 - 2.09	Ch. 61+505 (-60)	11
ML061-CP011	0.91 - 2.44	Ch. 61+840 (126)	4
ML061-CP012	0.90 - 2.36	Ch. 61+695 (103)	13
ML061-CR009	1.47 - 3.10	Ch. 61+190 (14)	13
ML061-CR012	0.54 - 1.52	Ch. 61+375 (54)	14
ML061-CR013	0.68 - 2.07	Ch. 61+300 (-5)	13
ML061-CR014	0.75 - 1.27	Ch. 61+500 (2)	3
ML061-RC017	0.38 - 0.99	Ch. 61+655 (24)	3
ML061-WS017	0.97 - 1.70	Ch. 61+590 (95)	2

The flood level, from the flood modelling exercise on the upstream and downstream of Sedrup North Culvert is 77.854 mAOD and 77.726 mAOD, respectively, with a flood direction along the downstream direction mostly towards the north. This indicates that all the flows remain in-bank during the design flood event, thus not affecting the mainline earthworks.

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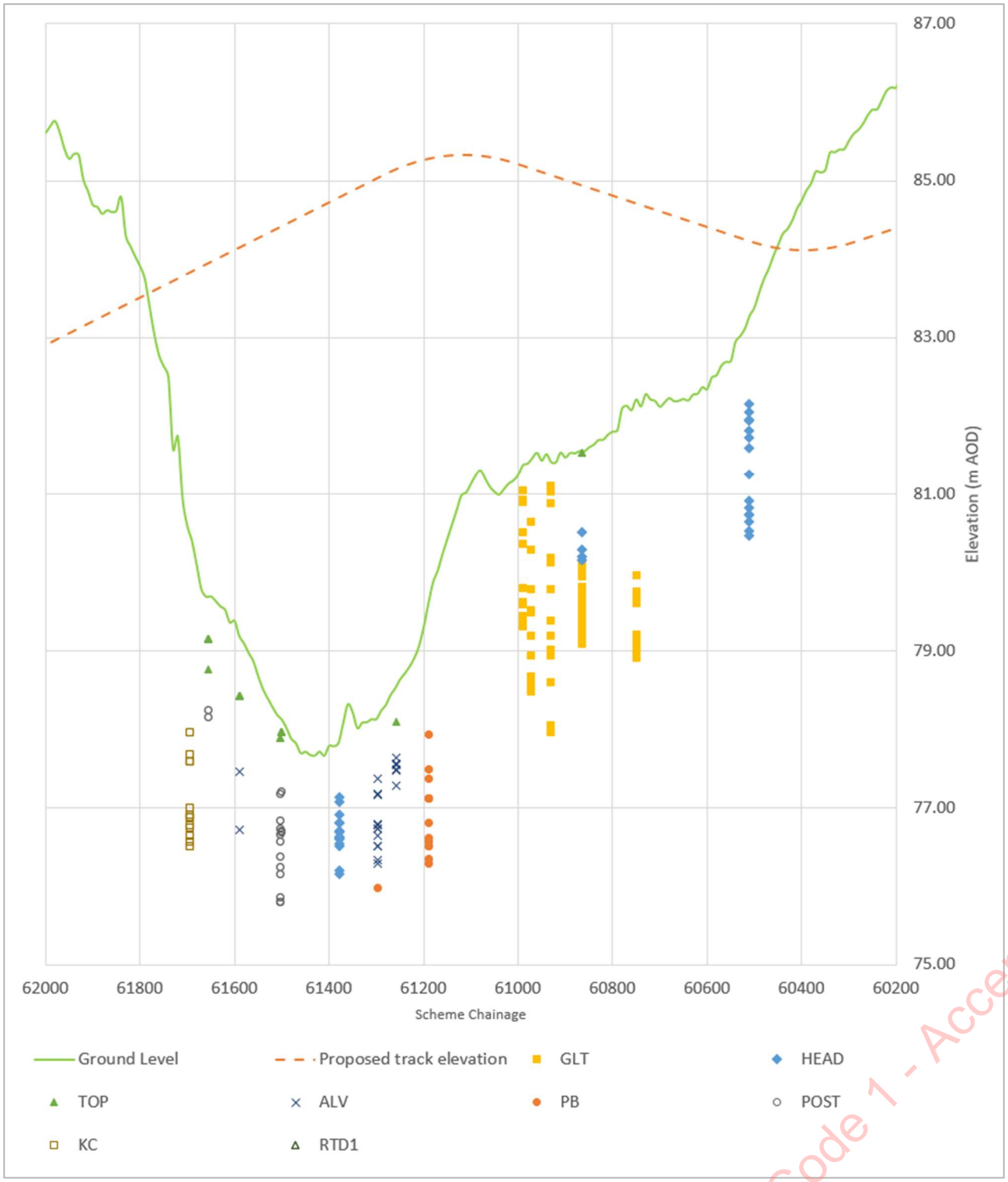


Figure 2-3: Summary of Groundwater Elevation Against Chainage

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2.4.7 Slope Stability

2.4.7.1 Basis of Geotechnical Design

The static global stability of earthworks has been calculated adopting the principles of limit state equilibrium in accordance with the HS2 Technical Standard - Earthworks Ref [3], Ground Engineering Design Basis Statement (GEDBS) Earthworks, Ref. [8] and Eurocode 7 Ref [4]. Specifically, Ultimate Limit State (ULS) Design Approach 1 Combination 1 (DA1 C1) and Combination 2 (DA1 C2) have been assessed adopting partial factors in accordance with Eurocode 7 Ref [4] with the requirement to achieve a Utilisation less than or equal to 100%.

The seismic global stability assessment is not required for Aylesbury Embankment in accordance with the HS2 Technical Standard – Civil Engineering Seismic Design Ref. [9].

Mainline embankment slopes of 1V:2H comprise of Class 1A1 (or Class 3A3 or Class 9M1) Fill, whereas, for landscape bund (1V:4H) slopes facing the trackside (Fill Zone 1), core fill (Zone 2) and external bund face (Zone 3) contains fill of material Class 2A3, Class 2A4 and Class 4A1 respectively.

The following assumptions have been adopted in design:

General:

- The Morgenstern-Price method of analysis was used.
- Slope stability was modelled using the specialist software SLOPE/W Ref [16].
- Circular slip surfaces were assessed adopting using the grid and radius method with ‘optimized’ methods of slip surface determination within the software to report the highest degree of utilisation.
- The stability assessment was carried out to achieve a target utilisation of less than 100%.
- Slope stability has assessed shallow (confined to embankment and bund slopes) and deeper (global) potential slip surfaces. The highest Utilisation or critical slip surface is reported.
- A depth of 0.3m was used as minimum slip surface depth in order to rule out insignificant, shallow slip surfaces.
- To model the effects of weathering and associated seasonal wetting and drying in the landscape bund cohesive slopes, a vadose zone has been modelled in the upper 2.0m of the landscape bunds (Cohesive fill) in the long-term drained analysis. A higher Ru coefficient of 0.2 and critical state parameters have been applied to this zone in accordance with the HS2 Earthworks Design Basis Statement Ref [8]. Further details are provided in Appendix B. The rest of the landscape fill below the 2.0m vadose zone uses a Ru coefficient of 0.1 and peak strength parameters.
- The short-term (undrained) and the long-term (drained) conditions were analysed for the static condition.
- Surcharge loading was applied at an offset of 0.5m from the crest of the mainline slopes.

2.4.7.2 Actions/ Loading

The loading applied in the embankment slope stability analyses is summarised below. Partial factors in accordance with Eurocode 7 Ref. [4] were applied to the static stability assessments.

The LM71 train loading (equivalent to uniformly distributed load of 57kPa) has been adopted in the slope stability assessments of the mainline earthworks in the long-term conditions (applied at finished embankment level, over 3m track widths). Details of the surcharge loads required for the design of the mainline HS2 earthworks are presented below and in Appendix I of this GDR.

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Load Case A (static – short-term condition):

- A variable surcharge of 20 kPa over the full width of the embankment to represent construction loading.
- A variable surcharge of 20 kPa over the full crest width of landscape bunds to represent construction loading.

Load Case B (static – long-term conditions):

- A variable surcharge of 10 kPa represents the operational/ maintenance loading over the crest of landscape bunds.
- A permanent surcharge of 15 kPa to represent the track load plus a variable surcharge of 57 kPa to represent the live train loading is applied over the slab track extent. Due to restrictions within the Slope/W software, these loadings have been modelled as a 72 kPa variable load due to adopting a conservative loading approach since variable loading requires more onerous partial factors.
- A variable surcharge of 10 kPa represents the operational/ maintenance loading within the cess.
- Favourable rail and track surcharging are ignored when checking the HS2-facing landscape bund slopes.

2.4.7.3 Materials

Five ground models have been assessed for Aylesbury Embankment, as summarised in Table 2-3. For the assessment of Ground Model 1 (GM1) and Ground Model 2 (GM2), the critical section from Ch. 61+000 was selected with the mainline embankment height of 3.5m and one landscape bund of 8.3m in height.

As mentioned in section 2.4.4, soft/compressible deposits were identified between Ch. 61+200 and Ch. 61+763 during site excavation. The reported in situ hand vane shear strength values do not comply with the termination strength criteria of a minimum of 50 kPa at the base of the foundation. For design purposes to represent the ground conditions observed at the site, there is an update to the c_u profile from Ch. 61+200 to Ch. 61+763 and applies for GM3 to GM5. A critical section at Ch. 61+420 was selected with an embankment height of 6.4m and landscape bund height of 13.0m. The conservative ground parameters based on these low-strength data received were adopted, as shown in Figure B-15 of Appendix B, irrespective of the ground model category they associate with. For GM3 to GM5, the stability assessments for mainline and landscape bunds were undertaken, and the mitigation solution developed for this cross-section shall be applied to the areas identified as Alluvium extents.

The material parameters outlined in Appendix B were adopted with partial factors in Eurocode 7 Ref [4] applied to static assessments.

Typical parameters for various fill materials Class 1A1 (Embankment Fill and Replacement Fill), Class 2A3 (Landscaping Fill Zone 1), Class 2A4 (Landscaping Fill Zone 2), and Class 4A1 (Landscaping Fill Zone 3) adopted within the calculations are outlined in Appendix B. The material parameters for Class 3A1, Class 3A3 and Class 9M1 (being considered as part of SNC-8291 and SNC-8293) are equal or better than those for Class 1A1 and as such calculations are considered representative for all materials.

A ‘worst-case’ groundwater level was taken at the existing ground level. Vadose zones within the landscape bunds slopes have been modelled on the inside slope facing the HS2 track.

2.4.7.4 Calculation Results

A summary of slope stability calculations with mitigation measures adopted in the form of Excavate and Replace is provided in Table 2-6. A selection of the calculation outputs is presented in Appendix C.

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The results presented in Table 2-6 show that a minimum of 2m Excavate and Replace mitigation is required to keep the mainline embankment slope stable. Where mainline earthworks are not buttressed, it is imperative to restrict the surcharge loads at an offset of 0.5m from the embankment crest to maintain the stability of the slope during construction (short-term).

From Ch. 61+200 to 61+763, localised soft/compressible soils are known to be present mostly within the footprint of the landscape earthworks. A conservative stability assessment of mainline embankment and landscape earthworks was carried out adopting 2m E&R with a reduced shear strength mentioned in Section 2.4.4 and Figure B-15 of Appendix B. Within the Alluvium extent, the 2m of E&R mitigation for the mainline embankment requires to be extended beyond the track support zone to cover the full width of the embankment and to remove the soft deposits within the earthwork base, as it would be considered as a good construction practice. The depth of E&R required underneath the bunds should be determined at the site, where a termination strength criterion of 50kPa for the formation ground is attained. The excavation should be backfilled by Class 2A1 material upon which bunds will be constructed.

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Table 2-6: Summary of DoU for critical sections Ch. 61+000 (Ground Model 1 and 2), and Ch. 61+420

Load Case	Partial Factor Combination	Slip Location	Degree of Utilisation (%) at			
			GM 1, Ch. 60+920	GM 1, Ch. 61+000	GM 2, Ch. 61+000	GM 3,4 & 5, Ch. 61+420 (2)
LCA – Short Term (Undrained)	DA1C1	Inner Landscape Bund (Facing HS2)	65	64	49	94
		Outer Landscape Bund (Away From HS2)	73	72	91	103 (3)
		Mainline Embankment – with E&R mitigation	75	75	70	84
	DA1C2	Inner Landscape Bund (Facing HS2)	60	60	69	94
		Outer Landscape Bund (Away From HS2)	72	71	93	103 (3)
		Mainline Embankment – with E&R mitigation	97	97 (1)	89 (1)	88 (1)
LCB – Long Term (Drained)	DA1C1	Inner Landscape Bund (Facing HS2)	71	81	78	87

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Load Case	Partial Factor Combination	Slip Location	Degree of Utilisation (%) at			
			GM 1, Ch. 60+920	GM 1, Ch. 61+000	GM 2, Ch. 61+000	GM 3,4 & 5, Ch. 61+420 (2)
		Outer Landscape Bund (Away From HS2)	64	78	70	75
		Mainline Embankment – with E&R mitigation	72	84	85	88
	DA1C2	Inner Landscape Bund (Facing HS2)	75	92	91	99
		Outer Landscape Bund (Away From HS2)	70	88	87	84
		Mainline Embankment – with E&R mitigation	85	95	98	99

Notes:

(1) The risk of these failures is reduced when the landscape bunds are constructed concurrently, or an exclusion zone of 0.5 m is enforced at the crest of the mainline embankment.
(2) DoU is reported for a 2m E&R fill underneath the landscape bunds. Section 61+420 requires mitigation to be undertaken at site. Excavate to a depth until a cu of 50kPa has been achieved and replace by Class 2A1 Fill.
(3) The stability of the outer landscape bund slope reported for 2m E&R, shows slope instability. Mitigation under landscape bunds must be provided by undertaking shear strength testing during site works as per Note 2.

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The summary of mitigation solution (depths and extents) required for ensuring the stability of the earthworks slope are provided in Table 2-7.

Table 2-7: Mitigation Measures for Aylesbury Embankment

Ground Model	Chainage Range	Mainline Embankment Mitigation (1)	Upside Landscaping Mitigation	Downside Landscaping Mitigation
GM1	60+500 to 61+000	2.0 m Excavate and Replace: Class 1A1 (or Class 3A1 or Class 3A3), within the track support zone (2)	None	
GM2	61+000 to 61+200	2.0 m Excavate and Replace: Class 1A1 (or Class 3A1 or Class 3A3), within the track support zone (2)	None	
GM3 to GM5	61+200 to 61+763	2.0 m Excavate and Replace: Class 1A1 (or Class 3A1 or Class 3A3), to be extended beyond the track support zone covering the full width of the embankment base (2)	Excavate until a minimum undrained shear strength of 50kPa has been achieved. Replace with Class 2A1 fill to the finished ground level.	

Notes:

(1) The landscape bunds are constructed concurrently with the mainline embankment, or an exclusion zone of 0.5 m is enforced at the crest of the mainline embankment.

(2) Refer to Aylesbury Embankment – Report of Cross-section and General Arrangement drawings for more clarity of mitigation extents.

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The mitigation requirements for slope stability purposes have been taken forward. A summary of the combined mitigation measures for Aylesbury Embankment is summarised in section 2.4.14.

2.4.8 Settlement Analysis

2.4.8.1 Introduction

Settlement is one of the key geotechnical risks identified as a result of the construction of medium embankments and large landscape earthworks above compressible soil, less permeable over-consolidated clays, ground of varied stiffness due to the presence of inferred faults, leading to differential settlement along the asset length. Mitigation design has been undertaken based on all Ground Investigation (GI) data available from the HS2 Main GI and Stage 2 GI (Data drop 30).

2.4.8.2 Basis of Geotechnical Design

Earthwork settlements along the mainline embankment have been assessed to satisfy compliance with deformation criteria set out in the HS2 Technical Standard – Earthworks Ref [3]. Total settlements (immediate, consolidation and creep/secondary) have been assessed. The settlement analyses have been undertaken in accordance with the design approach set out in Ground Engineering Design Basis Statement – Earthworks Ref [8]

This section covers the settlement of the mainline earthworks. Details of the transitions between the mainline earthworks and the Sedrup North Culvert can be found in Section 2.4.10.

The following assumptions have been adopted in the design:

- Settlements have been calculated considering permanent loading from the track slab, the mainline embankment self-weight and the overburden exerted by the adjacent landscape earthworks on the mainline; for details of the loading applied to the earthworks for settlement calculations, refer to Appendix I of this GDR.
- A Wait Period of two months is proposed for the entire asset length to observe the ground movements and validate the settlement predictions, thereby ensuring that the predictions are accurate.
- Compliance Periods of three months for ‘long’ earthworks and five months for ‘short’ earthworks have been assumed to calculate post-construction consolidation settlement.
- The Compliance Periods of three to five months are assumed to start once the embankments and adjacent bunds are constructed to the full height and the temporary surcharge is applied.
- Temporary surcharge loading equivalent to 15kPa, simulating the weight of the slab track, is required for the duration of the compliance period. No works, including track drainage, can be installed during the two-month Wait Period. Please refer to Section 2.4.2 for further discussion of constraints where a surcharge is present over the embankment.
- The mainline earthwork asset has been classified as ‘long’ earthwork from Ch. 60+500 to Ch. 61+330 and ‘short’ earthwork from Ch. 61+330 to Ch. 61+763 in accordance with the HS2 Technical Standard – Earthworks Ref. [3]. The presence of Sedrup North Culvert at Ch. 61+330 is considered as a discontinuity for the categorisation of earthworks above. The maximum allowable post-construction settlement for a ‘long’ earthwork is 60mm, and 30mm for a ‘short’ earthwork, Ref. [3].
- Two cross-sections have been selected at Ch. 61+180 and Ch. 61+220, both being 20m on either side of the fault line at Ch. 61+200. A difference in geology is noted south of the fault, which comprises of Gault Clay Formation followed by Portland Stone/Sand Formation, and north of the fault, the geology consists of Purbeck Group followed by Portland Stone/Sand Formation. The

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longitudinal distortion of the mainline embankment is assessed for variation in ground profile induced by faulting, and the results are reported.

- Mitigation measures have been determined to limit the long-term longitudinal distortion to less than 1V:1500H, in accordance with HS2 Technical Standard – Earthworks, Ref. [3].
- Vertical stress distribution underneath the mainline embankment has been calculated using the Boussinesq method using the specialist software Oasys PDisp, Ref. [18].
- In the absence of an incompressible layer at depth, settlements have been calculated in accordance with Eurocode 7 Ref. [4] which states the depth of compressible soil 'may normally be taken as the depth at which the effective vertical stress due to foundation load is about 20% of the effective overburden stress'.
- Based on the mechanical properties of the soil, the ground models have been grouped into two categories – M1 (Ch. 60+500 to Ch. 61+200) and M2 (Ch. 61+200 to Ch. 61+763) to generate a ground stiffness profile with depth. In situ ground properties and the ground stiffness profile used for the settlement analyses are provided in section 2.4.8.5.
- Material properties for the Embankment Fill and Excavate and Replace (E&R) zones (i.e., Class 1A1, Class 3A1, Class 3A3 or Class 9M1) are provided in Appendix B.
- Due to the low permeability of the underlying geology, one-way drainage has been assumed to determine the magnitude of the post-compliance settlements.
- Settlement within the Class 1A1 (or Class 3A1 or Class 3A3) Replacement Fill and Class 1A1 (or Class 3A3 or Class 9M1) Embankment Fill is anticipated to be immediate and to be completed within the construction period.

Fine-Grained Soils – One-dimensional Consolidation

The classic one-dimensional analysis has been adopted to calculate the total and consolidation settlement within over-consolidated and normally consolidated cohesive materials, following the recommendations of the ICE Manual of Geotechnical Engineering Ref. [19]:

$$S_{1xd} = \sum m_v \, \Delta\sigma_1 dz$$

where m_v is the coefficient of volume compressibility, $\Delta\sigma_1$ is the appropriate change in stress, and dz is the layer thickness.

For over-consolidated clays, $S_u \sim 0.5 S_t$ where $S_t \sim S_{1xd}$

For normally consolidated clays, $S_u \sim 0.1 S_t$ where $S_t \sim 1.1 S_{1xd}$.

The time for (vertical) consolidation settlement is then determined using the expression:

$$t = \frac{T_v d^2}{c_v}$$

where T_v is a time factor, d is the length of the drainage path, and c_v is the coefficient of consolidation (vertical)

The Degree of Consolidation (U_v) was determined from Figure 5.6 (Carter and Bentley), Ref. [20], for 'Case 3' (one-way drainage).

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Average Degree of Consolidation

The drainage for the dissipation of excess pore water pressure occurs simultaneously in both directions. The average Degree of Consolidation is expressed as:

$$U = 1 - (1 - U_v)(1 - U_r)$$

Where U_v is the Degree of Consolidation vertically, U_r is the Degree of Consolidation radially, and U is the average Degree of Consolidation for simultaneous vertical and radial drainage.

Replacement Fill – Elastic Theory

The immediate settlement of replacement fill has been calculated in accordance with Elastic Theory adopting an equation after Barnes Ref. [21]:

$$S_i = \frac{pBI_s}{E'_{md}} \text{ and } \alpha = \frac{kB}{E'_{md}}$$

where p is the foundation pressure, B is the width of the loaded area, I_s is a settlement influence factor, E'_{md0} is the drained elastic modulus at the founding level, k is the rate of modulus increase with depth, and α is an influence factor.

2.4.8.3 Settlement Analysis Methodology

The following methodology has been used to determine the predicted settlement and the required measures to mitigate the settlements along the embankment.

At each critical cross-section:

- Step 1: Calculate the post-construction consolidation settlement at the centre of the embankment base, assuming no mitigation scenario ('No' E&R and 'No" hold period defined). Permanent dead loads representing the track slab weight of 15 kPa are considered. This aids in improving the short-term settlement during construction.
- Step 2: Where post-construction consolidation settlements calculated in Step 1 exceed the settlement criteria for short and long earthworks, Step 1 is repeated by adopting the mitigation measure. The preferred mitigation measure is to Excavate and Replace the soft ground with a competent granular material (Class 1A1, Class 3A1 or Class 3A3). The combined mitigation depth governs the depth of mitigation at that location, which, in most cases for Aylesbury Embankment, is 2.0 mbgl.
- Step 3: Where consolidation settlements calculated in Step 2 do not yet comply with the HS2 settlement criteria, Step 2 is repeated, adopting a minimum compliance period of 3 months.
- Step 4: Where Compliance Periods stipulated in Step 4 are not sufficient to meet the settlement criteria, Step 3 is repeated by increasing the compliance period in increments of one month till the HS2 settlement criteria are met.
- Step 5: Present the settlement results in a table showing the calculated post-construction settlements for the various scenarios detailed above (no mitigation, E&R, and E&R plus hold period).
- Step 6: Check the longitudinal distortion and transverse distortion requirements with the mitigation solutions by adopting E&R depths and compliance period to ensure that long term requirement is met.

2.4.8.4 Actions/Loading

The loading applied in the settlement analyses is summarised below. Partial factors have not been applied for this Serviceability Limit State (SLS) assessment (EC7-1, Ref. [4]).

- Self-weight of the mainline embankment and the landscape bunds (permanent load case).

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- A permanent surcharge of 15kPa to represent the slab track loading. For the purposes of modelling within Oasys PDisp, Ref. [18], a 30° load spread (from the vertical) was assumed for an equivalent surcharge at the formation level, shown in Figure 2-4.

A permanent surcharge of 15 kPa to represent slab track loading. For the purposes of modelling within PDisp, a 30° load spread (from vertical) was assumed for an equivalent surcharge at the formation level, Shown in Figure 2-4.

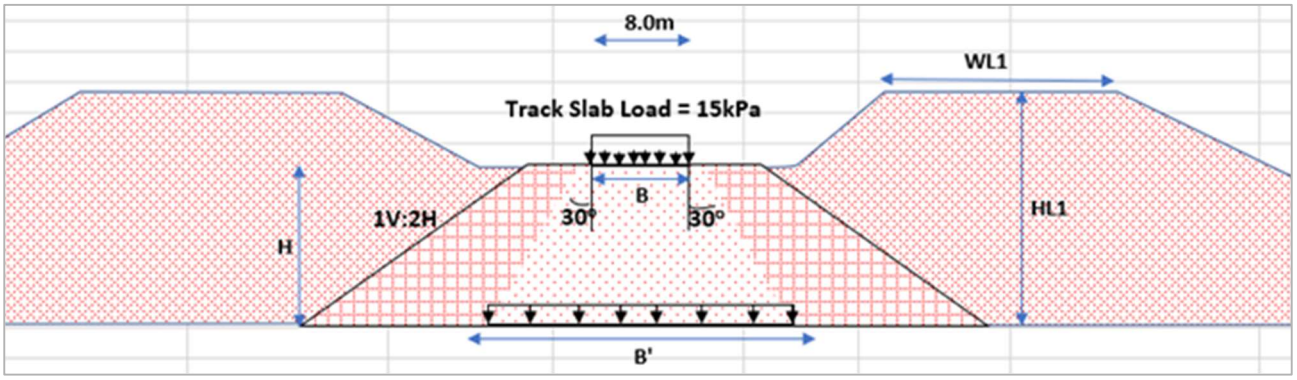


Figure 2-4: Typical Arrangement of Embankment Loading for Settlement Analysis (Not to Scale)

2.4.8.5 **Materials**

The ground stiffness profile has been determined by grouping the ground models based on their mechanical properties. Based on all the latest GI data available, two stiffness profiles have been developed by plotting the constrained modulus (Eoed) against depth. Figure 2-5 and Figure 2-6 show the stiffness profiles, M1 and M2, with the design line adopted for the settlement assessment.

In situ ground parameters and fill material parameters adopted for the settlement analyses are provided in Appendix B.

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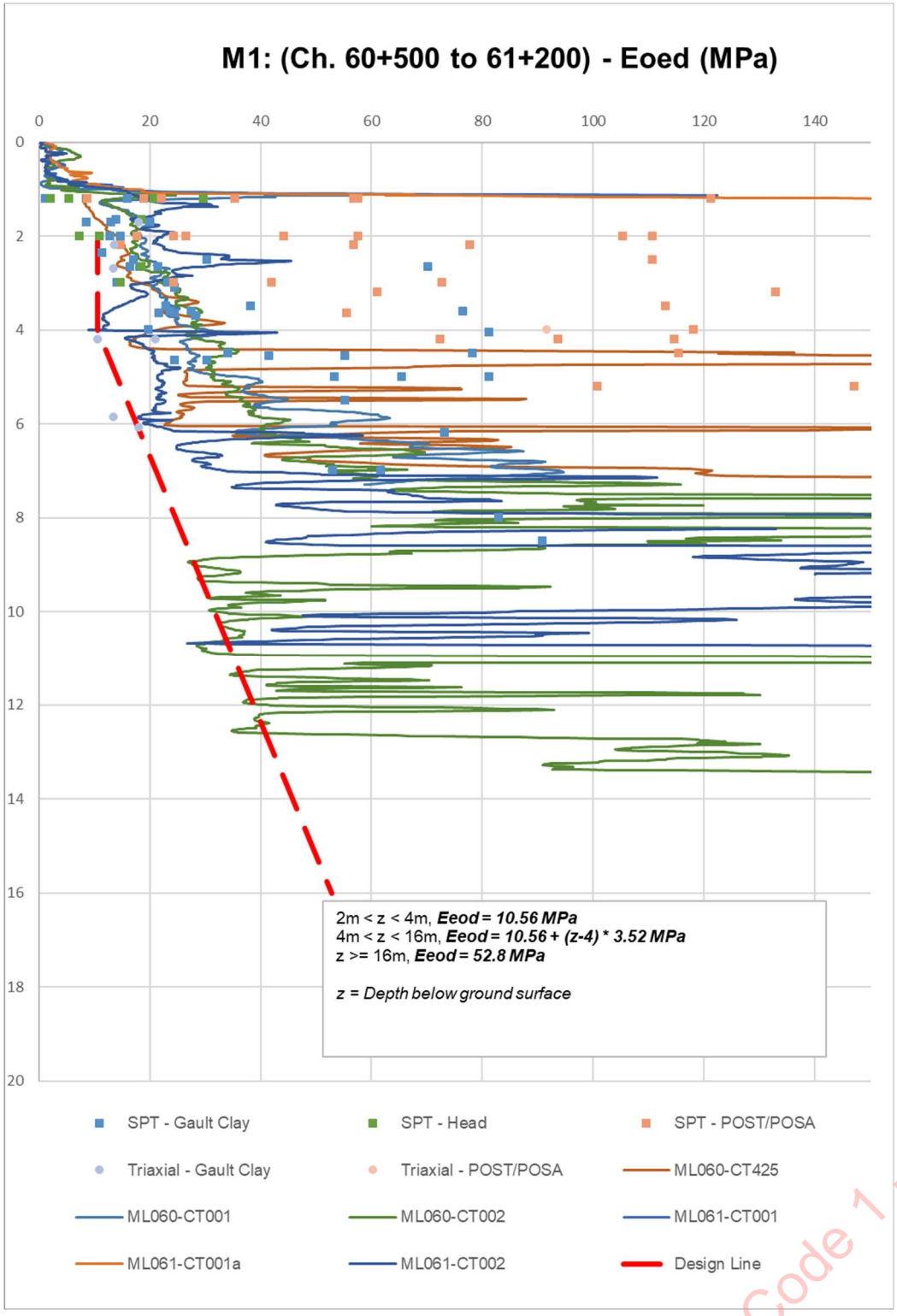


Figure 2-5: Stiffness Profile (M1) adopted from Ch. 60+500 to 61+200

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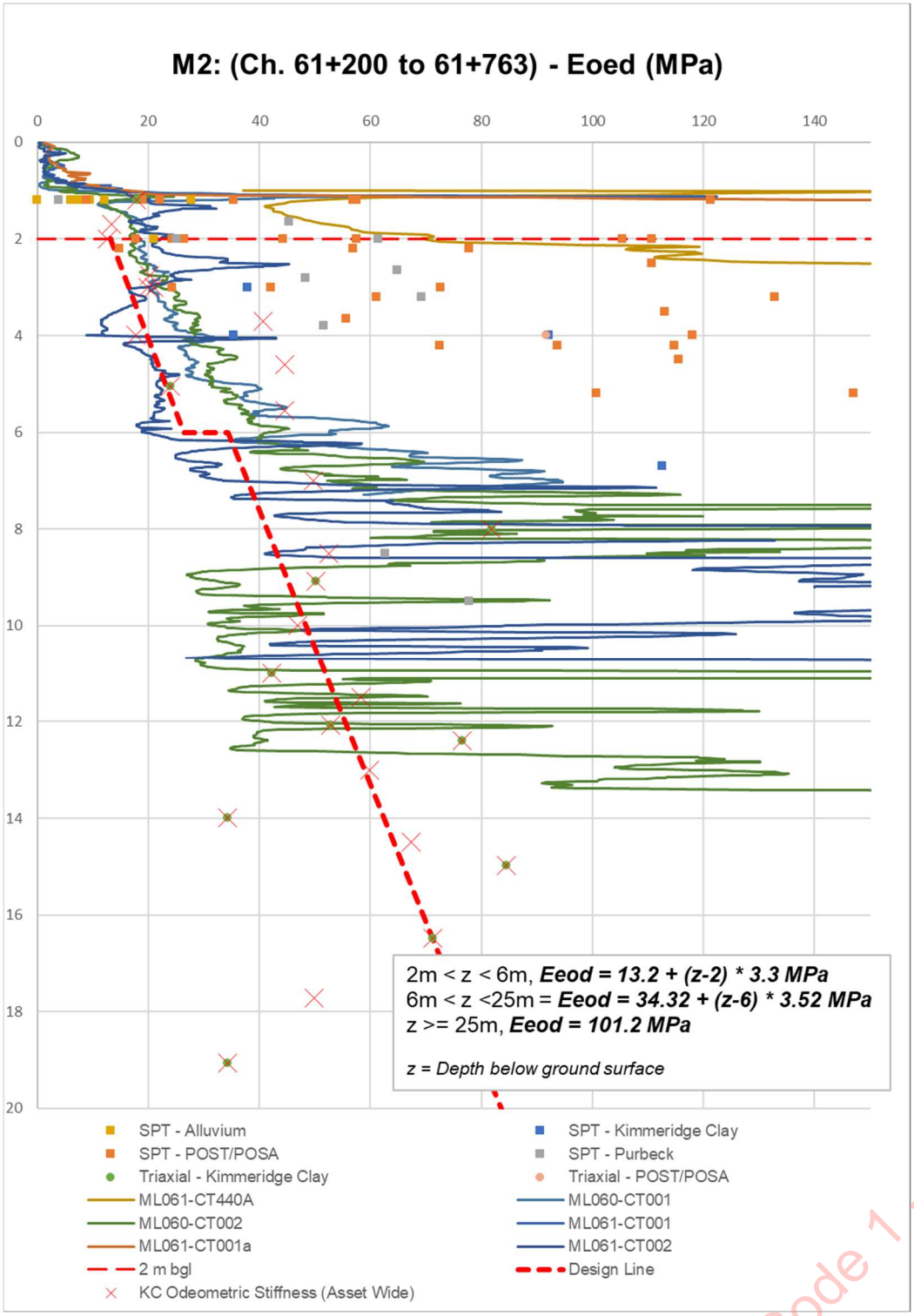


Figure 2-6: Stiffness Profile (M2) adopted from Ch. 61+200 to 61+763

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2.4.8.6 Critical cross-sections

A summary of the earthwork geometry for the six selected cross sections and the justifications for being assessed at these locations are presented in Table 2-8. A summary of the ground models used in settlement analysis is presented in section 2.4.5. A 'worst-case' groundwater level is considered at the existing ground level for settlement calculations.

Table 2-8: Summary of Critical Sections and Justifications

Critical HS2 Chainage	Ch 60+820	Ch 60+920	Ch 61+180	Ch 61+220	Ch 61+360	Ch 61+600
Earthwork Type	Long	Long	Long	Long	Short	Short
Mainline Embankment Height (m)	2.65	3.1	4.87	5.74	6.25	4.14
Mainline Embankment Width (m)	15.7	15.7	16.73	15.85	15.35	15.34
Max Landscape Bund Height (m)	7.64	8.82	9.26	13.08	13.02	11.09
Total Earthwork Width (1) (m)	108.58	111.00	96.76	106.79	110.56	101.34
Justification	Check longitudinal distortion due to the presence of a fault. The thickness of Gault Clay vary significantly on both side of the fault	Location of SBH19 Overbridge	South side of fault line at 61+200	North side of fault line at 61+200	Culvert and Highest Embankment and Highest Landscape Bund	Critical section within the Alluvium extent, north of the asset

Note:

(1) The total width is from the toe of the upline landscaping to the toe of the downline landscaping bund.

2.4.8.7 Calculation Results

A summary of the predicted settlements beneath the centre line of the embankments and the longitudinal distortion checks performed is provided in Table 2-9. The post-construction settlement without any mitigation measures (free-fall analyses) indicates that the predicted ground movements are higher than the settlement limits associated with short and long earthworks criteria specified by HS2. The settlement results following the application of mitigation measures and the Compliance Period, along with the surcharge loading, are presented in Table 2-9. Mitigation solutions in the form of E&R to a depth of 2.0 mbgl, along with a Compliance Period of three months for long earthwork and five months for short earthwork, are sufficient to meet the HS2 settlement criteria, longitudinal distortion, and transverse distortion criteria. A temporary surcharge is required for the entire duration of the Compliance Period.

Using the mitigations applied, the longitudinal distortion limit of 1V:1500H (6.0×10^{-4}) is also satisfied between all sections, which is presented in Table 2-9. The settlement values used to determine the longitudinal distortion are represented by the maximum difference between the mitigated settlement predictions at adjacent chainages.

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Transverse distortion has been assessed at Ch. 61+360 as ground conditions are anticipated to vary transversely. The resulting transverse distortions for both slab tracks are less than 1V:1000H (1.0×10^{-3}), satisfying the criteria in accordance with HS2 Technical Standard – Earthworks, Ref. [3].

A selection of calculation outputs is presented in Appendix C.

The construction of the landscape bund can precede the construction of the mainline embankment; however, the landscape bunds are to be constructed to the full height along with the mainline embankment. Following this, a temporary surcharge bund equivalent to 15 kPa is applied on the mainline embankment. It should be allowed to consolidate for a Compliance Period ranging from three to five months, depending on the earthwork type (long/short earthwork). The above-stated Compliance Period is inclusive of the two-month long Wait Period. A float period of one month is required at the end of the compliance period to verify the predicted settlements and carry out any mitigation, if necessary. A Hold Period of four months is recommended for long earthworks (Ch. 60+500 to 61+763) and six months for short earthworks (Ch.61+330 to 61+763) to comply with the HS2 requirements post construction.

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Table 2-9: Summary of Settlement Results

Estimated Settlement	Ch. 60+820	Ch. 60+920	Ch. 61+180	Ch. 61+220	Ch. 61+360	Ch. 61+600
No Mitigation/ Free Fall Analyses (1)						
Compliance Period, CP (months)	-	-	-	-	-	-
Immediate Settlement (mm)	27.6	35.4	54.8	44.3	48.4	30.5
Consolidation Settlement (mm)	52.3	63.5	90.2	85.3	92.6	56.2
Total Settlement (mm)	79.9	98.9	144.9	129.7	141.0	86.7
Post-construction Consolidation Settlement (mm) [2]	52.3 [3]	63.5	90.2	85.3	92.6	56.2
Excavate and Replace Mitigation with Compliance Period						
Compliance Period, CP (months) [5]	3	3	3	3	5	5
Immediate Settlement (mm)	46.5	59.1	91.1	88.8	92.7	57.4
Consolidation Settlement (mm)	36.4	45.4	50.3	39.2	42.8	27.3
Total Settlement (mm)	82.9	104.5	141.4	128.0	135.5	84.7
Post-construction Consolidation Settlement (mm) [4]	22 [3]	29.4	40	29.6	28.1	18.4

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Estimated Settlement	Ch. 60+820	Ch. 60+920	Ch. 61+180	Ch. 61+220	Ch. 61+360	Ch. 61+600
Settlement limits (mm)	60	60	60	60	30	30

Estimated Settlement	Ch. 60+820	Ch. 60+920	Ch. 61+180	Ch. 61+220	Ch. 61+360	Ch. 61+600
Longitudinal Distortion	7.4×10^{-5}		2.6×10^{-4}		4.04×10^{-5}	
		4.08×10^{-5}		1.07×10^{-5}		
Transverse Distortion					3.81×10^{-4}	

Notes:
[1] No Excavate and Replace, Compliance Period is assumed for the analyses.
[2] The post-construction consolidation settlement is same as the predicted consolidation settlement, as no compliance period is adopted.
[3] A consistent Compliance Period of 3 months with combined mitigation has been assessed from a construction point of view, even though the predicted settlement for 'No mitigation' case is within the HS2 limits.
[4] The Post-construction Consolidation settlement is calculated from the end of Compliance Period.
[5] The Compliance Period is assumed to start from the end of construction, i.e., when both mainline and landscaping earthworks are constructed to the full height.

Figure 2-7 shows the post-construction settlement (mm) against asset chainages. The Sedrup North Culvert location is schematically shown. The transitions between the culverts and the earthworks are discussed in Section 2.4.10.

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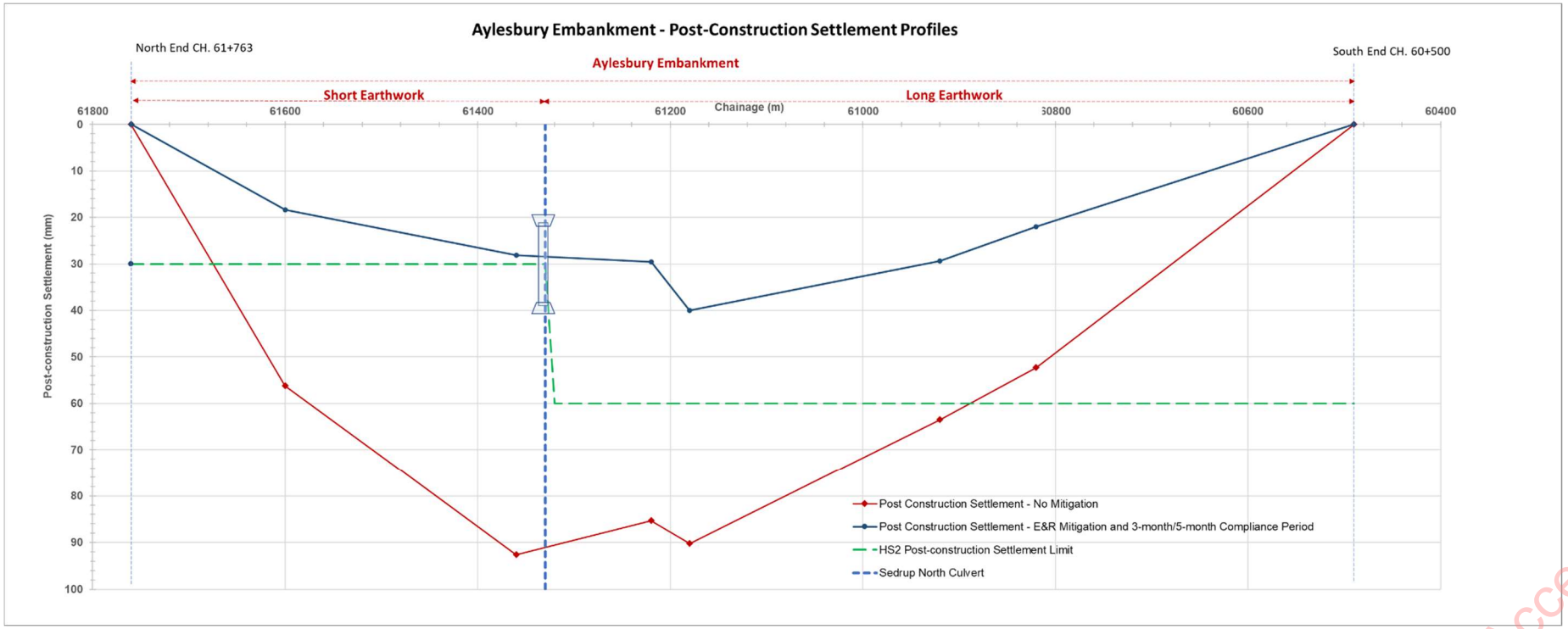


Figure 2-7: Variation of Post-construction Settlement along Mainline Embankment

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2.4.9 Heave due to Stress Relief

There is potential for some heave during the excavate and replacement works along the embankment. However, the duration of the unloading of the ground and the reversal of the ground movement due to the loading of the embankment renders heave as insignificant and is not considered further.

2.4.10 Transitions to Structures

The design of the mainline earthwork transition to the Sedrup North Culvert is detailed in this section. The Sedrup North culvert is proposed at Ch. 61+330 and will be constructed below the footprint of the mainline and landscaping earthworks in a transverse direction to the HS2 track. The settlement analysis and the required geometry for the approach embankments to the culvert are detailed in Appendix C.

It is a requirement that transitions for the high-speed mainline shall never be less than 20m in length and shall be significantly longer where needed to meet differential settlement requirements. To prevent unacceptable differential settlement, the longitudinal and transverse distortions along earthwork approaches (transition lengths) to the structure interface need to be less than 1V:1500H, as per the requirement of HS2 Technical Standard – Earthworks, Ref. [3]. The mitigation measures described in Appendix C are only required below the mainline earthwork (specifically within the full base width of the embankment) and are not required under the landscaping earthworks.

For a smooth transition within the upper embankment, material Class 6N12 (Enhanced Upper Embankment Fill, UEF*) is specified above and around the structures. The lower embankment fill does not apply to this culvert transition.

The settlement analysis has been undertaken in accordance with the design approach set out in the (GEDBS) Earthworks, Ref. [8]. A summary of the transition details for the Sedrup North Culvert is provided in Table 2-10 below. The mitigation requirements have been presented in Section 2.4.14. Refer to Drawing No. 1MC06-CEK-GT-DDE-CS04_CL18-000003 Ref [22] for transition details.

As part of the design update to incorporate SNC-8291 and SNC-8293 concerning the option to use of Class 3A1, Class 3A3 and/or Class 9M1 for the mainline Embankment Fill and E&R as appropriate, a transition between 'mainline earthworks foundation treatment' and culvert 'earthworks transition' is required as to not affect assurance of Sedrup North Culvert (including the earthworks transitions). If any Class 3A1, Class 3A3 or Class 9M1 fill is used outside of the culvert and earthworks transition zone, the mainline earthworks E&R and Embankment Fill shall smoothly transition into the Class 3A1, Class 3A3 or Class 9M1 fill (as appropriate) with a sloped gradient of 1H:2H, see the Figure below.

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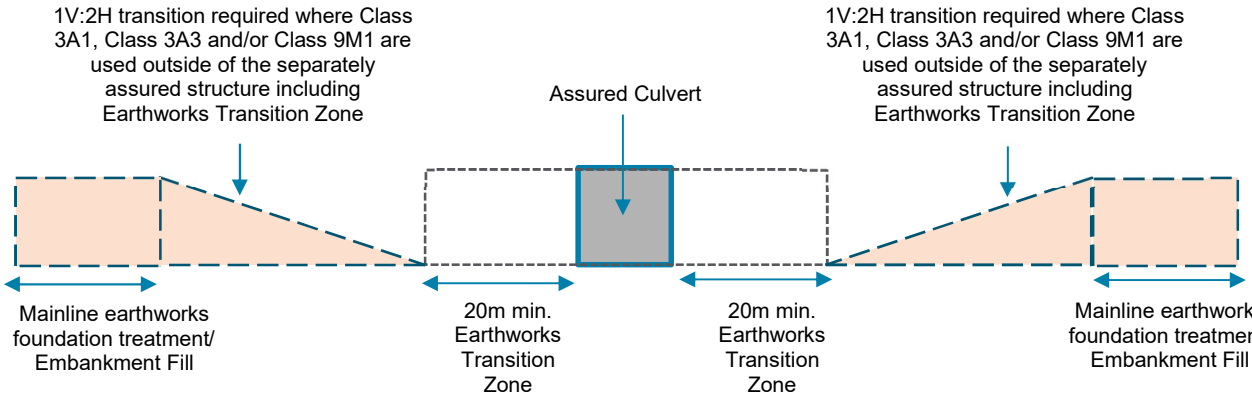


Figure 2-8: Diagram of transition between 'mainline earthworks foundation treatment' and culvert 'earthworks transition'

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Table 2-10: Transition Mitigation Measures

Structure	Structure Location	Structure Form	Transition Mitigation Extent	Transition mitigation
Sedrup North Culvert	Ch. 61+330	Box Culvert	60m (Ch. 61+301 to 61+359)	• The box culvert is 3.3m wide and 3.8m in height. The centreline of the culvert is at approximately Ch. 61+300. • The culvert will be founded on Class 1A2 fill with a thickness of 2.0m to satisfy the foundation design. • The foundation of the culvert will extend 6.75m on either side, towards both up and down chainages, from the centreline of the culvert. • A maximum post-Compliance differential settlement of less than 10mm was calculated between the approach earthworks and the culvert location, resulting in a minimum transition length of 20m being adopted on either side of the founding stratum: ○ CH. 61+301 to 61+319: Transition zone; E&R changes from 2.0 to 3.75mbgl ○ CH. 61+319 to 61+341: Founding stratum of the culvert; E&R of 3.75mbgl ○ CH. 61+341 to 61+359: Transition zone; E&R changes from 3.75 to 2.0mbgl • A five-month Compliance Period is required for the mainline earthworks within the transition mitigation extent. • The Compliance Period is assumed to start after the full construction of the culvert and its foundation, following the full height construction of the mainline, surrounding landscaping earthworks and temporary surcharge. • The transition details specified in this Section of the GDR should be applied to the full width of the embankment base. • Material Class 1A1 is proposed as the transition fill zone material. • For a smooth transition within the upper embankment, UEF* material is specified around and above the culvert. Refer to Appendix C for an elevation view of the transition details.

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2.4.11 Dynamic Performance (Rayleigh Waves)

2.4.11.1 Introduction

The design has been undertaken in accordance with the Ground Engineering Technical Note – Guidance on the Assessment of Dynamic Performance of Earthwork Structures Ref. [12]. This section briefly outlines the procedure that has been followed and is specific to this asset.

The Earthworks Technical Standard Ref. [3] Table 4.2/1 sets a minimum Rayleigh Wave velocity to be achieved at the formation level of 160 m/s (1.6 multiplied by 100 m/s, the design speed) Ref. [3]. This will be determined by pre-construction validation testing at the formation level and/or by asset-specific field trials.

2.4.11.2 Basis of Design

The dynamic performance of earthwork is dependent on the small strain stiffness of each layer, with layers closest to the surface having the largest influence on the performance, reducing to negligible at depths greater than about 10m. An assessment of the stiffness has been undertaken of each of the material types below the embankment and foundation to allow an assessment of the expected dynamic performance of the completed earthwork. Figure *2-9* summarises the design process for establishing the requirement for embankment foundation treatment to manage the potential low dynamic performance.

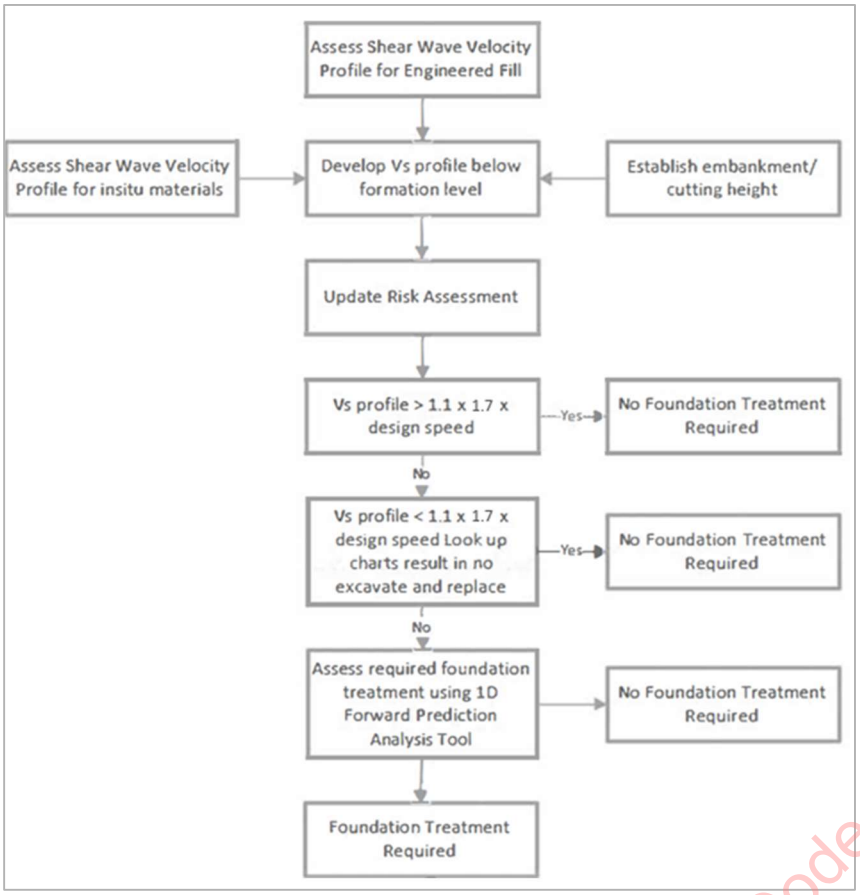


Figure 2-9: Dynamic Performance Foundation Treatment Design Process

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Initially the 1D tool assesses the Rayleigh Wave velocity at formation level to determine if excavate and replacement (E&R) is required. If the minimum Rayleigh Wave velocity is not achieved with the existing shear wave profile, the 1D tool will automatically run the model with E&R increasing in depth until the Rayleigh wave velocity is satisfied.

2.4.11.3 1D Forward Prediction Analysis Tool (1D Tool)

The 1D Forward Prediction Analysis Tool (1D Tool) is used to assess the dynamic performance of the ground below formation level, and to determine the requirement for Excavate and Replace mitigation.

The HS2 Rayleigh Wave Criterion has been determined based on the following assumptions:

- The Design Speed (DS) of the HS2 track = 100 m/s.
- The Rayleigh Wave velocity performance requirement from the Technical Standard Table 4/2.1 Ref. [12] is given as 1.6*DS = 160 m/s.
- A 1.1 factor is applied to the Rayleigh Wave velocity performance requirement to determine the target HS2 Rayleigh Wave Criterion, based on the Design Basis Statement for Dynamic Performance Ref. [12] = 176 m/s.

The parameters used for input into the 1D-Tool are presented in Table 2-11.

Table 2-11: Input Parameters for the 1D-Tool

Input Parameter	Input Value/Range
Frequency Range (Hz)	10 – 50
Replacement Fill Vs (m/s) (1)	250
HS2 Rayleigh Wave Criterion (m/s)	176
Poisson Ratio of Fill	0.25
Increment Step of the Replacement Material (m)	0.1
Version of the 1D-Tool	V1.0

Notes:

1. The Replacement Fill shear wave velocity design value is representative of Class 1A1, Class 3A1 and Class 3A3 materials

2.4.11.4 Asset Description and Ground Models

The ground model at Aylesbury Embankment has been subdivided into three separate models, shown in Figure 2-10, for the dynamic performance analysis. The northern ground model spreads from Ch. 61+763 to 61+500, the central ground model from Ch.61+500 to 60+900, and the southern ground model from Ch. 60+900 to 60+500, respectively. The embankment height at Aylesbury Embankment varies between 0.2 m and 6.4 m above existing ground level, as shown in Figure 2-10. An additional profile has been plotted, rounding down the topographic profile to the next 0.5 m, for comparison with the 1D tool output.

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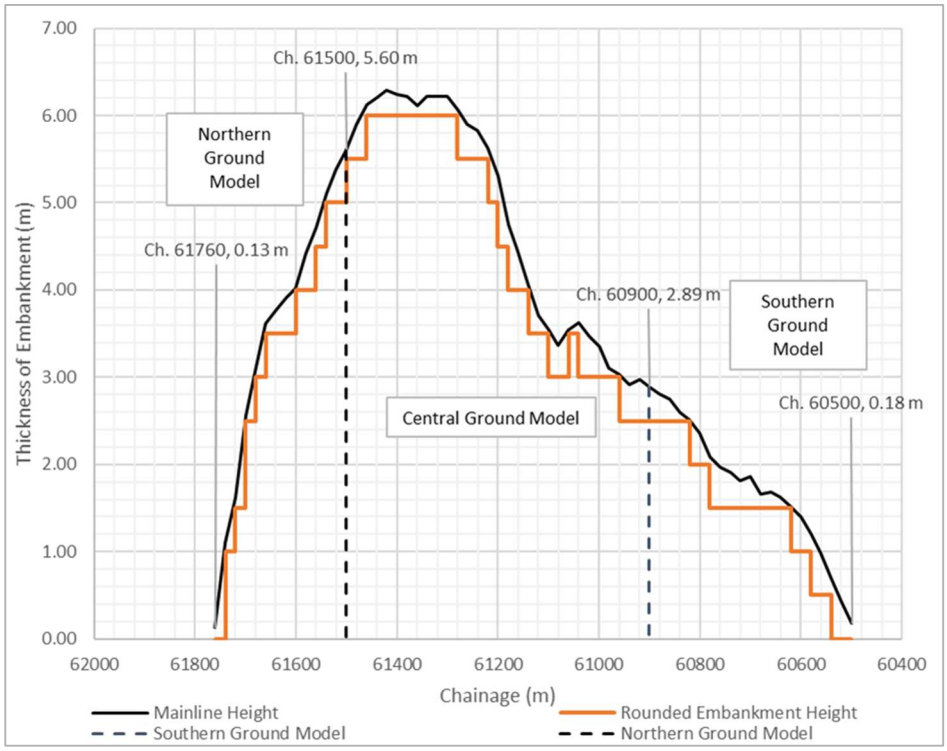


Figure 2-10: Mainline Embankment Thickness Profile of Aylesbury Embankment

At Aylesbury Embankment, nineteen MASW and four CPT direct testing data are available within the extent and immediate vicinity of the asset. Table 2-12 shows the geophysical testing locations used in the analysis.

Table 2-12: Summary of Geophysical Testing for Rayleigh Wave Design

Test Type	Location Reference	Ground Profile	Notes
Testing Within Asset Limits (Ch 60+500 – Ch 61+763)			
Seismic Cone Penetration Tests (SCPTs) (1)	ML060-CT422 ML060-CT425	Southern Ground Model	Direct measurement of V_s
	ML061-CT440A	Northern Ground Model	Direct measurement of V_s. Location consists of two readings.
Multi-channel Analysis of Surface Waves (MASW) (2)	ML060-MA426	Southern Ground Model	Direct measurement of V_s
	ML061-MW439	Northern Ground Model	Direct measurement of V_s
	ML060-MA400A to ML060-MA400N	Southern Ground Model	Direct measurement of V_s
Testing Outside of Asset Limits			

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Test Type	Location Reference	Ground Profile	Notes
Seismic Cone Penetration Tests (SCPTs)	ML060-CT422	Southern Ground Model	Direct measurement of V_s. 10 m offset (Chainage)
Multi-channel Analysis of Surface Waves (MASW)	ML061-MW441	Northern Ground Model	Direct measurement of V_s
	ML061-MA434 ML061-MA427	Northern Ground Model	Direct measurement of V_s. 152 m offset (Chainage) 55 m offset (Chainage)

Notes:
1. SCPT: Seismic Cone Penetration Tests
2. MASW: Multi-channel Analysis of Surface Waves
3. Combined Vs plots for the MASW data and CPT data are presented in Figure 2-11 and Figure 2-12

2.4.11.5 Shear Wave Velocity Profiles

The velocity profiles have been determined by plotting the geophysical shear wave velocity testing results against depth and taking a cautious estimate of the mean profile. Figure 2-11 shows the Southern Ground model with a design line based on both MASW and CPT data.

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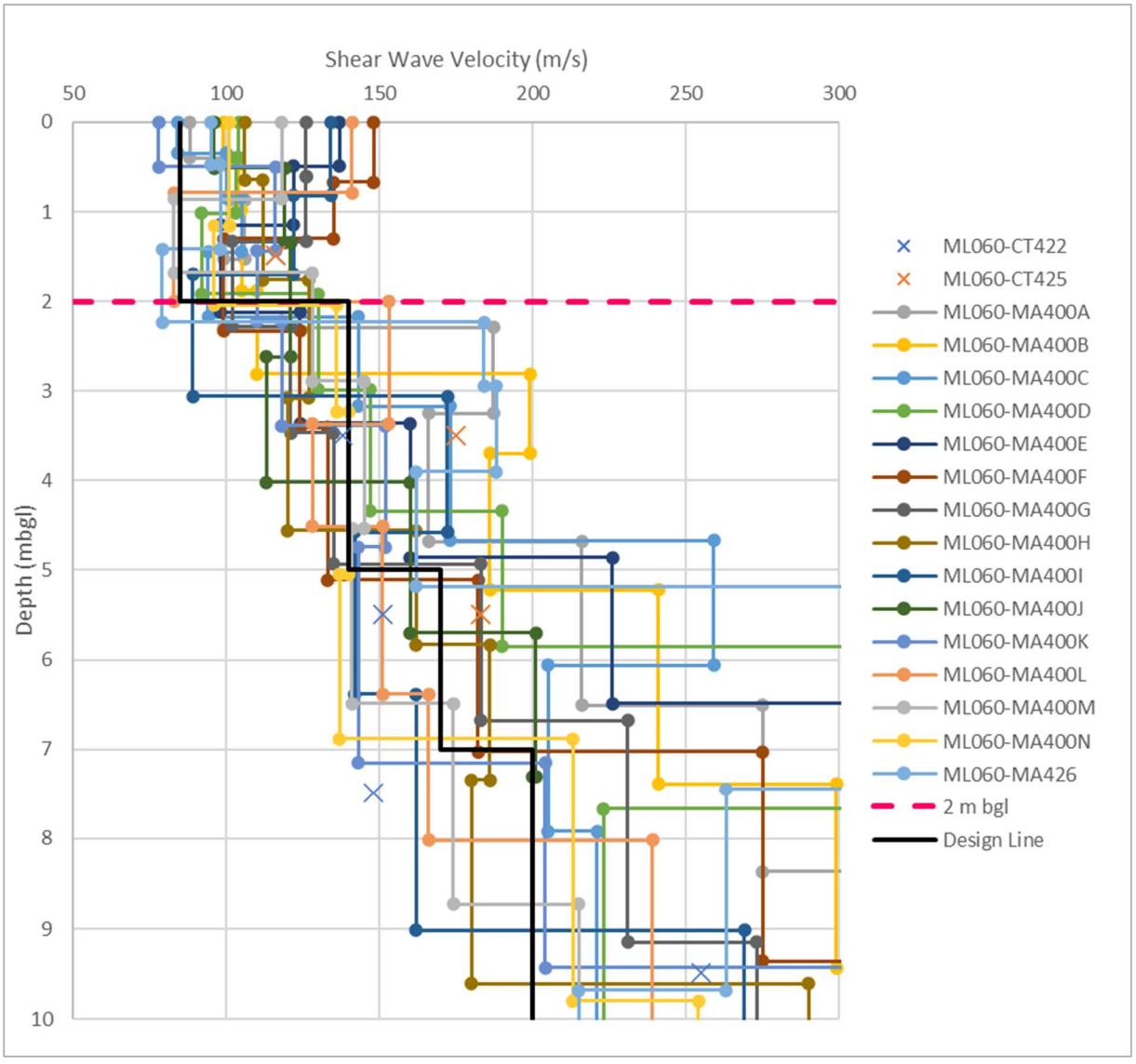


Figure 2-11 Shear Wave Velocity Profile of the Southern Ground Model

Figure 2-12 shows the Northern Ground model with a design line based on both MASW and CPT data. MASW data taken from borehole ML061-MA434 is located outside the boundaries of Aylesbury Embankment, but after a review of the borehole log, it is regarded as representative of the Northern Ground Model.

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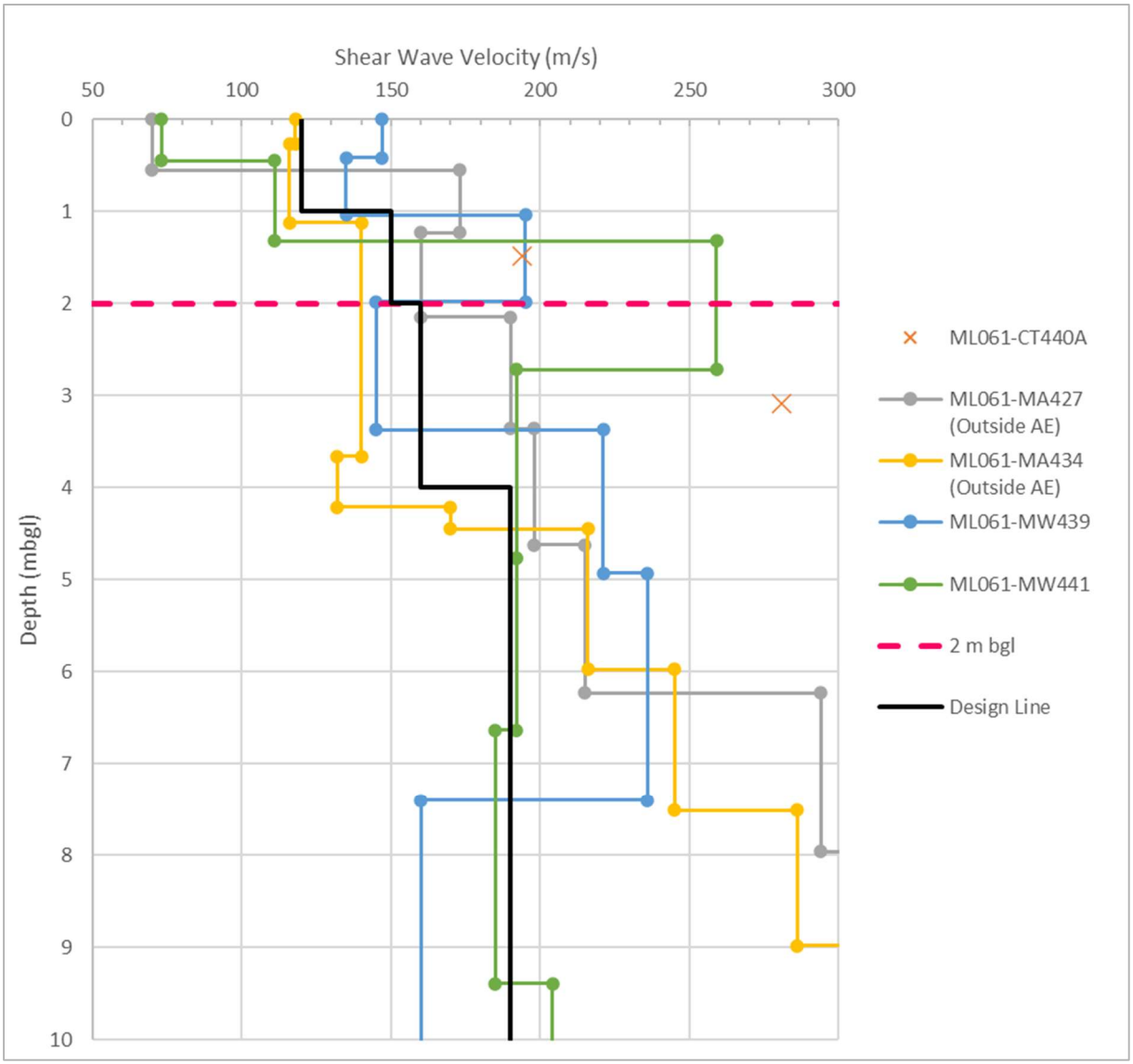


Figure 2-12 Shear Wave Velocity Profile of the Northern Ground Model

The central ground model has no direct testing. The ground profile in this central section consists of approximately 2.0 m of Alluvium over Gault, Purbeck, or Portland Sand/Stone formation in varying locations. In this area, the embankment ranges between 2.90 m and 5.60 m in thickness, representing a reasonable amount of material with a high transmission velocity (250 m/s) to disperse Rayleigh Waves emitted by the train.

To demonstrate the effectiveness of the embankment at dispersing Rayleigh waves, an overly conservative ground model was proposed using lower bound values from the neighbouring ground models and assigning the Alluvium a shear wave velocity that would be expected to be removed. The profile is tabulated in Table 2-13.

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Table 2-13: Shear Wave Velocity Profile for the Central Ground Model

Depth (m)	Geology	Shear Wave Velocity (m/s)	Justification
0 -1	Alluvium	70	Scheme design shear wave velocity is provided in the Rayleigh Wave Risk Assessment and Mitigation Design [22] as 70+10z m/s.
1 - 2		80	
2 – 4	Purbeck Formation	120	Selected as a lower bound value from the Gault Clay in the southern ground model, which ranges between 113 m/s and 221 m/s
4 - 6		150	The lower bound shear wave velocity in the Gault Clay increases with depth.
6 - 10		180	

The required depth of the foundation treatment for Rayleigh Wave depends on the small strain stiffness of the in situ material below the platform level and the stiffness of the replacement material.

2.4.11.6 Mitigation Solution

A sensitivity analysis was undertaken using the 1D tool for varying heights of the mainline embankment to assess the potential ranges of foundation treatment. The results are presented in Table 2-14.

Table 2-14: 1D Analysis Tool Output

Mainline Embankment Height (m)	Southern Ground Model (1) Excavation (mbgl)	Central Ground Model (2) Excavation (mbgl)	Northern Ground Model (3) Excavation (mbgl)
At-Grade	2.3	N/A	1.5
0.5 m Embankment	1.8	N/A	1.2
1.0 m Embankment	1.2	N/A	0.4
1.5 m Embankment	1.6	N/A	0.9
2.0 m Embankment	1.4	N/A	0.7
2.5 m Embankment	1.0	1.7	0.2
3.0 m Embankment	N/A	1.4	None
3.5m Embankment	N/A	1.1	None
4.0 m Embankment	N/A	0.9	None
4.5 m Embankment	N/A	0.6	None
5.0 m Embankment	N/A	0.3	None
5.5 m Embankment	N/A	None	N/A

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Mainline Embankment Height (m)	Southern Ground Model (1) Excavation (mbgl)	Central Ground Model (2) Excavation (mbgl)	Northern Ground Model (3) Excavation (mbgl)
6.0 m Embankment (4)	N/A	None	N/A

Notes:

1) The rounded embankment thickness in the southern ground model ranges between being at grade (0 m) and 2.5 m.
2) The rounded embankment thickness in the central ground model ranges between 2.5 m and 6.0 m.
3) The rounded embankment thickness in the northern ground model ranges between being at grade (0 m) and 5.0 m.
4) The highest modelled height of the embankment is rounded down to 6.0 m.

Figure 2-13 presents the Excavate and Replace (E&R) mitigation predicted by the 1D tool plotted against the height of the Mainline Embankment.

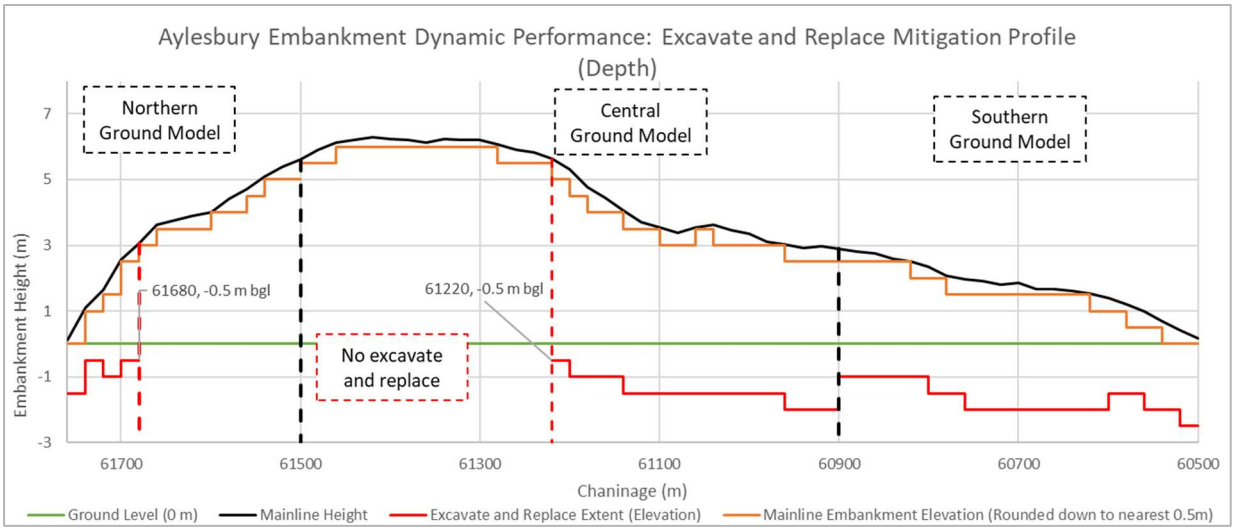


Figure 2-13: Excavate and Replace Predictions against Mainline Embankment Height.

Table 2-15 summarises the minimum excavate and replace mitigation required to meet the dynamic performance criteria per the Technical Standard. This table gives the maximum excavate and replace in the range of chainage given.

Mitigation solutions may be developed further (including opportunities) when additional information becomes available from advanced site trials, asset-specific demonstration trials and/or direct testing of formation (during construction). The requirements for demonstration trials, where various combinations of fill type, fill depth and sub-grade are tested, along with direct geophysical testing to be undertaken at the formation level to determine Rayleigh Wave velocity as per HS2 requirements, are detailed in the Series 600 Specification (App. 6/1 and App. 6/7 HSR).

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Table 2-15: Minimum Mitigation to Achieve Dynamic Performance Requirements in Relation to Mainline Embankment Height.

Chainages	Mainline Embankment Height (Existing to FL)	Minimum Dynamic Performance Mitigation (mbgl)
Ch. 60500 - Ch. 60540	0.00 m - 0.50 m	2.5
Ch. 60540 - Ch. 60580	0.50 m - 1.00 m	2.0
Ch. 60580 - Ch. 60620	1.00 m - 1.50 m	2.0
Ch. 60620 - Ch. 60780	1.50 m - 2.00 m	2.0
Ch. 60780 - Ch. 60820	2.00 m - 2.50 m	1.5
Ch. 60820 - Ch. 60960	2.50 m - 3.00 m	1.5
Ch. 60960 - Ch. 61040	3.00 m - 3.50 m	1.5
Ch. 61040 - Ch. 61060	3.50 m - 3.00 m	1.5
Ch. 61060 - Ch. 61100	3.00 m - 3.50 m	1.5
Ch. 61100 - Ch. 61140	3.50 m - 4.00 m	1.5
Ch. 61140 - Ch. 61180	4.00 m - 4.50 m	1.0
Ch. 61180 - Ch. 61200	4.50 m - 5.00 m	1.0
Ch. 61200 - Ch. 61220	5.00 m - 5.50 m	0.5
Ch. 61220 - Ch. 61280	5.50 m - 6.00 m	-
Ch. 61280 - Ch. 61460	6.00 m - 5.50 m	-
Ch. 61460 - Ch. 61500	5.50 m - 5.00 m	-
Ch. 61500 - Ch. 61540	5.00 m - 4.50 m	-
Ch. 61540 - Ch. 61560	4.50 m - 4.00 m	-
Ch. 61560 - Ch. 61600	4.00 m - 3.50 m	-
Ch. 61600 - Ch. 61660	3.50 m - 3.00 m	-
Ch. 61660 - Ch. 61680	3.00 m - 2.50 m	-
Ch. 61680 - Ch. 61700	2.50 m - 1.50 m	1.0
Ch. 61700 - Ch. 61720	1.50 m - 1.00 m	1.0
Ch. 61720 - Ch. 61740	1.00 m - 0.00 m	1.5

An E&R of between 0.5 and 2.5mbgl is required for dynamic performance mitigation for all earthworks within the Aylesbury Embankment asset. The 1-D Tool plots are presented in Appendix C. These requirements have been taken forward to the combined mitigation measures summarised in Section 2.4.14.

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2.4.12 Prepared Subgrade and Protection Layer

The subgrade analyses have been undertaken in accordance with the design approach set out in the Ground Engineering Design Basis Statement – Earthworks Ref. [8].

In accordance with the HS2 Technical Standard - Earthworks Ref. [3], mitigation measures are to be applied where the subgrade does not achieve the required static stiffness determined by Modulus of Deformation (E_{V2}). Key requirements from HS2 standards and associated proposed mitigation measures to be applied where required are listed in Table 2-16.

Table 2-16: Subgrade Requirements

Elements		Design Assumption (1)	Comments / Justifications
Subgrade Design	Existing foundation classification	QS0/QS1	Soft Superficial Deposit over Gault formation and Portland Sand/Stone formation, > 15% fines, E_{V2} < 60MPa (E_{V2} = modulus of deformation)
	Foundation treatment	Replace with QS2 stabilized to 2.0mbFL	Excavate and replace to achieve E_{V2} > 60MPa within materials 2m below formation level in accordance with Table 4.2/2 of HS2 Technical Standard - Earthworks Ref. [3]
	Prepared subgrade	0.40m (Minimum thickness)	Class 6F8 or 6F9 material in accordance with SCEW Series 600 Earthworks Ref. [23]. Thickness in accordance with Table 4.2/2 of HS2 Technical Standard - Earthworks Ref. [3]

Note:

(1) The materials are in accordance with 1MC06-CEK-GT-REP-C002-000029 Rev P04 - Definition of Scheme Design - Earthwork Material Re-Use [68].

In the absence of specific plate load test data from which the Modulus of Deformation (E_{v2}) may be measured directly, the E values reported in Appendix B suggest that the foundation materials will not achieve the required stiffness of 60MPa at depths of less than 2m below formation level.

It will, therefore, be necessary to Excavate and Replace the foundation materials with Class 1A1 (or Class 3A1 or Class 3A3) Fill material to a depth of 2m below formation level where the embankment height is less than 2m.

A summary of mitigation requirements based on this criterion is included in Table 2-17.

Table 2-17: Subgrade Mitigation

Chainage (From)	Chainage (To)	Depth of Excavation and Replace
61+763	61+740	2.0 mbFL (2.5 to 2.0 mbgl)
61+740	61+720	2.0 mbFL (2.0 to 1.5 mbgl)
61+720	61+700	2.0 mbFL (1.5 to 1.0 mbgl)

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Chainage (From)	Chainage (To)	Depth of Excavation and Replace
61+700	61+680	2.0 mbFL (1.0 to 0.0 mbgl)
61+680	60+900	None
60+900	60+820	2.0 mbFL (0.0 to 0.5 mbgl)
60+820	60+740	2.0 mbFL (0.5 to 1.0 mbgl)
60+740	60+600	2.0 mbFL (1.0 to 1.5 mbgl)
60+600	60+560	2.0 mbFL (1.5 to 2.0 mbgl)
60+560	60+500	2.0 mbFL (2.0 to 2.5 mbgl)

The requirements for subgrade mitigation have been taken forward into the combined mitigation measures summarised in Section 2.4.14.

2.4.13 Risk Management

2.4.13.1 Introduction

A summary of the key potential geotechnical geohazards that may affect Aylesbury Embankment is provided in Table 2-18. The geohazards have been identified from the following:

- Geotechnical Desk Study Ref [24]
- Section 1 GIR Ref [25]
- HS2 Geomorphology Assessments Ref [26]
- Supplementary GIR Ref [7]

Table 2-18: Geohazards at Aylesbury Embankment

Risks	Description
Faulting	5 No. potential faults are located at approximate Ch 60+650, 060+875, 061+075, 061+180, and 061+250. There is potential for further unrecorded faulting near the Sedrup North Culvert area.
Dissolution Features	The presence of limestone within the PB, POST/POSA within an area of an active water course at Ch. 61+375 may lead to dissolution features. There is no significant risk at Aylesbury Embankment.
Ground Gas	Strata encountered within this asset are known to produce hydrocarbons naturally.
Hard Ground / Flint Bands	Bands of limestone, sandstone and siltstone were recorded within shallow POST and KC, as well as bands of sandstone within POST and POSA geological units along the asset. Chert beds are noted within ML061-CR012 and ML061-WS025 to 2.20 mbgl. Potentially resulting in difficulties in excavation / piling of this material. Harder and softer ground can

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Risks	Description
	lead to the generation of hard spots due to their widely variable strength and consolidation characteristics.
High / Shallow Groundwater	Monitoring data records groundwater levels ranging between 0.0 and 4.1mbgl, potentially affecting temporary works and the permanent design.
Chemically Aggressive Ground	A potential risk for elevated sulphates, sulphides, chlorates, and chloride has been identified within the Kimmeridge Clay formation and Gault Clay Formation, which may attack buried concrete structures such as culverts. Conservative design undertaken and DS, ACEC classes assigned.
Rayleigh Waves	Amplification of ground displacements associated with ground borne vibrations can occur causing track deterioration, increased maintenance and may lead to track failure and derailment in extreme cases. Material descriptions at shallow depths along the whole of the asset indicate soft and firm clay which are susceptible to Rayleigh waves. The detailed design proposes mitigation in the form of Excavate and Replace, reducing the risk.
Relic Shear Surfaces	Fissured clays and polished surfaces have been identified in the geological long section across the asset within the Kimmeridge Clay Formation and Gault Clay Formation. Several Level 2 assets, namely the culverts, access roads, mainline embankment, and landscaping bunds, may be vulnerable to ground instability due to the presence of relict periglacial features. For mitigation measures, see details in section 2.4.13.2.
Shrink / Swell Potential	Potential for shrink / swell behaviour in the Kimmeridge Clay, Gault Clay and Head deposits identified in several Level 2 assets (culverts, overbridges, and embankment) due to changes in moisture content in high plasticity clays, which could cause unacceptable total or differential settlements. The detailed design proposes mitigation in the form of Excavate and Replace, reducing the risk.
Soft / Compressible Ground	Soft ground within the Head deposits, Alluvium, Gault Clay, POST/POSA and Kimmeridge Clay soils could cause instability of embankment slopes and/or unacceptable total or differential settlements/ insufficient stiffness/ dynamic performance issues. Potential reduction in the strength of Head deposits, Alluvium, Gault Clay, POST/POSA and Kimmeridge Clay soils is likely caused by exposure to rainfall during excavation works. The detailed design proposes mitigation in the form of Excavate and Replacing the soft/compressible ground, thus reducing the risk.
Seismicity	A number of faults are present within the Level 1 asset footprint that present a risk of seismic activity that could result in embankment and cutting instability.
Flood Risk	There is no risk of flooding to the embankment footprint as the Sedrup North watercourse is moved away at a minimum offset of 8m from the toe.
Loose / Unstable ground	There is the potential for loose sands and soft clays to be present. This could lead to the loss of foundation support and excavations to collapse. The detailed design proposes mitigation in the form of Excavate and Replacing the loose sands, thus reducing the risk.

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Risks	Description
Water Bearing Soils / Rock	Running sand conditions within Head deposits, Portland Sand Formation and the Purbeck Group combined with high groundwater strikes present a risk of instability and water ingress to excavations. Hydro-geological impacts are considered negligible at Aylesbury Embankment.
Groundwater Resources	Excavation may lead to groundwater drawdown in Portland Group which is designated as a principal/secondary aquifer. Hydro-geological impacts are considered negligible at Aylesbury Embankment.
Cryoturbation Features	The presence of cryoturbation can lead to soft and disturbed ground, weaken the in situ geology, and generate disturbed soil potentially with shear surfaces. For mitigation measures, see details in section 2.4.13.2.

Note:
Refer to section 2.4.3 for a description of settlements on a third-party gas main.

A few of the geohazards discussed in Table 2-18 are difficult to account for in design at this stage and before the commencement of the construction. These will need to be addressed as and when any additional information is obtained from the on-site observations and inspections, which will be routinely carried out during the construction stage, and for which the Contractor is responsible for the reporting and management. Where required, the Contractor will need to produce Risk Assessments and Method Statements (RAMS) along with any supplementary Contractor's Monitoring Plans (where there are instrumentation and monitoring proposals) for approval by the Designer.

The proposed mitigation measures, which are required to eliminate or reduce the above risks to an acceptable level, are summarised in Appendix D.

The following sections summarise the principal risks identified from our current understanding of the subsurface conditions along the asset and indicate the type of mitigation that may need to be deployed should the particular hazard be encountered during construction and identified as a risk to the design and performance of the asset.

2.4.13.2 Mitigation for Relict Shear/Slip Surfaces

The Aylesbury Embankment has been identified as 'at risk' from the possible presence of relict shear surfaces and, as a result, has been assigned a risk classification of 'Potential Level 1 – Moderate Risk' [25]. For a detailed account of the methodology of risk assessment reference should be made to the Supplementary GIR, Ref. [7].

Management of the risk of relict shear surfaces follows what is set out in the SMUGC document. The contractor shall submit Risk Assessments and Method Statements outlining the risk management for approval by the designer. Asset-specific solutions must be confirmed at site level, with the instrumentation and monitoring requirements to address the risk set out in the Designer Monitoring Plan (DMP), Ref. [27].

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The contractor will produce the Contractor Monitoring Plan (CMP). The following steps have been completed/need to be actioned with respect to the above:

1. Detailed studies of available information at Scheme Design – **Completed**.
2. Targeted Detailed Design ground investigation scope – **Completed**.
3. Development of route-wide/ standard design solutions to eliminate and/or minimise the risk to the asset from the effects of reduced shear strength layers (*i.e.*, E&R) – **Completed**.
4. Geological mapping during construction – **Not started**.
5. Construction ground investigation – **Not started**.
6. Review Detailed Design solutions where relict shear surfaces are encountered. Undertake further calculations to determine any further specific mitigation requirements (*e.g.*, E&R tracing relict shear surfaces beyond the Track Support Zone and embankment footprint), as appropriate – **Not started.**
7. Monitoring of ground movement during construction – **Not started**.
8. Monitoring of ground movement during operation (if required) – **Not started**.

As part of the assessments presented in this GDR, there have been no opportunities to further inspect the asset to understand the risks from relict shear surfaces. Therefore, the potential risk of encountering relict shear surfaces in the shallow weathered materials during construction suggests the need for further, as yet undefined, mitigations.

A typical solution for a relict shear surface located at depth may include constructing a stabilising blanket layer within the cutting slope profile (temporary slope of E&R). The mitigation solution and minimum dimensions associated with a deep potential relict shear surface, and the typical instrumentation and monitoring arrangement, are provided in the following Standard Earthworks drawings:

- G002 Typical HS2 Mainline Earthworks Section Embankment using Foundation Treatment for Relict Landslide Features (1), drawing no. 1MC06-CEK-GT-DSE-C002-000074 Ref [28]
- G003 Typical HS2 Mainline Earthworks Section Embankment Instrumentation and Monitoring Arrangement (1), drawing no. 1MC06-CEK-GT-DSE-C002-000076 [29]

An asset-specific solution will be required depending on the extent of the disturbed material and/or the space available to accommodate the solution. The final detail and extent of the mitigation solution will be subject to further review upon completion of site walkovers, additional construction GI and/or the observational approach during construction, with the latter under the supervision of a suitability qualified and experienced geotechnical engineer.

A summary of the relict slip assessment and mitigation provision is provided in Table 2-19.

Table 2-19: Relict Shear Surface Mitigation Summary

Chainage	Location	Risk Level	Mitigation Provision	Length of Mitigation Provision (m)
60+500 to 61+763	Embankment foundation	Potential Level 1	Provide stabilising blanket layer to the depth of the relict material identified	To be determined on-site during construction

2.4.13.3 Groundwater Management

The groundwater monitoring indicates shallow groundwater present across the asset. As mentioned previously, for the purpose of Detailed Design, the groundwater level has been assumed at the existing ground level. Groundwater ingress is not relevant to the embankment design. However, groundwater control

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(such as sump pumping) may need to be considered as part of the temporary works design for the E&R zones and any other 'below ground' construction activities.

2.4.13.4 CDM Risk Register

In accordance with the requirements of the Lead Designer under the CDM Regulations, 2015, a Designer's Risk Assessment (DRA) has been undertaken in connection with the geotechnical design of the mainline and landscaping earthworks.

Reference should be made to the CDM Risk Register, Ref. [30], for details of this earthwork design's general and asset-specific risks.

2.4.14 Design Solution

Table 2-20 provides a summary of the mitigation solutions presented in the previous sections for the Mainline Earthworks assets.

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Table 2-20: Summary of Proposed Mitigation Measures

Element Name	Mainline Embankment						UTX	Mainline Embankment								Culvert Transition	Sedrup North Culvert	Culvert Transition
Chainages	61+763 to 61+740	61+740 to 61+720	61+720 to 61+700	61+700 to 61+680	61+680 to 61+660	61+660 to 61+650	61+650	61+650 to 61+620	61+620 to 61+580	61+580 to 61+540	61+540 to 61+500	61+500 to 61+480	61+480 to 61+440	61+440 to 61+420	61+420 to 61+360	61+360 to 61+339	61+339 to 61+321	61+321 to 61+300
Mainline Embankment Height (1)	0.0m to 0.5m	0.5m to 1.0m	1.0 to 1.5m	1.5m to 2.5m	2.5m to 3.0m	3.0m to 3.5m		3.5m to 4.0m	4.0m to 4.5m	4.5m to 5.0m	5.0m to 5.5m	5.5m to 6.0m	6.0m to 6.5m	6.5m to 6.0m				
Mainline Embankment Slope Gradient	1V:2H																	
Mainline Embankment Slope Mitigation (2),(4)	2.0 mbgl E&R across full width of embankment																	
Alluvium Extents Mitigation within footprint of Mainline Embankment	None																	
Landscape Bund Slope Gradient	1V:4H																	
Landscape Bund Slope Mitigation (3)	Upside Landscape Bund (Ch. 61+200 to 61+763) and Downside Landscape Bund (Ch. 61+580 to 61+763): Excavate until a minimum undrained shear strength of 50kPa has been achieved. Replace with Class 2A1 fill to the finished ground level.																	
Subgrade Stiffness Mitigation (2)	2.0 mbFL [2.5 to 2.0 mbgl]	2.0 mbFL [2.0 to 1.5 mbgl]	2.0 mbFL [1.5 to 1.0 mbgl]	2.0 mbFL [1.0 to 0.0 mbgl]	None													
Settlement Mitigation (2),(5),(6),(7)	2.0 mbgl E&R with a minimum Compliance Period of **5 months.** Preload of 15kPa is required during the Compliance Period, following the complete construction of mainline and landscaping earthworks to full height.																	
Dynamic Performance Mitigation - Mainline (E&R) (2)	1.5 mbgl	1.5 mbgl	1.0 mbgl	1.0 mbgl	None													
Relic Shear Surfaces Mitigation (8)	Slope stabilisation for relict shear surfaces to be determined on site (observational method)																	
Mainline Earthworks Transition to Structures (2), (11), (12)	None															2.0 to 3.75 mbgl E&R with a minimum Compliance Period of **5 months.**	3.75 mbgl E&R	3.75 to 2.0 mbgl with a minimum Compliance Period of **5 months.**

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Element Name	Mainline Embankment		UTX	Mainline Embankment	Culvert Transition	Sedrup North Culvert	Culvert Transition
Combined Mitigation (2),(5),(6),(7),(9),(10)	2.5 to 2.0 mbgl E&R across full width of embankment with a Compliance Period of **five months.**	2.0 mbgl E&R across the full width of the embankment with a minimum Compliance Period of **five months.**	2.0 mbgl E&R across the full width of the embankment with a minimum Compliance Period of **five months**	2.0 mbgl E&R across the full width of the embankment with a minimum Compliance Period of **five months.**	2.0 to 3.75 mbgl E&R across full embankment width with a Compliance Period of **five months.**	3.75 mbgl E&R	3.75 to 2.0 mbgl E&R across full embankment width with a Compliance Period of **five months.**

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Table 2-20: Summary of Proposed Mitigation Measures (continued)

Element Name	Mainline Embankment							Bridleway SBH/19 Overbridge	Mainline Embankment								
Chainages	61+300 to 61+280	61+280 to 61+220	61+220 to 61+200	61+200 to 61+160	61+160 to 61+140	61+140 to 61+100	61+100 to 60+920	60+920	61+920 to 60+900	60+900 to 60+820	60+820 to 60+780	60+780 to 60+740	60+740 to 60+600	60+600 to 60+560	60+560 to 60+540	60+540 to 60+520	60+520 to 60+500
Mainline Embankment Height (1)	6.0m to 5.5m	5.5m to 5.0m	5.0m to 4.5m	4.5m to 4.0m	4.0m to 3.5m	3.5m to 3.0m	3.0m to 2.5m		2.5m to 2.0m	2.0m to 1.5m			1.5m to 1.0m	1.0m to 0.5m	0.5m to 0.0m		
Mainline Embankment Slope Gradient	1V:2H																
Mainline Embankment Slope Mitigation (2), (4)	2.0 mbgl E&R across full width of embankment			2.0 mbgl E&R within the track support zone			2.0 mbgl E&R within the track support zone										
Alluvium Extents Mitigation within footprint of Mainline Embankment (13)	None			Removal of Alluvium beyond the track support zone by extending the mitigation extents to the entire base of the embankment.			None										
Landscape Bund Slope Gradient	1V:4H																
Landscape Bund Slope Mitigation (3)				No mitigation required.													
Subgrade Stiffness Mitigation (2)	None								2.0 mbFL [0.0 to 0.5 mbgl]	2.0 mbFL [0.5 to 1.0 mbgl]			2.0 mbFL [1.0 to 1.5 mbgl]	2.0 mbFL [1.5 to 2.0 mbgl]	2.0 mbFL [2.0 to 2.5 mbgl]		
Settlement Mitigation (2), (5),(6),(7)	2.0 mbgl E&R with a minimum Compliance Period of 3 months. Preload of 15kPa is required during the Compliance Period, following the complete construction of mainline and landscaping earthworks to full height.																
Dynamic Performance Mitigation - Mainline (E&R) (2)	None	0.5 mbgl	1.0 mbgl	1.0 mbgl	1.5 mbgl	1.5 mbgl	1.5 mbgl	1.5 mbgl	1.5 mbgl	1.5 mbgl	2.0 mbgl	2.0 mbgl	2.0 mbgl	2.0 mbgl to 2.5 mbgl		2.5 mbgl	
Relic Shear Surfaces Mitigation (8)	Slope stabilisation for relict shear surfaces to be determined on site (observational method)																
Mainline Earthworks Transition to Structures	None																
Combined Mitigation (2), (5),(6),(7),(9),(10)	2.0 mbgl E&R across the full width of the embankment with a compliance period of **three months.**						2.0 mbgl E&R within the track support zone with a compliance period of **three months.**								2.0 to 2.5 mbgl E&R within the track support zone. A minimum of **three months** compliance period is required.		2.5 to 2.8 mbgl E&R within the track support zone. A minimum of **three months** compliance period is required.

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Notes:

(1) The Embankment height is the level difference between the top of the protection layer and the existing ground level, as indicated in the Report of Cross-sections, Ref. [31].
(2) The 'm bgl' and 'm bfl' refer to mitigations (E&R), presented as metres below ground level and metres below formation level, respectively.
(3) Within the extent of Alluvium from approximately Ch. 61+200 to 61+763, E&R mitigation is required for the landscaping earthworks. The depth of the mitigation depends on the termination criteria of a minimum undrained shear strength of 50 kPa. All the soft materials within the landscape bund footprint are removed by strength tests performed during site works and replaced by Class 2A1 fill over the full bund width.
(4) To ensure slope stability of the mainline earthworks in the short term, the loading should be applied with a minimum offset of 0.5m from the crest of the slope during construction, prior to adjacent landscaping slopes being built to at least the same height as the mainline earthworks.
(5) A Wait Period of two months is required for the entire asset.
(6) A Compliance Period of three months is required from Ch. 60+500 to 61+300 and five months from Ch. 61+300 to 61+763. It is assumed that the Wait/ Compliance Period starts after the full construction of the mainline and landscaping earthworks (to the full height) and the application of a temporary surcharge.
(7) Temporary surcharge of 15 kPa (equivalent to approximately 1m of soil surcharge), is required throughout the entire Compliance Period, where no track drainage installation can take place.
(8) Mitigation for relict slips and/ or shear surfaces is to be determined on-site by observational method during excavation and through visual monitoring as part of construction control. The contractor shall submit risk assessments and method statements outlining the management of the risk for approval by the designer. Typical treatment options are presented in the Standard Earthwork Drawings, 1MC06-CEK-GT-DSE-C002-000074, P03 Ref [28]and 1MC06-CEK-GT-DSE-C002-000076, C01 Ref. [29].
(9) All Mainline E&R is designed using material Class 1A1 (or Class 3A1 or Class 3A3). Refer to Appendix B for the geotechnical parameters of the fill materials adopted.
(10) Combined mitigation treatment details are presented in the Earthwork Report of Cross Sections, document number 1MC06-ASC-GT-DSE-CS04_CL18-000013, Ref. [31]
(11) E&R depths proposed are the minimum mitigation requirement. At the base of the E&R, a minimum Cu of 50kPa is required for the founding material compaction purposes. At the base of Sedrup North Culvert, a minimum stiffness parameter of Eu = 32 MPa is required below the founding material. This is localised to the natural soil below the founding stratum only. The natural ground below the culvert transition zones requires a minimum Cu of 50kPa, the same as the rest of the earthworks.
(12) The culvert transition details specified in this GDR are only required below the mainline earthworks (full width of the embankment). Despite the culvert running transversely below both the mainline and landscaping earthworks, no transition zone is required below the landscaping bunds. For details of backfill material requirements surrounding the culverts refer to Sections 2.4.10 of this GDR.
(13) The mitigation is extended beyond the track support zone, limiting to the embankment toe, to remove the loose materials within the mainline footprint.

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2.4.15 Drainage

The Aylesbury Embankment asset has four drainage catchments associated with it; these belong to two track drainage systems and one land drainage system:

- Land Drainage C2.L.26 Catchment;
- Land Drainage C2.L.22 Catchment;
- Track Drainage C2.T.21; and
- Track Drainage C2.T.23.

Further details are given in Section 6.

2.4.16 Durability

Few asset-specific samples were submitted for sulphate testing. Due to the limited Sulphate testing on the asset, generic results carried out at neighbouring assets were used to determine the design sulphate class and concrete classification. A summary of the route-wide geological unit testing is shown in Table 2-21.

Table 2-21: Summary of Soil Sulphate Chemistry

Geological Formation	Design Sulphate Class (DS)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) (%)	pH
Head Deposits	DS-1	AC-1s	0.20	7.7
Alluvium	No Testing Available			
Gault Clay Formation	DS-3	AC-3s	0.90	7.9
Whitchurch Sand Formation	DS-4	AC-3s	1.60	7.3
Purbeck Group	DS-4	AC-3s	1.5	7.4
Portland Sand/Stone Formation	DS-1	AC-1s	1.0	8.1
Kimmeridge Clay Formation	DS-5	AC-4s	12.2	7.6

Notes:

Design Sulphate (DS) classes and Concrete (ACEC) classes were determined based on Table C2 of BRE SD1, Ref. [32] for natural ground locations.

For suitability of treatment with lime/cement, the following guidance has been adopted for soils:

- TPS < 0.25% Low Risk
- TPS ≥ 1.0% Significant Risk

A review of Total Potential Sulphate (TPS) concentration results indicated that the Kimmeridge Clay Formation, Portland Sand/Stone Formation, Purbeck Group and Whitchurch Sand Formation possess significant risks (TPS > 1) and are therefore unsuitable for the use of stabilisation techniques for treatment with lime/cement, thus causing swelling. The buried concrete design should comply with the ACEC Class, as shown in Table 2-21.

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3 Non-mainline Earthworks

3.1 Introduction

This section of the report describes the approach adopted for the design of non-mainline earthworks including asset-specific information and a summary of the design solution. A summary of the non-mainline earthwork assets is shown below.

Table 3-1: Non-mainline Earthworks within Aylesbury Embankment.

Level 2 Non-mainline Asset ID	Level 2 Non-mainline Asset Name
HS2-000020RKL	Oat Close Bridleway (SBH/19) Realignment
HS2-000001403	Sedrup North Watercourse Diversion
HS2-00002BJSK	Maintenance Access Road to Pond (South Arm off Calley Farm)

3.2 General Definitions

Reference should be made to the calculation pack within Appendix E.1 for Maintenance Access Road to Pond (South Arm off Calley Farm) and Appendix E.2 for Oat Close Bridleway (SBH/19) Realignment for a full account of general definitions used.

3.3 Standards and Design Approach

Reference should be to the calculation pack within Appendix E for a full account of the design and the standards and ground models used for the design of highway earthworks.

3.4 Design Procedures

Reference should be made to the calculation pack within Appendix E for a full account of the design procedures and assumptions used for the design of highway earthworks.

3.5 IDR 208.4 - Maintenance Access Road to Pond (South Arm off Calley Farm – South)

3.5.1 Introduction

This section sets out an overview of the earthwork design undertaken for the non-mainline (highways) assets, to satisfy the requirements of the HS2 Technical Standard – Earthworks, Ref [3].

3.5.2 Standards and Design Approach

Reference should be made to the calculation pack within Appendix E.1 of this GDR for a full account of the design standards, design assumptions, constraints, ground models, design methodology and calculation output in associated with the design of highway earthworks.

3.5.3 Asset Description

This non-mainline highway assets is located between mainline Ch. 61+300 and 61+875m (SnakeGrid) within Section C2 (AA), centred at approximate National Grid Reference E 480731 N 211511 (OSGB) within the Aylesbury Embankment Level 1 Asset Group. Details of the asset are presented in Table 3-2.

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Table 3-2: Detailed Design Proposals – IDR208.4

Asset Description	Maintenance Access Road to Pond (South Arm Off Calley Farm - South)
Type	Carriageway
Start - End Mainline Chainage (approx.)	61+300 to 61+875m
Length (approx. m)	619
Carriageway/ Verge Width	3.5 / 1.5m
Max. Structure Height at Asset Chainage	2.5m at 61+825
Proposed Detailed Design Slope Angle	1V:3H
Landscape Bunds	Yes, left hand side of highway from Ch.61+580 to 61+850

The proposed extent of the assets is shown in Figure 3-1.

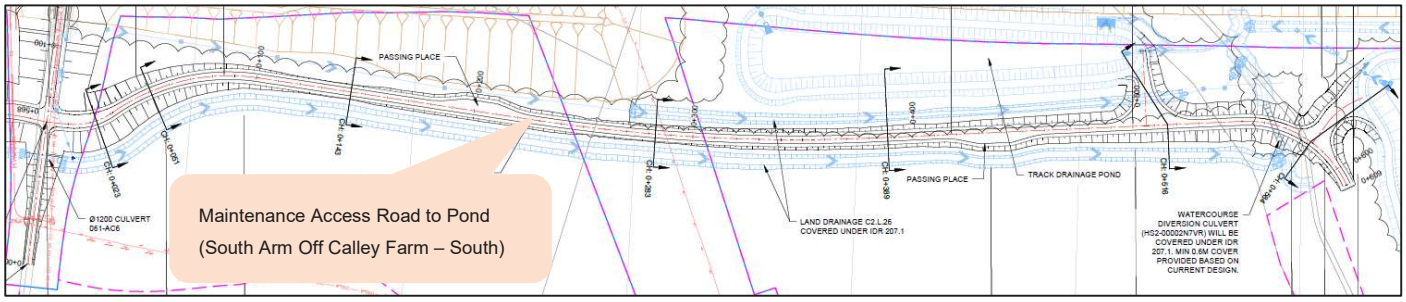


Figure 3-1: General Location Plan

3.5.4 Construction Method and Sequence

The construction sequence of the highway earthworks is yet to be confirmed by the Client. It is assumed that the highway earthworks and the adjoining landscape earthworks will be constructed simultaneously. If the highway earthworks are constructed first, then it is assumed that the adjoining landscape earthworks will be constructed within 2 years of the highway earthworks.

The highway earthworks are to be constructed using Class 2A4 / 7C3 materials.

3.5.5 3rd Party Asset Constraints

There is an existing high pressure gas main that passes beneath the Maintenance Access Road at approximate Ch 0+280m. At this location the highway is proposed to be at grade or on shallow embankments up to 0.75m in height. A 16m slab is proposed on top of the gas main under the proposed carriageway and drainage. For further details refer to the Aylesbury - FDR Pack (CSO-02-2202) drawings.

Aylesbury Embankment - Phase 2 Ground Movement Impact Assessment Report (1MC06-CEK-GT-REP-CS04_CL18-000042) provides further information regarding the ground movements within this asset.

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3.5.6 Ground Investigation

A total of 30 no. asset specific exploratory holes have been considered in the design for these assets, the location of which are shown on the plan included in the calculation pack in Appendix E.1. The L1 asset wide ground investigation data has also been considered during material parameter value derivation. The Uncertainty in Ground Conditions Classification for the asset group is Class 1, considering the available GI data and Geotechnical Category of the earthworks.

A list of the exploratory holes under consideration when deriving the ground model for this document are included in the tables in Appendix E.1.

3.5.7 Ground Models and Parameter values

The ground models were established considering the asset specific exploratory holes and the parameter values have been derived with reference to those adopted for the Aylesbury Embankment mainline. Plasticity Index was derived using the asset specific data. A summary of the ground models and parameter values developed for earthwork analyses is presented in Table 3-3 and Table 3-4.

Table 3-3: Ground Models

Ground Model	Strata	Depth Range (mbgl)
Ground Model 1 **Ch. 0+000 to 0+500**	Portland Group	0.00 – 7.50
	Kimmeridge Clay Formation	7.50 – 20.00+
Ground Model 2 **Ch. 0+500 to 0+619**	Alluvium	0.00 – 2.00
	Purbeck Group	2.00 – 4.50
	Portland Group	4.50 – 15.00
	Kimmeridge Clay Formation	15.00 – 20.00+

Notes:

Thin localised occurrences of Made Ground, Head and Alluvium deposits are encountered but is assumed will be excavated and replaced during construction

Table 3-4: Summary of Characteristic Parameter values for Slope Stability and Settlement Assessment

Strata/ Fill	PI (%)	Unit Weight γ (kN/m³)	c_u (kN/m²)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)
Alluvium	48[2]	18[3]	20[1 / 11]	21[5]	22[6]	0[7]
Purbeck Group	-	20[3]	80[1]	22[1]	24[1]	1[7]
Portland Group	25[2]	20[3]	45[4]	25[5]	27[6]	1[7]
Kimmeridge Clay Formation	35[2]	20[3]	40+8z (up to 150kNm²)[10]	23[5]	25[6]	1[7]
Class 2A4 / 7C3	-	20[8]	50 [10]	21[8]	23[8]	1

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Strata/ Fill	PI (%)	Unit Weight γ (kN/m³)	c_u (kN/m²)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)
Class 1A4	-	20[3]	-	-	35[9]	0

Notes:

1. Parameter values taken from Aylesbury Embankment due to limited available local data.
2. Plasticity Index derived using asset specific data, see Appendix E.1, Figure C1.
3. Unit weights (γ) of the in situ materials are determined based on the asset specific GI factual data and available supporting factual GI data, as appropriate. Reference should be made to the supplementary GIR / GDR.
4. Characteristic line from asset specific Cu data plot, see Appendix E.1, Figure C4. An f_1 (correlation factor between Cu and SPT 'N') value of 4.5 has been adopted for all cohesive materials, with reference to Stroud 1974.
5. Derived from BS8004:2015, Table 2, using the above-mentioned PI values.
6. ϕ'_{pk} values derived from ϕ'_{cv}, using a dilation value of 1° for Alluvium and 2° for Portland Group Kimmeridge Clay as per BS8004:2015.
7. Cohesion is considered to be 1kPa for cohesive soils and 0kPa for granular.
8. Class 2A4 / 7C3 Fill material consists of re-used Kimmeridge Clay Formation, parameter values based on L1 wide asset characteristic lines.
9. Class 1A4 values are as per the IDR 208.4 Specification (1MC06-CEK-AU-SPE-CS04-000003).
10. Z = depth measured from top of stratum.
11. soft soil which has a cu<20kPa is to be removed during construction.

3.5.8 Groundwater Conditions

There are 7 No. relevant monitoring locations. Considering the recorded high groundwater level, a design groundwater level has been assigned as at the existing ground level.

Further details of groundwater conditions across the Level 1 asset are presented in Section 2.4.6 of this GDR.

3.5.9 Geotechnical Risks

A summary of the key potential geotechnical geohazards that may affect the assets are provided in Table 3-5. A snapshot of the GRR can be found in Appendix D where more information and risk mitigations are provided. These risks were considered in the design.

Table 3-5: Summary of Geohazards

Risks	Description
Faulting	Potential Faulting identified at mainline chainages Ch 060+650, 060+875, 061+075, 061+180, and 061+250, resulting in locally weaker ground conditions 1.Based on results from ground investigation works, No faults are encountered in this area. Conservative parameter values have been adopted for design. And inspection of formation and validation testing required during construction of embankment.
Soft ground	Occurrences of very soft or loose material can lead to instability of embankments and excessive ground movement and low CBR values affecting pavement design. Any soft ground occurrences are considered to be localised and should be excavated and replaced in accordance with IDR Specification 208.4
Aggressive ground conditions	Occurrences of gypsum crystals can be associated with high sulphate content which can lead to chemical attacks on concrete structures. All concrete and fill that interface with the natural ground is to be designed adequately to BRE 363 considering available chemical testing.

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Risks	Description
Shallow groundwater	Potentially shallow groundwater present below embankments, leading to instability. Conservative groundwater level considered in the design.
Shrink / swell	Changes in soil volume by shrinking or swelling from changes in moisture content independent of loading leading to ground movements. Conservative ground parameters have been considered in the slope stability and settlement design including the adoption of a Vadose zone as set out in the HS2 Technical Standard – Earthworks (HS2-HS2-GT-STD-000-000001) for assessment of long-term slope stability.
Relic Shear Surfaces	Polished discontinuity surfaces were encountered within Kimmeridge Clay Formation within ML061-CR014 between 19.40-20.00 mbgl.
Contaminated Ground	Limited data available has been reviewed for potential contamination, however, this remains a potential risk from site-wide unforeseen contamination. The formation is to be inspected prior to placement of fill material of highway to ensure no Made Ground is present. Undertake a watching brief during construction, and isolate and test any materials that have the potential to pose a risk to human health and the environment.

3.5.10 Slope Stability Analysis

The static global stability of earthworks has been calculated adopting the principles of limit state equilibrium in accordance with BS EN 1997-1 Geotechnical Design (EC7) and the associated UK National Annex (NA+ A1: 2014 to BS EN 1997-1:2004+A1:2013). Specifically, Ultimate Limit State (ULS) Design Approach 1 Combination 1 (DA1 C1) and Combination 2 (DA1 C2) have been assessed adopting partial factors in accordance with EC 7 with the requirement to achieve a utilisation less than or equal to 1. Assessments have been undertaken to satisfy the requirements of the HS2 Technical Standard – Earthworks.

Where geogrid reinforced slopes have been adopted for the design of earthwork side slopes, the internal stability of the geogrid arrangement has been assessed in accordance with BS 8006-1:2010+A1:2016.

The tables below summarise the results of the stability analyses for the following assessments, with further results presented in Appendix E.1.

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Table 3-6: Slope Stability Analysis

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Ground Model	Analysis	DA1C2-Drained (Unreinforced) DoU	DA1C2-Undrained (Unreinforced) DoU	Soil Reinforced (BS8006) DoU	Soil Reinforced (DA1C2 - Drained) DoU
1	1.0	1V:3H	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)	0.874	0.315	N/A	
2	1.5	1V:3H	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)	0.968	0.360	N/A	
3	2.0	1V:3H	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)	1.035 (1)	0.404	N/A	
4	0.5	1V:3H	2A4	2	Unreinforced Embankment (MAR - South Arm off Calley Farm South) – non ditch interface	0.897	0.341	N/A	
5	1.0	1V:3H	2A4	2	Unreinforced Embankment (MAR - South Arm off Calley Farm South) – ditch interface	1.079 (2)	0.483	N/A	

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Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Ground Model	Analysis	DA1C2-Drained (Unreinforced) DoU	DA1C2-Undrained (Unreinforced) DoU	Soil Reinforced (BS8006) DoU	Soil Reinforced (DA1C2 - Drained) DoU
6	3.0	1V:3H	2A4 / 7C3	1	Reinforced Embankment (MAR - South Arm off Calley Farm South)	N/A		0.853	0.861
7	2.0	1V:3H	2A4 / 7C3	2	Reinforced Embankment (MAR - South Arm off Calley Farm South)	N/A		0.824	0.980
8	1.35	1V:3H	2A4	1	Pond C2.T.23-PND-0614 cutting adjacent to access road	0.662	0.226	N/A	
9	1.35	1V:3H	2A4	2	Pond C2.T.23-PND-0614 cutting adjacent to access road	0.701	0.321	N/A	
10	2.3 (1.2 Emb / 1.1 Cutting)	1V:3H	2A4	2	Pond C2.T.21-PND-0611 cutting adjacent to access road	0.874	0.429	N/A	
11	2.3 (Cutting)	1V:3H	2A4	2	Pond C2.T.21-PND-0611 cutting adjacent to access road	0.954	0.394	N/A	

Notes:

1) Analysis shows that maximum unsupported slope height for 1V:3H slopes is 1.5m (Ground Model 1). Slopes greater in height will be soil reinforced, refer to row 6 above for reinforcement calculation results, Table 3-7 defining where soil reinforcement is required and Section 5.2 for further details of soil reinforcement.

2) Analysis shows that maximum unsupported slope height for 1V:3H slopes is 0.5m (Ground Model 2). Slopes greater in height will be soil reinforced, refer to row 7 above for reinforcement calculation results, Table 3-7 defining where soil reinforcement is required and Section 5.2 for further details of soil reinforcement.

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3) All results are presented as DoU.

4) Results are not presented for DA1C1 as they do not govern the slope design.

5) A 2m deep vadose zone has been modelled within the highway embankment with a ru of 0.25 with the remaining embankment fill modelled with a ru of 0.1 for all embankment heights.

6) High utilisation shown above in red highlights the need for mitigation in the form of geogrid reinforcement.

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In summary:
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Ch0+000 to 0+500:

- Highway embankments with 1V:3H side slopes, less than 1.5m in height, can be constructed using Class 2A4 material for meeting the required HS2 acceptability criteria.
- Highway embankments with 1V:3H side slopes, 1.5m and greater in height, require geogrid reinforcement with Class 7C3 material in order to meet the required HS2 acceptability criteria.

Maintenance Access Road to Pond (South Arm off Calley Farm - South) Ch0+500 to 0+619:

- Highway embankments with 1V:3H side slopes, less than 0.5m in height, can be constructed using Class 2A4 material for meeting the required HS2 acceptability criteria.
- Highway embankments with 1V:3H side slopes, 0.5m and greater in height, require geogrid reinforcement with Class 7C3 material, in order to meet the required HS2 acceptability criteria

Reference should also be made to Section 5.2 of this GDR which presents the design of strengthened earthworks.

Table 3-7: Summary of Slope Stability Design

Left Hand Side		Right Hand Side	
Ch.	Solution	Ch.	Solution
0+000 to 0+048	Geogrid Reinforcement	0+000 to 0+048	Geogrid Reinforcement
0+048 to 0+500	Unreinforced	0+048 to 0+500	Unreinforced
0+500 to 0+600	Geogrid Reinforcement[2]	0+500 to 0+600	Geogrid Reinforcement[2]
0+600 to 0+619	Unreinforced	0+600 to 0+619	Unreinforced
0+000 to 0+034 (C2.T.23-PND-0614) Access	Unreinforced	0+000 to 0+034 (C2.T.23-PND-0614) Access	Reinforced
0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced	0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced

Notes:

1. Side of embankment taken as if looking south along chainage
2. Geogrid with 100kN/m tensile capacity is required at base of the highway embankment, as per Appendix 6/9 of the IDR2084. Specification(1MC06-CEK-AU-SPE-CS04-000003). A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid is also required.

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3.5.11 Settlement Analyses

Due the gentle change in gradient of highway embankments proposed, the rate of change of post construction ground movements are expected to be gradual and within tolerable limits for the proposed maintenance access road. No specific settlement assessment has been carried out for the highway design. Furthermore, no specific excavate and replace or hold periods will be required to manage the ground movements other than inspection by a qualified Geotechnical Engineer/ Engineering Geologist.

A specific settlement analysis will be carried out for the Culvert (061-AC3) earthworks and reported in Section 4 of this GDR.

3.5.12 Pavement Foundation Design

The pavement design has been undertaken in accordance with the DMRB, Volume 7: Pavement Design and Construction as well as the HS2 Technical Standard and the HS2 Highway Standard Details.

The road pavements are designed in accordance with CD225, whereas the footpaths/ bridleways are designed in accordance with CD239. Design California Bearing Ratio (CBR) values are derived for embankments and the existing ground based on the available in situ and laboratory testing as well as correlations with other derived parameters.

With reference to CD225, for road pavement, the lowest of the short-term CBR (for construction traffic loadings) and long-term equilibrium CBR (for long-term traffic loadings) should be adopted for design. The short-term CBR is based on either plate bearing tests or CBR testing, or by corelation with undrained strength testing within the near surface soils. The long-term equilibrium CBR value is based on the Plasticity Index (PI) of the natural subgrade or fill material, based on Table C1 of LR 1132.Table 3-8 provides a summary of the pavement design for the highway assets.

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Table 3-8: Summary of Pavement Foundation Design

Asset	Chainage		Permitted Pavement Option	Pavement Type	CBR (%)	Standard Pavement Thickness	GEO Input
	From	To					
Maintenance Access Road to Pond (South Arm off Calley Farm – South)	0	600	AT2	Unbound	2.0	250	Additional 100mm added to subbase with Type 1 (Total thickness = 350mm) [1 / 2]
	600	619			1.0		Additional 200mm added to subbase with Type 1 (Total thickness = 450mm) [1 /2]
Maintenance Access Road to Pond (South Arm off Calley Farm – South). C2.T.23-REG-0614 Pond Access	0	34			2.0		Additional 100mm added to subbase with Type 1 (Total Thickness = 350mm) [1 / 2]
Maintenance Access Road to Pond (South Arm off Calley Farm – South). Pond C2.T.21-REG-0611 Pond Access	0	37					

Notes:

[1] Assumes trafficking of 2 nos. HGV movements per annum.

[2] Subgrade mitigation has been specified to achieve a minimum subgrade modulus of 31MPa / CBR of 2.5% at the formation of the pavement (i.e., below subbase or capping).

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3.5.13 Design Solution: Maintenance Access Road to Pond (South Arm off Calley Farm – South)

Table 3-9 below summarise the geotechnical design along the alignment. For a full account of the design, reference should be made to the calculation pack within E.1 and the IDR208.4 specification.

Table 3-9: Summary of Geotechnical Input – Maintenance Access Road to Pond (South Arm off Calley Farm – South)

Element	Left Hand Side									Right Hand Side								
	0+000 to 0+048	0+048 to 0+062	0+062 to 0+438	0+0438 to 0+500	0+500 to 0+545	0+545 to 0+600	0+600 to 0+619	0+000 to 0+034 (Pond C2.T.23-PND-0614	0+000 to 0+037 (Pond C2.T.21-PND-0611	0+000 to 0+048	0+048 to 0+062	0+062 to 0+438	0+438 to 0+500	0+500 to 0+545	0+545 to 0+600	0+600 to 0+619	0+000 to 0+034 (Pond C2.T.23-PND-0614	0+000 to 0+037 (Pond C2.T.21-PND-0611
Embankment Heights (m)	2.5 – 1.5	1.5 – 0.75	<0.75	0.75 – 1.5	1.5 - 2.0	2.0 - 0.75	<0.50	<2.0	<1.5	2.5 – 2.0	2.0 – 0.75	<0.75	0.75 – 1.50	1.50 – 2.0	2.0 – 0.75	<0.5	<2.0	<1.5
Side Slope Angle	1V:3H									1V:3H								
Side Slope Design	Reinforced	Soil Slope			Reinforced[5]		Soil Slope	Soil Slope	Reinforced[5]	Reinforced	Soil Slope			Reinforced[5]		Soil Slope	Reinforced[5]	
Embankment Fill Class	Class 2A4 / 7C3	Class 2A4	Class 1A4 / 2A4	Class 2A4	Class 2A4 / 7C3		Class 1A4 / 2A4	Class 2A4	Class 2A4 / 7C3	Class 2A4 / 7C3	Class 2A4		Class 1A4 / 2A4	Class 2A4 / 7C3		Class 1A4 / 2A4	Class 2A4 / 7C3	
Subgrade DSG Assumed	Class 2A4		Portland Group	Class 2A4			Alluvium	Class 2A4		Class 2A4		Portland Group	Class 2A4			Alluvium	Class 2A4	
Subgrade DSG CBR Value (%)	2.0						1.0	2.0		2.0						1.0	2.0	
Subbase Thickening Layer	Yes (100mm)						Yes (200mm)	Yes (100mm)		Yes (100mm)						Yes (200mm)	Yes (100mm)	
Pavement thickness (mm)	250									250								
Mitigation SED	G103 G114	G103	G102	G103	G103 G114		G102	G103	G103 G114	G104 G114	G103	G102	G103	G103 G114		G102	G103 G114	

Notes:

1. Details of pavement design are provided in Section 3.5.12.
2. Geotextile separation layers are to be placed below all embankments in accordance with SED's, G103, G104 (1MC06-CEK-GT-DSE-CS002-000104), G106 (1MC06-CEK-GT-DSE-CS002-000106) and G114 (1MC06-CEK-GT-DSE-CS002-000111).
3. Topsoil strip required where embankments < 3m in height.
4. Starter layer required (300mm Class 6C1) – Ch. 0+500 to 0+600
5. Geogrid with 100kN/m tensile capacity required at base of the highway embankment. A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.
6. During construction, the formation level beneath the pavement, from Ch.0+000 to 0+500 is to be exposed and ground conditions inspected/tested. Soils with an undrained shear strength of <45 kPa, equivalent to a short-term CBR of 2.0% should be excavated and replaced (E&R) in accordance with the standard earthwork drawing SED G106, 1MC06-GT-DSE-C022-000106, and the specification verification of pavement design assumptions.
7. During construction, the formation level beneath the pavement, from Ch.0+500 to 0+619 is to be exposed and ground conditions inspected/tested. Soils with an undrained shear strength of <20 kPa, equivalent to a short-term CBR of 1.0% should be excavated and replaced (E&R) in accordance with the standard earthwork drawing SED G106, 1MC06-GT-DSE-C022-000106, and the specification verification of pavement design assumptions.

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3.5.14 Ground Treatment

Excavate and Replace (E&R) will be carried out below earthworks where soft spots have been identified, in accordance with the Specification and SED G106 (Drawing No. 1MC06-CEK-GT-DSE-C002-000106). It is noted that the contractor may substitute Class 1 material with Class 2A4 / 2B4 for replacement material unless specified as LWA.

For earthworks at grade or up to 0.75m in height ((Ch.0+000 to 0+500), the formation level (below the top soil strip) is to be exposed and ground conditions inspected/tested such that encountered soils with an undrained shear strength of <45kPa, equivalent to a short term CBR of <2.0%, should be excavated and replaced accordingly.

For earthworks at grade or up to 0.5m in height ((Ch.0+500 to 0+619), the formation level (below the top soil strip) is to be exposed and ground conditions inspected/tested such that encountered soils with an undrained shear strength of <20kPa, equivalent to a short term CBR of <1.0%, should be excavated and replaced accordingly.

A high strength basal reinforcement layer, with 100kN/m tensile capacity, has been specified between chainages 0+500 to 0+600, as per Appendix 6/9 of the IDR 208.4 Specification(1MC06-CEK-AU-SPE-CS04-000003). The purpose of the inclusion of a high strength geotextile basal reinforcement is to prevent potential failure surfaces into an existing drainage ditch from extending beneath the proposed low height highway embankment and will control any potential differential settlements. A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

3.5.15 Risk Management

3.5.15.1 Introduction

A number of hazards discussed in Section 3.5.9 are difficult to account for at this stage of Detailed Design due to a lack of information on the presence, location and extent of the hazard or design consideration. These will need to be addressed as and when additional information is obtained from further ground investigation and through the observational method during construction.

The subsequent sections summarise the risks and provide an indication of the type of mitigation that may need to be deployed should the particular hazard be encountered during construction.

3.5.16 Geotechnical Risks

For a full account of the geotechnical risks, refer to IDR 208.4 GRR (1MC06-CEK-GT-REG-CS04-000003). An extract of the GRR for the non-mainline highway earthwork assets is included in Appendix D.

3.5.16.1 CDM Risks

For a full account of the CDM risks, refer to IDR 208.4 CDM Risk Register (1MC06-CEK-AU-REG-CS04-000009).

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3.5.17 Reuse of Materials

Kimmeridge Clay will be sourced from the HS2 Mainline cutting excavation works adjacent to these assets and will provide suitable materials for the Highway embankments in the form of Class 2A4 and Class 7C3 materials in accordance with the IDR 208.4 Specification (1MC06-CEK-AU-SPE-CS04-000003).
A summary of the materials likely to be produced on site is as follows:

- A nominal 300mm topsoil strip is anticipated for the entire highway alignment, to be reused as Class 5.
- In areas of minor cutting, Portland Stone Formation and Portland Sand Formation may be recovered and is assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface material. In areas of minor cutting, Alluvium Formation may be recovered and is assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface materials.

3.6 Sedrup North Watercourse Diversion (Ch. 61+330)

The Sedrup North Culvert Watercourse is located along the HS2 Aylesbury Embankment, as illustrated in Figure 3-2, which is an extract from the Sedrup North Watercourse Diversion Layout Plan (Drawing No.: 1MC06-CEK-DR-DPL-CS04_CL18-000001). The watercourse is located at approximate HS2 Chainage (Ch.) 61+330. Salix CocoBN erosion control matting (or similar approved) forms the liner material for the open channels, whereas the Flex MSE sandbags (or similar approved) form the liner material for the headwall sections and the transition to the headwalls.

Open channel slopes beyond the downstream end of the Sedrup North Culvert have not been subjected to geotechnical stability checks and are designed only as per the Watercourse Standard. Culvert transition slopes and some of the open channels have been analysed according to HS2 Earthwork Standards. The applied design standards have been summarised in Table 3-10.

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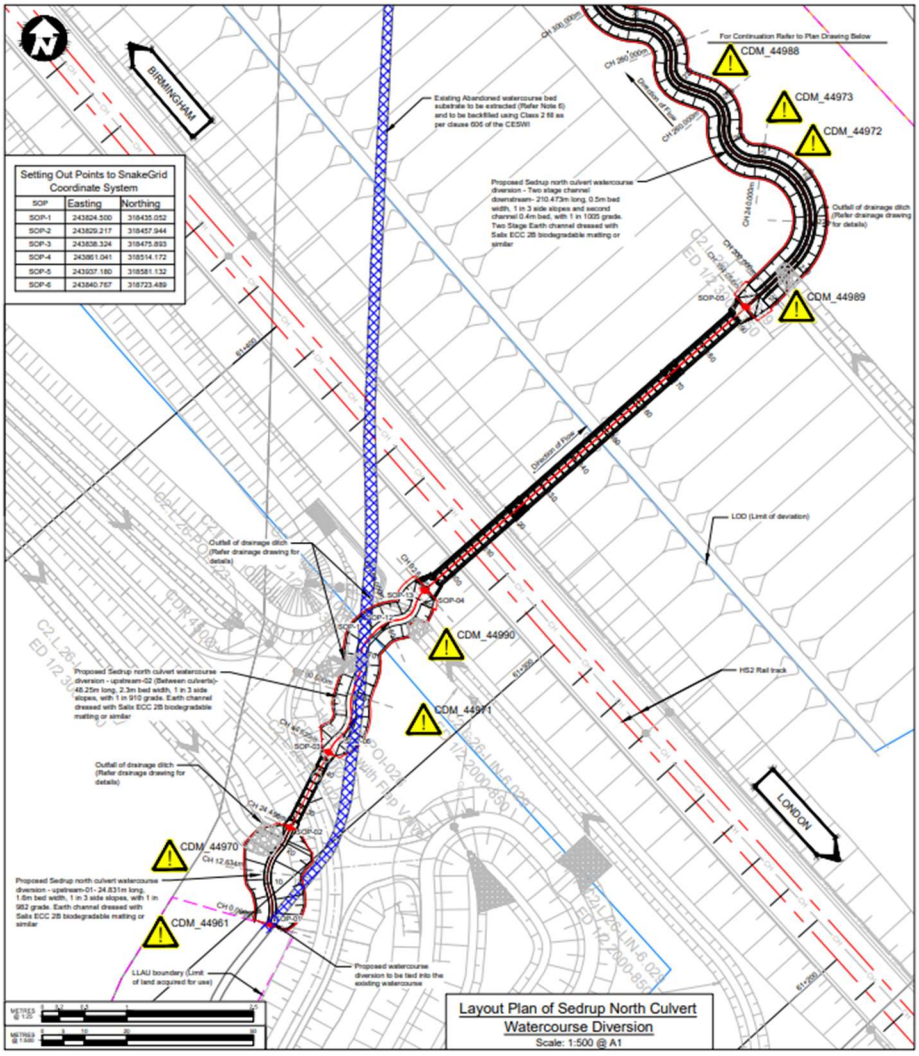


Figure 3-2 Sedrup North Watercourse Plan View

Table 3-10: Design Standard Summary

		Applied Design Standard	
Chainage (m)	Function	Left bank	Right bank
0 – 21	Open channel	Earthwork Standards	Earthwork Standard
21 – 24.498	Open channel to culvert transition	Earthwork Standard	Earthwork Standard
44.625 – 47.249	Culvert to open channel transition	Earthwork Standard	Earthwork Standard
47.249 – 80	Open channel	Watercourse Standard	Earthwork Standard
80 – 87.68	Open channel	Earthwork Standard	Earthwork Standard

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		Applied Design Standard	
Chainage (m)	Function	Left bank	Right bank
87.68 – 92.68	Open channel to culvert transition	Earthwork Standard	Earthwork Standard
194.056 - 199.056	Culvert to open channel transition	Earthwork Standard	Earthwork Standard
199.056 – 408	Open channel	Watercourse Standard	Watercourse Standard

Sections that adopt the Watercourse Standard (HS2-HS2-EV-STD-000-000014) are not subjected to geotechnical stability checks. Sections that are subjected to the HS2 Earthworks Standards have been assessed for geotechnical stability checks.

3.6.1 Assumptions

The following general assumptions have been made for the slope stability calculation:

- For the short-term and long-term analyses, pore water pressures are considered hydrostatic from a conservative assessment of existing groundwater levels.
- For the short-term and long-term analyses, the groundwater profile was assumed to run behind the granular backfill to the toe of the vegetated retaining wall system at the side slopes.
- Design calculations have currently been based on the geotechnical properties of the Flex MSE system. If an alternative system is used, then a verification of the geotechnical design will be required based on the proposed solution to ensure that the current design codes and standards are satisfied.
- The slope stability assessment has been undertaken using the GeoStudio software Slope/W and slip surfaces were assessed based on the Morgenstern-Price method. These slip surfaces were then defined using the entry and exit method.
- A 5kPa (variable unfavourable) surcharge has been adopted along the crest of the ditch as it is not accessible to vehicles. A 10kPa (variable unfavourable) surcharge has been adopted along the crest of the maintenance access strip load at Ch. 12.63.
- Rapid drawdown conditions are considered not applicable for this case, as based on flood risk assessment the duration for the water during a potential flood event is expected to recede over an extremely short period. In addition, new open channels are formed within granular fill material that is considered a free-draining material.
- Geogrid reinforcement of minimum ultimate tensile strength of 17kN/m and a factored capacity of 15kN/m is to be utilised.

3.6.2 Ground Model

The derived ground models and geotechnical parameters are based on Data Drop 24 and are presented as follows:

- Ground Model 1: Between Ch. 0 and Ch. 12.63
- Ground Model 2: Between Ch 60 and Ch. 80.
- Ground Model 3: Between Ch. 260 and Ch. 300
- Ground Model 4: To Ch. 400.

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Table 3-11: Ground Model 1

Strata	Top of Layer (m AOD)	Base Depth (m bgl)	Design Depth (m bgl)	Remarks
Alluvium	78.91 – 77.76	1.25 – 2.20	1.50	Soft gravelly sandy CLAY
Portland Sand / Stone Formation	77.76 – 76.72	>5.00	6.00	Firm to very stiff gravelly sandy CLAY

Table 3-12: Ground Model 2

Strata	Top of Layer (m AOD)	Base Depth (m bgl)	Design Depth (m bgl)	Remarks
Alluvium	78.06 – 77.76	0.30 – 2.00	2.00	Soft gravelly sandy CLAY
Purbeck Group	77.84 – 76.72	>4.50	5.00	Recovered as a medium-strong to strong Limestone
Portland Sand / Stone Formation	73.56	12.5 – 16.0	8.00	Recovered as a strong Limestone

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Table 3-13: Ground Model 3

Strata	Top of Layer (m AOD)	Base Depth m bgl)	Design Depth (m bgl)	Remarks
Alluvium	77.68 – 77.96	0.50 – 2.20	2.00	Soft gravelly sandy CLAY
Portland Sand / Stone Formation	77.41 – 76.68	>5.00	6.00	Firm sandy SILT becoming a weak Limestone

Table 3-14: Ground Model 4

Strata	Top of Layer (m AOD)	Base Depth (m bgl)	Design Depth (m bgl)	Remarks
Alluvium	77.96—77.46	0.50 – 2.20	1.50	Soft gravelly sandy CLAY
Portland Sand / Stone Formation	76.76 – 76.26	>7.00	7.00	Recovered as a strong Limestone

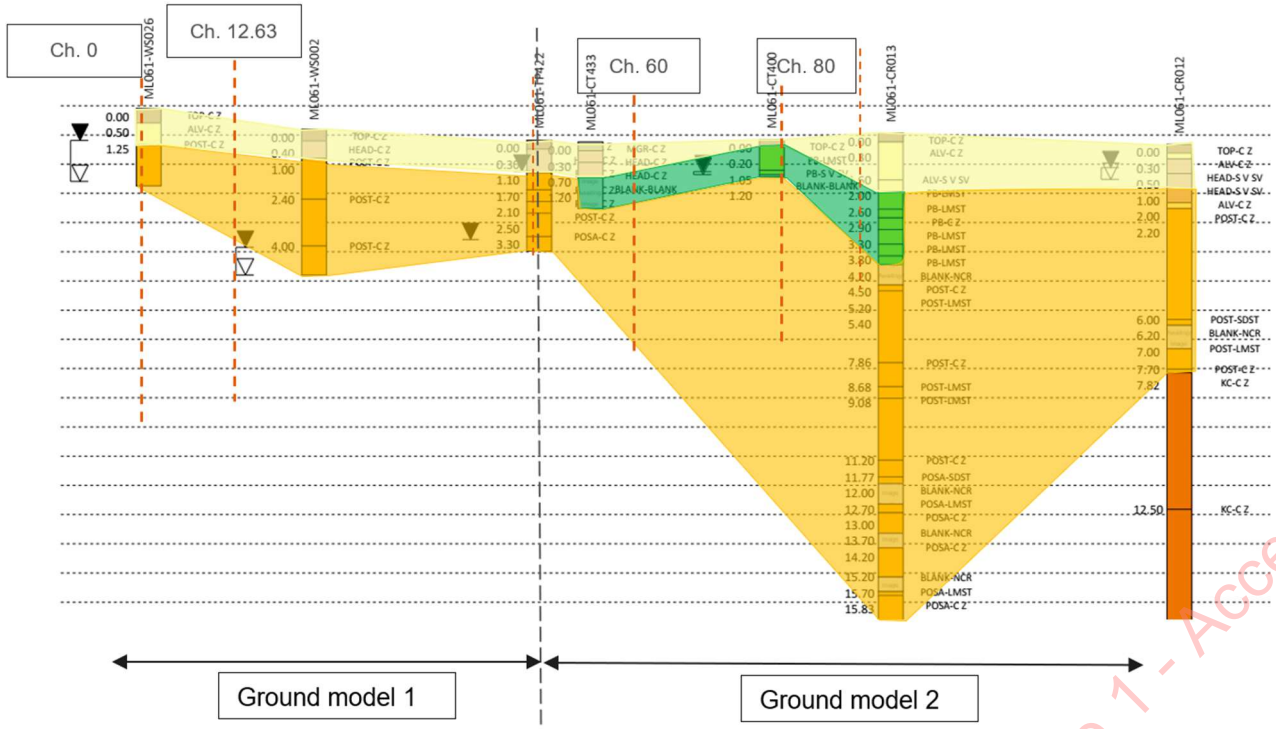


Figure 3-3: Ground Models 1 and 2

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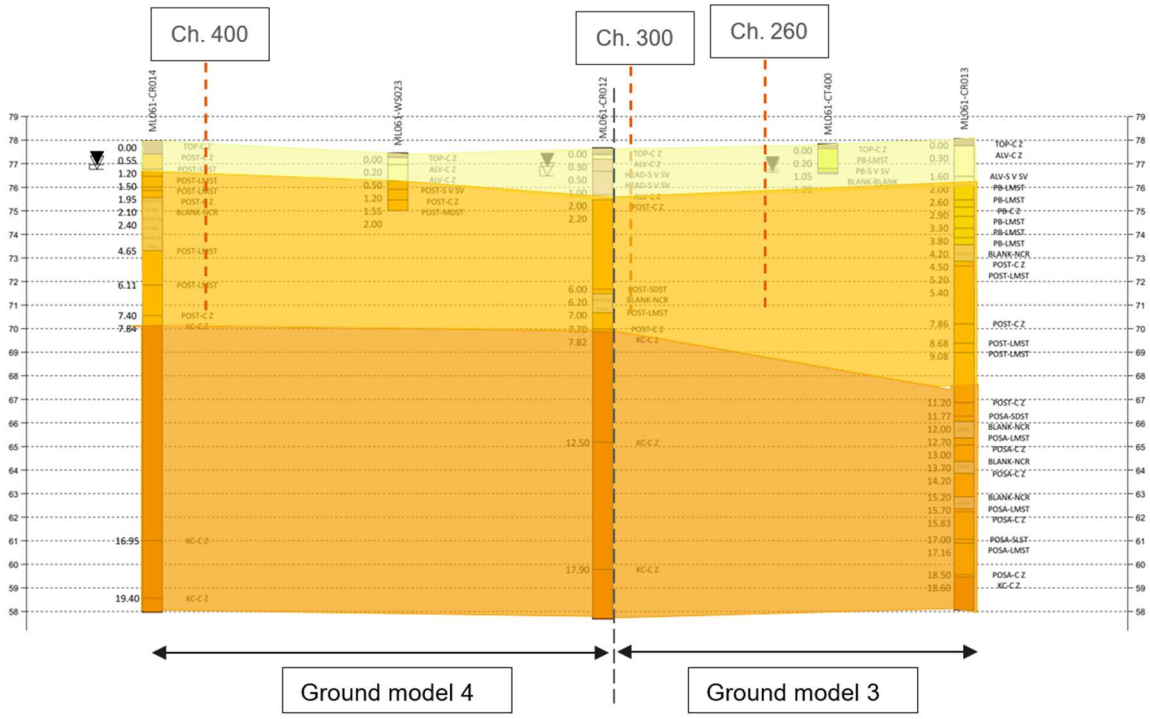


Figure 3-4: Ground Models 3 and 4

3.6.3 Geotechnical Parameters

The ground parameters presented in Table 3-15 were adopted for the slope stability analyses and have been adopted from Table B-1. Table 3-16 presents the geotechnical parameters adopted for the adopted fill materials.

Table 3-15: Characteristic Geotechnical Parameters

Strata	γb (kN/m³)	c'(kPa)	PI (%)	SPT-N_{60}	$\varphi' cv$ (°)	$\varphi' pk$ (°)	Cu (kPa)
Alluvium	18.0	0	48	0 – 22	21	21	20
Portland Sand / Stone Formation (0m< z<2m)	20.0	1	35	5 + 11z	23	25	50
Portland Sand / Stone Formation (z>2.0m)	20.0	2	20	9 + 13.5z	26	28	50 + 10z (limited to a maximum value of 130 kPa)
Purbeck Group	20.0	2.5	Non-plastic	35 – 50	22	24	-

z is the depth (m) measured from the top of the stratum.

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Table 3-16: Characteristic Geotechnical Parameters - Fill Materials

Strata	γ above water (kN/m^3)	γ below water (kN/m^3)	c'(kPa)	$\varphi' pk$ (°)	$\varphi' cv$ (°)	Cu (kPa)
Class 6I1/6J1 Fill	20.0	22.0	0	35	-	N/A
Class 1A1 Fill	20.0	22.0	0	35	-	N/A
Class 2A Highway Fill	19.0	19.0	0	27	25	50
Class 4 Landscape Fill	20.0	20.0	0	25	23	50

3.6.4 Groundwater

Groundwater within this asset is anticipated to be shallow. The groundwater monitoring readings range from 0.38m to 3.1m bgl. Therefore, the design groundwater level has been taken as an average level of 0.7m below the crest (existing ground level).

Table 3-17: Groundwater Monitoring Results

Location	No. of Readings	Highest (m bgl)	Lowest (m bgl)	Highest (m AOD)	Lowest (m AOD)
ML061-CP004	11	2.09	0.71	75.8	77.18
ML061-CP012	13	2.36	0.9	76.51	77.97
ML061-CR009	13	3.10	1.47	76.3	77.93
ML061-CR012	14	1.52	0.54	76.16	77.14
ML061-CR013	13	2.07	0.68	75.99	77.38
ML061-RC017	3	0.99	0.38	78.16	78.77

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3.6.5 Slope Stability Analysis

3.6.5.1 Mitigations along open channels

The results of the slope stability analyses for the Sedrup North watercourse open channels are summarised in Table 3-18. Please refer to Appendix G Figures G-1 – Figures G-6.

Table 3-18: Slope Stability Assessment Results - Open Channels

Chainage (Ch.)	Watercourse Slope Gradient Left Bank	Watercourse Slope Gradient Right Bank	DA1C2 ODF Left Bank (1)	DA1C2 ODF Right Bank (2)	Mitigation measure to maintain slope stability (if required)
12.63	1V:3.0H	1V:3.0H	1.017	1.052	0.7m wide and minimum 1.5m deep Excavate & Replace with Class 1A1 for Left Bank and1.0m wide and minimum 1.5m deep E&R with Class 1A1 for Right Bank
60	1V:3.0H	1V:3.0H	1.012	1.006	2.0m wide and minimum 1.1m deep Excavate & Replace with Class 1A1 fill for Right Bank
80	1V:2.3H	1V:3.0H	1.042	1.005	0.7m wide and minimum 1.5m deep Excavate & Replace with Class 1A1 for Left Bank and 2.0m wide and minimum 1.4m deep E&R with Class 1A1 for Right Bank

(1) (2) Over-design factor (ODF) in accordance with BS EN 1997-1:2004+A1:2013 Eurocode 7 – Geotechnical Design for Design Approach 1 Combination 2 (DA1C2) for the left and right banks.

3.6.5.2 Mitigation Measure at Transitions to Culvert Inlet & Outlet

The results of slope stability analyses for the reinforced earth transitions to the culvert inlets & outlets along Sedrup North watercourse are summarised in Table 3-19. Please refer to Appendix G, Figure G-7 and Figure G-8.

Table 3-19: Slope Stability Assessments Results - Headwall Sections

Headwall ID	Mitigation application range (local watercourse chainage)	Proposed Gradient (left / right sides)	DA1C2 ODF Left and Right Bank (3)	Mitigation measure to maintain slope stability (if required)
Sedrup North Culvert Headwalls	87.68 - 92.68 & 194.056 – 199.056	Vertical / Vertical	1.325	For both banks: 3.1m long geogrids (3.86m including length within the Flex MSE bags) at 0.28m vertical spacing (2 bags spacing) with 3.60m wide Class 6I1/6J1 backfill. The foundation soil needs to be excavated and replaced by 1m thick Class 1A fill
061-AC3 Ancillary	22.198 – 24.498 &	Vertical / Vertical	1.476	For both banks: 3.0m long geogrids (3.76m including length within the Flex MSE bags) at 0.42m vertical spacing (3

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Headwall ID	Mitigation application range (local watercourse chainage)	Proposed Gradient (left / right sides)	DA1C2 ODF Left and Right Bank (3)	Mitigation measure to maintain slope stability (if required)
Culvert Headwalls	44.625 – 47.249			bags spacing) with 3.5 wide Class 6I1/6J1 backfill.

(3) Over-design factor (ODF) in accordance with BS EN 1997-1:2004+A1:2013 Eurocode 7 – Geotechnical Design for Design Approach 1 Combination 2 (DA1C2) for the left and right banks.

3.6.6 Summary of Internal/External Stability Results

As per BS 8006-1:2010, the internal/external stability of the vegetated Flex MSE retaining wall solution needs to be assessed. The results of external stability assessments of the reinforced soil block for the Sedrup North Culvert Headwalls have been summarised in Table 3-20. The results of the internal stability assessments have been summarised in Table 3-21.

Table 3-20: Summary of external stability assessment of reinforced soil block for Sedrup North Culvert Headwalls

Failure mode	Max Utilisation (%) – Drained Case
Bearing Capacity Failure	73
Base Sliding Stability	62.39

Table 3-21: Summary of internal stability assessment of reinforced soil block for Sedrup North Culvert Headwalls

Failure mode	Max Utilisation (%)
Rupture Stability	94.16 (Serviceability Limit State)
Adherence Stability	27.87
Wedge Stability	9.69
Geogrid/Bags pull out	73.97

The results of the external stability assessment of the reinforced soil block for 061-AC3 Culvert Headwalls have been summarised in Table 3-22. The results of the internal stability assessments have been summarised in Table 3-23.

Table 3-22: Summary of external stability assessment of reinforced soil block for 061-AC3 Culvert Headwalls

Failure Mode	Max Utilisation (%) – Drained Case
Bearing Capacity Failure	70
Base Sliding Stability	57.21

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Table 3-23: Summary of internal stability assessment of reinforced soil block for 061-AC3 Culvert Headwalls

Failure Mode	Max Utilisation (%)
Rupture Stability	57.05
Adherence Stability	23.62
Wedge Stability	10.19
Geogrid/Bags pull out	84.07

The results of the above assessments have been presented in tables at the end of the Appendix G.

3.6.7 Abandoned Watercourse

The abandoned existing watercourse along the proposed Sedrup North Watercourse shall be rehabilitated according to Specification (1MC06-CEK-AU-SPE-CS04-000005) Appendix 6/3, Section 3.4 Watercourses. It is required that the rehabilitation of the abandoned watercourse also fulfil the requirements of earthworks design.

Where any areas of old watercourses (infilled or otherwise) occur under a section of new embankment any soft soils (undrained shear strength <50kPa) or soils containing organic matter shall be excavated and replaced with Class 6C1 material, in accordance with Contract-specific Table 6/1 for mainline embankments within the track support zone or other Engineered Fill as shown on the Contract Drawings for other areas.

3.6.8 Risk Management

Geotechnical risks relevant to the watercourse are highlighted in the GRR doc no 1MC06-CEK-GT-REG-C002-000076. Residual risks are registered to Digital CDM and an extract is provided in doc no 1MC06-CEK-AU-REG-CS04_CL18-000042.

3.7 Oat Close SBH/19 Realignment

3.7.1 Introduction

This section sets out an overview of the assumptions for earthworks design and presents a summary of the calculations undertaken. The Design has been undertaken to satisfy the performance requirements of the HS2 Technical Standard - Earthworks Ref [4].

For details of the ground model, constraints, design methodology and calculation output, reference should be made to Appendix E of this GDR.

3.7.2 Asset Information

This section summarises the arrangement of the asset, including length, embankment height and interaction with other elements.

The Oat Close (SBH/19) realignment is a Public Right of Way (PRoW) where the Bridleway SBH/19 realignment running approximately northwest to southeast across the proposed route of the HS2 mainline. The bridleway is surrounded by open fields which will provide access to areas on both sides of the HS2 alignment near Ch 60+920 via a new overbridge and embankments on both sides of the HS2 mainline.

The proposed realignment of the highway comprises three distinct sections. Firstly, the Bridleway SBH/19 realignment is positioned on the northeast, spanning an approximate length of 381m. This segment is situated along an embankment ranging in height from 0m to 10.35m.

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The second section, Bridleway SBH/19 Link, extends on the southwest with a length of approximately 413m. It is strategically positioned on an embankment varying in height from 0m to 8.35m.

Lastly, the third section involves the realignment of Footpath SBH/27 on the northwest, covering a length of approximately 384m. This realignment is positioned along an embankment that varies in height from 0m to 8.35m.

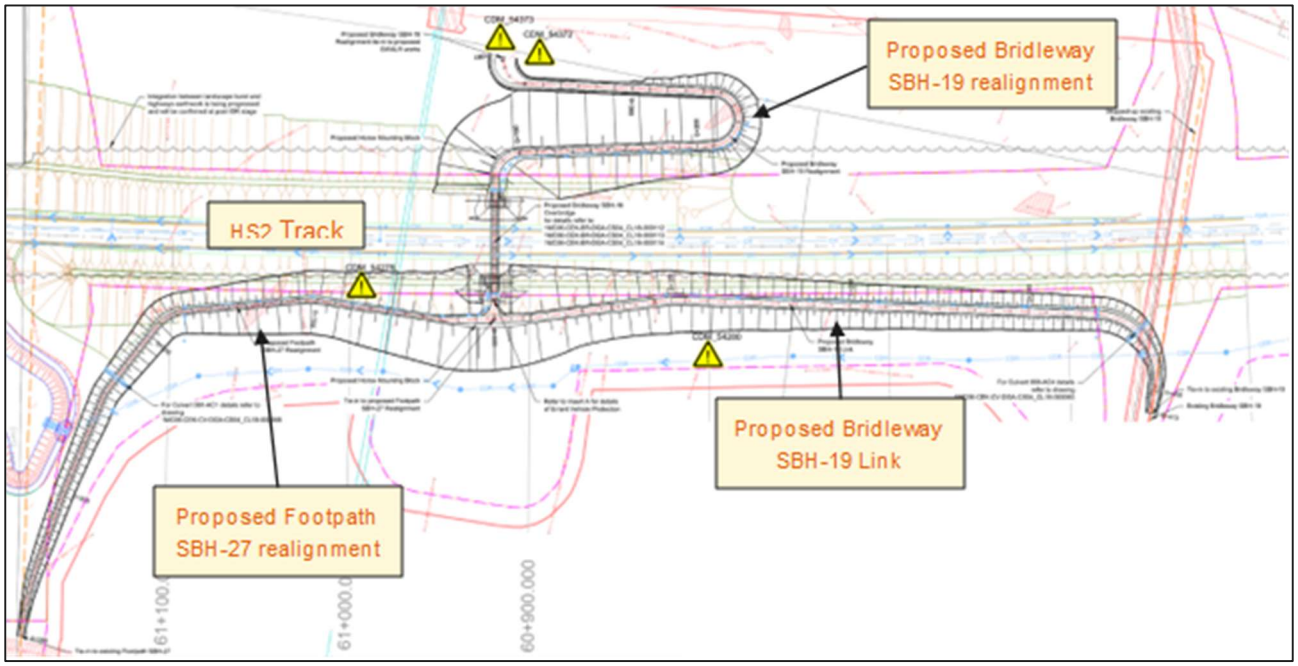


Figure 3-5: Bridleway SBH/19 Realignment Asset Plan and Location

3.7.2.1 Site constraints

In addition to the descriptions above the following constraints have been identified and considered as part of the earthworks design.

Table 3-24: Constraints to Highway Earthworks Design - Oat Close SBH-19 Realignment

Asset	Feature	Location	Chainage (mainline)
Oat Close SBH-19 Realignment	Land drainage pond (C2.L.22-PND-0606) and (C2.L.22-PND-0609)	southwest	60+875m to 60+975m And 60+550 to 60+637
	Truck drainage attenuation Pond (C2.T.21-PND-0611)	northwest	60+984 to 61+620
	Highway drainage (C2.L.22 land drainage ditch)	Both sides	60+550 to 60+637
	Ancillary Structures (Culverts 060-AC4 and 061-AC1),	Both sides	60+550 m and 61+150m
	Landscape Earthworks	Both sides	60+550m to 61+150m

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3.7.3 Construction Method and Sequence

The Aylesbury Embankment and highways earthwork will be constructed following the geotechnical design documents. Upon completion of the mainline embankment and highways earthwork, the permanent landscape bunds are expected to be built within 2 years of the highway earthwork.
The RIA (Appendix J) indicates no hold period is required between the installation of abutments and wingwalls and the installation of the bridge deck for Oat Close Bridleway SBH/19 Overbridge, Importantly, the construction sequence of the bridge is not expected to impact the design of the proposed highway earthworks.

3.7.4 Ground Investigation and Field Trials

A list of the GI information considered for the assets listed above is included in Appendix E and corresponds with Data Drop 39 (shared November 2023). This includes GI across Aylesbury Embankment nearby the footprints of the assets. The ground conditions have been assessed considering GI from previous phases of investigation as well as the Stage 2 site investigation which is still ongoing at the time of this submission.

Following completion of the earthworks design for these assets, no further ground investigation has been conducted (other than that mentioned above). This GI includes 18 no. exploratory holes.

Considering the consistency in ground conditions presently encountered as well as the robust design approach for the earthworks, the ‘Uncertainty in Ground Conditions Classification System’ (in accordance with the Strategy for Managing Uncertainty in Ground Conditions (1MC06-CEK-GT-REP-C002-000127) is considered as Class 1.

3.7.5 Ground Conditions

A summary of the ground models and parameters developed for earthworks analyses relating to the Oat Close Bridleway SBH/19 Realignment assets is presented below. The ground models have been selected considering the asset specific exploratory points and the parameters have been selected in accordance with Appendix E and the IDR 207.2 Specification (1MC06-CEK-AU-SPE-CS04_CL18-000003).

The figure and tables below present the ground models applicable across the site, with Ground Model 1 covers the highway earthworks for Footpath SBH/27 realignment, whilst Ground Model 2 covers the highways earthworks for Bridleway SBH/19 Link, furthermore, due to the limited ground investigation data to the east of the SBH/19 Overbridge, both GM1 and GM2 have been applied.

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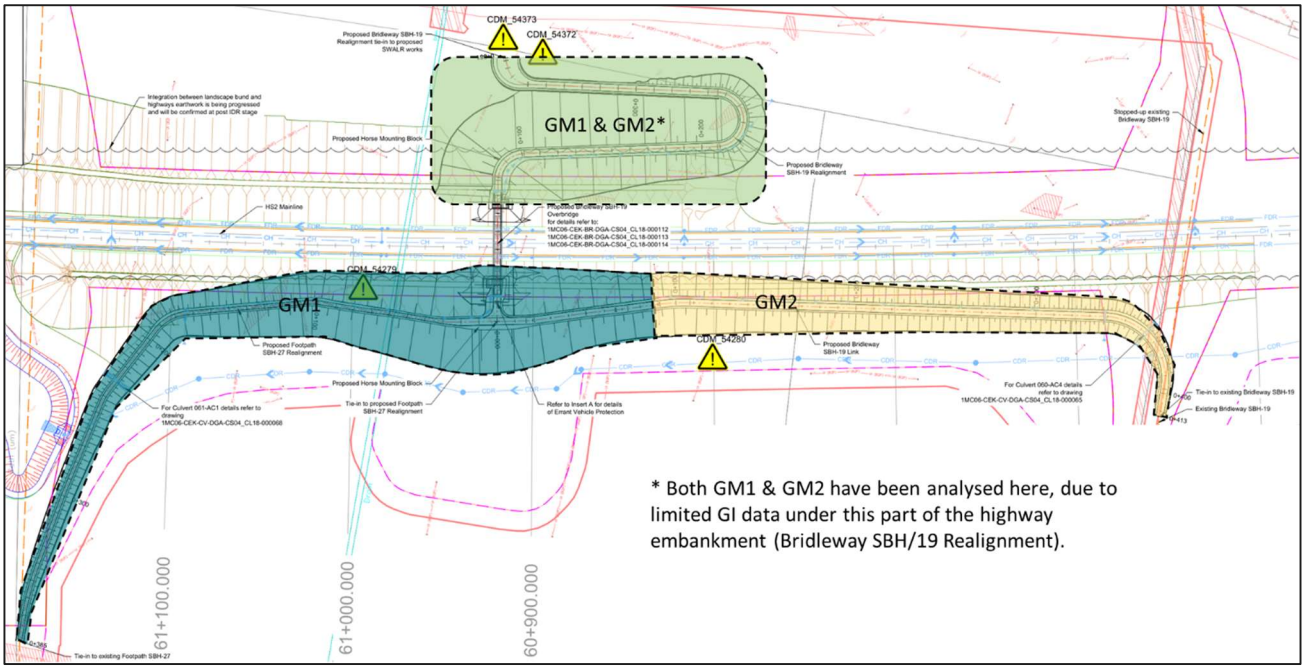


Figure 3-6: Ground models across site - Oat Close SBH-19 Realignment

Table 3-25: Ground Model 1 and Typical Geological Descriptions - Oat Close SBH-19 Realignment

Stratigraphy	Depth range (mbgl)	Summary Description
Gault Clay Formation (GLT – C Z)	1.70 – 16.35	Firm, becoming stiff to very stiff, grey and brown slightly sandy CLAY. Occasionally extremely weak grey sandy MUDSTONE, recovered as a firm, occasionally stiff, CLAY.
Whitchurch Sand Formation (WHS – C Z)	1.47	Firm to stiff grey occasionally slightly sandy silty CLAY with occasional shell fragments.

Table 3-26: Ground Model 2 and Typical Geological Descriptions - Oat Close SBH-19 Realignment

Stratigraphy	Depth range (mbgl)	Summary Description
Head (HEAD – C Z)	0.80 – 2.30	Soft to firm brown slightly sandy slightly gravelly CLAY
Gault Clay Formation (GLT – C Z)	2.00 - 7.55	Same as GM1
Whitchurch Sand Formation (WHS – C Z)	1.50	Firm grey slightly sandy slightly gravelly CLAY
Purbeck Group (PB – LMST)	1.85 – 2.30	Moderately weak to medium strong grey shelly LIMESTONE.

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Stratigraphy	Depth range (mbgl)	Summary Description
Portland Stone / Portland Sand Formations (POST – SLST/POSA – LMST)	3.90 – 10.10	Extremely weak to very weak, occasionally medium strong, grey SILTSTONE. Moderately weak to medium strong LIMESTONE.

Notes:

1) Occurrences of Made Ground are encountered in localised areas. It is assumed at these locations the formation will be inspected and soft spots will be highlighted. It is also assumed these will be removed during construction and replaced in line with the IDR 207.2 Specification (1MC06-CEK-AU-SPE-CS04_CL18-000003).

Table 3-27: Summary of Characteristic Parameters for Slope Stability and Settlement Assessment – Oat Close SBH/19 Realignment

Stratigraphy	γ (kN/m³)	PI(%)	ϕ' peak [ϕ'_{cv}] (°)	c' (kN/m²)	c_u (kN/m²)	E' (MPa)	E_u (MPa)	v'	v_u
Class 2A4 / 7C3	19 (1)		23 [21] (4 & 5) (**)	1	50 (12)	-	-	0.2	-
Head	18 (2)	40 (3)	22 [22] (4 & 5)	1(*)	40 (6)	9.6 (7)	12 (8)	0.2	0.5
Gault Clay Formation	19(2)	47 (3)	23 [21] (4 & 5)	1(*)	0 – 3m = 50 +3m = 50+10z <150 (9)	0 - 3m = 12 +3m = 12+2.4z (7)	0 - 3m = 15 +3m = 15+3z (8)	0.2	0.5
Whitchurch Sand Formation (***)	20 (2)	20 (11)	-	-	50 (9)	24 (7)	30 (8)	0.2	0.5

Notes:

(*) The cohesion value in the Head and the Gault Clay is assumed as 1kPa to consider the affect from rooting and negative pore pressure.

(**) Design assumption for drained parameters based on TQ4280.

(***) Parameters for the Whitchurch Sand Formation are only applicable where the Whitchurch Sand Formation is encountered at shallow depth due to geological faulting.

Z is the depth from 3.00m bgl.

(1) Characteristic unit weight obtained from Figure 1 in BS 8004:2015+A1 2020.

(2) Characteristic unit weight obtained from Figure 2 in BS 8004:2015+A1 2020.

(3) Derived using characteristic plasticity index using L2 asset specific design line for Head and the Gault Clay Formation (see plasticity plots)

(4) For cohesive material, the φ'cv has been derived the following equation:

$$\varphi' cv = (42° - 12.5 log 10 Ip)$$

Where Ip the soils plasticity index.

(5) For cohesive material, the peak angle of friction is based on the following equation:

$$\varphi' pk = \varphi' cv + \varphi' dil$$

Where $\varphi' dil$ is 2° for the Gault Clay Formation

Where $\varphi' dil$ is 0° for the Head

(6) Derived using characteristic Cu using L2 asset specific design line for Head (see Cu plot)

(7) E' is based on 0.8Eu.

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[8] Eu is based on relationship between Eu and Cu (after Jamiolkowski et al.).

For Head a PI of 40, Eu/Cu=300.

For the Gault Clay Formation a PI of 50, Eu/Cu=300.

For the Whitchurch Sand Formation a PI of 20, Eu/Cu=600

[9] Cu for the Gault Clay Formation and the Whitchurch Sand Formation have been taken from the Mainline Design in Aylesbury Embankment GDR for (1MC06-CEK-GT-REP-CS04_CL18-000037), based on the following rules:

[a] The Cu is adopted from Chainage 60+500 to 61+200, based on the Mainline Design Cu plot B-14 in the GDR.

[10] The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. Based on the geological descriptions, these Formations are assumed to be rigid stratum with a E'=100MPa as per the overbridge design.

[11] Parameters have been assessed using local ground investigation data or in the absence of data have been taken from Aylesbury Embankment GDR.

[12] Based on Table 6/1 in Series 600 for Class 2A4.

3.7.6 Groundwater Level

Groundwater was encountered during drilling within some exploratory hole positions across the mainline asset. Nearby groundwater monitoring shows the presence of a high variability in the water levels encountered during previous phases of GI.

The majority of monitoring indicates recorded groundwater levels ranging between 0.48m (81.10m AOD) and 4.13m bgl (78.49m AOD).

Based on these data, for the purposes of geotechnical design for stability and settlement, a conservative design groundwater level has been taken at existing ground level.

3.7.7 Slope Stability Analysis

The static global stability of earthworks has been calculated adopting the principles of limit state equilibrium in accordance with Eurocode 7. Specifically, Ultimate Limit State (ULS) Design Approach 1 Combination 1 (DA1 C1) and Combination 2 (DA1 C2) have been assessed adopting partial factors in accordance with Eurocode 7 with the requirement to achieve a utilisation less than or equal to 1.000.

Assessments have also been undertaken to satisfy the requirements of the HS2 Technical Standard – Earthworks. The table below summarises the results of the stability analyses for the following assessments:

- Analysis 1 to 4, 13, 14, 23 and 24 – Ground model 1 & 2 – Typical embankments with Class 2A4 fill, unsupported, varied slope.
- Analysis 5 to 12 – Ground model 1 & 2 – Embankment critical cross sections of Class 2A4 fill unsupported, varied slope.
- Analysis 15 to 22 – Ground model 1 & 2 – Class 2A4/7C3 fill embankment, soil reinforced, varied slope.
- Analysis 25 and 26 – Ground model 1 & 2 – Embankments with Class 2A4 fill, unsupported, 1V:3H, Interim scenario.
- Analysis 27 and 28 – Ground model 1 & 2 – Highways embankment interface with drainage ponds, unsupported, varied slope.

The following table presents a summary of the results for selected analysis, with further results presented in Appendix E.

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Table 3-28 - Summary of Slope Stability Results – Unreinforced Embankments – Oat Close SBH/19 Realignment

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
1	2.00	1V:3H	Class 2A4	0.966	0.326
2	2.50	1V:3H	Class 2A4	1.000	0.355
3	2.50	1V:3H	Class 2A4	1.000	0.253
4	3.00	1V:3H	Class 2A4	1.029 (4)	0.403
5 (Section AA)	8.10	1V:3.75H	Class 2A4	0.949	0.473
6 (Section AA)	8.10	1V:3.75H	Class 2A4	0.949	0.619
7 (Section CC)	3.40	1V:3H	Class 2A4/	1.071 (5)	0.294
8 (Section CC)	3.40	1V:3H	Class 2A4	1.136 (5)	0.415
9 (Section GG)	4.70	1V:3.9H	Class 2A4	0.868	0.299
10 (Section HH)	8.40	1V:3.3H	Class 2A4	1.064 (5)	0.493
11 (Section II)	7.00	1V:2.5H	Class 2A4	1.395 (5)	0.571
12 (Section JJ)	4.10	1V:4H	Class 2A4	0.901	0.419
13	9.60	1V:3H	Class 2A4	1.178 (5)	0.649
14	9.60	1V:3H	Class 2A4/	1.179 (5)	0.847
23	9.60	1V:3.5H	Class 2A4	1.034 (5)	0.544
24	9.60	1V:3.6H	Class 2A4	0.984	0.533

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

3) The results for the construction (short term) analysis are presented as the DOU for the undrained scenario. Whereas the long-term analysis are presented as the DOU for the drained scenario.

4) Analysis shows the maximum height a 1V:3H slope can sustain whilst been unsupported is 3.0m, thus soil reinforcement is required for 1V:3H slopes above 3.0m, see Table 3-29 for results of soil reinforcement analyses.

5) Analysis shows that slope is not stable for the analysed slope gradient and height, soil reinforcement will be required at this Section location, see Table 3-29 for results of soil reinforcement analyses.

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Based on the analysis from the table above the following is established:

- **Bridleway SBH/19 Realignment**:
  - The left-hand side slope, between Chainages 0+085 and 0+110, require geogrid reinforcement due sides slopes having a proposed gradient of 1V:3H and the earthwork heights between 9.00m and 10.00m.
  - Additionally, the right-hand side slope, which is no longer constrained by the Landscape Bund, between Chainages 0+220 and 0+240, require geogrid reinforcement due to the proposed gradient of 1V:3H and the earthwork heights exceeding 3.00m.
  - The left-hand side slope between Chainages 0+110 and 0+220 does not require geogrid reinforcement, due to the side slopes having a proposed gradient greater then 1V:3.5H.

- **Footpath SBH/27 Realignment**:
  - The left-hand side slope of Footpath SBH/27 realignment requires geogrid reinforcement between Chainages 0+000 and 0+090, due to the slope gradients being between 1V:3H and 1V:3.5H, and slope heights in excess of 3.00m.
  - Beyond Chainage 0+090, on the left-hand side, the slope gradient is greater than 1V:3.5H, so geogrid reinforcement is not required. Similarly, beyond Chainage 0+225, on the right-hand side, the highway earthwork is no longer constrained by the Landscape Bund and is less than 3.00m in height, therefore geogrid reinforcement is not required, despite the slope gradient of 1V:3H.

- **Bridleway SBH/19 Link**:
  - The right-hand side slope of Bridleway SBH/19 Link requires geogrid reinforcement between Chainages 0+000 and 0+080, due to the slope gradients being between 1V:2.5H and 1V:3.5H, and slope heights in excess of 3.00m.
  - Beyond Chainage 0+080, on the right-hand side, the slope gradient is greater than 1V:3.5H, so geogrid reinforcement is not required. Similarly, beyond Chainage 0+360, on the left-hand side, the highway earthwork is no longer constrained by the Landscape Bund and is less than 3.00m in height, therefore geogrid reinforcement is not required, despite the slope gradient of 1V:3H.

Table 3-29 - Summary of Slope Stability Results - Reinforced Embankments (Class 2A4/7C3) – Oat Close SBH/19 Realignment

Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	DoU (As Per BS8006)	DoU (As Per DA1 C2)
15 (Section CC)	3.40	1V:3H	3 (Primary Reinforcement) 3 (Secondary Reinforcement)	0.854	0.942
16 (Section CC)	3.40	1V:3H	3 (Primary Reinforcement) 3 (Secondary Reinforcement)	0.879	0.990
17 (Section HH)	8.40	1V:3.3H	7 (Primary Reinforcement) 7 (Secondary Reinforcement)	0.774	0.918
18 (Section II)	6.40	1V:2.5H	6 (Primary Reinforcement) 5 (Secondary Reinforcement)	0.907	1.016 (4)
19 (Section II)	6.40	1V:2.5H	1 Basal Reinforcement 5 (Primary Reinforcement) 5 (Secondary Reinforcement)	0.922	0.985
20	9.60	1V:3H	9 (Primary Reinforcement) 9 (Secondary Reinforcement)	0.856	0.993

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Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	DoU (As Per BS8006)	DoU (As Per DA1 C2)
21	9.60	1V:3H	9 (Primary Reinforcement) 9 (Secondary Reinforcement)	0.867	1.034 (5)
22	9.60	1V:3H	1 Basal Reinforcement 8 (Primary Reinforcement)	0.851	0.986

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

3) A 2m deep vadose zone has been modelled within the highway embankment with an r_u of 0.2 with the remaining embankment fill modelled with an r_u of 0.1 for all embankment heights.

4) The results for Analysis 18 (GM1) show that for slope gradients of between 1V:2.5H and 1V:3H, a basal geogrid reinforcement layer is required in order to maintain the stability of the side slope. Adding a basal geogrid reinforcement layer will decrease the DOU to <1.000 in DA1 C2, as shown by Analysis 19 (GM1).

5) The results for Analysis 21 (GM2) show that for slope gradients of 1V:3H at 9.60m in height, a basal geogrid reinforcement layer is required in order to maintain the stability of the side slope. Adding a basal geogrid reinforcement layer will decrease the DOU to <1.000 in DA1 C2, as shown by Analysis 22 (GM1).

6) Details of geogrids considered as part of the analysis are included in Section 5.3.

Table 3-30: Slope Stability Results – Temporary Condition – Oat Close SBH/19 Realignment

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
25	10.3	1V:3H	Class 2A4	1.000	0.642
26	10.3	1V:3H	Class 2A4	1.000	0.835

Notes:
1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

Table 3-31: Slope Stability Results – Pond Analysis – Oat Close SBH/19 Realignment

Analysis No.	Highway Embankment Height (m)	Pond Slope Gradient	Pond Distance from Embankment Toe (m)	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
27	0.42	1V:3H	3.80	0.989	0.173
28	1.64	1V:5H	1.60	0.942	0.185

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

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Based on the results presented above, the following tables describes the slope stability design along the alignment of the asset. For further details of the soil reinforcement refer to Section 5.3.

Table 3-32: Summary of Slope Stability Design

Left Hand Side		Right Hand Side	
Ch.	Solution	Ch.	Solution
Bridleway SBH/19 Realignment			
0+000 to 0+015	Supported by the Landscape Bund	0+000 to 0+015	Supported by the Landscape Bund
0+015 to 0+071	Overbridge	0+015 to 0+071	Overbridge
0+071 to 0+085	Supported by the Landscape Bund		
0+085 to 0+110	Geogrid Reinforcement	0+071 to 0+220	Supported by the Landscape Bund
		0+220 to 0+240	Geogrid Reinforcement
0+110 to 0+220	Unsupported	0+240 to 0+381	Unsupported
Bridleway SBH/19 Link			
0+000 to 0+360	Supported by the Landscape Bund	0+000 to 0+080	Geogrid Reinforcement
0+360 to 413	Unsupported	0+080 to 0+413	Unsupported
Footpath SBH/27 Realignment			
0+000 to 0+090	Geogrid Reinforcement	0+000 to 0+225	Supported by the Landscape Bund
0+090 to 0+385	Unsupported	0+225 to 0+385	Unsupported

Notes:

Side of embankment taken as if looking up chainage.

3.7.8 Ground Movement

3.7.8.1 Settlement Analyses

Ground movement has been assessed in the form of settlement below the base of the embankments. There are no cuttings as part of the highway earthworks thus there is no potential for heave.

Settlement analysis has been carried out at the location of eastern embankment for Bridleway SBH/19 Realignment Approach Embankment, to determine:

- the total settlement,
- the immediate settlement (assumed to occur during construction of the embankment), and

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- the consolidation settlement (assumed to occur in the long term after the immediate settlement has ceased) to assess the impact of total and differential settlements.

This portion of embankment is considered to be the critical case and therefore, no analysis was carried out on the highway embankments of Bridleway SBH/19 Link and Footpath SBH/27 Realignment due to the low embankments and its lack of proximity to any structures.

The results of settlement analysis for Bridleway SBH/19 Realignment are presented in Table 3-33 below.

Table 3-33: Summary of Settlement Results – Oat Close SBH/19 Realignment

Longitudinal Zone No.	Height (m)	Immediate Settlement (Si) (mm)	Consolidation Settlement (Sc) (mm)	Total Settlement (St) (mm)
Load 15 - OBr1	10	69	88	158
Load 16 - BRA1	10	64	114	179
Load 19 - BRA2	9	56	106	162
Load 25 - BRA3	8	48	99	147
Load 34 - BRA4	7	40	88	128
Load 44 - BRA5	6	32	77	109
Load 51 - BRA6	5	25	65	90
Load 63 - BRA7	4.5	21	52	73
Load 64 - BRA8	3	14	34	48

3.7.8.2 Transition to Structures

Settlement of non-mainline highway earthworks can result in differential movement between structures and approach earthworks leading to an impact on the quality and can be a hazard to pedestrians.
Currently there is no guidance provided by HS2 on the permissible limits on settlement for approach earthworks to non-mainline structures. In the absence of any guidance, the following limits are proposed:

Bridleways and Footpaths:

- Criterion 1: Between 0 and 10m back from a structure – post construction (consolidation) settlement should not exceed 100mm.
- Criterion 2: 0 to 20m back from a structure – post construction differential settlement should not exceed 1V:100H (1.0%)
- Criterion 3: >20m Along alignment – post construction differential settlement should not exceed 1V:100H (1.0%) from design gradient.

The settlement results (see Table 3-33 summary of Settlement Results and Figure 3-7 below) show a maximum consolidation settlement of 114mm, which is slightly above the 100mm criteria limit. Considering that the consolidation settlement will commence as soon as the construction work begins and the embankment construction will take place over a period of months, no specific hold period is required before construction of the permanent works, including the pavement. Additionally, it should also be considered that the Overbridge is expected to settle at the same time.

As shown on the chart below (red line) the predicted differential settlement is less than the 1V:100H allowable differential settlement limit (black line) at a distance of 0m to 10m back from the overbridge.

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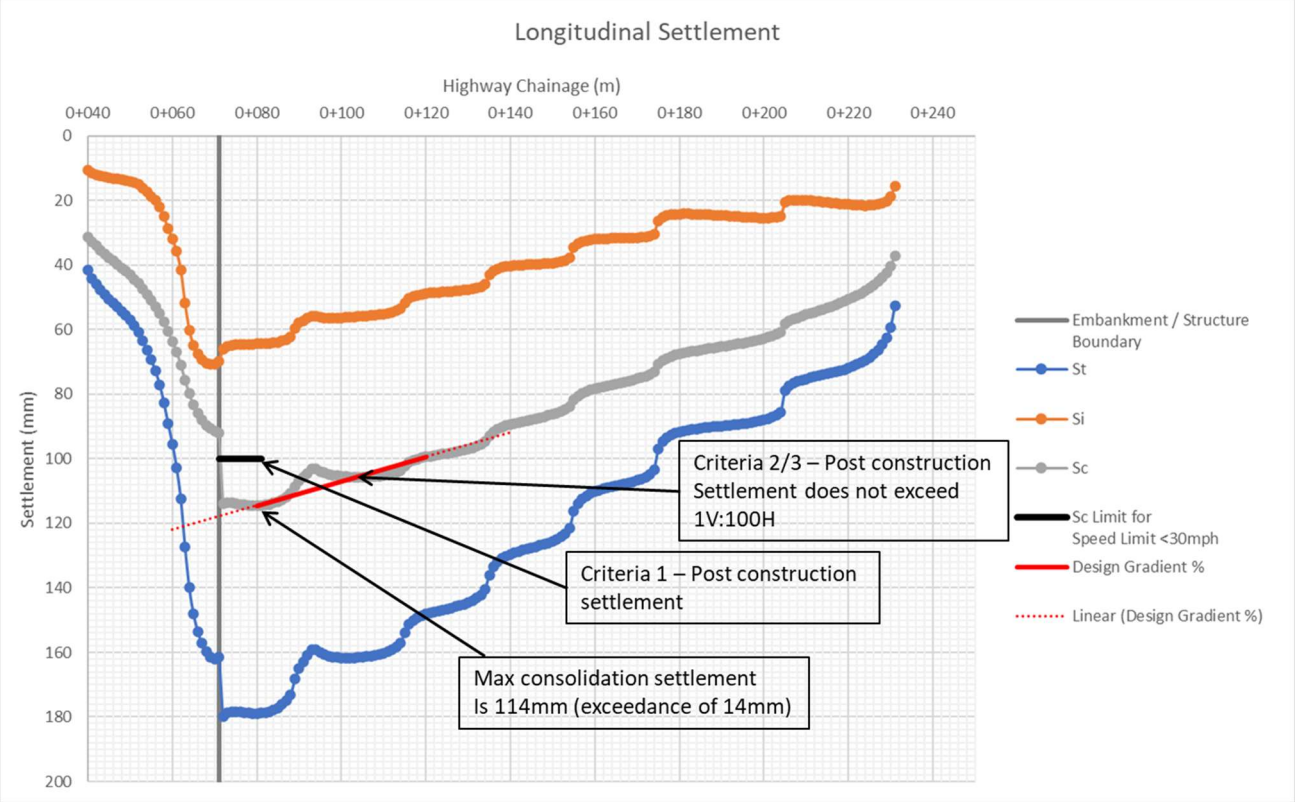


Figure 3-7- Longitudinal Settlement – Oat Close SBH/19 Realignment

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3.7.9 Pavement and Subgrade Design

The pavement foundation subgrade varies along the alignment based on the changing ground conditions (for at grade roads or low embankments) as well as the type of embankment fill. The table below summarises the design subgrade CBR (which is based on the lowest of the short term and long equilibrium values in accordance with CD225).

Table 3-34: Summary of Pavement Foundation Design – Oat Close SBH/19 Realignment

Permitted Pavement Options								
Description (e.g., mainline)	Chainage (m)		Permitted Pavement Option	Pavement Type	CBR (%)	Total Thickness without GEO input (mm)	Geotechnical Input	Total Thickness with GEO input (mm)
	From	To						
Bridleway SBH/19 Realignment (South)	0	5	AT2	Unbound	2.0	250	None [1]	-
	5	20	C1	Bound	2.0	245	None [2]	-
Bridleway SBH/19 Realignment (North)	65	85	C1	Bound	2.0	245	None [2]	-
	80	381	AT2	Unbound	2.0	250	None [1]	-
Bridleway SBH/19 Link	0	414	AT2	Unbound	2.0	250	None [1]	-
Footpath SBH/27 Realignment	0	387	AT2	Unbound	2.0	250	None [1]	-

Notes:

[1] Based on the use of the asset as a bridleway and footpath, the currently proposed pavement thickness will be sufficient to support the anticipated loadings thus no further GEO input is required.

[2] Based on the overbridge design, the pavement type C1 will be founded on either Class 6N6 or Class 1A4.

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3.7.10 Summary of Highway Design

The following table summarises the geotechnical design along the alignment.

Table 3-35: Summary of Design – Bridleway SBH/19 Realignment

Element	Left Hand Side							Right Hand Side						
	0+000 to 0+015	0+015 to 0+071	0+071 to 0+085	0+085 to 0+110	0+110 to 0+230	0+230 to 0+290	0+290 to 0+381	0+000 to 0+015	0+015 to 0+071	0+071 to 0+220	0+220 to 0+240	0+240 to 0+270	0+270 to 0+290	0+290 to 0+381
Embankment Heights (m)	8.63 – 9.72		10.35 – 9.57	9.57 – 8.66	8.66 – 3.39	3.39 - <0.75	<0.75	8.63 – 9.72		10.35 – 3.94	3.94 – 2.91	2.91 – 1.94	1.94 - <0.75	<0.75
Side Slope Angle	1V:3H		1V:3H	1V:3H to 1V:3.5H	>1V:3.5H	>1V:3.5H to 1V:2H [2]	1V:2H to 1V:1H [2]	1V:3H		1V:3H				1V:3H to >1V:3H
Side Slope Design	Soil slope supported by the Landscape Bund		Soil slope supported by the Landscape Bund	Reinforced	Soil Slope			Soil slope supported by the Landscape Bund		Soil slope supported by the Landscape Bund	Reinforced	Soil Slope		
Embankment Fill Class	Class 2A4		Class 2A4	Class 2A4 / 7C3	Class 2A4		Class 1A4 or 2A4	Class 2A4		Class 2A4	Class 2A4 / 7C3	Class 2A4		Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4 / Class 6N6 or 1A4 [1]		Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4		GLT [3]	Class 2A4 / Class 6N6 or 1A4 [1]		Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4			GLT [3]
Subgrade DSG CBR Value (%)	2.0	Overbridge	2.0					2.0	Overbridge	2.0				
Subgrade Improvement Layer	No		No					No		No				
Pavement thickness (mm)	250		250					250		250				
Earthwork Geotextile Separator Layer	Yes		Yes					Yes		Yes				
Topsoil Strip	No		No			Yes [4]		No		No		Yes		
Starter Layer	Yes		Yes			No [5]		Yes		Yes		No		
Excavate and Replace	Yes [6]		Yes [6]	No				Yes [6]		Yes [6]	No			
Horizontal Drainage Layers	Yes		Yes[7]	No				Yes		Yes [7]	No			
Mitigation SED	G105		G105	G104	G103, G104	G102		G105		G105, G104	G104, G103	G103	G103, G102	G102

Notes:
1 – The Class 6N6 or 1A4 is a requirement from the overbridge design. Refer to Section 7.3 for extent of Class 6N6 or 1A4.
2 – Due to the geometry of the bridleway, there is a small gully on the left-hand side slope. Beyond Ch. 0+260, the right-hand slope of this small gully is 0.30m high with slope gradients of between 1V:2H and 1V:1H (Ch. 360).
3 - GLT = Gault Clay Formation.
4 – The topsoil strip will extend from Chainage 0+240, due to the embankment height being less than 3.00m
5 – Beyond Ch. 0+240, a starter layer is not required, due to the embankment height being less than 3.00m
6 – Excavate and Replace is required below the abutments as per the Overbridge design.
7 – The horizontal drainage layer only extends to Ch. 0+110 on left-hand side and to Ch. 0+220 on the right-hand side of the embankment to accelerate consolidation. Refer to standard earthwork details G105.

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Table 3-36: Summary of Design –Bridleway SBH/19 Link

Element	Left Hand Side			Right Hand Side					
	0+000 to 0+360	0+360 to 0+390	0+390 to 0+413	0+000 to 0+020	0+020 to 0+080	0+080 to 0+100	0+100 to 0+190	0+190 to 0+390	0+390 to 0+413
Embankment Heights (m)	8.63 – 1.74	1.74 – <0.75	<0.75	8.63 – 8.36	8.36 – 4.97	4.97 – 4.20	4.20 – 2.99	2.99 – <0.75	<0.75
Side Slope Angle	1V:3H			1V:2.9H – 1V:3.1H	1V:2.5H to 1V:3.3H	1V:3.7H to 1V:4H	1V:4H		
Side Slope Design	Soil slope supported by the Landscape Bund	Soil Slope		Reinforced		Soil Slope			
Embankment Fill Class	Class 2A4 [1]		Class 1A4 or 2A4	Class 2A4 / 7C3		Class 2A4			Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4		GLT [2]	Class 2A4		Class 2A4			GLT [2]
Subgrade DSG CBR Value (%)	2.0			2.0					
Subgrade Improvement Layer	No			No					
Pavement thickness (mm)	250			250					
Earthwork Geotextile Separator Layer	Yes			Yes					
Topsoil Strip	No [5]	Yes		No				Yes	
Starter Layer	Yes [6]	No		Yes				No	
Excavate and Replace	No			No					
Horizontal Drainage Layers	Yes [3]	No		Yes [3]	No				
Mitigation SED	G103, G104 and G105	G103	G102	G105	G104			G103	G102

Notes:
1 – The fill material highway embankment between Ch. 0+000 and 0+010 will include the granular 6N Fill used for the Overbridge design.
2 - GLT = Gault Clay Formation.
3 - Horizontal drainage layer required from Ch. 0+000 to 0+020 on the left-hand side and 0+000 to 0+360 on the right-hand side of the embankment to accelerate consolidation (15m back from overbridge abutment).
4 – For details of the pavement design (unbound) refer to Page 39 of Appendix E.2.
5 – The topsoil strip will extend from Chainage 0+190, due to the embankment height being less than 3.00m
6 – Beyond Ch. 0+190, a starter layer is not required, due to the embankment height being less than 3.00m

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Table 3-37: Summary of Design –Footpath SBH/27 Realignment

Element	Left Hand Side							Right Hand Side	
	0+000 to 0+020	0+020 to 0+060	0+060 to 0+090	0.090 to 0+100	0+100 to 0+170	0+170 to 0+360	0+360 to 0+385	0+000 to 0+360	0+360 to 0+385
Embankment Heights (m)	8.63 – 8.73	8.73 – 6.49	6.49 – 5.07	5.07 – 4.65	4.65 – 2.85	2.85 - 0.75	<0.75	8.63 – 0.75	<0.75
Side Slope Angle	1V:3.1H	1V:3.3H	1V:3.5H	1V:3.7H	1V:4H			1V:3H	
Side Slope Design	Reinforced			Soil Slope				Soil slope supported by the Landscape Bund [4]	Soil Slope
Embankment Fill Class	Class 2A4/7C3			Class 2A4			Class 1A4 or 2A4	Class 2A4 [5]	Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4						GLT [1]	Class 2A4	GLT [1]
Subgrade DSG CBR Value (%)	2.0							2.0	
Subgrade Improvement Layer	No							No	
Pavement thickness (mm)	250							250	
Earthwork Geotextile Separator Layer	Yes							Yes	
Topsoil Strip	No					Yes		No [6]	Yes
Starter Layer	Yes					No		Yes [7]	No
Excavate and Replace	No							No	
Horizontal Drainage Layers	Yes [2]	No						Yes [8]	No
Mitigation SED	G105	G104			G103 G104	G103	G102	G103, G104, G105	G102

Notes:
1 – GLT = Gault Clay Formation.
2 - Horizontal drainage layer required from Ch. 0+000 to 0+020 on the left-hand side of the embankment to accelerate consolidation (15m back from overbridge abutment).
3 – For details of the pavement design (unbound) refer to Page 39 of Appendix E.2.
4 – Highway earthwork to be supported by the landscape bund from the edge of the abutment to Ch. 0+225, beyond which the right-hand side is an unsupported soil slope.
5 – The fill material highway embankment between Ch. 0+000 and 0+010 will include the granular 6N Fill used for the Overbridge design.
6 – Beyond Ch. 0+170, a topsoil strip is required due to the embankment height being less than 3.00m.
7 - Beyond Ch. 0+170, a starter layer is not required due to the embankment height being less than 3.00m.
8 – A drainage layer is required between Ch. 0+000 and 0+360) on the right-hand side of the embankment to accelerate consolidation.

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3.7.11 Risk Management

3.7.11.1 Introduction

The geotechnical hazards associated with this asset are high/shallow groundwater, soft and compressible ground, Made Ground/Topsoil Material, flood, and shrink/ swell potential.

3.7.11.2 Soft/ Compressible Ground

Soft ground within Made Ground, Head deposits and weathered clay in the upper Gault Clay Formation could cause instability of embankment slopes and/or unacceptable total or differential settlements/ insufficient stiffness. Potential reduction in the strength of Made Ground, Head deposits and weathered clay in the upper Gault Clay likely caused by exposure to rainfall during excavation works. The formation of assets is to be tested prior to construction of the earthworks and where soft ground is encountered it is to be treated in line with the IDR 207.2 Specification (1MC06-CEK-AU-SPE-CS04_CL18-000003).

3.7.11.3 Chemically aggressive ground

The Gault Clay Formation is anticipated to contain heightened levels of sulphates and sulphides, indicating the potential necessity for sulphate-resistant concrete in foundations that come into contact with it.

3.7.11.4 Shrink and Swell Potential

Potential for shrink / swell behaviour in the Gault Clay Formation due to changes in moisture content in high plasticity clays, which could cause unacceptable total or differential settlements. The shrink swell potential has been taken into account in the design parameters and construction of earthworks should be avoided during poor weather.

3.7.11.5 Groundwater Management

High or perched groundwater levels may be encountered in excavations during construction. The latest AGS data at the time of writing this document reports groundwater levels ranging between 0.48m (81.10m AOD) and 4.13m bgl (78.49m AOD).

3.7.11.6 Variable weathering profile

Potential variable weathering profiles beneath the formation level may exist, introducing potential disparities in strength and compressibility within the foundation soils. These variations pose an elevated risk of excessive settlement and instability.

3.7.11.7 Geological faulting

The presence of faulting may lead to an incorrect ground model adopted for Oat Close Bridleway (SBH/19) Realignment.

Faulting may result in highly variable ground conditions, increased risks associated with ground stability, settlement and high groundwater flows in the area which could affect the foundation design for Oat Close Bridleway (SBH/19) Realignment. Differential movements are more likely in a faulted zone.

3.7.11.8 Relict periglacial features

Low strength relic shear surfaces may be present in the superficial Head Deposits and the Gault Clay beneath the embankment.

Possible slickenside surfaces were encountered in the Gault Clay in borehole ML060-CR011 at depths of between 14.23m bgl and 16.65m bgl.

Reactivation of these low strength shear surfaces may result in local slips which may destabilise the embankment.

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3.7.12 Reuse of Material

A summary of the material likely to be produced on site:

- A nominal 300mm topsoil strip is anticipated for the entire highway alignment, to be reused as Class 5.
- In areas of minor cutting where Head is encountered, it will be recovered and is assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface materials.
- Made Ground if encountered shall be stockpiled and tested to determine its suitability for re-use in the Design.

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4 Structures

4.1 Introduction

This section of the report summarises the design for the following assets;

- Sedrup North Culvert.
- Bridleway SBH/19 Overbridge
- Ancillary culverts

4.2 General Definitions

Not applicable.

4.3 Standards and Design Approach

Please refer to the details in the structure specific sections of this report and the asset specific geotechnical calculation report for details of the standard and design approach of each individual structure.

4.4 Design Procedures

Please refer to the details in the structure specific sections of this report and the asset specific geotechnical calculation report for details of the methods of analysis and design of each individual structure.

4.5 Sedrup North Culvert

The geotechnical design for Sedrup North Culvert is included in this section.

4.5.1 Asset Description

The proposed culvert will be a box culvert with 3.3 x 3.8m external dimensions and the total length of 94.09m. The culvert's inlet level is to be constructed at an elevation of 76.746m AOD and with an outlet level of 76.644m AOD (based on the drawing 1MC06-CEK-CV-DSE-CS04_CL18-000031).

4.5.2 Construction Method and Sequence

It has been assumed that earthworks above the culvert (mainline and landscape bund earthworks) will be constructed concurrently. No allowance has been made in the design for sequencing the earthworks construction along the culvert.

The Sedrup North culvert geotechnical design requires a mitigation measure consisting of a minimum of 2m of excavate and replacement (E&R) below the culvert base using Class 1A2 material.

After excavation for E&R up to formation level, i.e., a shallow excavation of 11.3m x 105m size and 2m deep below culvert founding level, a hold point has been proposed for geological mapping of the excavation to ascertain the location of the potential fault line as per Appendix 6/3 of 1MC06-CEK-AU-SPE-CS04_CL18-000002, Ref. [65]

During this hold point, a qualified and competent Geotechnical Engineer shall inspect the excavation to confirm the stratigraphy & location of the potential fault line. The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level. If the potential fault line is confirmed the location, then the direction of the fault line, the properties of the soils within the fault line and the site photos etc., shall be included in the report.

The undrained stiffness (Eu ≥25MPa) or undrained shear strength (Cu ≥85kPa) shall be confirmed at the excavation base along the footprint of the culvert in cohesive soil. Confirmation of stiffness shall be completed by a plate bearing test in accordance with BS 1377, with a minimum 600mm diameter plate. Confirmation of undrained shear strength shall be completed by hand vane shear tests. A qualified and competent geologist

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shall carry out field identification of rock as per Table 5 of BS EN ISO 14689, Ref [66] to identify the rock as weak to medium strong rock.

A detailed description of the culvert to earthworks transition design is presented in Section 2.4.10

According to the EKFB Physical Infrastructure Diagram drawing with number 1MC12-EKF-SY-SKE-C000-000001 dated 04/10/2022, Ref. [65], there are no switches and crossings 150m before or after Sedrup North Culvert.

4.5.3 3rd Party Asset Constraints on Design

Not applicable.

4.5.4 Ground Investigation and Field Trials

A summary of the boreholes used to determine the ground model specific to the asset has been tabulated in Table 4-1.

The ground models adopted for the design were built based on the nearby boreholes (i.e., as encountered in ML061-CR012, ML061-CR462 and ML061-CR013). The maximum depth of local boreholes was 20.2m.

Table 4-1: Relevant boreholes for Sedrup North Culvert

Ground Investigation ID	Type	Top of Hole (m AOD)	Eastings	Northings	Final Depth (m)	Distance From Asset (m)	GI Status
ML061-CR012	CP+RC	77.68	243882	318594	20.00	55	Final
ML061-CR013	CP+RC	78.06	243890	318497	20.00	47	Final
ML061-WS020	WLS	78.40	243902	318449	4.00	84	Final
ML061-WS023	WLS	77.46	243818	318596	2.45	78	Final
ML061-WS025	WLS	77.87	243747	318535	5.00	20	Final
ML061-CT400	CPT	77.84	243869	318536	1.27	10.6	Final
ML061-CR462	**RC**	**77.87**	**243902**	**318550**	**20.20**	**At Asset**	**Final**

To examine the variation in the strata at the culvert, i.e. to verify the potential fault in the vicinity of the culvert, the following tabulated exploratory holes are also used. These exploratory holes are approximately within 200m of the upline and downline from the Culvert along the mainline.

Ground Investigation ID	Type	Top of Hole (m AOD)	Eastings	Northings	Final Depth (m)	Distance From Asset (m)	GI Status
ML061-CR014	CP+RC	77.96	243762	318654	20.00	170	Final
ML061-CR442	CP+RC	77.56	243777	318608	20.30	125	Final
ML061-CR008	CP+RC	79.93	243965	318403	25.00	155	Final
ML061-CR009	CP+RC	79.40	243976	318428	25.00	140	Final

Notes:

CP- Cable Percussion
CP+RC – Cable Percussion and Rotary Core
TP – Trial Pit
WLS – Dynamic (Windowless) Sampling

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4.5.5 Ground Conditions

The variation in stratigraphy encountered between ML061-CR012 and ML061-CR013 suggested the presence of the potential fault line within the area of the Sedrup North Culvert.

The exploratory borehole ML061-CT400 (CPT) was proposed at the location of the Sedrup North culvert ASC detailed GI program. However, the CPT could not penetrate any past 1.27m; thus, no information regarding the deeper strata and presence of the potential fault line was gained.

Therefore, to mitigate the risk of uncertainty on the ground, an additional GI ML061-CR462 was proposed in EW-1283 dt-8th Oct 2021 & TQ-1402 dt-13th Sept 2021. The additional GI was made available in Data Drop 30. The exploratory hole ML061-CR462 validated the stratigraphy recorded in borehole ML061-CR012, which includes a superficial deposit of topsoil, a head deposit, and an Alluvium formation overlying the Portland formation, followed by the Kimmeridge clay formation.

However, the asset-specific exploratory hole ML061-CR462 and CPT ML061-CT400 provided validation of variation in strata along the culvert length. As a result, when boreholes in the 200m vicinity on each side of the culvert and along the mainline are studied (Refer to Figure H 2.3 and Figure H 2.4 in Appendix H), the following details are observed,

- Boreholes ML061-CR462, ML061-CR012 and ML061-CR014 demonstrated similar stratigraphy, which has a superficial deposit followed by Portland clayey formation up to ~7mbgl, which is overlying the Kimmeridge Clay formation.
- Boreholes ML061-CR013, ML061-CR442 and ML061-CR008 demonstrated similar stratigraphy, which has superficial deposit followed by Portland/Purbeck Limestone formation up to ~18mbgl overlying the Kimmeridge clay formation. The CPT ML061-CT400 terminated at a shallow depth, unable to penetrate the limestone formation.

This confirms the presence of the potential fault line at the footprint of the Sedrup North Culvert.

Consequently, two ground models are developed for the current design,

- **Ground Model 1 (GM1)** - Based on exploratory holes ML061-CR012 and ML061-CR462 stratification,
- **Ground model 2 (GM2)** - Based on exploratory hole ML061-CR013

Although the possibility of the potential fault line below the culvert location has been confirmed, its precise location remains uncertain. Therefore, for the current design, the most pessimistic location, i.e., adjacent to borehole ML061-CR462, is being considered.

Table 4-2 and Table 4-3 summarise the design ground models GM1 and GM2 for Sedrup North Culvert.

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Table 4-2: Ground Model-1 (GM-1) at Sedrup North Culvert

Geological Unit	Top of Stratum Range (m AOD)	Design Top Level (m AOD)	Range of Thickness (m)	Design Thickness (m)
Topsoil (TOP-CZ)	77.87-77.38	77.70	0-0.3	0.30
Alluvium (ALV-CZ)	77.57-75.48	77.40	0.3-2.2	1.00
Head Deposits (HEAD-S V SV)	77.18-75.68	76.40	0.5-2.0	1.00
Portland Stone Formation (POST-CZ)	75.48-69.86	75.40	2.2-7.82	3.70
Kimmeridge Clay Formation (KC-CZ)	71.35-57.67	71.70	6.52-20.2	21.7*

* The maximum depth of investigation data available is 20.2m. *Kimmeridge clay depth is assumed to be extended 50mAOD.

Table 4-3 Ground Model-2 (GM-2) at Sedrup North Culvert

Geological Unit	Top of Stratum Range (m AOD)	Design Top Level (m AOD)	Range of Thickness (m)	Design Thickness (m)
Topsoil (TOP-CZ)	78.06-77.76	77.70	0-0.3	0.30
Alluvium (ALV-CZ)	77.76-76.06	77.40	0.3-2	1.40
Purbeck Limestone (PB-CZ)	76.06-72.86	76.00	2-5.2	3.10
Portland Stone Formation (POST-LMST)	72.86-65.36	72.90	5.2-12.7	7.60
Portland Sand Formation (POSA-CZ)	65.36-59.46	65.30	12.7-18.6	5.90
Kimmeridge Clay Formation (KC-CZ)	59.46-50.0	59.50	18.6-20*	9.5*

* The maximum depth of investigation data available is 20.2m. *Kimmeridge clay depth is assumed to be extended 50mAOD.

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4.5.6 Groundwater Conditions

The groundwater monitoring was carried out in ML061-CR012 and ML061-CR013 between July 2016 and March 2018. The groundwater level (GWL) varied between 0.54m bgl and 2.07m bgl. The design GWL was considered at the Existing ground level.

4.5.7 Geotechnical Model

The characteristic geotechnical parameters adopted in culvert design and their justification of derivations are provided in Appendix H-1 of this report. A summary of the Geotechnical Parameters used in the design is shown in Table 4-4.

Table 4-4: Summary of the Geotechnical Parameters

Geological Unit	γ_b kN/m^3	ϕ'_{cv} °	c' kPa	c_u kPa	E' MPa	Eu MPa	v_d -	UCS MPa	PI %	SPT N range	Permeability (m/s) kx	Permeability (m/s) ky
Alluvium (ALV-CZ)	19	21	-	30	5.4	9	0.15	-	48	5-22	1.0E-10	1.0E-10
Head Deposits (HEAD-S V SV)	19.5	30	-	-	16	-	0.25	-	18	17	6.9E-06	7.0E-06
Portland Stone Formation (POST-CZ)	21	21	-	60+44z (150)	25	18+12z (45)	0.15	-	30	-	1.0E-08	1.0E-08
Kimmeridge Clay (KC-CZ)	19.5	25	10	110+15.4z (250)	35.2+4.9z (80)	44+6.1z (100)	0.15	-	26	17-37	1.0E-08	2.0E-09
Purbeck Formation (PB-LMST)	21	25	-	-	250	-	0.15	1.5	-	-	6.9E-06	7.0E-06
Portland Stone Formation (POST-LMST)	21	25	-	-	480	-	0.15	22.6	-	-	6.9E-06	7.0E-06

Note: z is taken from the top of respective ground strata

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4.5.8 Culvert Design

The design was carried out in line with PD6694 [39] and BS EN 1997 Part 1 [4], adopting partial factors applicable to design approaches DA1-C1 and DA1-C2.

The design assessed temporary and permanent stability, spring stiffnesses, total settlement and damage through horizontal separation of adjacent culvert segments. Refer to Appendix H for detailed calculation and corresponding loads considered in the design of Sedrup North Culvert.

The analysis has been carried out considering two different ground models as presented in Section 4.5.5, i.e. GM1 and GM2, due to the potential fault line at the footprint of the culvert.

The analysis assumed that a minimum of 2m of soil/rock would be excavated below the culvert founding level and would be replaced with Class 1A2 fill. The culvert will be founded directly on the replacement material. The undrained stiffness parameter (Eu = 25MPa) for the culvert founding material will be confirmed on-site.

The tolerance for out-of-balance placement of fill material to the sides of the culvert is 250mm on each side. Placement of fill with differential levels greater than 250mm on either side of the culvert requires a temporary works design.

It has been assumed that earthworks above the culvert (mainline and landscape bunds earthworks) will be constructed concurrently. No allowance has been made in the design for sequencing the earthworks construction along the culvert.

A minimum of 0.8m cover over the culvert shall be installed before heavy construction vehicle movements (e.g. A60H vehicles) may cross the culvert location.

The transition detail for the culvert is shown on drawing 1MC06-CEK-GT-DDE-CS04_CL18-000003.

4.5.8.1 Spring Stiffness

The calculations of subgrade reaction/spring stiffness of the ground below the culvert are required by the structures team to assess the effects of load applied on the culvert. Spring stiffness was determined by modelling the culvert, existing ground and embankment conditions in Plaxis2D.

The embankment self-weight, permanent rail loads and LM71 loads are applied over the culvert length at existing ground level in the Plaxis model, and displacements are estimated at every 2m intervals. The maximum and minimum spring stiffness values have been recommended for structural assessment.

The settlement assessment is carried out with a combination of GM1 and GM2 to address the uncertainty in the location of the potential fault line. Accordingly, Table 4-5 presents the maximum and minimum spring stiffness values obtained over 2m intervals throughout the culvert length.

Table 4-5: Spring Stiffness considered for structural at inlet/outlet and the centre of the culvert

Spring stiffness (kN/m²/m)	
Minimum	Maximum
1879	5558

4.5.8.2 Stability Analysis

The culvert has been designed in accordance with the requirements of Eurocode 7 and PD 6694-1:2011, Ref. [39] to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

The detailed stability calculations shall be referred to the following Appendices:

- Appendix H-5A Sedrup North Culvert Stability Design Sheet -Stage 1 - No cover

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- Appendix H-5B Sedrup North Culvert Stability Design Sheet -Stage 2A
- Appendix H-5C Sedrup North Culvert Stability Design Sheet -Stage 2B
- Appendix H-5D Sedrup North Culvert Stability Design Sheet -Stage 2C
- Appendix H-5E Sedrup North Culvert Stability Design Sheet -Stage 2D
- Appendix H-5F Sedrup North Culvert Stability Design Sheet -Stage 3 - HS2 in Service

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C), Ref [67]. The partial factors on are taken from the National Annex to BS EN 1997 Table A.NA.4, Ref [4].

The coefficient of earth pressure values K values is taken from Table B.5 of PD 6694-1:2011, Ref [39], Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004, Ref [4]. The most onerous loading conditions which the culvert structures are likely to experience have been taken into consideration for design.

The GM1 has relatively less stiff strata than GM2; consequently, the GM1 is critical for stability assessment of the culvert. Therefore, the stability of the culvert is carried out considering GM1 and results are presented in the following tables.

PD 6694-1:2011 Clause 10.6, ref. [39] requires the following stages to be analysed in the design:

- Stage 1 - No Cover

Stage 1 represents the case when the culvert has been placed after excavation to the required level. During the construction stage, the culvert is filled with imported fill (Class 6N2) up to the top of the culvert, while there is no soil on top of the culvert (namely the cover). This results in high horizontal loads which creates the worst-case scenario for sliding with minimum vertical loads and respective resistance. The Detailed Design calculations shall be referred to in Appendix H-5.

Table 4-6 and Table 4-7 summarise the results of the sliding check carried out under conditions of Stage 1.

Table 4-6: Stage 1 - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	89.17	105.2	85%	PASS
DA1 C2	84.10	134.3	63%	PASS
SLS	55.54	108.2	51%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-7: Stage 1 - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	178.29	221.1	81%	PASS
DA1 C2	166.65	274.9	61%	PASS
SLS	110.55	219.9	50%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-8 summarises the result of the overturning check carried out under conditions of Stage 1.

Table 4-8: Stage 1 - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	129.7	171.6	76%	PASS
DA1 C2	123.7	244.2	51%	PASS
SLS	81.2	202.5	40%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-9 summarises the result of the bearing check carried under conditions of Stage 1 for granular subgrade and natural subgrade material.

Table 4-9: Stage 1- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	2%	10%	PASS
DA1 C2	3%	12%	PASS

- Stage 2 - Construction Stage

It is proposed to use A60H vehicles during construction, and a 0.8m cover is proposed. The vehicles will track directly on top of 0.8m engineering fill placed over the culvert and the embankment on either side. The weight of A60H vehicles will surcharge the culvert and the ground at the side of the culverts. Also, any breaking and acceleration of vehicles on the culvert will also result in horizontal forces. The following four substages of the construction stage have been assessed considering the arrangement of the A60H vehicle.

- Stage 2A

Arrangement of vehicles to give the maximum axle load applied directly behind the culvert to give the maximum horizontal force at the side of the culvert. The Detailed Design calculations shall be referred to in Appendix H-5.

Table 4-10 and Table 4-11 summarise the result of the sliding check carried out under conditions of Stage 2A.

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Table 4-10: Stage 2A - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	216.92	344.6	63%	PASS
DA1 C2	214.94	310.7	69%	PASS
SLS	138.88	171.4	81%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-11:Stage 2A - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	313.42	637.6	49%	PASS
DA1 C2	302.51	568.5	53%	PASS
SLS	197.85	295.9	67%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-12 summarises the result of the overturning check carried out under conditions of Stage 2A.

Table 4-12: Stage 2A - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	455.0	594.6	77%	PASS
DA1 C2	465.1	584.7	80%	PASS
SLS	294.7	358.9	82%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-13 summarises the result of the bearing check carried out under conditions of Stage 2A for granular subgrade and natural subgrade material.

Table 4-13: Stage 2A- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	5%	18%	PASS
DA1 C2	15%	28%	PASS

- Stage 2B

Arrangement of vehicles to give the maximum breaking and acceleration horizontal with vehicles going in opposite directions. The Detailed Design calculations shall be referred to in Appendix H-5. Table 4-14 and Table 4-15 summarise the result of the sliding check carried out under conditions of Stage 2B.

Table 4-14: Stage 2B - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	174.29	428.6	41%	PASS
DA1 C2	168.63	383.9	44%	PASS
SLS	110.46	259.8	43%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-15: Stage 2B - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	270.79	701.4	39%	PASS
DA1 C2	256.20	621.3	41%	PASS
SLS	169.43	363.0	47%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-16 summarises the result of the overturning check carried out under conditions of Stage 2B.

Table 4-16: Stage 2B - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	334.4	869.0	38%	PASS
DA1 C2	326.8	873.5	37%	PASS
SLS	214.3	647.7	33%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-17 summarises the result of the bearing check carried under conditions of Stage 2B for granular subgrade and natural subgrade material.

Table 4-17: Stage 2B- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	3%	17%	PASS
DA1 C2	6%	20%	PASS

- Stage 2C

Arrangement of vehicles to give the maximum bearing pressure. The Detailed Design calculations shall be referred to in Appendix H-5.

Table 4-18 and Table 4-19 summarise the result of the sliding check carried out under conditions of Stage 2C.

Table 4-18: Stage 2C - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	138.03	428.6	32%	PASS
DA1 C2	131.78	383.9	34%	PASS
SLS	86.29	259.8	33%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-19: Stage 2C - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	234.53	701.4	33%	PASS
DA1 C2	219.35	621.3	35%	PASS
SLS	145.26	363.0	40%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-20 summarises the result for overturning check carried under conditions of Stage 2C.

Table 4-20: Stage 2C - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	226.1	869.0	26%	PASS
DA1 C2	220.1	873.5	25%	PASS
SLS	142.1	647.7	22%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-21 summarises the result for the bearing check carried under conditions of Stage 2C for granular subgrade and natural subgrade material.

Table 4-21: Stage 2C- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	3%	17%	PASS
DA1 C2	5%	19%	PASS

- Stage 2D

Arrangement of vehicles to give the maximum braking and acceleration horizontally together with the corresponding maximum horizontal force on the side of the culvert due to surcharging from vehicles at the side of the culvert. The Detailed Design calculations shall be referred to in Appendix H-5.

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Table 4-22 and Table 4-23 summarise the result of the sliding check carried out under conditions of Stage 2D.

Table 4-22: Stage 2D - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	155.70	512.6	30%	PASS
DA1 C2	142.25	457.0	31%	PASS
SLS	98.06	348.2	28%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-23: Stage 2D - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	252.19	765.2	33%	PASS
DA1 C2	229.81	674.2	34%	PASS
SLS	157.04	430.2	37%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-24 summarises the result for overturning check carried under conditions of Stage 2D.

Table 4-24: Stage 2D - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	355.8	1143.3	31%	PASS
DA1 C2	315.6	1162.2	27%	PASS
SLS	228.5	936.4	24%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-25 summarises the result for bearing check carried under conditions of Stage 2D for granular subgrade and natural subgrade material.

Table 4-25: Stage 2D- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	4%	26%	PASS
DA1 C2	8%	30%	PASS

- Stage 3 – In service condition

This stage considers stability analysis of culvert when it has been covered with the maximum height of backfill and experiences the HS2 rail loads on top of the embankment. The Detailed Design calculations shall be referred to Appendix H-5.

Table 4-26 and Table 4-27 summarises the result for sliding check carried out under conditions of Stage 3.

Table 4-26: Stage 3 - Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	364.07	772.0	47%	PASS
DA1 C2	355.03	682.5	52%	PASS
SLS	230.58	392.9	59%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-27: Stage 3 - Results of Sliding Check - Granular Fill / Subgrade Interface

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	597.87	1221.7	49%	PASS
DA1 C2	580.17	1073.8	54%	PASS
SLS	377.72	562.3	67%	PASS

Note: All forces in kN/m and moments in kN-m/m

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Table 4-28 summarises the result for overturning check carried under conditions of Stage 3.

Table 4-28: Stage 3 - Results of Overturning Check

Approach	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	Remarks
DA1 C1	652.0	1528.0	43%	PASS
DA1 C2	638.5	1449.7	44%	PASS
SLS	413.8	907.8	46%	PASS

Note: All forces in kN/m and moments in kN-m/m

Table 4-29 summarises the result for bearing check carried under conditions of Stage 3 for granular subgrade and natural subgrade material.

Table 4-29: Stage 3- Results of Bearing Check

Approach	Utilisation for Granular Subgrade	Utilisation for Natural Subgrade	Remarks
DA1 C1	4%	32%	PASS
DA1 C2	6%	32%	PASS

The above checks conclude that a minimum 0.8m engineering fill cover and a 2.0m excavation and replacement using Class 1A2 fill below the culvert founding level at the mainline embankment would satisfy all stability requirements.

4.5.8.3 Settlement Analysis and damage assessment

The settlement analysis was carried out using the Finite Element Software Plaxis 2D program, estimating displacements of a soil mass due to the loads applied showing the predicted soil settlement pattern beneath the loaded area. In the Plaxis2D model, the embankment loads, and railway loads are applied at the existing ground level conservatively.

For Horizontal separation assessment, the pessimistic approach is adopted by considering combination of GM1 and GM2, and the presence of the potential fault line at adjacent of ML061-CR462 which also has maximum embankment Surcharge on culvert foundation. The horizontal separation between each adjacent culvert segment is presented in Figure 4-2. The total settlement with combination of GM1 and GM2 is presented in Figure 4-1.

The total settlement of the culvert segments after 120 years due to embankment fill and railway loadings by considering 2m Excavation and replacement (E&R) below culvert with class 1A2 material is shown in Figure 4-1 below, which is 137mm and within the allowable limits of 175mm.

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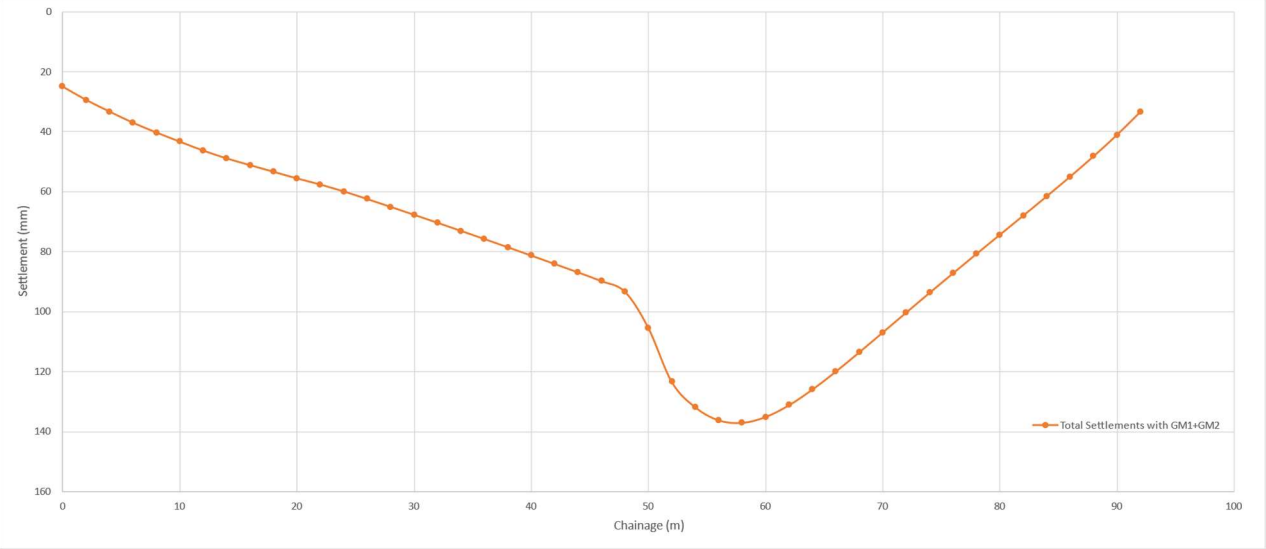


Figure 4-1: Settlement along length of culvert

A damage assessment was undertaken to ensure the structural performance of the culvert under the applied loads *i.e.*, the separation of culvert segments is within the adopted limit.

Table 4-30 presents the summary of the analysis carried out for the horizontal movement where the results are within the adopted limit.

Table 4-30: Result for segment separation

Particular	Max localised horizontal movement (mm)
Calculated value	18.2
Adopted limit	10
Remark	Exceeding permissible limit, require risk mitigation measurement

The horizontal separation along the culvert segments is estimated and shown below in Figure 4-2.

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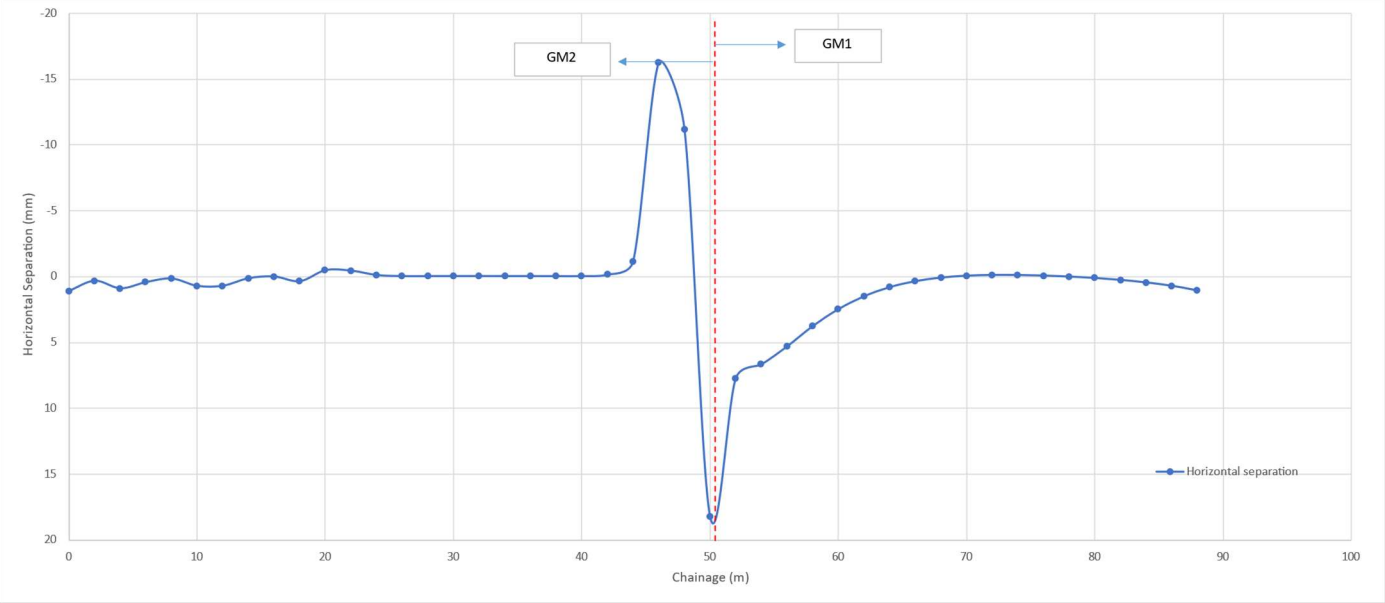


Figure 4-2: Horizontal separation along length of culvert

The horizontal separation is >10mm and is applicable for central 12m length of culvert at the potential fault line (i.e three culvert segments on each side of the potential fault line). Therefore, EW-3114 dated-14th Feb 2023 was issued to EKFB to discuss the mitigation measure for excessive horizontal separation of segments. Following to EW-3114, two Risk Reduction Meetings (Dt.28/02/2023 and Dt.15/03/2023) were conducted with EKFB to present and discuss various Geotechnical and Structural solutions to mitigate the risk of excessive horizontal separation of segments. Out of multiple options, EKFB selected the solution of lining at the potential fault line location after construction of Embankment. To identify the location of the potential fault line, the shallow excavation has been proposed in drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000024.

It is proposed that a qualified and competent Geotechnical Engineer shall inspect the excavation to confirm the stratigraphy & location of the potential fault line. The size of shallow excavation shall be 11.3m x 105m and 2m deep below culvert founding level. Refer drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000069 for liner details, the 9 joints (i.e. 8 culvert segments) are proposed to be provided with special treatment of liner.

4.5.9 Headwall Design

The following stability checks have been undertaken for Sedrup North Culvert Headwalls:

- Sliding
- Overturning
- Bearing
- Settlement

For sliding and overturning analysis, Oasys Greta software has been used. The tool provides the Factor of Safety (FoS) for stability and overturning as per Eurocode 7 – DA1 C1 and DA1 C2 partial factors. The bearing capacity calculations have been undertaken using an in-house spreadsheet. The settlement calculations for a shallow foundation were undertaken using Oasys Pdisp software.

The details of the headwall design are provided in Appendix H-6 of this report.

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4.5.10 Durability

No geotechnical chemical testing was carried out in the boreholes located in the direct vicinity to the proposed structure. For this reason, the design Sulphate and ACEC class was taken as that presented for the wider Aylesbury Embankment area as shown in Table 2-21.

Table 4-31: Concrete Design Classes for Aylesbury Embankment

Geological Formation	Design Sulphate Class (DS)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) (%)	pH
Head Deposits	DS-1	AC-1s	0.20	7.7
Alluvium	No Testing Available			
Purbeck Group	DS-4	AC-3s	1.5	7.4
Portland Sand/Stone Formation	DS-1	AC-1s	1.0	8.1
Kimmeridge Clay Formation	DS-5	AC-4s	12.2	7.6

4.5.11 Geotechnical Risk Management

A number of geotechnical risks relevant to this structure are highlighted in the GRR Doc no. 1MC06-CEK-GT-REG-CS04_CL18-000005.

- The available Geotechnical investigation (GI) is sufficient to support the development of ground model and derivation of geotechnical parameters.
- 'The geological section at asset chainage indicates the presence of the potential fault line within the footprint of the Culvert. Therefore, pre-construction GI, i.e., a shallow excavation of width 11.3m, length 105m and depth 2m below culvert founding level is required to be carried out at the culvert location to identify the location of the potential fault line before construction of the culvert. A qualified and competent Geotechnical Engineer shall inspect the excavation to confirm the stratigraphy & location of the potential fault line. The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level. If the potential fault line is confirmed the location, then the direction of the fault line, the properties of the soils within the fault line and the site photos etc. shall be included in the report.
- Along with preconstruction GI, the existing GI is sufficient to undertake the detailed design and to proceed with L3 validation.

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4.6 Bridleway SBH/19 Overbridge

4.6.1 Introduction

This section of the GDR covers the design of Bridleway SBH/19 Overbridge located within the Aylesbury Embankment L1 Asset Group. This L1 Asset Group is located within Delivery Package AA and sits between the Aylesbury South Cutting Asset Group to the south and the Aylesbury North Cutting Group to the north. The proposed structure will be located on the outskirts of Aylesbury in Buckinghamshire, carrying Public Rights of Way (PRoW): Bridleway SBH/19/7. The bridge provides a route over the HS2 mainline, with new approach embankments (Aylesbury Embankment) on both the east and west sides of the crossing.

Bridleway SBH/19 Overbridge was originally located at main Snake Grid chainage 060+544 within the Aylesbury Embankment Ground Level 1 asset. The location of Bridleway SBH/19 Overbridge was later relocated 386 metres north of its original position. The new position for Bridleway SBH/19 Overbridge is Snake Grid chainage 060+930. However, the Bridleway SBH/19 Overbridge remains within the Aylesbury Embankment Ground Level 1 asset.

Due to the late relocation of this overbridge, no Stage II Ground Investigation (GI) at Bridleway SBH/19 Overbridge was originally available at the time of carrying out the design of the overbridge. Therefore, GI from the HS2 Main GI 2016 and Stage II GI (at the vicinity of the asset) was used for the detailed design. Since then, the Stage II GI data has become available a Rapid Impact Assessment has been carried out and included in Appendix .J Results from the Stage II GI data has been used to update the Uncertainty in Ground Conditions (Section 4.6.4), the Ground Conditions (Section 4.6.7) and the Settlement Results (Section 4.6.11.4)for this asset – Refer to Appendix J for these updated sections. The final bridge design, drawings and all related deliverables for Bridleway SBH/19 Overbridge have been updated with the results presented in the Rapid Impact Assessment.

This section of the GDR focuses on the key performance considerations for the design of Bridleway SBH/19 Overbridge and defines the geometry and the construction measures together with the risks and opportunities associated with the earthworks and geotechnical design.

This report includes the geotechnical design of Bridleway SBH/19 Overbridge based on the available Ground Investigation (GI) at detailed design. The GI data is:

- GI, laboratory testing and monitoring data from the HS2 Main GI 2016 – 2018;
- GI, laboratory testing and monitoring data from the Stage II GI (supplementary GI) up to September 2021;
- data available from the HS2 Main and Stage II GI within approximately 150m from the proposed bridge location.
- New Stage II GI data at the asset location (ML060-RC432 and ML060-RC433) which has been included in the Rapid Impact Assessment

This report follows the principles described in the Footpath Bridleway SBH/19 Overbridge Approval in Principle (AIP) Document No 1MC06-CEK-BR-REP-CS04_CL18-000048 C02 Ref [40] and Footbridge Type WS-PWT Single Common Design Basis Approval in Principle (AIP) Document No 1MC06-CEK-BR-REP-C002-000039 C02 Ref [41].

4.6.2 Asset Description

The proposed Bridleway SBH/19 Overbridge (UAID HS2-000001293) is a Level 2 asset contained within the Level 1 asset Aylesbury Embankment (HS2-00002C5SN – Snake Grid chainage 60+500 to 61+763) within Section C2 (Aylesbury Area).

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Bridleway SBH/19 is a Public Right of Way, running approximately east-west across the proposed route of HS2. The terrain is a flat section of valley bottom surrounded by open fields with the hamlet of Bishopstone to the south and the town of Aylesbury to the north.

The proposed Bridleway SBH/19 Overbridge structure is a simply supported overbridge. The bridge's single span measures 42m between bearing centrelines. The deck is supported by warren trusses along its two longitudinal edges. The superstructure is perpendicular to its abutments. The bank seat abutments are constructed from reinforced concrete.

4.6.3 Geotechnical Category

The Geotechnical Category (GC) for HS2 earthwork cases (EW 1 to EW 9), and the category of check required (CAT I to III) are presented in Table 2.2/2 of Technical Standard – Earthworks (HS2 (2020). Technical Standard – Earthworks, Document No. HS2-HS2-GT-STD-000-000001, Rev. P08). The Earthworks Category and category of checking associated with Aylesbury Embankment are shown in Table 4-32.

Table 4-32 Geotechnical Category for Bridleway SBH/19 Overbridge

Asset Type	Location	Geotechnical Category	Level of Checking
Asset 3rd Party Type – Bridleway SBH/19 Overbridge	HS2 Mainline/Offline	GC3	CAT III

4.6.4 Uncertainty In Ground Conditions Classification

The level of uncertainty in ground conditions prior to IDR and prior to receipt of all Stage II GI data followed the Ground Engineering Design Basis Statement – Managing Uncertainty in Ground Conditions (SMUGC) for the permanent works design 1MC06-CEK-GT-REP-C002-000127 Rev. C02 Ref [1]. This document introduces five classes that are used to communicate the level of uncertainty in ground conditions which varies between Level 1 (i.e., sufficient GI and field trials to support detailed design and proposed optimisations) and Level 5 (i.e., insufficient GI and/or field trials to develop worst credible geotechnical model for detailed design).

The level of uncertainty in ground conditions along this asset has been assessed based on an initial review of the GI data that is available. The level of uncertainty is driven by knowledge of the distribution of materials, characterisation of parameters and identification and mitigation of geotechnical hazards. The current classification for the Level of Uncertainty in Ground Conditions is summarised in the below table for this asset.

Table 4-33 Level of Uncertainty in Ground Conditions for SBH/19 Overbridge prior to IDR

Level 2 Asset	Ground Condition Uncertainty Classification	Justification
Bridleway SBH/19 Overbridge	3 (Low – Medium Note 1)	Classification based on the quality and quantity of the available GI (both HS2 Main GI and Stage II GI) and the following contributing factors: • Presence of unsuitable ground at formation level of the earthworks at the overbridge location, such as Head Deposits/Alluvium and soft clays at shallow depth • Insufficient GI coverage to support detailed design & EC7 Compliance since there are no exploratory holes at the abutment locations. • Potential variable ground conditions • GRR maximum risk rating.

Note 1 – Risk of change in detailed design assumptions following receipt of further GI and field trial data

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Initially, the potential mitigation options for detailed design development based on the previous level of insufficient GI data to support detailed design could include the following:

- Option 1 – Complete proposed GI. Postpone detailed design and construction where necessary.
- Option 2 – Proceed with developing an **efficient design** using characteristic values/ground conditions as a cautious estimate of the value governing a limit state. This carries an elevated risk of change once the results of proposed GI and Field Trials are received.
- Option 3 – Proceed with developing a **less efficient design using a more pessimistic (conservative), estimate of the characteristic values/ground conditions** governing a limit state. This aims to reduce the risk of change once the results of proposed GI and Field Trials are received.

Several additional options to mitigate the late delivery of Stage II GI and Field Trials were introduced by SMUGC, and form subcategories to Options 2 and 3 above. They are set out below in the general order of preference:

- A – Proceed through IDR and L3 based on a limited scope of GI with no change to the design (i.e. no increase in robustness to account for uncertainty)
- B – Proceed through IDR and L3 with design elements placed into abeyance (clouded) where the outstanding GI and/or field trials are required to support the design.
- C – Adjust the design to be more cautious to address the uncertainty in ground conditions and rely upon the existing limited scope of GI.

Based on the initial level of ground condition uncertainty for this asset, the Detailed Design approach initially adopted for Bridleway SBH/19 Overbridge was **Option 2C**.

The initial risk mitigation measures regarding uncertainties of ground conditions developed for this asset included:

- To account for the lack of boreholes at the bridge location, boreholes located approximately 150m from the asset Bridleway SBH/19 Overbridge could be used to reduce the risk associated with uncertainty of ground conditions. Post-IDR, a first pass impact assessment (RIA) will be carried out to reduce the risk associated with the ground conditions when the Stage II GI becomes available.
- Exploratory holes located within 150m from the bridge location indicate the potential for soft ground (Head Deposits / Alluvium) immediately under and/or the proximity of the earthworks blister for the bridge abutments. This has the potential to generate relatively large settlements under the abutments. Therefore a 1.5m deep "Dig & Replace" mitigation measure has been included under the earthworks blister in order to manage this risk.
- A place-holder of 6-month post-construction hold period will need to be introduced between the installation of the abutments and wingwalls and the installation of the bridge deck. This hold period will be reviewed once the results of the Stage II GI become available.

Since then, and as discussed in Section 4.1, all Stage II GI is now available. The ground model and the parameters, geohazards and Ground Condition Uncertainty Class have been reviewed and reported in the Rapid Impact Assessment, which is included in Appendix J. Therefore, initial risk mitigation measures have been updated as follows:

- Post-IDR, a first pass impact assessment (RIA) was carried out and its results have been included in the final bridge design, drawings and all related deliverables for Bridleway SBH/19 Overbridge.
- A 1.5m deep "Dig & Replace" mitigation measure is still required for this asset.
- A holder of 6-month post-construction hold period will not need to be implement and any references to a hold period has been removed from the design.

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4.6.5 Construction Sequence

The assumed construction sequence is bottom-up. The following sequence was assumed during detailed design:

- Prepare the formation. Immediately under the earthworks blister, excavate to a maximum depth of 1.5m. Subsequently, backfill to formation level using General Fill Class 1 or Class 2.
- Construct the false cutting as required. The embankment may need to be temporarily extended to create a safe and level working platform.
- Regrade the surrounding approach earthworks at the location of the earthworks blisters to construct these using selected granular fill.
- Construct the earthworks blisters that will support the bankseat abutments.
- Construct reinforced concrete bank seat abutments and wing walls.
- Backfill behind bankseat abutments.
- Install the superstructure.

4.6.6 Ground Investigations

The following GI documents that were available are the following:

- HS2 Main Ground Investigation Factual Report for Amersham to Calvert for Area ACD Ref [42].
- Stage I Ground Investigation Report (GIR) Section 1 Ref [25].
- Supplementary Ground Investigation Report (GIR) Ref [7].

The layout of the boreholes and the trial pits is shown in Appendix F. A summary of the available GI is presented in Table 4-34 below. The exploratory holes in Table 4-34 are located within 150m from Bridleway SBH/19 Overbridge. A geotechnical cross section and a larger section for the entire Level 1 asset are provided in in Appendix F. It should be noted that additional GI has now become available and included in Appendix J. Results of all GI has been considered in the final design and explained in Appendix J.

Table 4-34 Summary of available GI for SBH/19 Overbridge.

In situ hole ID	Borehole Type	Ground Level (mAOD)	Final Depth (m)	Distance From Asset (m)	Easting (Snake Grid)	Northing (Snake Grid)	Contract Area
ML060-CP002	CP	82.62	10.0	110	244041.17	318193.32	ACD
ML060-CR004	RC	81.63	20.0	170	244286.23	318117.26	ACD
ML060- CR011	RC	81.64	20.0	70	244075.81	318248.80	ACD
ML060-WS010	WS	83.19	4.0	110	244082.90	318141.56	ACD
ML060-TP427	TP	80.40	2.0	135	244178.80	318279.95	Stage II GI
ML060-CT425	CT	81.44	7.2	30	244164.28	318212.50	Stage II GI
ML060-CR424	RC	81.53	25	65	244187.03	318180.80	Stage II GI
	CP – Cable Percussion; RC – Rotary Coring; WS – Window sample, TP- Trial Pit, CT – Cone Penetration						

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4.6.7 Ground Conditions

The available GIs show Bridleway SBH/19 Overbridge to be predominately underlain by Head Deposits (HEAD). These deposits typically comprise a soft to firm, brown sandy gravelly CLAY.

Underlying the Head Deposits, clays of the Gault Formation (GLT) are present, comprising firm to very stiff, grey CLAY, which in turn is underlain by the Whitchurch Sand Formation (WHS).

The Whitchurch Sand Formation was reported in the Supplementary Ground Investigation Report (GIR) Ref [7] to be present at 16.65mbgl. However, the WHS was present at 7.0mbgl in borehole ML060-CR424 (Stage II GI outstanding in the Supplementary GIR). The WHS was described as very stiff sandy gravelly SILT and extremely weak grey SANDSTONE.

The Purbeck Formation (PB) underlies the Whitchurch Sand Formation. The Purbeck Formation (PB) was described as extremely weak to medium strong grey LIMESTONE and MUDSTONE. Occasionally recovered as soft to very stiff grey laminated calcareous CLAY.

Portland Stone Formation (POST) underlies the PB. The POST was described as medium strong to very strong thinly bedded grey LIMESTONE and extremely weak to weak calcareous SILTSTONE/MUDSTONE.

The Portland Stone (POST) formation was underlain by the Portland Sand Formation (POSA), which was described as extremely weak to medium strong SILTSTONE.

The PB, POST and POSA formations have been considered as the solid stratum for the design of the footbridge due to the relative high stiffness of these material when compared with the GLT formation.

The scheduled Phase II ground investigation for this bridge has not been completed prior to the commencement of detailed design.

Table 4-35 summarises the design ground model proposed for Footbridge SBH/19 Overbridge.

Table 4-35 Stratigraphy at SBH/19 Overbridge

Geological Unit	Top of Stratum Range Level (mAOD)	Thickness Range (m)	Notes
Topsoil	83.44 – 81.44	0.20 – 0.30	Observed at all locations in Table 4-34.
Soft to Firm CLAY (HEAD)	81.43 – 81.14	1.00 – 1.30	Observed at some locations in Table 4-34.
Firm to very Stiff CLAY (Gault Formation - GLT)	83.19 – 80.13	3.75 (1) – 16.35	Observed at all locations in Table 4-34.
Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS) (1) (2)	74.53 – 63.99	1.28 – 1.50	Observed at two locations in Table 4-34.

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Geological Unit	Top of Stratum Range Level (mAOD)	Thickness Range (m)	Notes
(Purbeck Group - PB) (2)	73.03 – 63.52	1.55 – 1.85	Observed at two locations in Table 4-34
(Portland Stone Formation - POST) (2)	71.73 – 61.97	4.25 – 10.10	Observed at three locations in Table 4-34
(Portland Sand Formation - POSA) (2)	64.48	2.90	Observed at one location in Table 4-34

(1) Thickness not proven in some shallow exploratory holes in Table 4-34.

(2) The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. PB, POST and POSA Formations assumed to be a rigid stratum (E'=75MPa) for the settlement calculation.

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4.6.8 Groundwater Conditions

Groundwater has been recorded at fourteen locations under the proposed Aylesbury Embankment. Four out of the fourteen locations are ML060-CP002, ML060-CR004, ML060-CR011 and ML060-CR424 included in Table 4-34. Monitoring for ML060-CP002, ML060-CR004, ML060-CR011 was carried out monthly between February 2017 and March 2018, and monitoring for ML060-CR424 was carried out monthly between July 2021 and February 2022. All these boreholes were screened in both Head Deposits (when present) or the Gault Formation.

The groundwater monitoring recorded depths to groundwater between 0.60mbgl and 4.13mbgl. The average groundwater level recorded was 2.12mbgl. Therefore, groundwater records indicate that groundwater may be present just above the "Dig & Replace" mitigation measure excavation level.

For design purpose, the groundwater is conservatively considered 1.0m below proposed ground level.

4.6.9 Design Geotechnical Parameters, Ground Model and Ground Water Model

This section of the report details the stratigraphy and geotechnical parameters assumed in the design of the proposed Bridleway SBH/19 Overbridge.

As described in Section 4.6.4 above, Bridleway SBH/19 Overbridge has been attributed a Ground Condition Uncertainty Classification level of 3 and the geotechnical model has followed the approach "Option 2C" in accordance with SMUGC for the substructure design. This represents a more cautious estimate of the characteristic values/conditions governing a limit state based on the limited GI and Field Trial information, under the assumption that the outstanding scope of supplementary GI and Field Trials will validate the design assumptions.

4.6.9.1 Design Geotechnical Ground Model

The design stratigraphy presented in the tables below has been assumed, for the purpose of the geotechnical foundation design (See Appendix F – Cross section).

Table 4-36 Geotechnical Design Profile for East Abutment of Bridleway SBH/19 Overbridge

Geological Unit	Design Top of Stratum (mAOD)	Design Thickness (m)	Notes
Class 1A4/Class 1B4/Class 2A and 6N/6P	Variable depending on abutment and founding level of earthworks blister.		Non-HSR General Fill for the "Dig & Replace" mitigation measure and the approach embankment. Selected granular fill (Class 6N/6P) for the earthworks blister approach embankment.
Topsoil	81.5	0.3	Topsoil will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.
Soft to Firm CLAY (HEAD)	81.2	1.2	HEAD will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth (removed under bridge abutments only).
Firm to very Stiff CLAY (Gault Formation - GLT)	80.0	5.5	None

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Geological Unit	Design Top of Stratum (mAOD)	Design Thickness (m)	Notes
Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS)	74.5	1.5	The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. PB, POST and POSA Formations assumed to be a rigid stratum (E'=75MPa)

Table 4-37 Geotechnical Design Profile for West Abutment of Bridleway SBH/19 Overbridge

Geological Unit	Design Top of Stratum (mAOD)	Design Thickness (m)	Notes
Class 1A4/Class 1B4//Class 2A and 6N/6P	Variable depending on abutment and founding level of earthworks blister.		Non-HSR General Fill for the "Dig & Replace" mitigation measure and the approach embankment. Selective granular fill (6N/6P) for the earthworks blister approach embankment.
Topsoil	81.6	0.3	Topsoil will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.
Firm to very Stiff CLAY (Gault Formation – GLT)	81.3	16.35	Top 1.2m will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.
Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS)	64.0	1.28	The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. PB, POST and POSA Formations assumed to be a rigid stratum (E'=75MPa)

4.6.9.2 Geotechnical Parameter values

Characteristic values for geotechnical parameters proposed for the design of the are summarised in Table 4-38 below.

The characteristic parameter values of the Selected Granular Fill (Class 6N/6P) and those of the general fills for the Dig & Replace (Class 1A and Cass 1B) layer are taken as minimum specified parameter values for the earthworks design.

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Table 4-38 Summary of Characteristic Geotechnical Parameters Assumed for The Geotechnical Design in Bridleway SBH/19 Overbridge

Geological Unit	γ_b (kN/m^3)	φ'_{peak} (°)	φ'_{cv} (°)	c' (kPa)	PI (%)	c_u (kPa)	E' (MPa)	E_u (MPa)	E_{oed} [7] (MPa)	ν_u (-)	ν' (-)
Topsoil	N/A. This will not be used in design as topsoil will be removed.										
Class 1A4 or 1B4 (Non HSR General Fill)	20	35.0	-	-	-	-	45	-	50	-	0.20
Class 2A General Fill	20	24.0	-	-	-	50	45	-	50	-	0.20
RIP-RAP	20	45.0	-	-	-	-	-	-	-	-	-
Class 6N5/6P5 (Wingwall Backfill)	20	35.0	-	-	-	-	45	-	50	-	0.20
Class 6N6/6P6 (fill for earthworks blister)	20	40.0	-	-	-	-	45	-	50	-	0.20
Soft to Firm CLAY (HEAD)	19.5 [1]	22.0 [2]	22.0 [2]	0 [3]	40	25+12.5z [4]	6+3z [5]	7.5+3.75z [6]	6.66+3.33z	0.50	0.20
Firm to very Stiff CLAY (Gault Formation - GLT) [8]	19.5 [1]	23.0 [2]	21.0 [2]	0 [3]	50	40.0+7.5z [4] 160kPa max	9.6+1.8z [5] 38.4MPa max	12+2.25z [6] 48MPa max	10.6+2z	0.50	0.20
Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS) [8]	19.5 [9]	22.0 [9]	22.0 [9]	0	-	150kPa [9]	36MPa	45MPa	40MPa	0.50	0.20

Notes:
z is the depth from top of the corresponding stratum
[1] Characteristic unit weight above and below groundwater obtained from test data. Design values are based on average density values and BS 8004:2015.
[2] ϕ' of HEAD based on PI of 40%. Section 4.3.1.4.8 of BS 8004:2015 Ref [33] and Table 2 give ϕ'cv=22° for PI=40. No shear box tests were carried out on HEAD. ϕ'dil=0° has been assumed. **ϕ'pk=22° and ϕ'cv=22° have been adopted as characteristic values for the design**.
ϕ' of GLT based on PI of 50%. Section 4.3.1.4.8 of BS 8004:2015 Ref [33] and Table 2 give ϕ'cv=21° for PI=50. ϕ'dil=2° has been assumed (OCR>>1). Therefore ϕ'pk=21°+2°=23°. No shear box tests were carried out on GLT. 4No. triaxial tests were carried out on GLT (average ϕ'pk=23° c'pk =13kPa;). **ϕ'pk=23° and ϕ'cv=21° have been adopted as characteristic values for the design**.
[3] c' has been assumed as zero for the HEAD and GLT.
[4] Cu obtained from triaxial tests and correlation with SPTs based on a PI of 40% and 50% which gives f_1 value of 4.2 (Figure 7-12 of HS2 Technical Note – Guidance on the Assessment of Settlement Ref [11]).
[5] E' is based on 0.8Eu.
[6] Eu is based on relationship between Eu and Cu (after Jamiolkowski et al.). For PI=40-50, Eu/Cu=300. Note that this correlation is based on the high strain level applicable to spread foundations.
[7] Eoed = E' (1- ν') / {(1+ ν')(1-2ν')}. Assuming ν' = 0,2, then Eoed = 1.11E'
[8] The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. PB, POST and POSA Formations assumed to be a rigid stratum with a E'=75MPa.
[9] Design values are based on average values and BS 8004:2015. Stiffness value based on material description (Very Stiff SILT). Eu is based on Eu/Cu=300. Note that this correlation is based on the high strain level applicable to spread foundations.

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4.6.9.3 Design Groundwater Model

The design groundwater level for the earthworks blister and abutments is conservatively 1.0m below proposed ground level for the ULS cases.

4.6.10 Design Approach and Partial Factors for The Abutment Design

4.6.10.1 Ultimate Limit States

The abutments are conventional reinforced concrete retaining walls which support the loading of the deck structure, the horizonal earth pressures and the pedestrian and maintenance traffic surcharge pressures acting behind the abutment. The design of the abutments is based on a limit equilibrium analysis of conventional retaining walls to balance the forces and moments at foundation level in the ultimate limit state (both DA1C1 and DA1C2) and serviceability limit state. The horizontal earth pressures are calculated in accordance with Annex C in EC7-1. The abutments are ground bearing.

The overbridge foundations have been designed in accordance with Design Approach 1 (DA1) of EC7-1 and its UK National Annex, which requires that both GEO and STR limit states should be checked for combinations DA1C1 (or ULS Set B) and DA1C2 (or ULS Set C) in the manner shown in the table below.

Table 4-39 Design Approach 1 for STR and GEO Limit States for Retaining Walls – Eurocode 7.

Design Approach	Combination 1	Combination 2
	DA1 C1 (ULS B)	DA1 C2 (ULS C)
Design Approach 1	A1 + M1 +R1	A2 + M2 +R1

The partial factors on materials required to satisfy the worst of the two design combinations DA1C1 (ULS Set B) and DA1C2 (ULS Set C) are shown in the table below.

Table 4-40 Material Partial Factors for Design Approach 1 Combinations 1 and 2 for Retaining Walls (Table A4 in UK NA to BS EN 1997-1)

Parameter	Symbol	DA1 C1	DA1 C2
		M1	M2
Shearing Resistance	$g_{\phi'}$	1.0	1.25
Effective Cohesion	$g_{c'}$	1.0	1.25
Undrained Shear Strength	g_{cu}	1.0	1.4
Density	g_{g}	1.0	1.0

Both Combination 1 and Combination 2 are assessed with respect to the calculation of the horizonal earth pressures and surcharge pressures acting behind the abutment. These pressures are then resolved to foundation level (foundation soffit of the abutment).

Partial factors applying to actions or effects of actions (A1 and A2 in Table 4-39) are applied according to Tables NA.A2.4 (B) and (C) of NA to BS EN 1990 [7].

For the calculation of the horizontal earth pressures under EC7, partial factors (as shown in Table 4-39) are applied to both design combinations DA1C1 and DA1C2.

For the limit equilibrium analysis in the GEO ultimate limit state, the horizontal earth pressures were calculated using the corresponding Ka coefficient reflecting the development of a fully active state. The Ka value has been calculated from the analysis procedures given in Annex C of EC-7.

For the limit equilibrium analysis in the STR ultimate limit state, relatively small movements of the abutments are expected when compared to the GEO ultimate limit state. For small movements insufficient to mobilise a fully active state, the value of horizontal earth pressure expected to act at the back of the abutments will lie between the active pressure (derived from K_a) and the at-rest pressure (derived from K_o). However, the

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horizontal earth pressures behind the abutments have been conservatively calculated using a design value of earth pressure coefficient K_o. The at-rest pressure coefficient K_o has been calculated in accordance with Clause 9.5.2 in EC-7.

4.6.10.2 Serviceability Limit State

Displacement calculations, such as settlement or lateral movements, adopt partial factors set to 1.0.

4.6.10.3 Compaction Pressures

Compaction pressures have been calculated based on 800kg/m dead-weight of roller and have been applied to STR ultimate limit state only. This is due to the relief of compaction pressures with horizontal movements of the abutments and therefore it is expected that compaction pressures will not be maintained at the GEO ultimate limit state.

4.6.10.4 Accidental

When choosing partial factors for material strength and resistances in accidental loading situations, the square root of values given for persistent and transient loading have been used as per Table NA.1 of the UK National Annex to EC7-1.

4.6.10.5 Seismic

HS2 Technical Standard – Civil Engineering Seismic Design (TSCESD) defines seismic design processes and specific requirements generally in accordance with BS EN 1998 1 2004 (EC8) but, in some aspects, overrides EC8.

The HS2 Technical Standard for Retaining Walls requires seismic design to be undertaken for all abutment structures in accordance with the requirements of EC8.

TSCESD states that the UK shall be considered as an area of low seismicity and that the low seismicity provisions in EC8 shall apply. TSCESD sets out the two-level design approach following EC8-1 and the UK National Annex: ULS and DLS.

- Damage Limitation State, DLS – 1 in 475 years (10% probability of exceedance in 50 years)
- Ultimate Limit State, ULS – 1 in 2500 years (2% probability of exceedance in 50 years)

TSCESD provides Seismic Hazard maps in its Appendix A for the 475year and 2500year return periods. The Importance Classes are set out in Table 1 in Section 3.4 of the TSCESD and supersede those stated in EC8-1. Footbridges that span over the main-line structures and earth retaining structures, embankments and cuttings near the HS2 track where potential damage could affect the HS2 track or service are considered structures of Importance Class II. For Importance Class II, only ULS must be checked.

EC8 (BS EN 1998-5) defines general partial factors that should be applied to characteristic ground property values. These partial factors are given in Clause 3.1(3) of EC8 and the UK National Annex of EC8 (Part 5) confirms these values in its Table NA.1 (NA to EC8). The partial material factors for the seismic check are the partial factors summarised in Table 4-40 (same values as per DA1C2 in the table).

In EC8-1, the seismic hazard is also described in terms of a single parameter, i.e., the value of the reference peak ground acceleration on Ground Type A, α_{gR}. The ground acceleration for a specific return period, αg, is obtained by multiplying the reference ground acceleration by the importance factor. In the case of HS2, the importance factor has been set to 1 for all structures.

Table 4-41 presents the reference peak and design ground accelerations considered in Seismic ULS.

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Table 4-41 Reference peak and design ground accelerations considered in Seismic ULS

Seismic Limit State	Return Period	PGA on Ground Type A, α_{gR}	Importance Factor	Reference peak acceleration on Ground Type A, α_g	Soil Type	Soil Factor	Design ground acceleration on Ground Type C, α_g-S
Ultimate Limit State, ULS	1 in 2500 years	0.05g	1.0	0.05g	C	1.5	0.075g

Modal spectral analysis has been used to calculate the structural effects of earthquakes and quantify the inertia forces transmitted to the foundations.

TSCESD recommends Type 2 Spectrum elastic response spectra to be adopted in the design. The shape of the elastic response spectrum Se(T) is defined as a function of the Ground Type. At T=0, the spectral acceleration given by a certain spectrum equals the design ground acceleration on Ground Type A multiplied by the corresponding soil factor S.

The average undrained shear strength along the first 30m in depth is in the range of 70-250kPa, therefore Ground Type C has been assumed at this site. The corresponding soil factor S is 1.50 for the recommended Type 2 elastic response spectra.

4.6.10.6 Surcharge Behind Abutment

A pedestrian surcharge of 5kPa has been included in assessment of the stability of the footbridge abutment.

4.6.11 Design of Bridleway SBH/19 Overbridge

4.6.11.1 Seismic Design for The Abutments of SBH/19

The seismic forces acting on an abutment wall have been determined by pseudo-static analysis. The basic model for pseudo-static analysis consists of the abutment wall and shallow footing, a soil wedge behind the structure supposed to be in a state of active limit equilibrium and any surcharge loading acting on the soil wedge.

For retaining walls, the geotechnical analysis for the seismic case requires the calculation of revised active earth pressure coefficients which reflect the static and dynamic response of the ground when subjected to a seismic event.

For the definition of the horizontal seismic coefficient (k_h), the factor *r* shall be taken from Table 3 of the TSCESD, which supersedes Table 7.1 in EC8-5. For bridge abutments, the factor *r* is taken as 1.0 for Importance Class II.

The unitless horizontal seismic coefficient k_h can be determined as $k_h = 0.5\alpha.S/r$ and the vertical seismic coefficient k_v is equal to $\pm 0.5k_h$ if a_{vg}/a_g is larger than 0.6 and $\pm 0.33k_h$ otherwise. Given that the value of a_{vg}/a_g is 0.45 for a Type 2 Spectrum in NA.1 of NA to EC8-1, Therefore $k_v = \pm 0.33k_h$. The table below presents the horizontal and vertical seismic coefficients adopted in Seismic ULS ($k_h = 0.5\alpha.S/r = 0.5x0.05x1.5/1 = 0.0375$; $k_v = 0.33k_h = 0.01238$).

Table 4-42 Reference peak and design ground accelerations considered in Seismic ULS.

Seismic Limit State	Horizontal seismic coefficient (k_h)	Vertical seismic coefficient (k_v)
Ultimate Limit State, ULS	±0.0375	±0.01238

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4.6.11.2 Loading Acting on The Abutments of SBH/19

The 3.5m wide carriageway abutment is approximately 3.26m wide and 6.060m length. The height of the abutment is 2.67m. Dimensions of the abutment are shown on drawing 1MC06-CEK-BR-DDE-C002-000293 Ref [43], which presents details for a standard 3.5m wide abutment.

The superstructure loading has been computed using the software LUSAS and resolved at a single point located at the centre of the deck at bearing level (at point O in figure below). This software allows the load cases to be combined into load combinations in line with the requirements of the Eurocodes.

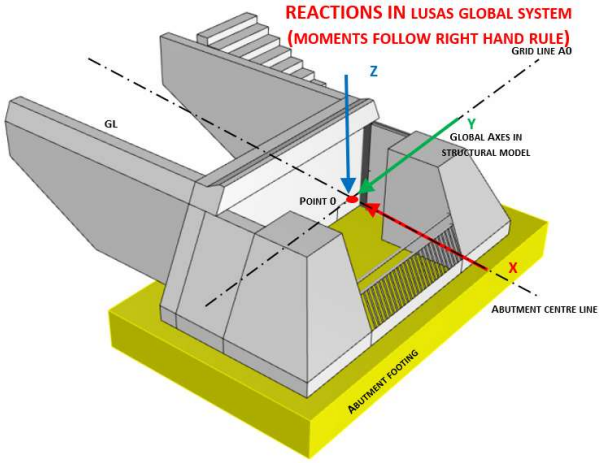


Figure 4-3 Sketch of Bridge Abutment and Sign Convection For Superstructure Reactions.

The software LUSAS also generates load envelopes, maximizing and minimizing each different component of the forces {Fx, Fy, Fz, Mx, My, Mz} to produce the most onerous combinations. Tables with the results for most onerous ULS combinations for the abutments are detailed below. The self-weight of the abutment has not been included in the table below.

Table 4-43 ULS SET B Reactions from Superstructure at Point 0 For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN	kN	kN	kN.m	kN.m	kN.m
ULS SET B	Fxmax	129	23	441	-22	23	239
	Fxmin	-129	-23	917	22	-23	-239
	Fymax	35	197	438	-236	6	65
	Fymin	-35	-197	920	236	-6	-65
	Fzmax	88	101	1100	-88	16	164
	Fzmin	0	-178	261	210	0	0
	Mxmax	-18	-188	832	308	-3	-33
	Mxmin	18	187	349	-308	3	33
	Mymax	129	23	441	-22	23	239
	Mymin	-129	-23	917	22	-23	-239

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Table 4-44 ULS SET C Reactions from Superstructure at Point 0 For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN	kN	kN	kN.m	kN.m	kN.m
ULS SET C	Fxmax	110	20	454	-19	19	204
	Fxmin	-110	-20	775	19	-19	-204
	Fymax	30	168	451	-202	5	56
	Fymin	-30	-168	778	202	-5	-56
	Fzmax	75	86	931	-75	13	139
	Fzmin	0	-45	294	48	0	0
	Mxmax	-15	-160	703	263	-3	-28
	Mxmin	15	159	376	-263	3	28
	Mymax	110	20	454	-19	19	204
	Mymin	-110	-20	775	19	-19	-204

Table 4-45 SLS Reactions from Superstructure at Point 0 For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN	kN	kN	kN.m	kN.m	kN.m
SLS	Fxmax	82	36	725	-22	15	152
	Fxmin	-82	-36	857	22	-15	-152
	Fymax	26	119	478	-140	5	49
	Fymin	-26	-119	712	140	-5	-49
	Fzmax	65	67	859	-56	12	121
	Fzmin	0	-104	347	123	0	0
	Mxmax	-13	-112	647	194	-2	-24
	Mxmin	13	111	412	-194	2	24
	Mymax	82	36	725	-22	15	152
	Mymin	-82	-36	857	22	-15	-152

Table 4-46 Seismic Reactions from Superstructure at Point 0 For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN	kN	kN	kN.m	kN.m	kN.m
ASL - Seismic	Fxmax	122	52	499	-29	22	227
	Fxmin	-122	-52	429	29	-22	-227
	Fymax	68	59	502	-32	12	126
	Fymin	-68	-59	426	32	-12	-126
	Fzmax	68	59	553	-32	12	126
	Fzmin	-68	-59	376	32	-12	-126
	Mxmax	-68	-59	426	32	-12	-126
	Mxmin	68	59	502	-32	12	126
	Mymax	122	52	499	-29	22	227
	Mymin	-122	-52	429	29	-22	-227

In order to calculate the loads at the base of the abutment, the superstructure loads are transferred to the base of the abutment and the self-weight of the abutment (approximately 680kN or 113kN/m) and the earth pressures are added to the superstructure loading. The calculation of the earth pressures has been carried out as described in Section 4.6.10.1. Results of the superstructure loads, abutment weight and earth pressures at the middle of the abutment (Point C) are presented in Table 4-48 to

Table 4-51 expressed in kN per metre run of wall.

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For the calculation of the horizontal earth pressures, the following geotechnical properties for the backfill material have been assumed.

Table 4-47 Assumed Characteristic Strength Parameters for the Backfill Material behind Wingwalls and Abutments

Geological Unit	γ_b (kN/m^3)	φ'_{peak} (°)	c' (kPa)	K_0 (-)	K_a (-)
Class 6N5/6P5 (Wingwall / abutment Backfill)	20	35.0	0	0.43	0.27

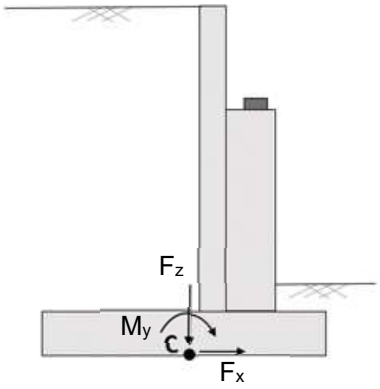


Figure 4-4 Schematic Diagram of Load Transfer at The Centre of The Base of Bridge Abutment

Table 4-48 ULS SET B Loads from Superstructure, Abutment Weight, Earth Pressures, Compaction and Surcharges at Point C For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx kN/m	Fy kN/m	Fz kN/m	Mx kN.m/m	My kN.m/m	Mz kN.m/m
ULS SET B	Fx_{max}	63	4	187	-4	114	40
	Fx_{min}	7	-4	306	4	11	-40
	Fz_{max}	59	17	337	-15	139	27
	Fz_{min}	42	-30	157	35	43	0
	My_{max}	65	4	187	-4	114	40
	My_{min}	7	-4	306	4	11	-40

Table 4-49 ULS SET C Loads from Superstructure, Abutment Weight, Earth Pressures and Surcharges at Point C For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx kN/m	Fy kN/m	Fz kN/m	Mx kN.m/m	My kN.m/m	Mz kN.m/m
ULS SET C	Fx_{max}	59	3	189	-3	106	34
	Fx_{min}	16	-3	243	3	20	-34
	Fz_{max}	53	14	269	-13	123	23
	Fz_{min}	40	-8	163	8	42	0
	My_{max}	59	3	189	-3	104	34
	My_{min}	16	-3	243	3	20	-34

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Table 4-50: SLS Loads from Superstructure, Abutment Weight, Earth Pressures and Surcharges at Point C For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN/m	kN/m	kN/m	kN.m/m	kN.m/m	kN.m/m
SLS	Fx_{max}	61	6	236	-4	135	25
	Fx_{min}	27	-6	258	4	60	-25
	Fz_{max}	58	11	259	-9	137	20
	Fz_{min}	47	-17	172	21	69	0
	My_{max}	61	6	236	-4	135	25
	My_{min}	27	-6	258	4	60	-25

Table 4-51: Seismic Loads from Superstructure, Abutment Weight, Earth Pressures and Surcharges at Point C For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Fx	Fy	Fz	Mx	My	Mz
		kN/m	kN/m	kN/m	kN.m/m	kN.m/m	kN.m/m
ASL - Seismic	Fx_{max}	55	9	199	-5	119	38
	Fx_{min}	9	-9	187	5	-5	-38
	Fz_{max}	45	10	208	-5	97	21
	Fz_{min}	22	-10	178	5	22	-21
	My_{max}	55	9	199	-5	118	38
	My_{min}	9	-9	187	5	-5	-38

4.6.11.3 Design Checks

Overturning Check

The overturning stability of the abutments is concerned with the combination of vertical and horizontal forces in order to maintain equilibrium of the abutments. The total factored restoring and destabilising moments for the abutment for Bridleway SBH/19 Overbridge have been presented in tables below. By inspection of the factored moments, it can be seen than the restoring moments are considerably greater than the destabilising moments for all load cases. Therefore, the stability of the abutment for Bridleway SBH/19 Overbridge is verified against overturning failure.

Table 4-52: ULS SET B Overturning Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Destabilising Moments	Restoring Moments	FoS
		kNm/m	kNm/m	-
ULS SET B	Fx_{max}	37	228	6.2
	Fx_{min}	21	510	24.0
	Fz_{max}	37	447	12.1
	Fz_{min}	37	251	6.7
	My_{max}	37	228	6.2
	My_{min}	21	510	24.0

Table 4-53: ULS SET C Overturning Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Destabilising Moments	Restoring Moments	FoS
		kNm/m	kNm/m	-
ULS SET C	Fx_{max}	36	239	6.6
	Fx_{min}	27	403	15.0
	Fz_{max}	34	350	10.2
	Fz_{min}	34	257	7.5
	My_{max}	34	239	7.0
	My_{min}	27	403	15.0

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Table 4-54: ULS Seismic Overturning Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Destabilising Moments kNm/m	Restoring Moments kNm/m	FoS -
ASL - Seismic	Fx_{max}	30	236	7.8
	Fx_{min}	24	335	13.9
	Fz_{max}	30	272	9.2
	Fz_{min}	30	299	10.1
	My_{max}	30	235	8.0
	My_{min}	24	335	13.9

Sliding Check

The sliding stability of the abutments has been considered in accordance with the methodology provided in EC7. The sliding resistance (R_d) has been calculated based on the properties of the Class 6N/6P material for the earthworks blister included in Table 4-38. This sliding resistance has then been compared with the factored destabilising horizontal loading (H_d) acting on abutments, to verify the satisfaction of the following equation $H_d < R_d$. Results are presented in the tables below. It can be seen than the sliding resistance is greater than the destabilising horizontal loading for all load cases. Therefore, the stability of the abutment for Bridleway SBH/19 Overbridge is verified against sliding failure.

Table 4-55: ULS SET B Sliding Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Horizontal Forces kN/m	Sliding Resistance kN/m	FoS -
ULS SET B	Fx_{max}	90	117	1.3
	Fx_{min}	34	191	5.7
	Fz_{max}	78	210	2.7
	Fz_{min}	42	98	2.3
	My_{max}	92	117	1.3
	My_{min}	34	191	5.7

Table 4-56: ULS SET C Sliding Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Horizontal Forces kN/m	Sliding Resistance kN/m	FoS -
ULS SET C	Fx_{max}	82	97	1.2
	Fx_{min}	39	124	3.2
	Fz_{max}	69	137	2.0
	Fz_{min}	40	83	2.0
	My_{max}	82	97	1.2
	My_{min}	39	124	3.2

Table 4-57: ULS Seismic Sliding Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Horizontal Forces kN/m	Sliding Resistance kN/m	FoS -
ASL - Seismic	Fx_{max}	86	111	1.3
	Fx_{min}	40	104	2.6
	Fz_{max}	62	116	1.8
	Fz_{min}	39	99	2.5
	My_{max}	86	111	1.3
	My_{min}	40	104	2.6

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Bearing Capacity Check

The bearing capacity has been calculated based on the properties of the Class 6N/6P material for the earthworks blister included in Table 4-38. The bearing check has been carried out in according to Appendix D in EC7 for drained materials (this includes the calculation of bearing capacity factors, inclination factors and shape factors for the abutment). The bearing capacity factors have been reduced due to the abutment being in the vicinity of a slope. The reduction of the bearing capacity followed the recommendations of BS 8002:2015 Code of Practice for Earth Retaining Structures for the calculation of the bearing capacity (Clause 5.7.2.).

Table 4-58: ULS SET B Bearing Capacity Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Eccentr. / B'	Fz	Ave. Total Bearing Pressure	Ultimate Bearing Capacity	FoS
		m/m	kN/m	kN/m	kN/m	-
ULS SET B	Fx_{max}	0.61 / 2.04	187	92	261	2.8
	Fx_{min}	0.04 / 3.19	306	96	983	10.2
	Fz_{max}	0.41 / 2.44	337	138	515	3.7
	Fz_{min}	0.27 / 2.72	157	58	406	7.1
	My_{max}	0.61 / 2.04	187	92	250	2.7
	My_{min}	0.04 / 3.19	306	96	983	10.2

Table 4-59: ULS SET C Bearing Capacity Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Eccentr. / B'	Fz	Ave. Total Bearing Pressure	Ultimate Bearing Capacity	FoS
		m/m	kN/m	kN/m	kN/m	-
ULS SET C	Fx_{max}	0.56 / 2.15	189	88	88	1.0
	Fx_{min}	0.08 / 3.10	243	78	214	2.7
	Fz_{max}	0.46 / 2.35	269	114	130	1.1
	Fz_{min}	0.26 / 2.75	163	59	118	2.0
	My_{max}	0.55 / 2.17	189	87	88	1.0
	My_{min}	0.08 / 3.10	243	78	214	2.7

Table 4-60: SLS Bearing Capacity Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Eccentr. / B'	Fz	Ave. Total Bearing Pressure	Ultimate Bearing Capacity	FoS
		m/m	kN/m	kN/m	kN/m	-
SLS	Fx_{max}	0.41 / 2.13	236	111	252	2.3
	Fx_{min}	0.72 / 2.80	258	92	458	5.0
	Fz_{max}	0.47 / 2.21	259	117	285	2.4
	Fz_{min}	0.38 / 2.47	172	70	256	3.7
	My_{max}	0.41 / 2.13	236	111	252	2.3
	My_{min}	0.72 / 2.80	258	92	458	5.0

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Table 4-61: ULS Seismic Bearing Capacity Factor of Safety For 3.5m Carriageway Abutment.

Conbination Type	Force Maximised	Eccentr. / B'	Fz	Ave. Total Bearing Pressure	Ultimate Bearing Capacity	FoS
		m/m	kN/m	kN/m	kN/m	-
ASL - Seismic	Fx_{max}	0.60 / 2.10	199	96	149	1.5
	Fx_{min}	0.03 / 3.21	187	58	357	6.1
	Fz_{max}	0.47 / 2.33	208	89	187	2.1
	Fz_{min}	0.12 / 3.02	178	59	278	4.7
	My_{max}	0.59 / 2.08	199	96	149	1.5
	My_{min}	0.03 / 3.21	187	58	357	6.12

4.6.11.4 Settlement Analysis

The Serviceability Limit State (SLS) assesses the movements of the earthworks blister and bridge abutments, which have been assessed with the application of partial factor to actions set to unity (such as pedestrian/ maintenance loading) and materials.

Earthwork settlements have been assessed to satisfy compliance with deformation criteria set out in the HS2 Technical Standard - Earthworks. Total settlements (immediate/undrained and long-term/drained movements) have been assessed. The settlement analyses have been undertaken in accordance with the design approach set out in Ground Engineering Design Basis Statement – Earthworks.

This section covers the settlement of the earthworks blister supporting the bridge abutment only. Settlement of the approach embankment will be covered in the GDR of the main Level 1 Asset.

The following assumptions have been adopted in design:

i. Longitudinal distortion along the bridge alignment has been limited to a long-term maximum value of 1V:250H.

ii. The stress distribution has been calculated using the Boussinesq method in the specialist software Oasys PDisp.

iii. A rigid boundary of 75MPa of stiffness has been assumed for the settlement calculations at the base of the Whitchurch Sand Formation.

iv. Material properties for Non HSR General Fill for the "Dig & Replace" under the earthworks blister are provided in Section 4.6.9.2.

v. One-way drainage has been assumed given the underlying geology.

vi. Settlement within earthworks blister (selective granular fill Class 6N/6P) is anticipated to be completed within the construction period. Therefore, settlement within the earthworks blister has not been included in the settlement calculation.

vii. A Dig & Replace layer of 1.5m depth is required under the earthworks blister to remove topsoil and relatively soft materials that may be encountered (HEAD and soft clays of the Gault Formation).

viii. The total settlement is calculated using the equation $S_t = \Sigma(\Delta\sigma'_i/E_{oed\ i})*H_i$ for each layer. Initial settlement and consolidation settlement are calculated as a relative proportion of the total settlement adopting a Skempton and Bjerrum geological factor μ_g of 0.6 for overconsolidated clays.

ix. Results of the PDisp analyses for the concrete abutments and earthworks at the toe of the earthworks blister (closest point to rail track) are reported at existing ground level. This is due to the settlement of the earthworks blister being completed within the construction period.

x. Due to the potential difference in thickness of the stiff Gault clay under each abutment, a PDisp analysis was undertaken assuming a thick clay layer of the Gault Formation at the west abutment

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whereas a thin clay layer was assumed at the east abutment. This was to check the impact of the ground model variation in terms of differential settlement

xi. Three different cases have been assessed during the settlement analysis in PDisp due to uncertainty on the ground conditions immediately under the abutments (it should be noted that the deepest boreholes available at the time of producing the design were located more than 70m from the bridge location):

- Case 1: 16.35m thick Gault Clay under both east and west abutments;
- Case 2: 5.5m thick Gault Clay under both east and west abutments; and
- Case 3: 16.35m thick Gault Clay under the west abutment and a 5.5m thick Gault Clay under the east abutment.

Actions / Loading

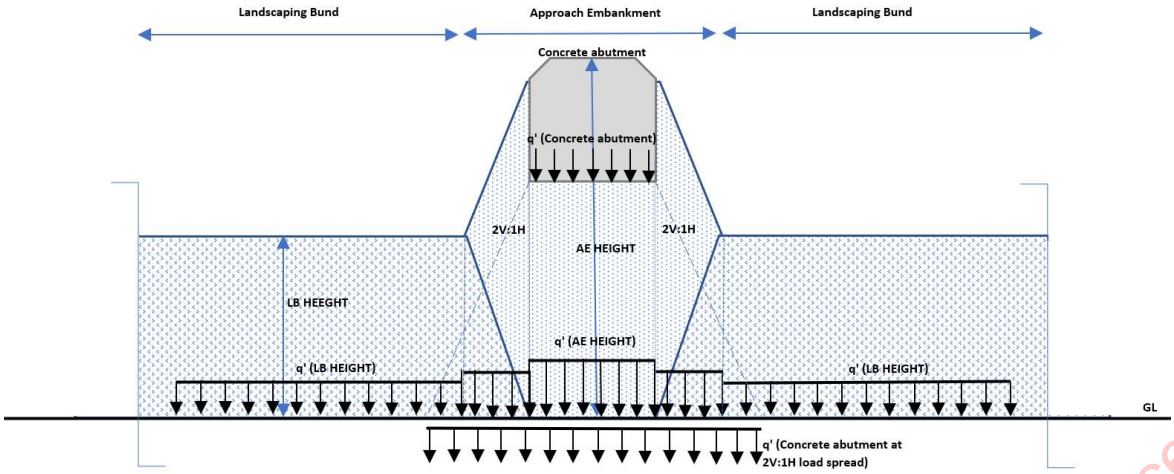
For the settlement calculation, the loads included in the analysis are as follows. All loads were applied at existing ground level (base of the proposed new embankment approach – see Appendix F for the general arrangement drawing) for the settlement analysis:

i. The load due to the imposed weight of the approach embankment was taken as a UDL of 200kPa (approximately max.10.0m height embankment) applied at existing ground level.

ii. The load due to the imposed weight of the landscaping bunds either side of the approach embankment has been applied as UDL applied at existing ground level. The landscaping bunds were assumed to be infinite in length in the plain strain direction (along the direction of the HS2 track as shown in sketch below):

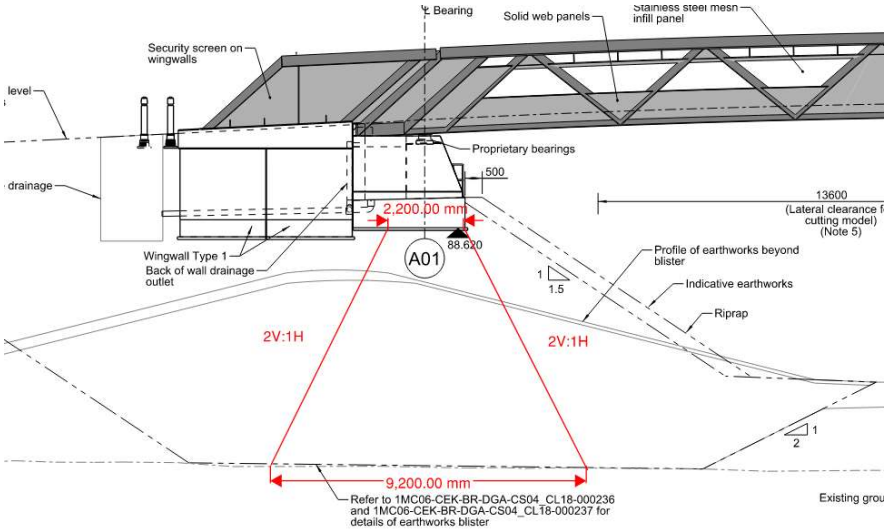
a. Western landscaping bund (west abutment): 125kPa (6.2m high embankment).

b. Eastern landscaping bund (east abutment): 175kPa (8.6m high embankment).



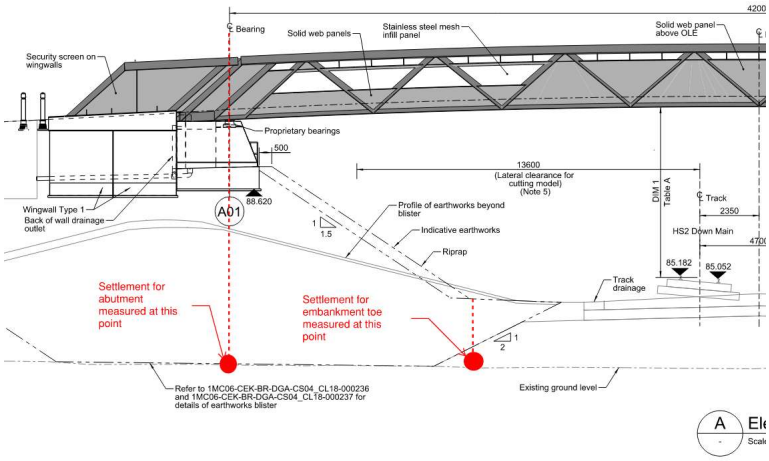
iii. An additional imposed load due to the bearing pressures acting at the base of the concrete abutments has also been included. This load has been transferred at existing ground level as an equivalent UDL at a 2V:1H load spread (see previous sketch). The worst average net bearing pressure of the abutment is approximately 117kPa over a length of 2.2m width at the base of the abutment (Load Case Fz_{max} in Table 4-60). In order to transfer this bearing pressure at existing ground level, the load has been spread at a 2V:1H load spread for a 7.0m height embankment (measured from base of abutment to ground level). Therefore, the bearing pressure of the abutment spreads over approximately 9.2m at existing ground level. The bearing pressure can then be applied at existing ground level as a UDL of approximately 30kPa.

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Settlement Results

A summary of the predicted settlement of the earthworks blister and bridge abutments at existing ground level is provided in Table 4-62 below. Results of the PDisp analyses for the concrete abutments and earthworks at the toe of the earthworks blister are reported at existing ground level (see sketch below location at where settlement are measured). Settlements of the earthworks blister supporting the concrete abutments have been excluded of the calculations because these settlements are immediate settlements occurring during the construction period of the earthworks blisters.



The following cases were found to be critical:

1. Case 1 (16.35m thick Gault Clay under both east and west abutments) presented the maximum total Post Construction Settlement with a value of 81mm at the east abutment. This is due to the higher landscaping bund running along the east abutment.
2. Case 3 (16.35m thick Gault Clay under the west abutment and a 5.5m thick Gault Clay under the east abutment) presented the maximum post construction differential settlement between abutments. For Case 3, the difference in thickness of the Gault Clay under the abutments is approximately 11.0m, which resulted in a maximum differential settlement of 15mm between abutments. This is less than

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the allowable 25mm of differential settlement specified for the bridge abutments, as prescribed the Footpath SBH/19 Accommodation Overbridge Approval in Principle (AIP) Document No 1MC06-CEK-BR-REP-CS04_CL18-000048 C02 [40] and Footbridge Type WS-PWT Single Common Design Basis Approval in Principle (AIP) Document No 1MC06-CEK-BR-REP-C002-000039 C02 [41].

3. Case 1 produced maximum post construction settlements of 81mm at the east abutment whereas Case 3 produced maximum post construction settlements of 78mm at the west abutments. These are the two governing cases.

Table 4-62: Settlement analyses results for SBH/19.

Case	Location (1, 2)	Landscape bund Load	Immediate Settlement	Consolidation Settlement	Total Settlement	Post-Construction Settlement	Allowable Post–Construction Settlement (3)
		kPa	mm	mm	mm	mm	mm
Case 1 (16.35m thick Gault Clay, both abutments)	East Abutment	175	54	81	135	81	85
	East Embank.		12	18	30	18	20
	West Abutment	125	52	78	130	78	85
	West Embank.		12	18	30	18	20
Case 2 (5.5m thick Gault Clay, both abutments)	East Abutment	175	42	63	105	63	85
	East Embank.		8	12	20	12	20
	West Abutment	125	40	60	100	60	85
	West Embank.		8	12	20	12	20
Case 3 (16.35m Gault Clay at west and 5.5m thick Gault Clay at east abutment)	East Abutment	175	42	63	105	63	85
	East Embank.		8	12	20	12	20
	West Abutment	125	52	78	130	78	85
	West Embank.		12	18	30	18	20

NOTES

(1) Settlements at the abutment were estimated at the vertical of the centre of the abutment base.

(2) Settlements at the embankment were estimated at the toe of the embankment.

(3) Allowable limit of 85mm set for the bridge abutments by the bridge designer.

The maximum calculated differential post-construction settlement of 15mm between the East and West abutments (Case 3 in Table 4-62 of 78mm at the west abutments) is within the allowable limit of 25mm.

However, due to the uncertainty in the ground model at the bridge location, total post construction settlements and differential settlement could be exceed. Therefore, a place-holder for a 6-month post-construction hold period is need to be introduced between the installation of the abutments and wingwalls and the installation of the bridge deck.

As mentioned in Sections 4.6.1 and 4.6.2, new GI data has become available since the initial submission of this GDR. Results of the new data are included in the First Pass Rapid Impact Assessment in Appendix J. After reviewing the GI data, a 6-month post-construction hold period is no longer required and references to any hold period has been removed from the final design of Bridleway SBH/19 Overbridge.

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4.6.12 Earthworks Stability Check

The slope stability analysis for the proposed earthworks and earthworks blister supporting the bridge abutments for Bridleway SBH/19 Overbridge is assessed and developed in accordance with the requirements detailed in Eurocode 7, UK National Annex, and recommendations in BS6031:2009.

Given anticipated ground condition variations between the East and West abutments at footbridge SBH/19, slope stability analyses were undertaken for both abutments.

4.6.12.1 Load Combination

Eurocode 7 requires partial factors on actions and materials to be applied to the GEO ultimate limit state to check failure or excessive deformation of the ground. Partial factors on resistance are equal to the unity, so they will not affect the ultimate limit state. Eurocode 7 requires that the GEO ultimate limit state for the overall slope stability be carried out for Design Approach 1 which includes Combination 1 and Combination 2 (DA1 C1 and DA1 C2; see Table 4-39 and Table 4-40).

However, for the slope stability checks, DA1 C2 normally governs since material partial factors for DA1C1 are set to 1 (see partial factors in Table 4-40). Therefore, only results for DA1 C2 combination have been presented in this report. The partial factors for each combination in the slope stability check are shown in Table 4-39 and Table 4-40.

The seismic global stability was also assessed in accordance with HS2 Technical Standard – Civil Engineering Seismic Design (TSCESD) and EC8. This was done as described in Sections 4.6.10.5 and 4.6.11.1.

4.6.12.2 Basis of The Geotechnical Design and Model

In the long term, a minimum uniformly distributed load (UDL) of 10kPa has been applied at the top surface of the embankment (behind the bridge abutment). In accordance with the recommendations provided in BS 6031:2009, this UDL of 10kPa has been treated as a permanent load and appropriate partial factors have been applied to this surcharge. This is conservative since the pedestrian surcharge behind the abutment is considered as 5kPa.

In the short term, a variable surcharge of 10kPa as a UDL was applied to represent the variable operational/maintenance loading behind the abutment. Therefore, the partial factor for variable unfavourable action has been applied to this UDL.

The benefit of the loading due to the railway track has not been considered in any of the stability calculation.

The analyses have been carried out by using the Morgenstern-Price method. The slope stability analyses considered both drained and undrained parameters in SLOPE/W. The long-term drained behaviour has been calculated according to Mohr-Coulomb equations, where effective friction angles and cohesion for the materials have been defined. The short-term undrained behaviour has been assessed considering the undrained properties of the embankment fill and fine grained materials underlying the earthworks.

Circular and non-circular slip surfaces were assessed. A depth of 0.3m was used as minimum slip surface depth in order to rule out insignificant, shallow slip surfaces.

Ground model and groundwater model as defined in Section 4.6.9.

For the seismic analysis, pseudo-static methods were adopted in the seismic ULS case and therefore the Damage Limitation State (DLS) was deemed satisfied. The seismic loads have been modelled in two sets which are as follows (as described in Section 4.6.11.1), where W is the weight of the sliding mass according to section 4.1.3.3 of BS EN 1998-5:

i. Set 1: Fh = 0.0375*W and Fv = 0.0123*W

ii. Set 2: Fh = 0.0375*W and Fv = -0.0123*W

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For the static case, a UDL of 114kN/m (worst case of average net bearing pressure of the bearing capacity check – Load Case Fzmax over a length of B' = 2.35m, see Table 4-59) has been used with a horizontal load of 59kN/m.

For the seismic case, a ULD of 96kN/m (worst case of average net bearing pressure of the bearing capacity check – Load Case Fxmax over a length of B' = 2.10m, see Table 4-61), has been used with a horizontal load of 55kN/m. Furthermore, an additional horizontal load of 4.25kN/m was added in order to represent the horizontal seismic inertia of the concrete abutment.

4.6.12.3 Slope Stability Results

The results of the slope stability assessment are shown in Table 4-63 below and in Appendix F. A minimum overdesign factor (ODF) of 1 is required to satisfy the GEO limit state, where the ODF is the ratio of design resistance to the corresponding design effect of actions.

The East abutment location was found to be the more critical of the two due to presence of soft Head Clay layer and ODF of the east and west abutment results are presented in Table 4-63.

Table 4-63: Results of Slope Stability of The Earthworks Blister for Bridleway SBH/19 Overbridge (DA1 C2 – GEO Limit State).

Abutment	Drainage Conditions	Failure Type	Location of Failure	ODF (1)	Figure in Appendix F
East Abutment	Drained	Circular	Through Earthworks	1.12	1
	Undrained	Circular	Through Earthworks	1.08	2
	Undrained – Seismic Kv-	Circular	Through Earthworks	1.01	3
	Undrained – Seismic Kv+	Circular	Through Earthworks	1.00	4
West Abutment	Drained	Circular	Through Earthworks	1.13	5
	Undrained	Circular	Through Earthworks	1.12	6
	Undrained – Seismic Kv-	Circular	Through Earthworks	1.06	7
	Undrained – Seismic Kv+	Circular	Through Earthworks	1.02	8

(1) The lowest ODF value for each analysis case is presented.

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4.7 Ancillary Culvert 060-AC4

4.7.1 Introduction

This section presents the geotechnical design of “Aylesbury Embankment Ancillary Culvert 060-AC4” (HS2-00002N7VP), also referred as “AE 060-AC4”, as part of IDR 207.2. The following sections summarise the geotechnical detailed design for the culvert units and headwalls with the details of geotechnical calculations presented in Appendix K.

4.7.2 Asset Description

The AE 060-AC4 is to be located offline at the approximate HS2 Chainage 060+575 on the upline side of the Aylesbury Embankment Level 1 Asset Group. The culvert is approximately offset 60m from the mainline earthworks.

The ancillary culvert crosses underneath the Oat Close Bridleway (SBH/19) Realignment (asset ID: HS2-000020RKL), at approximate highways Chainage 0+360 and connects eastern and western channels of the C2.L.22 land drainage catchment (asset ID: HS2-00002CT4W). A location plan of the 060-AC4 is shown in Figure 4-5

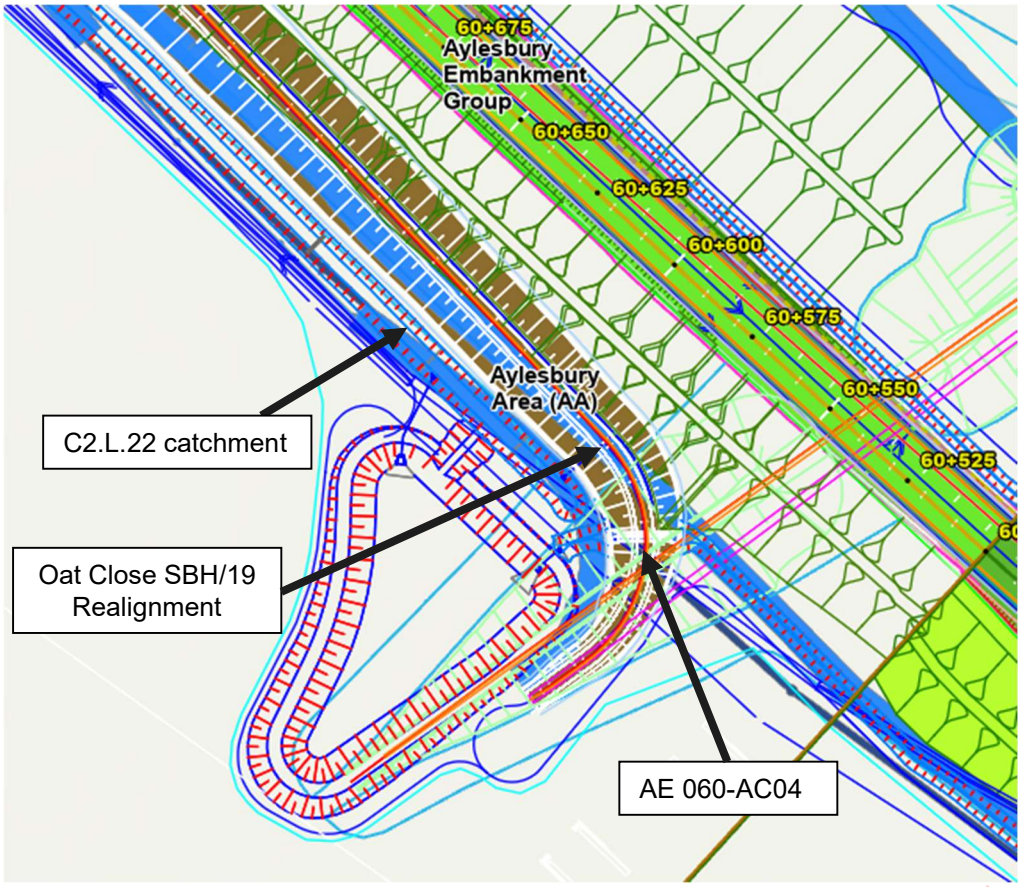


Figure 4-5: Location Plan of Aylesbury Embankment 060-AC4

060-AC4 is a pipe culvert with an internal diameter of 1500mm and a concrete pipe thickness of 155mm. The culvert pipe will be surrounded by Type S granular bedding. The pipe section is proposed to be 12.06m in length, leading to inlet and outlet headwalls. It comprises 3 Nos. 2.5m and 4 Nos. 1.25m precast concrete pipe culvert segments.

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The inlet and outlet invert levels are to be constructed at elevations of 81.993m and 81.968m AOD respectively, which is slightly lower elevation from the existing ground level.

It is anticipated that the pipe culvert and headwalls will be founded on the Head deposits, as identified in boreholes ML060-TT429, ML060-CT002, ML060-CR009 and ML060-WS006.

4.7.2.1 Relevant Drawings

Full details of the ancillary culvert are shown on the following drawings:

- 1MC06-CEK-CV-DGA-CS04_CL18-000262 - Aylesbury Embankment 060-AC4 - General Arrangement
- 1MC06-CEK-CV-DGA-CS04_CL18-000261 -Aylesbury Embankment 060-AC4 - Headwall and Wingwall General Arrangement
- 1MC06-CEK-CV-DSE-CS04_CL18-000072 - Aylesbury Embankment 060-AC4 - Elevations and Sections
- 1MC06-CEK-CV-DGA-CS04_CL18-000246 - Aylesbury Embankment 060-AC4 – General Notes

4.7.3 Design Standards

The geotechnical design is carried out for the stability of the headwalls. The design of the ancillary culvert has been undertaken to satisfy the requirements of:

i) HS2 Technical Standard - Culvert and Underpasses HS2-HS2-BR-STD-000-000005 Ref [74]
ii) BS EN 1991–2: 2003 Ref [70]
iii) BS EN 1997-1: 2004 + A1: 2013 Ref [4]
iv) PD6694-1: 2002+A1: 2020 Ref [39]
v) BRE SD1: 2005 Ref [32].
vi) AA – Ancillary Culvert Design Basis Report 1MC06-CEK-CV-REP-C002-000035 [75]

4.7.4 Construction Method and Sequence

The steps below provide the construction sequence that has been assumed in the design:

- Excavate and Replace with Class 6N6 material beneath the headwall and culvert foundations.
- Lay concrete blinding layer for culvert pipe and headwalls.
- Construction of cast-in situ headwall and lowering of each culvert unit into position.
- Prepare unit joints in accordance with manufacturer's design details.
- Apply waterproofing system as per specifications.
- Backfill trench to top of the culvert pipe with Type S surround material, working equally and alternatively on both sides using granular material. Subsequent filling with requirement to provide a minimum 1000mm cover above the roof slab of the culvert for the accommodation of construction traffic, prior to completion of main earthworks.
- Complete other works including guardrails, fences, accesses, etc.

4.7.5 Ground Model and Soil Parameters

The local GI data (Data Drop 40, October 2023), including ML060-TT429, ML060-CT002, ML060-CR009 and ML060-WS006 were considered for establishing the ground model of AE 060-AC4. A maximum of 1.8m bgl of Head deposits was encountered. Due to the presence of existing watercourses in the vicinity of the culvert, Alluvium may be present, despite not been identified in any of the local GI data. Therefore, the superficial geology has been referred to as Head deposits (possibly Alluvium) and conservative parameters have been used in the culvert design. Solid geology comprises the clay of the Gault Formation.

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The ground model and geotechnical parameters of the in situ materials are presented in Table 4-64 and Table 4-65 respectively. The parameters of granular fill at back of headwalls and foundation treatment are summarised in Table 4-66.

Table 4-64: Ground Model adopted for Detailed Design of AE Culvert 060-AC4

Geological Unit/ Description	Top of Stratum (mAOD)	Design Thickness (m)	Description
Head/Alluvium	83.17	2.55	Soft to firm brown slightly sandy slightly gravelly CLAY
Gault Formation	80.62	10.25	Firm, becoming stiff to very stiff, grey, and brown slightly sandy CLAY. Occasionally extremely weak grey sandy MUDSTONE, recovered as a firm, occasionally stiff, CLAY.
Portland Stone/Sand Formation (POST/POSA)	70.37	Not proven	Extremely weak to very weak, occasionally medium strong, grey SILTSTONE. Moderately weak to medium strong LIMESTONE.

Notes:
1. Alluvium may be present due to the presence of various watercourses around the culvert area.

Table 4-65: Geotechnical Parameters of In situ Ground adopted for Detailed Design of AE Culvert 060-AC4

Strata	Unit weight (γ) kN/m³	c_u (kPa)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)	Eu (MPa)	E' (MPa)	v'	v_u
HEAD Deposits (possibly Alluvium)	18	30 (0-2 m) 40 (2+ m)	21	23	0	9	7.2	0.2	0.5
Gault Formation	19	50+10z (up to 130 kN/m^2)	21	23	8	15+3z (up to 39MPa)	12+2.4z (up to 31MPa)	0.2	0.5

Notes:
1. z is depth (m) measured from the top of individual stratum.
2. Topsoil and subsoil will be stripped off during site clearance.
3. Made Ground, if encountered, will be excavated, and replaced by engineering fill before culvert construction.

Table 4-66: Geotechnical Parameters of Granular Fill for AE Culvert 060-AC4

Stratigraphy	Unit weight (γ) kN/m³		ϕ'_p	ϕ'_{cv}	c' (kPa)	Young's modulus E' (MPa)	Poisson's Ratio ν
	Above Groundwater Table	Below Groundwater Table					
Class 6N6	20	22	40	34	0	60	0.2

4.7.6 Groundwater

Table 2-5 and Figure 2-3 summarise the water strike and monitoring data across the Level 1 asset. Flows shall remain in-bank during operation. As the fill behind the headwalls is class 6N6 material, which will be

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free draining, the groundwater level has generally been taken at the top of the base slab for stability checks. For settlement checks, it is assumed the pipe is full.

4.7.7 Culvert Design

The culvert pipe is considered to be stable while it is buried in the embankment, therefore external stability checks (sliding and overturning) are not deemed required for the culvert sections. Bearing capacity check was also not deemed necessary in view of the Type S granular bedding surround that provides a suitable base stratum for the precast culvert pipe units. Details of the headwall can be found in Section 4.7.8.

Settlement calculations have been undertaken for the culvert pipe section and adjoining headwalls. The modelling of settlement/ ground displacement associated with the construction of the ancillary culvert is undertaken using Oasys Pdisp Version 20.1 software.

The culvert has been assessed for rotational pull-out failure (of the interlinking concrete units in the longitudinal direction of the culvert).

4.7.7.1 Design Loads

i. Live loads have been considered only for immediate settlement calculations.

ii. Permanent case traffic loads have not been considered provided that the edge of the carriageway lies outside the active wedge e.g. a distance greater than 45+φ/2 away from the headwall.

iii. Loading considered in the settlement calculation comprise of live load and dead load considered as follows,

- Live Load – the live load has conservatively been applied at the Culvert founding level and is in accordance with Table 2 BS EN 1991 – 2: 2003, Ref [70]
- Dead Load – Dead load comprises the self-weight of the soil fill above the culvert, self-weight of the culvert, the weight of removed soil subtracted from the original culvert self-weight, weight of water in the Culvert.

4.7.7.2 Settlement Check

The maximum settlement along the profile of culvert pipe and headwalls with no ground improvement is anticipated to be 17mm with max. horizontal separation from rotation of approx. 0.22mm and maximum relative displacement of around 7mm (less than 10mm limit).

Table 4-67: Settlement Result of AE Culvert 060-AC4

Analysis	Estimated Settlement (mm)
Maximum Pipe Structure Settlement on Head Deposits (possibly Alluvium) over Gault Formation	17

Table 4-68: Rotational and Displacement Analysis of Pipe Structure of AE Culvert 060-AC4

Rotation Check	Minimum	Maximum	Tolerance	Pass / Fail
Rotation of Segments (°)	-0.19	0.22	N/A	Pass
Relative Displacement at Top (mm)	-5.97	6.91	10	Pass

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The settlement results for the pipe structure are also shown in Figure 4-6 and Figure 4-7.

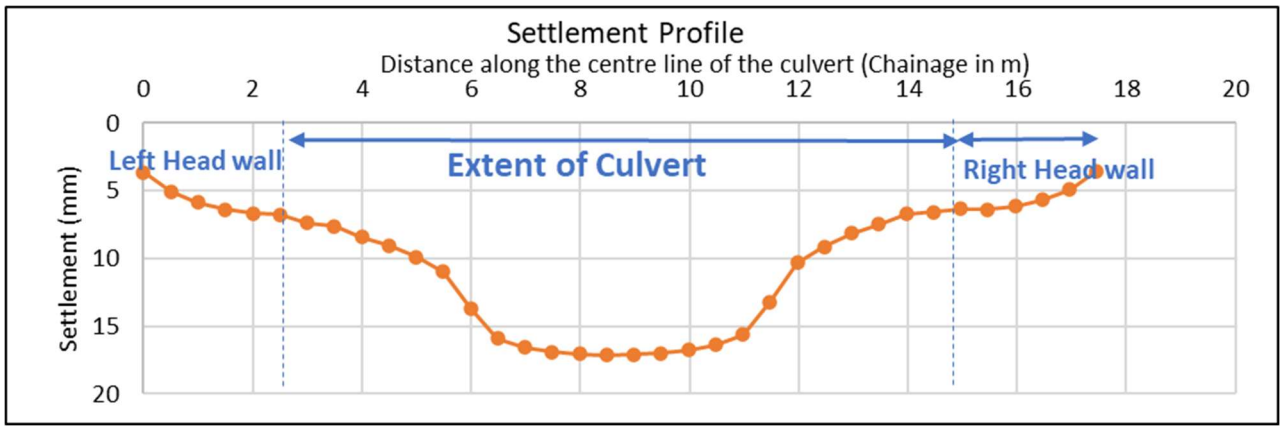


Figure 4-6: Aylesbury Embankment 060-AC4 – Settlement along Pipe Structure

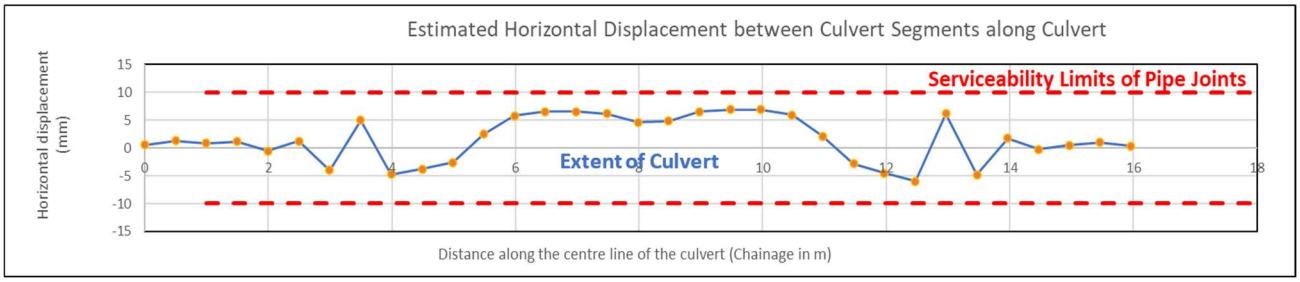


Figure 4-7: Aylesbury Embankment 060-AC4 – Horizontal Displacement along Pipe Structure

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4.7.8 Headwall Design

The following stability checks have been undertaken for the headwalls of AE Culvert 060-AC4:

a) Sliding
b) Overturning
c) Bearing
d) Settlement (see Section 4.7.7.2)

4.7.8.1 Design Loads

Three loading conditions have been adopted for the sliding, overturning, and bearing checks of the culvert headwalls:

- Case 1 - Construction condition with 20kPa variable surcharge, β = 0 degrees
- Case 1a - Construction condition with 10kPa variable surcharge, β = 18 degrees
- Case 2 - In-service condition with 10kPa permanent surcharge, β =18 degrees

Case 1 analysis with 20kPa variable surcharge on a level ground has been considered in the view that heavy construction plant (20kPa) cannot operate on sloping ground.

Case 1a analysis has considered for the conditions during slope construction, assuming lighter plant operation (10kPa variable surcharge) on slope.

Case 2 in service condition of the culvert, the nominal surcharge loading (10kPa permanent surcharge) was considered.

4.7.8.2 Sliding Check

A minimum of 600mm Excavate and Replace with Class 6N6 material below the headwall base slab achieves satisfactory degrees of utilisation for sliding for all Cases 1, 1a and 2. A summary of the headwall sliding check results are presented in Table 4-69 below.

Table 4-69: Sliding Check Outcome of Headwalls of AE Culvert 060-AC4

Headwall Sliding Check	Utilisation (%)		
	Foundation on 600mm of Class 6N6 (check at Granular / Subgrade Interface)		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	96%	90%	89%
DA1C2	74%	68%	66%
SLS	64%	60%	60%

4.7.8.3 Overturning Check

The degrees of utilisation for overturning are less than 100% for the headwalls for all Cases 1, 1a and 2 with 600mm Excavate and Replace. A summary of the headwall overturning check results are presented in Table 4-70 below.

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Table 4-70: Overturning Check Outcome of Headwalls of AE Culvert 060-AC4

Headwall Overturning Check	Utilisation (%)		
	Foundation on 600mm of Class 6N6 (Check against shear key toe Point 'A')		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	84%	75%	73%
DA1C2	66%	57%	55%
SLS	49%	44%	44%

4.7.8.4 Bearing Check

The degree of utilisations for the bearing checks are less than 100% for the headwalls for all Cases 1, 1a and 2 with 600mm Excavate and Replace. A summary of the headwall bearing check results is presented in Table 4-71 below.

Table 4-71: Bearing Resistance Check Outcome of Headwalls of AE Culvert 060-AC4

Headwall Bearing Check	Utilisation (%)		
	Foundation on 600mm of Class 6N6		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	6%	5%	5%
DA1C2	11%	8%	8%

4.7.9 Ground Aggressivity Assessment

This section presents the Design Sulphate (DS) and Aggressive Chemical Environment for Concrete (ACEC) classifications for the concrete culvert structure.

Due to the limited chemical testing results available for the specific location of 060-AC4 pipe culvert, all the chemical testing data for the entire Aylesbury Embankment (GI up to Data Drop 40, i.e., Oct 2023) were considered for the Ground aggressivity assessment was undertaken based on Build Research Establishment (BRE) Special Digest 1 (SD1) suite Ref [32].

Analysis of chemical testing data was undertaken, and a classification linked to brownfield location (presence of existing highway) is presented in Table 4-72 below. It is recommended that the concrete structure is designed with Design Sulphate (DSE) of DS-3 and Concrete Classification (ACEC) of AC-3.

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Table 4-72: Summary of Durability Test Results adopted for AE Culvert 060-AC4

Geological Formation	Design Sulphate	Concrete Classification	Total Potential Sulphate (TPS) %	pH	Water Soluble SO_4 (mg/l)
Head Deposits	DS-1	AC-1	0.03	7.41	179.27
Gault Clay	DS-3	AC-3	0.9	7.9	-

Notes:

- Characteristic Total Potential Sulphate and Water Soluble SO_4 values were taken based on the guidelines from clause C5. Step 1a of BRE SD1 Ref. [32] for brownfield location.

Design Sulphate (DS) and Concrete (ACEC) classes were determined based on Table C2 of BRE SD1 Ref. [32] for brownfield location.

4.7.10 Designer Risk Assessment (during Construction)

Several CDM risks for AE Culvert 060-AC4 have been identified. As a result, mitigation has been proposed to eliminate and reduce these risks. The residual risks are shown on the construction drawings for the contractor to be addressed during construction. These risks include:

- Instability of temporary excavations,
- Construction vehicles causing damage to culvert and headwalls,
- Working in proximity of existing buried and overhead utilities,
- Shallow groundwater table.

4.7.11 Conclusions and Recommendations

To ensure the external stability of the Headwall associated with the AE Culvert 062-AC1, a foundation treatment in the form of excavate and replace (E&R) is required. It involves a minimum of 600mm E&R with Class 6N6 material below the base slab of headwall.

Details of foundation treatment below the headwall are shown on the Aylesbury Embankment 060-AC4 drawings listed in Section 4.7.2.1 and the relevant Appendices to the IDR Package 207.2 Specification (document number: 1MC06-CEK-AU-SPE-CS04_CL18-000003) Ref [71].

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4.8 Ancillary Culvert 061-AC1

4.8.1 Introduction

This section presents the geotechnical design of “Aylesbury Embankment Ancillary Culvert 061-AC1” (HS2-00002N7VQ), also referred as “AE 061-AC1”, as part of IDR 207.2. The following sections summarise the geotechnical detailed design for the culvert units and headwalls with the details of geotechnical calculations presented in Appendix L.

4.8.2 Asset Description

The AE 061-AC1 is to be located offline at the approximate HS2 Chainage 061+128 within the Aylesbury Embankment Level 1 Asset Group. The culvert is to be at approximate 70m offset from the mainline earthworks.

The ancillary culvert crosses underneath the Oat Close Bridleway (SBH/19) Realignment (asset ID: HS2-000020RKL), at approximate highways chainage 0+230 and connects eastern and western channels of the C2.L.22 land drainage catchment (asset ID: HS2-00002CT4W). A location plan of the 061-AC1 is shown in Figure 4-8.

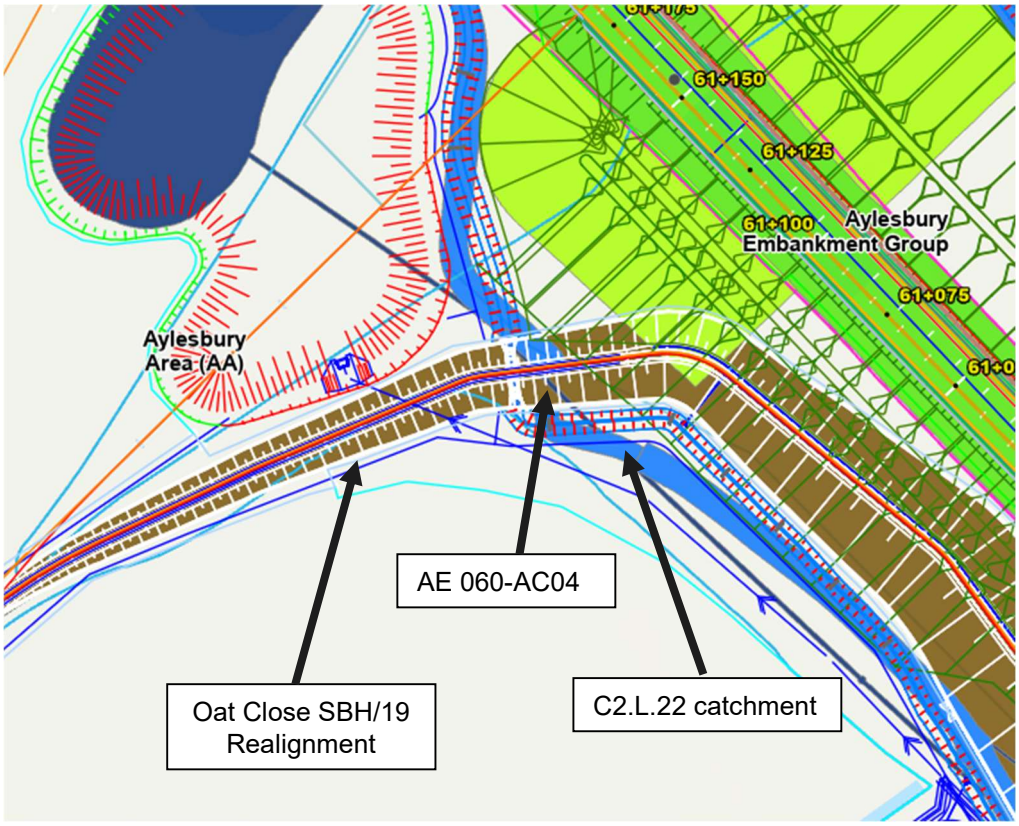


Figure 4-8: Location Plan of Aylesbury Embankment 061-AC1

061-AC1 is a pipe culvert with an internal diameter of 1500mm and a concrete pipe thickness of 155mm. The culvert pipe will be surrounded by Type S granular bedding. The pipe section is proposed to be 12.06m in length, leading to inlet and outlet headwalls. It comprises 3 Nos. 2.5m and 4 Nos. 1.25m precast concrete pipe culvert segments.

It is anticipated that the culvert and headwalls will be founded on the Gault Clay, as identified in boreholes ML061-WS018,ML061-TP435 and ML061-TP436.

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4.8.2.1 Relevant Drawings

Full details of the ancillary culvert are shown on the following drawings:

- 1MC06-CEK-CV-DGA-CS04_CL18-000264 - Aylesbury Embankment 061-AC1 -General Arrangement
- 1MC06-CEK-CV-DGA-CS04_CL18-000263-Aylesbury Embankment 061-AC1 - Headwall and Wingwall General Arrangement
- 1MC06-CEK-CV-DSE-CS04_CL18-000073- Aylesbury Embankment 061-AC1 - Elevations and Sections
- 1MC06-CEK-CV-DSE-CS04_CL18-000247- Aylesbury Embankment 061-AC1 – General Notes

4.8.3 Design Standards

The geotechnical design is carried out for the stability of the headwalls. The design of the ancillary culvert has been undertaken to satisfy the requirements of:

i) HS2 Technical Standard - Culvert and Underpasses HS2-HS2-BR-STD-000-000005 Ref [74]
ii) BS EN 1991–2: 2003 Ref [70]
iii) BS EN 1997-1: 2004 + A1: 2013 Ref [4]
iv) PD6694-1:2002+A1:2020 Ref [39]
v) BRE SD1: 2005 Ref [32].
vi) AA – Ancillary Culvert Design Basis Report 1MC06-CEK-CV-REP-C002-000035 Ref [75]

4.8.4 Construction Method and Sequence

The steps below provide indicative construction sequence that is assumed by the designer:

1. Excavate and replace with 6N6 material beneath the headwall and culvert foundations
2. Lay concrete blinding layer for culvert pipe and headwalls
3. Construction of cast-in situ headwalls and lowering of each culvert unit into position
4. Prepare unit joints in accordance with manufacturer's design details
5. Apply waterproofing system as per specifications
6. Backfill trench to top of the culvert pipe with Type S surround material, working equally and alternatively on both sides using granular material. Subsequent filling with requirement to provide a minimum 1000mm cover above the roof slab of the culvert for the accommodation of construction traffic, prior to completion of main earthworks
7. Complete other works including guardrails, fences, accesses, etc.

4.8.5 Ground Model and Soil Parameters

The local GI data (Data Drop 40, October 2023), including ML061-TP451, ML061-WS018, ML061-WS019, ML061-TP435, ML061-TP436, ML061-TP400 and ML061-TP450 were considered for establishing the ground model of AE 061-AC1. Solid geology comprises of Gault Formation below the culvert.

The ground model and geotechnical parameters of the in situ materials are presented in Table 4-73 and Table 4-74 respectively. The parameters of granular fill at back of headwalls and foundation treatment are summarised in Table 4-75.

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Table 4-73: Ground Model adopted for Detailed Design of AE Culvert 061-AC1

Geological Unit/ Description	Top of Stratum (m AOD)	Design Thickness (m)	Description
Gault Clay Formation	79.94	6.25	Firm, becoming stiff to very stiff, grey, and brown slightly sandy CLAY Occasionally extremely weak grey sandy MUDSTONE, recovered as a firm, occasionally stiff, CLAY
Portland Stone/ Sand Formation (POST/POSA)	72.69	Not proven	Extremely weak to very weak, occasionally medium strong, grey SILTSTONE Moderately weak to medium strong LIMESTONE

Table 4-74: Geotechnical Parameters of In situ Ground adopted for Detailed Design of AE Culvert 061-AC1

Strata	Unit weight (γ) kN/m³	c_u (kPa)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)	Eu (MPa)	E' (MPa)	v'	v_u
Gault Clay Formation	19	50+10z (up to 150 kPa)	21	23	8	15+3z (up to 45MPa)	12+2.4z (up to 36MPa)	0.2	0.5
POSA- CZ	19	50+10z (up to 130 kPa)	21	23	8	15+3z (up to 39MPa)	12+2.4z (up to 31MPa)	0.2	0.5

Notes:
1 z is depth (m) measured from the top of individual stratum.
2 Culvert is founded on GLT formation at 78.94 m AOD.
3 Topsoil and subsoil will be stripped off during site clearance.
4 Made Ground, if encountered, will be excavated, and replaced by engineering fill before culvert construction.

Table 4-75: Geotechnical Parameters of Granular Fill for AE Culvert 061-AC1

Stratigraphy	Unit weight (γ) kN/m³		ϕ'_p	ϕ'_{cv}	c' (kPa)	Young's modulus E' (MPa)	Poisson's Ratio ν
	Above Groundwater Table	Below Groundwater Table					
Class 6N6	20	22	40	34	0	60	0.2

4.8.6 Groundwater

Table 2-5 and Figure 2-3 summarise the water strike and monitoring data across the Level 1 asset; groundwater monitoring recorded depths between 1.0 m and 2.5m bgl. Flows shall remain in-bank during operation. As the fill behind the headwalls is class 6N6 material, which will be free draining, the groundwater level has generally been taken at the top of the base slab for stability checks. For settlement checks, it is assumed the pipe is full.

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4.8.7 Culvert Design

The culvert pipe is considered to be stable while it is buried in the embankment, therefore external stability checks (sliding and overturning) are not deemed required for the culvert sections. Bearing capacity check was also not deemed necessary in view of the Type S granular bedding surround that provides a suitable base stratum for the precast pipe culvert pipe units. Details of the headwall stability checks and foundation treatment design can be found in Section 4.8.8.

Settlement calculations have been undertaken for the culvert pipe section and adjoining headwalls. The modelling of settlement/ ground displacement associated with the construction of the ancillary culvert is undertaken using Oasys Pdisp Version 20.1 software.

The culvert has been assessed for rotational pull-out failure (of the interlinking concrete units in the longitudinal direction of the culvert).

4.8.7.1 Design Loads

i. Live loads have been considered only for immediate settlement calculations.

ii. Permanent case traffic loads have not been considered provided that the edge of the carriageway lies outside the active wedge e.g. a distance greater than 45+φ/2 away from the headwall.

iii. Loading considered in the settlement calculation comprise of live load and dead load considered as follows,

- Live Load – the live load has conservatively been applied at the Culvert founding level and is in accordance with Table 2 BS EN 1991 – 2: 2003, Ref [70]
- Dead Load – Dead load comprises the self-weight of the soil fill above the culvert, self-weight of the culvert, the weight of removed soil subtracted from the original culvert self-weight, weight of water in the Culvert.

4.8.7.2 Settlement Check

The settlement results are summarised in Table 4-76 and Table 4-77.

Table 4-76: Settlement Result of AE Culvert 061-AC1

Analysis	Estimated Settlement (mm)	Tolerance (mm)
Maximum Pipe Structure Settlement on Alluvium (possibly) over Gault Formation	15.0	NA

Table 4-77: Rotational and Displacement Analysis of Pipe Structure of AE Culvert 061-AC1

Rotation Check	Minimum	Maximum	Tolerance	Pass / Fail
Rotation of Segments (º)	-0.14	0.07	NA	Pass
Relative Displacement at Top (mm)	-4.30	2.10	10	Pass

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The settlement results for the pipe structure are also shown in Figure 4-9 and Figure 4-10.

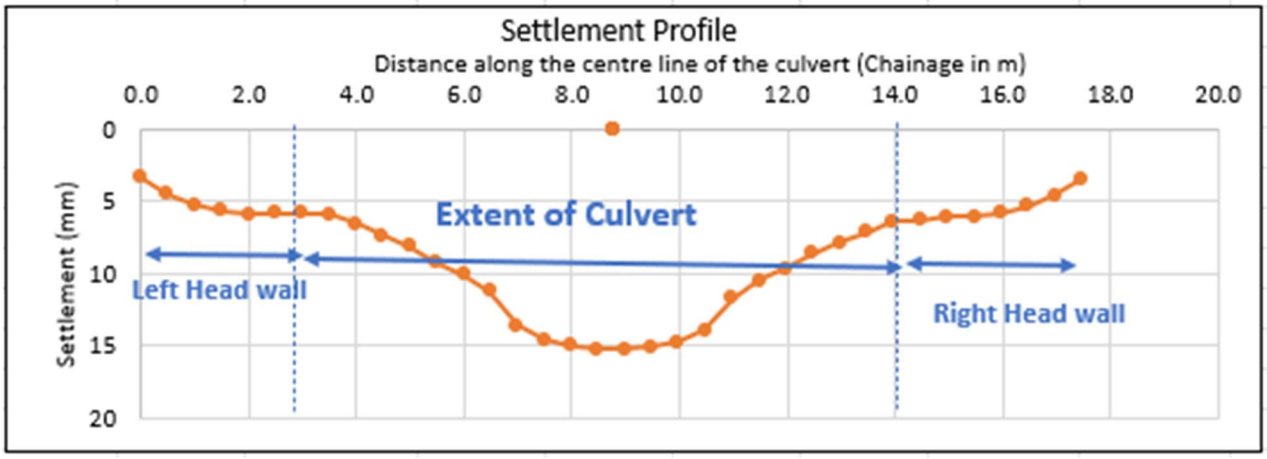


Figure 4-9: Aylesbury Embankment 061-AC1 – Settlement along Pipe Structure

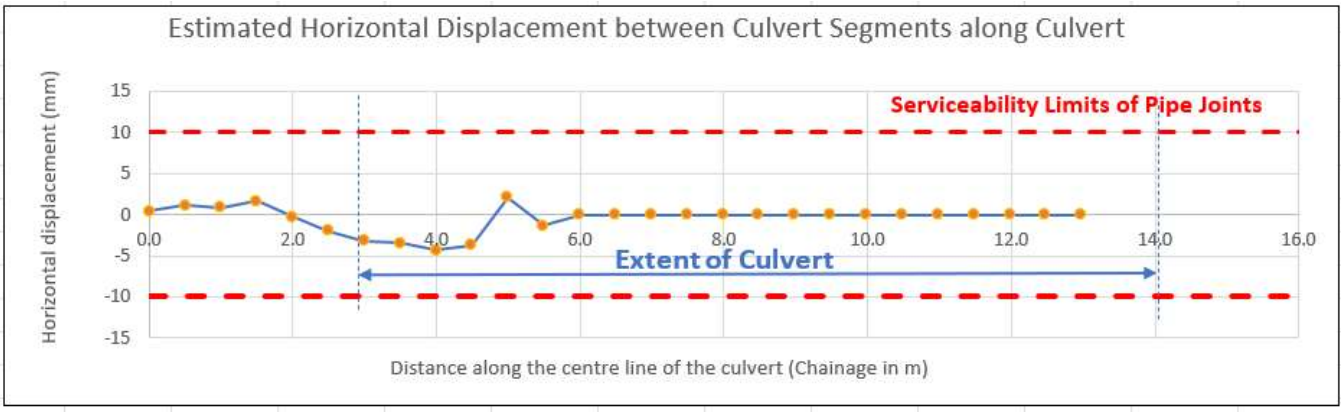


Figure 4-10: Aylesbury Embankment 061-AC1 – Horizontal Displacement along Pipe Structure

4.8.8 Headwall Design

The following stability checks have been undertaken for the headwalls of Culvert 061-AC1:

a) Sliding
b) Overturning
c) Bearing
d) Settlement (see section above)

4.8.8.1 Design Loads

Three loading conditions have been adopted for the sliding, overturning, and bearing checks of the culvert headwalls:

- Case 1 - Construction condition with 20kPa surcharge, β = 0 degrees
- Case 1a - Construction condition with 10kPa variable surcharge, β = 18 degrees
- Case 2 - In-service condition with 10kPa permanent surcharge, β =18 degrees

Case 1 analysis with 20kPa variable surcharge on a level ground has been considered in the view that heavy construction plant (20kPa) cannot operate on sloping ground.

Case 1a analysis has considered for the conditions during slope construction, assuming lighter plant operation (10kPa variable surcharge) on slope.

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Case 2 in service condition of the culvert, the nominal surcharge loading (10kPa permanent surcharge) was considered.

4.8.8.2 Sliding Check

A minimum of 600mm excavate and replace with Class 6N6 material below the headwall base slab to achieves satisfactory degrees of utilisation for sliding stability. A summary of the headwall sliding check results is presented in Table 4-78 below.

Table 4-78: Sliding Check Outcome of Headwalls of AE Culvert 061-AC1

Headwall Sliding Check	Utilisation (%)		
	Foundation on 600mm of Class 6N6 (check at Granular / Subgrade Interface)		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	95%	97%	97%
DA1C2	74%	74%	74%
SLS	62%	62%	62%

4.8.8.3 Overturning Check

The degrees of utilisation for overturning are less than 100% for the headwalls for all Cases 1, 1a and 2 with 600mm Excavate and Replace. A summary of the headwall overturning check results are presented in Table 4-79 below.

Table 4-79: Overturning Check Outcome of Headwalls of AE Culvert 061-AC1

Headwall Overturning Check	Utilisation (%)		
	Check against Shear Key Toe		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	78%	73%	73%
DA1C2	61%	56%	56%
SLS	45%	41%	41%

4.8.8.4 Bearing Check

The degree of utilisations for the bearing checks are less than 100% for the headwalls for all Cases 1, 1a and 2 with 600mm Excavate and Replace. A summary of the headwall bearing check results is presented in Table 4-80 below.

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Table 4-80: Bearing Resistance Check Outcome of Headwalls of AE Culvert 061-AC1

Headwall Bearing Check	Utilisation (%)		
	Foundation on 0.6m of Class 6N6		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	6%	5%	5%
DA1C2	11%	9%	9%

4.8.9 Ground Aggressivity Assessment

This section presents the Design Sulphate (DS) and Aggressive Chemical Environment for Concrete (ACEC) classifications for the concrete culvert structure.

Due to the limited chemical testing results available for the specific location of 061-AC1 pipe culvert, all the chemical testing data for the entire Aylesbury Embankment (GI up to Data Drop 40, i.e., Oct 2023) were considered for the Ground aggressivity assessment was undertaken based on Build Research Establishment (BRE) Special Digest 1 (SD1) suite Ref [32].

Summary of durability test rest results are presented in Table 4-81 below. Analysis of chemical testing data was undertaken, and a classification linked to the brownfield location (presence of existing highway) is presented below. It is recommended that the concrete structure is designed with Design Sulphate (DSE) of DS-3 and Concrete Classification (ACEC) of AC-3.

Table 4-81: Summary of Durability Test Results adopted for AE Culvert 061-AC1

Geological Formation	Design Sulphate	Concrete Classification	Total Potential Sulphate (TPS) %	pH	Water Soluble SO_4 (mg/l)
Gault Clay	DS-3	AC-3	0.90	7.9	-

4.8.10 Designer Risk Assessment (during Construction)

Several CDM risks for AE Culvert 061-AC1 have been identified. As a result, mitigation has been proposed to eliminate and reduce these risks. The residual risks are shown on the construction drawings for the contractor to be addressed during construction. These risks include:

- Instability of temporary excavation,
- Construction vehicles causing damage to culvert and headwalls,
- Working in proximity of existing buried and overhead utilities,
- Shallow groundwater table.

4.8.11 Conclusions and Recommendations

To ensure the external stability of the Headwall associated with the AE Culvert 061-AC1, a foundation treatment in the form of excavate and replace (E&R) is required. It involves a minimum of 600mm E&R with Class 6N6 material below the base slab of headwall.

Details of foundation treatment below the headwall are shown on the Aylesbury Embankment 061-AC1 drawings listed in Section 4.8.2.1 and relevant Appendices to the IDR Package 207.2 Specification (document number: 1MC06-CEK-AU-SPE-CS04_CL18-000003), Ref [71].

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4.9 Ancillary Culvert 061-AC3

4.9.1 Introduction

This section presents the geotechnical design of “Aylesbury Embankment Ancillary Culvert 061-AC3” (HS2-00002N7VR), also referred as “AE 061-AC3”. The following sections summarise the geotechnical detailed design for the culvert unit and headwalls with the details of geotechnical calculations presented in Appendix M.

4.9.2 Asset Description

The AE 061-AC3 culvert is to be located offline at the approximate HS2 Chainage 061+300 on the down-line side of the Aylesbury Embankment Level 1 Asset Group. The culvert is offset approximately 50m from the mainline earthworks.

The ancillary culvert crosses beneath the maintenance access road to pond (south arm off Calley Farm-south) (asset ID: HS2-00002BJSK, part of IDR 208.4) and allows Sedrup North Watercourse diversion (Asset ID: HS2-00002CT5D) to flow beneath the maintenance access road. A location plan of Culvert 061-AC3 is shown in Figure 4-11.

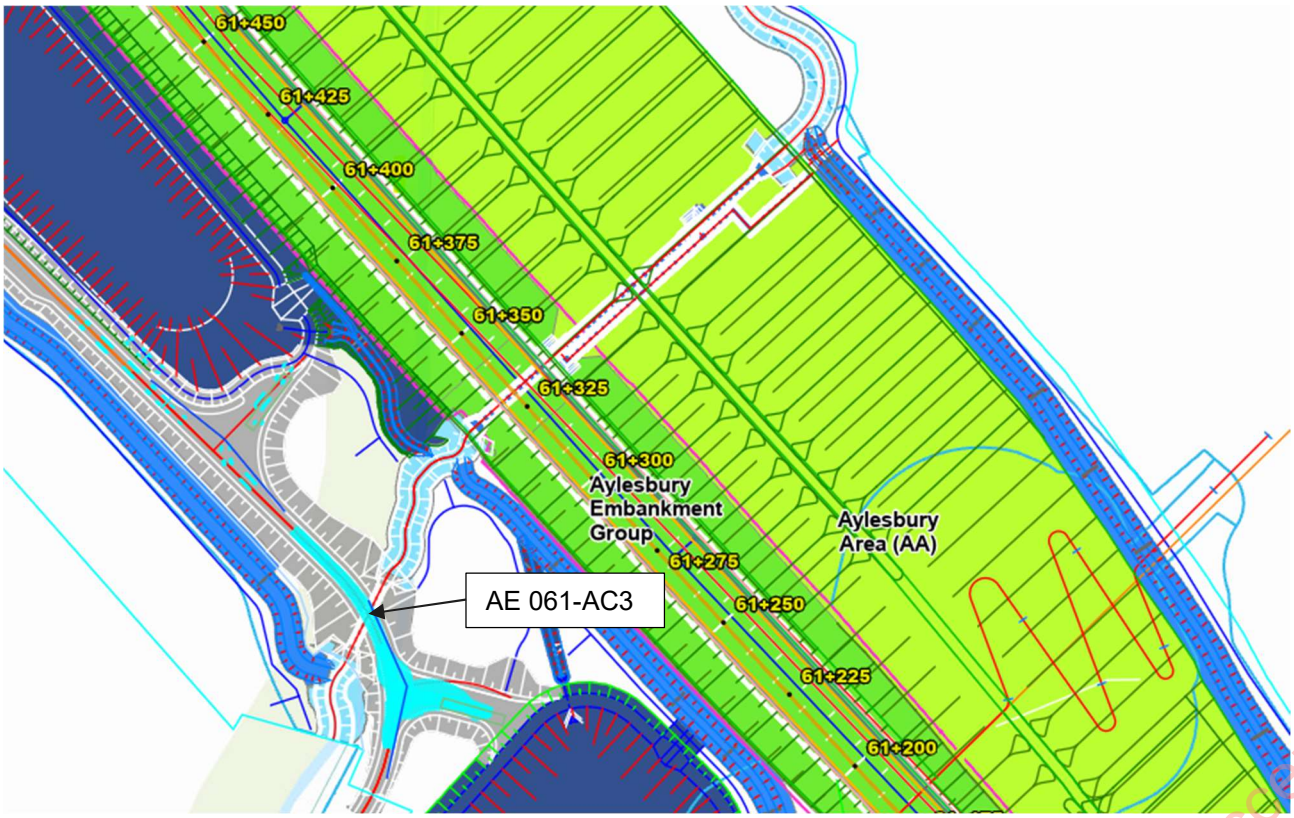


Figure 4-11: Aylesbury Embankment 061-AC3 Culvert General Arrangement Plan

AE 061-AC3 is a box culvert with size of 2.0m x 2.0m with inlet and outlet headwall. The length of box culvert is 16.09m and consists of 8nos of 2m box segments. The inlet and outlet headwall unit consists of wingwalls.

The inlet and outlet invert levels are at elevations of 76.823m and 76.805m AOD respectively, which is around 0.9m lower than existing ground level.

It is anticipated that the box culvert and headwalls will be founded on the Portland Stone/sand formation., as identified in boreholes ML061-TP447, ML061-TP448 and ML061-CR013.

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4.9.3 Relevant Drawings

Full details of the ancillary culvert are shown on the following drawings:

- 1MC06-CEK-CV-DSE-CS04_CL18-000050 - Aylesbury Embankment 061-AC3 - Elevations and Sections
- 1MC06-CEK-CV-DGA-CS04_CL18-000071 - Aylesbury Embankment 061-AC3- General Arrangement
- 1MC06-CEK-CV-DGA-CS04_CL18-000073 - Aylesbury Embankment 061-AC3– Inlet Headwall and Wingwall General Arrangement
- 1MC06-CEK-CV-DGA-CS04_CL18-000248 - Aylesbury Embankment 061-AC3– General Notes

4.9.4 Design Standards

The geotechnical design has been carried out for the stability of the culvert and headwalls. The design of the ancillary culvert has been undertaken to satisfy the requirements of:

- HS2 Technical Standard - Culvert and Underpasses HS2-HS2-BR-STD-000-000005 Ref [74]
- BS EN 1991–2: 2003 Ref [70]
- BS EN 1997-1: 2004 + A1: 2013 Ref [4]
- PD6694-1: 2002+A1: 2020 Ref [39]
- BRE SD1: 2005 Ref [32].
- AA – Ancillary Culvert Design Basis Report 1MC06-CEK-CV-REP-C002-000035 [75]

4.9.5 Construction Method and Sequence

The steps below provide the construction sequence that has been assumed in the design:

- Excavate and Replace with Class 6N6 material beneath the headwall and culvert foundations.
- Lay concrete blinding layer for culvert box and headwalls.
- Construction of cast-in situ headwall and lowering of each culvert unit into position.
- Prepare unit joints in accordance with manufacturer's design details.
- Apply waterproofing system as per specifications.
- Backfill trench to top of the culvert box with Class 6N6 fill material. Subsequent filling with requirement to provide a minimum 1000mm cover above the roof slab of the culvert prior to construction trafficking (Volvo A60H articulated haulers).
- Complete other works including guardrails, fences, accesses, etc.

4.9.6 Ground Model and Soil Parameters

The local GI data (Data Drop 42, December 2023), which includes ML061-CR013, ML061-TP447 and ML061-TP448, were considered for establishing the ground model of AE 061-AC3. The solid geology consists of Portland Sand/Stone and Kimmeridge Clay Formation.

The ground model and geotechnical parameters of the in situ materials are presented in Table 4-82 and Table 4-83 respectively. The parameters for the granular fill behind the headwalls and culvert units, as well as for the foundation treatment, are summarised in Table 4-84.

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Table 4-82: Ground Model adopted for Detailed Design of AE Culvert 061-AC3

Geological Unit/ Description	Design Range (m)	Description
Topsoil	0.0 to 0.3	Firm brown slightly gravelly slightly sandy silty CLAY. Occasional roots and rootlets. Sand is fine to coarse. Gravel is angular to subrounded fine to coarse of chert.
Portland Sand /Stone Formation (POSA/POST)	0.3m to 18.20m	Firm and stiff orangish brown mottled grey and brown slightly sandy CLAY with rare pockets of black possibly manganese oxides and occasional pockets of orangish brown silty sand.
Kimmeridge Clay Formation (KC)	18.20m to 19.60m	Stiff dark grey slightly calcareous silty CLAY with frequent white shell fragments.

Table 4-83: Geotechnical Parameters of In situ Ground *adopted for Detailed Design of AE Culvert 061-AC3*

Strata	Unit Weight (γ) kN/m³	cu (kPa)	ϕ'cv (°)	ϕ'pk (°)	c' (kPa)	Eu (MPa)	E' (MPa)	v'	vu
Portland Sand /Stone Formation (POSA/POST)	20	30 (0 – 2 m bgl) 40 (2- 4.5 m bgl) 40+8z depth > 4.5 m bgl	23	25	0	6 (0 – 2 m bgl) 2.0 m + 15+3z up to 39	4.8 (0 – 2 m bgl) 2.0 m + 12+2.4z up to 31	0.2	0.5

Notes:

1. z is depth (m) measured from the top of individual stratum.
2. Topsoil and subsoil will be stripped off during site clearance.
3. Made Ground, if encountered, will be excavated, and replaced by engineering fill before culvert construction.

Table 4-84: Geotechnical Parameters of Granular Fill for AE Culvert 061-AC3

Stratigraphy	Unit weight (γ) kN/m³		ϕ'p	ϕ'cv	c' (kPa)	Young's modulus E' (MPa)	Poisson's Ratio ν
	Above Groundwater Table	Below Groundwater Table					
Class 6N6	20	22	40	34	0	60	0.2

4.9.7 Groundwater

Table 2-5 and Figure 2-3 summarise the water strike and monitoring data across the Level 1 asset. Groundwater has been taken at existing ground level for design of the culvert. For the headwall, the groundwater level has generally been taken at the top of the base slab for stability checks, as the fill behind the headwalls is class 6N6 material and will be free draining.

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4.9.8 Culvert Design

Stability checks including sliding, overturning, bearing, and settlement have been conducted for AE 061-AC3 Culvert. Settlement calculations have been undertaken for the culvert box section and adjoining headwalls. The modelling of settlement/ ground displacement was undertaken using Oasys PDisp Version 20.1 software. The culvert has undergone an assessment for potential rotational pull-out failure, specifically focusing on the interlinking concrete units along the longitudinal direction of the culvert.

Two different design stages have been considered for the design of the box culvert: the service stage and the construction stage.

4.9.8.1 Design Loads

Service Stage

i. Live loads from traffic have been calculated based on EN 1991-2-2003 [79] and PD 6694 cl.10.2.
ii. Loading considered in the settlement calculation comprises live load and dead load considered as follows:
    - Live Load – the live load has been applied at the Culvert founding level and is in accordance with Table 2 BS EN 1991 – 2: 2003 [79]
    - Dead Load – Dead load comprises the self-weight of the soil fill above the culvert, self-weight of the culvert and weight of water in the culvert.

Construction Stage

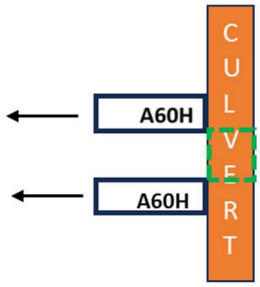
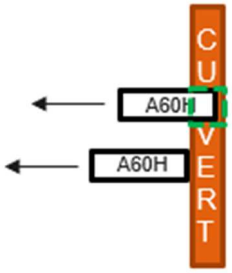
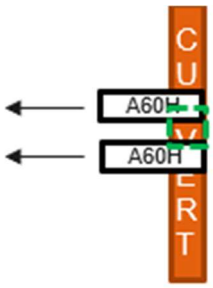
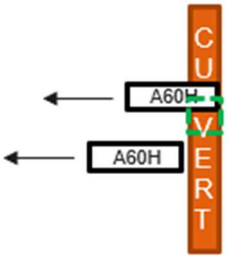
During construction, A60H vehicles have been taken into account for the calculations. These vehicles will traverse directly over the culvert and the embankment on either side. The weight of the A60H vehicles will exert a surcharge on the culvert and the ground adjacent to the culverts. Additionally, any braking and traction forces from vehicles on or close to the culvert will generate horizontal forces. Four load cases were analysed based on the arrangement of the vehicles.

i. Case A: Arrangement of vehicles to give the maximum axle load applied directly behind the culvert to give the maximum horizontal force at the side of the culvert.
ii. Case B: Arrangement of vehicles to give the maximum braking and traction force together with the corresponding maximum horizontal force on the side of the culvert from vehicles adjacent to the culvert.
iii. Case C: Arrangement of vehicles to give the maximum bearing pressure.
iv. Case D: Alternative arrangement of vehicles considered (see below).

The schematic arrangement of vehicle for each load cases are presented in Table 4-85.

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Table 4-85: Summary of construction load cases

Cases	Schematic Plan of Truck Layout
Case A	 One Axle directly behind culvert
Case B	 One Axle on culvert and one directly behind the culvert. (Only one wheel on any single 2m length culvert segment)
Case C	 Two Axles on the culvert
Case D	 Two Axles on the culvert. (Only one wheel fom each axle on any single 2m length culvert segment)

Where,

Worst case position of culvert segment analysed

Stability check results in service stage and construction stage are summarised in Table 4-86 and Table 4-87.

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Table 4-86: Summary of External Stability Checks in Service Stage for AE Culvert 061-AC3

Analyses	Utilisation (%)			
	Sliding		Overturning	Bearing
	Culvert/Granular Fill Interface (with 0.8m E&R)	Granular Fill / Natural Ground Interface		On natural Ground*
DA1C1	45	38	63	76
DA1C2	38	33	51	78

*Notes: * Design cautiously assumes culvert founding directly on natural ground and no benefit taken from E&R. If utilisation is greater than 100%, then a punching shear check has been undertaken.*

Table 4-87: Summary of External Stability Checks in Construction Stage for AE Culvert 061-AC3

Analyses	Utilisation (%)															
	Sliding								Overturning				Bearing			
	Culvert / Granular Fill Interface (with 0.8m E&R)				Granular Fill / Subgrade Interface								Foundation on Natural Ground			
	A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
DA1C1	45	54	24	29	35	41	22	25	74	98	28	32	65	37*	68	61
DA1C2	36	44	20	24	29	34	18	21	56	75	22	25	52	34*	66	59
SLS	83	73	35	47	69	58	35	44	90	90	27	38	NA	NA	NA	NA

*Notes: * Results for punching shear check with 0.8m E&R using Class 6N6 fill*

4.9.8.2 Settlement Check

The anticipated maximum settlement along the culvert box and headwalls profile, with ground improvement involving excavating and replacing with a 0.8m thickness of class 6N6 fill, is estimated to be approximately 23mm. Settlement analysis results are summarised in Table 4-88 and Table 4-89.

Table 4-88: Settlement Result for AE Culvert 061-AC3

Analysis	Estimated Settlement (mm)
Maximum Box Structure Settlement on Class 6N6 fill underlaid Portland Sand /Stone Formation over Kimmeridge Clay Formation	23

Table 4-89: Rotational and Displacement Analysis of Box Structure for AE Culvert 061-AC3

Rotation Check	Minimum	Maximum	Tolerance	Pass / Fail
Rotation of Segments (º)	-0.08	0.19	0.23	Pass

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Rotation Check	Minimum	Maximum	Tolerance	Pass / Fail
Relative Displacement at Top (mm) (joint opening/closing)	-3.55	8.11	10	Pass

The settlement profile for the culvert structure is shown in Figure 4-12 and Figure 4-13.

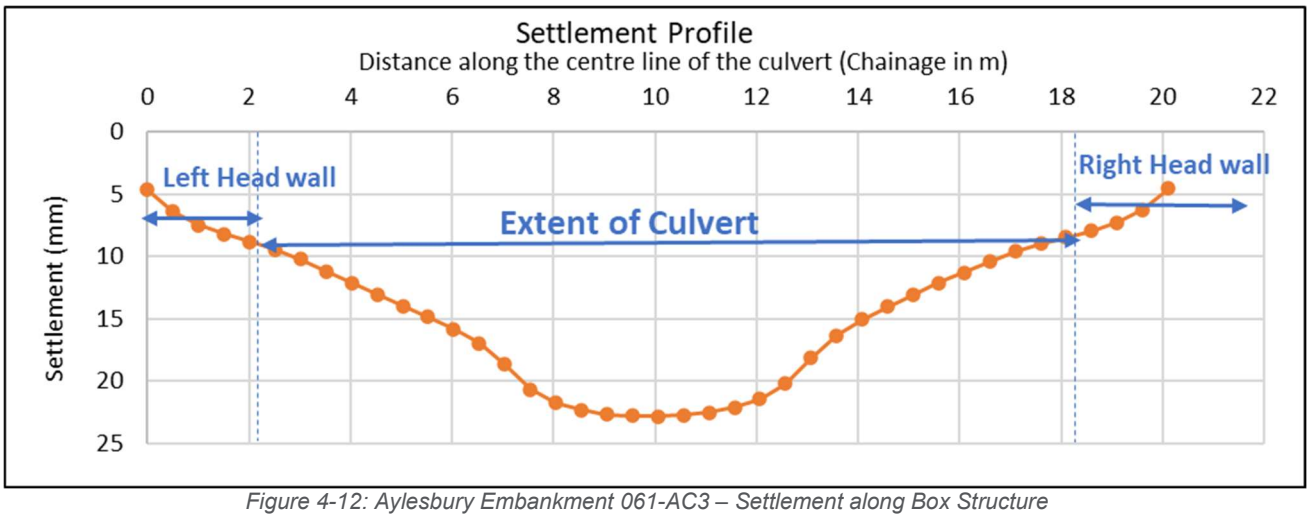


Figure 4-12: Aylesbury Embankment 061-AC3 – Settlement along Box Structure

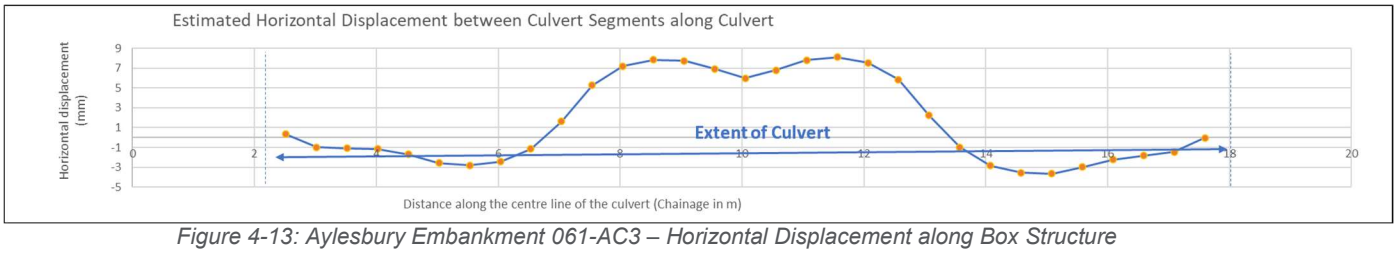


Figure 4-13: Aylesbury Embankment 061-AC3 – Horizontal Displacement along Box Structure

4.9.9 Headwall Design

Stability checks, including sliding, overturning, bearing, and settlement have been conducted for the headwalls of AE Culvert 061-AC3.

4.9.9.1 Headwall Design

Three loading conditions have been adopted for the sliding, overturning, and bearing checks of the culvert headwalls:

- Case 1 - Construction condition with 20kPa variable surcharge, β = 0 degrees
- Case 1a - Construction condition with 10kPa variable surcharge, β = 18 degrees
- Case 2 - In-service condition with 10kPa permanent surcharge, β =18 degrees

Case 1 analysis with 20kPa variable surcharge on level ground has been considered in view of heavy construction plant (20kPa variable surcharge) being unable to operate on significantly sloping ground.

Case 1a analysis has considered site conditions during slope construction, assuming lighter plant operating (10kPa variable surcharge) on the permanent works slope profile.

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Case 2 represents the in-service condition for the culvert, with nominal surcharge loading (10kPa permanent surcharge) being considered.

4.9.9.2 Sliding Check

Without excavate and replace beneath the headwall, degree of utilisation for sliding for the headwall in Cases 1 on the natural ground is above 100%. Hence to prevent occurrence of limit state, 0.8m of excavate and replace has been proposed. A summary of the headwall sliding check results are presented in Table 4-90 below.

Table 4-90: Headwalls Sliding Check for AE Culvert 061-AC3

Headwall Sliding Check	Utilisation (%)	Utilisation (%)		
	Foundation on Natural Ground	Foundation on 800 mm of Class 6N6		
Design Condition	Case 1	Case 1	Case 1a	Case 2
DA1C1	145%	53%	56%	86%
DA1C2	53%	41%	42%	64%
SLS	113%	35%	35%	57%

4.9.9.3 Overturning Check

The degree of utilisation for overturning is less than 100% for the headwalls for Cases 1, 1a and 2. A summary of the headwall overturning check results are presented in Table 4-91 below.

Table 4-91: Headwalls Overturning Check for AE Culvert 061-AC3

Headwall Overturning Check	Utilisation (%)		
	Foundation on Natural Ground/ Granular Fill		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	39	39	39
DA1C2	30	30	29
SLS	25	25	25

4.9.9.4 Bearing Check

The degree of utilisation for the bearing checks is less than 100% for the headwalls for all Cases when founding on the natural ground. A summary of the headwall bearing check results is presented in Table 4-92.

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Table 4-92: Headwalls Bearing Resistance Check for AE Culvert 061-AC3

Headwall Bearing Check	Utilisation (%)		
	Foundation on Natural Ground		
Design Condition	Case 1	Case 1a	Case 2
DA1C1	59	64	49
DA1C2	80	89	69

Notes: No allowance is made for enhancement of bearing resistance from the granular fill.

4.9.10 Ground Aggressivity Assessment

This section presents the Design Sulphate (DS) and Aggressive Chemical Environment for Concrete (ACEC) classifications for the culvert structure, including culvert units, and head walls.

Due to limited GI and chemical testing data available for the specific location of the box culvert, all the latest chemical testing for the Maintenance Access Road to Pond (South Arm off Calley Farm - South) Level 2 Asset Group were considered for this assessment. Ground aggressivity assessment was undertaken based on Building Research Establishment (BRE) Special Digest 1 (SD1) suite [32]. Due to the construction activities of Maintenance Access Road to Pond (South Arm off Calley Farm - South), brownfield conditions were considered for the assessment. It was conservatively assumed that the groundwater in these sites is mobile

The culvert / head walls are not directly in contact with the natural ground but there is a risk that the sulphide ions in the pyrite could oxidise causing a concentration of sulphate ions in the groundwater.

The results of the chemical tests indicate that Kimmeridge Clay can be classified as Design Sulphate class DS-4 and concrete classification AC-4 based on its pyritic characteristics. Therefore, it is recommended that an overall classification of Design Sulphate Class DS-4 and concrete classification of AC-4 be adopted for all new buried concrete associated with this asset. A summary of analysis of chemical testing data is presented in Table 4-93 below.

Table 4-93: Durability Test Results adopted for AE Culvert 061-AC3

Geological Formation	Design Sulphate	Concrete Classification	Total Potential Sulphate (TPS) %	pH	Water Soluble SO_4 (mg/l)
POSA/POST	DS-1	AC-1	0.01	8.0	329
Kimmeridge Clay	DS-4	AC-4	7.32	7.3	889

Notes:

- Characteristic Total Potential Sulphate and Water Soluble SO_4 values were taken based on the guidelines from clause C5. Step 1a of BRE SD1 [32] for brownfield location.
- Design Sulphate (DS) and Aggressive Chemical Environment for Concrete (ACEC) classes were determined based on Table C2 of BRE SD1 [32] for brownfield location.

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4.9.11 Designer Risk Assessment (during construction)

Several CDM risks for AE Culvert 061-AC3 have been identified. As a result, mitigation has been proposed to eliminate and reduce these risks. The residual risks are shown on the construction drawings for the contractor to be addressed during construction. These risks include:

- Instability of temporary excavations,
- Construction vehicles causing damage/instability to culvert and headwalls,
- Working in proximity of existing buried and overhead utilities,
- Shallow groundwater table.

4.9.12 Conclusions and Recommendations

A minimum of 800mm E&R with Class 6N6 material is required beneath the base slab of the box culvert and the culvert headwall to ensure the external stability and settlement/joint rotation/pullout remains within the tolerable limits.

Details of foundation treatment below the headwall are shown on the Aylesbury Embankment 061-AC3 drawings listed in section 4.9.3 and the relevant Appendices to the he relevant Appendices to the IDR Package 207.2 Specification Document number: 1MC06-CEK-AU-SPE-CS04_CL18-000003 [71].

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4.10 Ditch Slab – Aylesbury Embankment Pedestrian Ditch Slab No.1

4.10.1 General Details

This section presents the geotechnical design of Aylesbury Embankment Pedestrian Ditch Slab No.1 (asset ID: HS2-00002YD79) as part of IDR 212.5.

The ditch slab is located offline at the approximate HS2 chainage of 61+325 on the upline side of the Aylesbury Embankment Level 1 Asset Group. The ditch slab is part of the maintenance access strip. It spans over the C2.L.22 Catchment (UAID: HS2-00002CT4W) presented in IDR 207.2.

The geotechnical design is carried out for the shallow foundations of the ditch slab in accordance with the requirements of Eurocode 7, Ref[4], to check that sufficient bearing resistance is available for foundations at each end of the ditch slab. An overturning and sliding check have also been undertaken to confirm the stability of the ditch slab foundation. Settlement calculations were also undertaken for the foundations.

The foundation design of Aylesbury Embankment Pedestrian Ditch Slab No.1 is based on the drawing titled Aylesbury Embankment Pedestrian Ditch Slab No.1 Elevations and Sections, drawing number: 1MC06-CEK-CV-DSE-CS04_CL18-000064.

4.10.2 Construction Sequence

The construction sequence assumes the crossing slabs will be constructed as bottom up in advance of the watercourse/ditch channel earthworks taking place. The construction sequence of the ditch slab includes temporary excavations to the base of the foundation. The excavation is to be taken deeper if the formation is found either in made ground or the natural ground has an undrained shear strength of less than 40kPa. In the event of a deeper excavation, the excavation is to be backfilled with Class 6N6 material to the underside of foundation level.

Class 6N6 material is also required behind each foundation as detailed and presented in the ditch slab drawings (Aylesbury Embankment Pedestrian Ditch Slab No.1 - General Notes, drawing number: 1MC06-CEK-CV-DGA-CS04_CL18-000346 and Aylesbury Embankment Pedestrian Ditch Slab No.1 Elevations and Section, drawing number: 1MC06-CEK-CV-DSE-CS04_CL18-000064).

Details of the backfill requirements are presented in relevant Appendices to the IDR Package 212.5 – Specification (document number: 1MC06-CEK-AU-SPE-C002-000058).

4.10.3 Ground Model and Geotechnical Parameter values

The local GI data within approximately 100m radius of the ditch slab were used to validate the suitability of Aylesbury Embankment ground model for Aylesbury Embankment Pedestrian Ditch Slab No.1. The ground model based on the local GI data is summarised in Table 4-94.

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Table 4-94: Ground Model for Detailed Design of the Aylesbury Embankment Pedestrian Ditch Slab No.1

Stratigraphy	Depth to base (thickness) (m bgl)	Summary Description
Head / Alluvium (HEAD / ALV)	1.9	Soft orangish brown mottled grey sandy gravelly CLAY
Soft Portland Sand / Stone Formation (POST / POSA)	-	Soft orangish brown and grey mottled orange and brown slightly sandy CLAY OR Soft grey slightly sandy SILT

The characteristic parameter values in Appendix B of this GDR, as used for the mainline design of Aylesbury Embankment Level 1 Asset- HS2-00002C5SN, have been considered against the local data and are acceptable for Aylesbury Embankment Pedestrian Ditch Slab No.1 and were therefore adopted for its foundation design. The parameter values are summarised in Table 4-95.

Table 4-95: Characteristic Geotechnical Parameter values of in situ ground for Detailed Design of the Aylesbury Embankment Pedestrian Ditch Slab No.1

Stratigraphy	Unit weight (γ) kN/m³	Cu (kN/m²)	ϕ'		c' (kPa)	Young's modulus (MPa)		Poisson's ratio	
			ϕ'p	ϕ'cv		Young's modulus - Eu (MPa)	Young's modulus- E' (MPa)	υ	υ_u
HEAD / Alluvium (HEAD / ALV)	18.0	20	23	21	0	6	4.8	0.2	0.5
Soft Portland Sand / Stone Formation (POST / POSA)	20.0	20	25	23	0	6	4.8	0.2	0.5

The characteristic parameter values for Class 6N6 and Class 1A4 material used in the design and summarised in Table 4-96 have been adopted from Table 6/1 of the Earthworks (Series 600) Specification (Ref: 1MC06-CEK-GT-REP-C002-000154).

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Table 4-96: Characteristic Geotechnical Parameters of Granular Fill

Stratigraphy	Unit weight (γ) kN/m³	Cu (kN/m²)	ϕ'		c' (kPa)	Young's modulus (MPa)		Poisson's ratio	
			ϕ'_p	ϕ'_{cv}		Young's modulus - Eu (MPa)	Young's modulus – E' (MPa)	v´	v_u
Class 6N6	20	-	40	35	0	-	60	0.2	-
Class 1A4	20	-	35	32	0	-	60	0.2	-

4.10.4 Groundwater

Based on the available groundwater monitoring data, design groundwater is assumed at Finished Ground Level, i.e. at top of the Ditch Slab. This is consistent with other Ditch Slabs assets designed within AA. Additionally, the presence of land drainage and watercourses in the vicinity of the ditch slab supports this design groundwater level. Flood levels were not available at the time of writing this calculation, the adopted groundwater level accounts for flooding that may be associated with the location of the ditch slab.

4.10.5 Ditch Slab Design

4.10.5.1 Overview of the design

The following aspects were checked for the foundations of the ditch slab:

- Bearing capacity was checked below each of the foundations in accordance with BS EN 1997-1:2004 and the UK National Annex. The ULS design load combination has been derived following BS EN 1990:2004 and respective UK National Annex and using characteristic loads provided by the Ancillary Civils team.
- A high-level check for overturning has been undertaken, calculating the destabilising and restoring moments. The degree of utilisation from this has indicated a low utilisation and therefore a detailed overturning check is not required.
- A high-level check for sliding has been undertaken, calculating the destabilising and restoring horizontal forces. The degree of utilisation from this has indicated a low utilisation and therefore a detailed sliding check is not required.
- Settlement calculations were undertaken using an empirical approach suggested by Tomlinson for small foundations on clay.
- Design Sulphate Classification and ACEC class were determined for the foundation in Head / Alluvium deposits in accordance to the BRE Special Digest 1 Ref.[32].
- The design undertakes two cases:
    1. Short term – stability of the foundation from live load (pedestrian loading) and dead load (self-weight of the slab and foundation) using undrained parameter values. EC7 partial factors applied appropriately to action and geotechnical parameter values.
    2. Accidental – stability of the foundation from accidental loads along with dead load from the self-weight of the slab and the foundation using undrained parameter values. No partial factors are applied to the actions and the square root of DA1C2 material partial factors are applied for accidental as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013.

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4.10.5.2 Assumptions

The following assumptions were used in the design:

- The detailed design of ditch slab foundations is based on the geometry and loadings provided by the ASC Ancillary structures team. Any change to the loadings or slab geometry will need to be communicated to the ASC geotechnical design team and will require design review and validation.
- Scour protection is required on the slopes and at the base of the watercourse ditch, adjacent to the ditch slab foundations. This is to prevent erosion of the soil surrounding the foundations of the ditch slab.
- This geotechnical design is only for the permanent and transient condition of the ditch slab foundations. Temporary design for construction is beyond the scope of ASC's work and is not part of this assessment.
- In terms of the reinforced concrete unit weight, a superior unit weight of 25kN/m^3 has been adopted for the bearing capacity checks and an inferior unit weight of 24 kN/m^{3}has been adopted for sliding and overturning checks.
- The partial factors for actions on bridges are used for foundation design of the ditch slab as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013. Reference is made to Set B and Set C partial factors (Tables NA.A2.4(B) and NA.A2.4(C)) of NA to BS EN 1990:2002+A1:2005, Ref.[4].
- The self-weight of the handrails is insignificant as compared to the ditch slab and associated foundations self-weight and therefore, has been ignored in the calculations.
- Based on the available groundwater monitoring data, design groundwater is assumed at the existing ground level. This is consistent with other Level 2 assets designed within AA. Additionally, the presence of land drainage and watercourses in the vicinity of the ditch slab supports this design groundwater level. Flood levels were not available at the time of writing this calculation, the adopted groundwater level accounts for flooding that may be associated with the location of the ditch slab.
- As the ditch slab is dowelled to the foundations, it can transmit shear but not moment. Therefore, braking and traction loads applied to the deck will induce horizontal and moment loadings on the foundations. Braking and traction loads on the deck are split equally as horizontal loads acting on the top of the foundations.
- Active earth pressures are acting on the left-hand side of left foundation and left side of right foundation. Passive resistance is generated on the right side of right foundation. The passive resistance on the right side of left foundation has been conservatively ignored in the design calculations because of the presence of the ditch.
- Large movements are required to mobilise the passive earth resistance and the associated strains may not be compatible with the onset of bearing failure i.e., by the time the limiting passive has mobilised, a bearing failure may well have initiated. As such, a strain-compatibility-limited proportion of the passive resistance has been used for the bearing capacity checks.
- Sliding has been checked for the entire structure, i.e., the two foundations and the ditch slab. However, as there is no moment connection between the slab and foundations, an overturning check on individual foundations is required as all of the components of horizontal force act at a lever arm above the base of the foundations.
- Based on the information provided by the ancillary civils team, the foundations for the ditch slab will be cast in situ. Therefore, the design friction angle, δ_d between the foundation and natural ground is assumed to be equal to design value of the effective critical state angle of shearing resistance, $\phi'_{cv,d}$, for sliding checks.

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4.10.5.3 Design Outcomes

Bearing

Table 4-97: Bearing Outcome for Aylesbury Embankment Pedestrian Ditch Slab No.1

Ground Conditions	Utilisation (%) DA1. C1	Utilisation (%) DA1. C2	Utilisation (%) Accidental
Natural Ground (Head / Alluvium Deposits)	67	67	77

**Partial factors adopted for the accidental case apply a factor of 1.0 to actions and square root the material partial factors prescribed in EC7.*

The results from the bearing calculations indicate that the foundation is stable and does not require any further mitigation.

Sliding and Overturning

The destabilising forces and moments were found to be less than the restoring forces and moments. Therefore, the foundations of Aylesbury Embankment Pedestrian Ditch Slab No.1 are stable in sliding and overturning.

For overturning, the restoring moments from the passive resistance on the right foundation from Class 6N6 fill transmitted to the overturning foundation (left foundation) via the propping action of the slab has been conservatively ignored. For sliding passive resistance from soil on ditch side has also been conservatively ignored.

Settlement

Table 4-98: Settlement Result for Aylesbury Embankment Pedestrian Ditch Slab No.1

Analysis	Settlement (mm)
Maximum Foundation Settlement on Head / Alluvium Deposits	9

The Crossing slab – Approval In Principle (AIP) Ref.[82], informing the design of the ditch slabs does not prescribe a total settlement limit. A differential settlement limit of 25mm has been outlined, however due to the symmetry of the ground conditions and design it is expected that the ditch slab foundations will settle uniformly in the long term. The settlement reported in Table 4-98 is presented for completion of a full foundation design.

Chemical Aggressivity Classification

Analysis of chemical testing has been undertaken and a classification linked to the HEAD/ Alluvium deposits is presented in Table 4-99 below.

Due to the lack of Level 2 asset specific testing for the ditch slab, results available from the entire Level 1 asset group were used to determine the potential for attack on buried concrete. Mobile water conditions were assumed for the assessment Ref.[32].

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Table 4-99: Summary of Durability Test Results Adopted for Aylesbury Embankment Pedestrian Ditch Slab No.1 Location

Geological Formation	Design Sulphate (DSE)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) %	Water Soluble SO_4 (mg/l)
Head / Alluvium Deposits	DS-4	AC-4	5.5	1246

Notes:

1.Characteristic Total Potential Sulphate and Water Soluble SO_4 values were taken as the highest values in the range of results.

2. Design Sulphate (DS) and Concrete (ACEC) classes were determined based on Table C2 of BRE SD1 Ref.[32] for natural locations containing pyrite.

4.10.6 **Conclusions and Recommendations**

Based on the stability checks, a 1m wide x 1m deep foundation with a length of 2.4m founding on the existing natural ground, i.e., Alluvium Deposits provides adequate bearing capacity for the proposed ditch slab foundations, according to the EC7 design stability checks.

The contractor needs to ensure that the founding material (natural ground) has a minimum undrained shear strength (cu ≥ 40kPa) prior to construction of the ditch slab foundations. The contractor should confirm the stratigraphy and verify by hand shear vane testing in cohesive formations that the undrained shear strength at the base of the excavation, (cu ≥ 40kPa) .

Unsuitable founding material (cu<40 kPa, wet, deleterious soil, contaminated soil, made ground) is to be excavated until cu ≥ 40kPa is achieved. The level should be brought up to design founding level by placing and compacting class 6N6 fill. The Contractor may need to improve the formation in order to ensure adequate compaction of Class 6N6.

The BRE SD1 assessment results in a design sulphate and ACEC classification of DS-4 and AC-4

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4.11 Ditch Slab – Aylesbury Embankment Vehicular Ditch Slab No.1

4.11.1 General Details

This section presents the geotechnical design of Aylesbury Embankment Vehicular Ditch Slab No.1 (asset ID: HS2-00002YD7F) as part of IDR 212.5.

The ditch slab is located offline at the approximate HS2 chainage of 61+528 on the upline side of the Aylesbury Embankment Level 1 Asset Group. The ditch slab is part of maintenance access strip. It spans over the C2.L.26 Catchment (UAID: HS2-00002BVF6) presented in IDR 207.1.

The geotechnical design is carried out for the shallow foundations of the ditch slab in accordance with the requirements of Eurocode 7, Ref[4], to check that sufficient bearing resistance is available for foundations at each end of the ditch slab. An overturning and sliding check have also been undertaken to confirm the stability of the ditch slab foundation. Settlement calculations were also undertaken for the foundations.

The foundation design of Aylesbury Embankment Vehicular Ditch Slab No.1 is based on the drawing titled Aylesbury Embankment Vehicular Ditch Slab No.1 Elevations and Sections, drawing number: 1MC06-CEK-CV-DSE-CS04_CL18-000070.

4.11.2 Construction Sequence

The construction sequence assumes the crossing slabs will be constructed as bottom up in advance of the watercourse/ditch channel earthworks taking place. The Construction sequence of the ditch slab includes temporary excavations to the base of the foundation. The excavation is to be taken deeper if the formation is found either to be made ground or in the natural ground has an undrained shear strength of less than 40kPa. In the event of a deeper excavation, the excavation is to be backfilled with Class 6N6 material to the underside of foundation level.

Class 6N6 material is also required behind each foundation as detailed and presented in the ditch slab drawings (Aylesbury Embankment Vehicular Ditch Slab No.1 - General Notes, drawing number: 1MC06-CEK-CV-DGA-CS04_CL18-000340 and Aylesbury Embankment Vehicular Ditch Slab No.1 Elevations and Section, drawing number: 1MC06-CEK-CV-DSE-CS04_CL18-000070).

Details of the backfill requirements are presented in relevant Appendices to the IDR Package 212.5 – Specification (document number: 1MC06-CEK-AU-SPE-C002-000058)

4.11.3 Ground Model and Geotechnical Parameters

The local GI data within approximately 100m radius of the ditch slab were used to validate the suitability of Aylesbury Embankment ground model for Aylesbury Embankment Vehicular Ditch Slab No.1. The ground model based on the local GI data is summarised in Table 4-100

Table 4-100: Ground Model for Detailed Design of the Aylesbury Embankment Vehicular Ditch Slab No.1

Stratigraphy	Depth to base (thickness) (m bgl)	Summary Description
Head / Alluvium (HEAD / ALV)	1.9	Soft orangish brown mottled grey sandy gravelly CLAY

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Stratigraphy	Depth to base (thickness) (m bgl)	Summary Description
Soft Portland Sand / Stone Formation (POST / POSA)	-	Soft orangish brown and grey mottled orange and brown slightly sandy CLAY OR Soft grey slightly sandy SILT

The characteristic parameter values in Appendix B of this GDR, as used for the mainline design of Aylesbury Embankment Level 1 Asset- HS2-00002C5SN, have been considered appropriate to Aylesbury Embankment Vehicular Ditch Slab No.1 and were therefore adopted for its foundation design. The parameter values are summarised in Table 4-101.

Table 4-101: Characteristic Geotechnical Parameter values of in situ ground for Detailed Design of the Aylesbury Embankment Vehicular Ditch Slab No.1

Stratigraphy	Unit weight (γ) kN/m³	Cu (kN/m²)	ϕ'		c' (kPa)	Young's modulus (MPa)		Poisson's ratio	
			ϕ'p	ϕ'cv		Young's modulus - Eu (MPa)	Young's modulus- E' (MPa)	υ	υ_u
HEAD / Alluvium (HEAD / ALV)	18.0	20	23	21	0	6	4.8	0.2	0.5
Soft Portland Sand / Stone Formation (POST / POSA)	20.0	20	25	23	0	6	4.8	0.2	0.5

The characteristic parameter values for Class 6N6 and Class 1A4 material used in the design and summarised in Table 4-102 have been adopted from Table 6/1 of the Earthworks (Series 600) Specification (Ref: 1MC06-CEK-GT-REP-C002-000154).

Table 4-102: Characteristic Geotechnical Parameter values of Granular Fill

Stratigraphy	Unit weight (γ) kN/m³	Cu (kN/m²)	ϕ'		c' (kPa)	Young's modulus (MPa)		Poisson's ratio	
			ϕ'_p	ϕ'_{cv}		Young's modulus - Eu (MPa)	Young's modulus – E' (MPa)	v´	v_u
Class 6N6	20	-	40	35	0	-	60	0.2	-
Class 1A4	20	-	35	32	0	-	60	0.2	-

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4.11.4 Groundwater

Based on the available groundwater monitoring data, design groundwater is assumed at Finished Ground Level, i.e. at top of the Ditch Slab. This is consistent with other Ditch Slabs assets designed within AA. Additionally, the presence of land drainage and watercourses in the vicinity of the ditch slab supports this design groundwater level. Flood levels were not available at the time of writing this calculation, the adopted groundwater level accounts for flooding that may be associated with the location of the ditch slab.

4.11.5 Ditch Slab Design

4.11.5.1 Overview of the design

The following aspects were checked for the foundations of the ditch slab:

- Bearing capacity was checked below each of the foundations in accordance with BS EN 1997-1:2004 and the UK National Annex. The ULS design load combination has been derived following BS EN 1990:2004 and respective UK National Annex and using characteristic loads provided by the Ancillary Civils team.
- A high-level check for overturning has been undertaken, by accounting for the destabilising and restoring moments. The degree of utilisation from this has indicated a low utilisation and therefore a detailed overturning check is not required.
- A high-level check for sliding has been undertaken, by accounting for the destabilising and restoring horizontal forces. The degree of utilisation from this has been found to be less than 100% and therefore a detailed sliding check is not required.
- Settlement calculations were undertaken using an empirical approach suggested by Tomlinson for small foundations on clay.
- Design Sulphate Classification and ACEC class were determined for the foundation in Head / Alluvium deposits in accordance with the BRE Special Digest 1 Ref[32].
- The design undertakes to cases:
    1. Short term – stability of the foundation from live load (traffic loading) and dead load (self-weight of the slab and foundation) using undrained parameters. EC7 partial factors applied appropriately to action and geotechnical parameters.
    2. Accidental – stability of the foundation from accidental loads along with dead load from the self-weight of the slab and the foundation using undrained parameters. No partial factors are applied to the actions and the square root of DA1C2 material partial factors are applied for accidental as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013.

4.11.5.2 Assumptions

The following assumptions were used in the design:

- The detailed design of ditch slab foundations is based on the geometry and loadings provided by the ASC Ancillary civils team. Any change to the loadings or slab geometry will need to be communicated to the ASC geotechnical design team and will require design review and validation.
- Scour protection is required on the slopes and at the base of the watercourse ditch, adjacent to the ditch slab foundations. This is to prevent erosion of the soil surrounding the foundations of the ditch slab.
- This geotechnical design is only for the permanent and transient condition of the ditch slab foundations. Temporary design for construction is beyond the scope of ASC's work and is not part of this assessment.

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- In terms of the reinforced concrete unit weight, a superior unit weight of 25kN/m3 has been adopted for the bearing capacity checks and an inferior unit weight of 24kN/m3 has been adopted for sliding and overturning checks.
- The partial factors for actions on bridges are used for foundation design of the ditch slab as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013. Reference is made to Set B and Set C partial factors (Tables NA.A2.4(B) and NA.A2.4(C)) of NA to BS EN 1990:2002+A1:2005, Ref.[4].
- The self-weight of the handrails is insignificant as compared to the ditch slab and associated foundations self-weight and therefore, has been ignored in the calculations.
- Based on the available groundwater monitoring data, design groundwater is assumed at the existing ground level. This is consistent with other Level 2 assets designed within AA. Additionally, the presence of land drainage and watercourses in the vicinity of the ditch slab supports this design groundwater level. Flood levels were not available at the time of writing this calculation, the adopted groundwater level accounts for flooding that may be associated with the location of the ditch slab.
- As the ditch slab is dowelled to the foundations, it can transmit shear but not moment. Therefore, braking and traction loads applied to the deck will induce horizontal and moment loadings on the foundations. Braking and traction loads on the deck are split equally as horizontal loads acting on the top of the foundations.
- Active earth pressures are acting on the left-hand side of left foundation and left side of right foundation. Passive resistance is generated on the right side of right foundation. The passive resistance on the right side of left foundation has been conservatively ignored in the design calculations because of the presence of the ditch.
- Large movements are required to mobilise the passive earth resistance and the associated strains may not be compatible with the onset of bearing failure i.e., by the time the limiting passive has mobilised, a bearing failure may well have initiated. As such, a strain-compatibility-limited proportion of the passive resistance has been used for the bearing capacity checks.
- Sliding has been checked for the entire structure, i.e., the two foundations and the ditch slab. However, as there is no moment connection between the slab and foundations, an overturning check on individual foundations is required as all of the components of horizontal force act at a lever arm above the base of the foundations.
- Based on the information provided by the ancillary structures team, the foundations for the ditch slab will be cast in situ. Therefore, the design friction angle, δ_d between the foundation and natural ground is assumed to be equal to design value of the effective critical state angle of shearing resistance, $\phi'_{cv,d}$, for sliding checks.

4.11.5.3 Design Outcomes

Bearing

Table 4-103: Bearing Outcome for Aylesbury Embankment Vehicular Ditch Slab No.1

Ground Conditions	Utilisation (%) DA1. C1	Utilisation (%) DA1. C2	Utilisation (%) Accidental
Natural Ground (Head / Alluvium Deposits)	147	161	188

**Partial factors adopted for the accidental case apply a factor of 1.0 to actions and square root the material partial factors prescribed in EC7.*

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The results from the bearing calculations indicate that the foundation is not stable on existing ground and that the bearing capacity of the existing natural soil is not sufficient to support the applied loads. Therefore, ground treatment (excavate and replace with granular 6N6 fill) is required to satisfy the bearing capacity checks.

Table 4-104: Bearing Outcome post E&R for Aylesbury Embankment Vehicular Ditch Slab No.1

Ground Conditions	Utilisation (%) DA1. C1	Utilisation (%) DA1. C2	Utilisation (%) Accidental
Engineering Fill (0.5m of Class 6N6 E&R)	26	53	58

**Partial factors adopted for the accidental case apply a factor of 1.0 to actions and square root of the material partial factors prescribed in EC7.*

The results from the bearing calculations on engineering fill indicates that the foundation is stable on 0.5m of Class 6N6 Excavate and Replace.

Sliding and Overturning

The Destabilising forces and moments were found to be less than the restoring forces and moments. Therefore, the foundations of Aylesbury Embankment Vehicular Ditch Slab No.1 are stable in sliding and overturning.

For overturning, the restoring moments from the passive resistance on the right foundation from Class 6N6 fill transmitted to the overturning foundation (left foundation) via the propping action of the slab has been conservatively ignored. For sliding passive resistance from soil on ditch side has also been conservatively ignored.

Settlement

Table 4-105: Settlement Result for Aylesbury Embankment Vehicular Ditch Slab No.1

Analysis	Settlement (mm)
Maximum Foundation Settlement on Head / Alluvium Deposits	19

The Crossing slab – Approval In Principle (AIP) Ref.[82], informing the design of the ditch slabs does not prescribe a total settlement limit. A differential settlement limit of 25mm has been outlined, however due to the symmetry of the ground conditions and design it is expected that the ditch slab foundations will settle uniformly in the long term. The settlement reported in Table 4-105 is presented for completion of a full foundation design. On the basis of the calculated settlement, the differential settlement is expected to be 18mm +/- 25% or 9mm across the Ditch slab width or length.

Chemical Aggressivity Classification

Analysis of chemical testing has been undertaken and a classification linked to the HEAD/ Alluvium deposits is presented in Table 4-106 below.

Due to the lack of Level 2 asset specific testing for the ditch slab, results available from the entire Level 1 asset group were used to determine the potential for attack on buried concrete. Mobile water conditions were assumed for the assessment Ref.[32].

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Table 4-106: Summary of Durability Test Results Adopted for Aylesbury Embankment Vehicular Ditch Slab No.1 Location

Geological Formation	Design Sulphate (DSE)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) %	Water Soluble SO_4 (mg/l)
Head/Alluvium Deposits	DS-4	AC-4	5.5	1246

Notes:

1.Characteristic Total Potential Sulphate and Water Soluble SO_4 values were taken as the highest values in the range of results.

2. Design Sulphate (DS) and Concrete (ACEC) classes were determined based on Table C2 of BRE SD1 Ref.[32] for natural locations containing pyrite.

4.11.6 Conclusions and Recommendations

Based on the stability checks carried out in accordance with the requirements of BS EN 1997-1:2004+2013 - Geotechnical Design: Eurocode 7, a 1m deep, 1m wide foundation on a 0.50m thick bed of Class 6N6 with a minimum peak angle of shearing resistance of 40 degrees is required.

Prior to construction of the ditch slab foundations, the contractor needs to ensure that the founding material (natural ground) below the Class 6N6 fill that forms the 0.50m thick E&R has a minimum undrained shear strength of 40kPa. The contractor should confirm the stratigraphy and verify by hand shear vane testing in cohesive formations that the undrained shear strength at the base of the excavation, (cu ≥ 40kPa) . Unsuitable founding material (cu<40 kPa, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced with additional Class 6N6 fill. The Contractor may need to improve the formation in order to ensure adequate compaction of Class 6N6.

The BRE SD1 assessment results in a design sulphate and ACEC classification of DS-4 and AC-4.

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5 Special Geotechnical Measures

5.1 Mainline

Not Applicable.

5.2 IDR 208.4

The design of the soil reinforced slopes for assets as part of IDR 208.4 is summarised below. Further details are included within Appendix E.1 which includes outputs of analysis as well as a section covering Special Geotechnical Measures in accordance with the requirements of the GDR template for HS2. Details of the properties and construction of the soil reinforced slopes are defined in the IDR 208.4 Specification (Document No. 1MC06-CEK-AU-SPE-CS04_000003).

Table 5-1: Non-Mainline assets with soil reinforcement

HS2 ID	Asset Name	Zone of Soil Reinforcement	
		From	To
HS2-0000BJSK	Maintenance Access Road to Pond (South Arm off Calley Farm – South) LHS	0+000	0+048
		0+500	0+600
	Maintenance Access Road to Pond (South Arm off Calley Farm – South) RHS	0+000	0+048
		0+500	0+600

Notes:
1 – Chainages are relative to the specific asset name

5.2.1 Design approach

Slope stability analyses have been carried out for varying heights of embankments and the varying ground conditions encountered along the asset alignment. The proposed embankment slopes within this section of the GDR have slope gradients of 1V:3H along the whole length of embankment.

The main geotechnical risks associated with the reinforced soil slopes are considered to be:

- Global stability and failure of the slope.
- Internal stability of the reinforced soil section
- Excess settlement of the reinforced soil section and differential settlement between reinforced and non-reinforced slopes.

The above risks have been mitigated within the design in accordance with BS8006-1:2010+A1:2016 following the adoption of the soil parameters and ground models as presented within this GDR.

The internal stability of the reinforced soil slopes has been determined in accordance with BS8006-1:2010+A1:2016 'Code of Practice for strengthened / reinforced soils and other fills'.

The limit state design philosophy for reinforced soil walls and abutments involves increasing soil weight and live loading by the appropriate partial load factors and reducing the soil properties and reinforcement base strength by appropriate partial material factors.

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The partial factors to be used in the design for both internal and external stability are given in Table 21 of BS8006-1:2010+A1:2016.

For the purposes of this GDR, the soil reinforced embankments are considered as Category 2 (in accordance with Table 9 'Category of Structure Depending Upon Ramification of Failure' within BS8006-1:2010+A1:2016) and thus applied with a partial factor of 1.0. The geogrid slope is to be designed following guidance within the Standard Earthwork Drawing (SED) 1MC06-CEK-GT-DSE-C002-000111.

5.2.2 Design Requirements

The following design requirements are applicable:

- The strengthened earthworks will be slopes constructed to a maximum design gradient of 1V:3H, soft faced structures with layers of high strength geogrid inserted typically at 1.0m vertical spacing. Geogrid reinforcement will need to be installed in all embankment side slopes constructed to a gradient of 1V:3H, greater than 1.5m in height from ch.0+000 to 0+500 and greater than 0.5m in height from ch.0+500 to 0+600.
- The proposed embankments are be constructed as a core of Class 2A4 materials, with the embankment side slopes constructed of a Class 7C3 material. Both of these materials are assumed to be recovered Kimmeridge Clay sourced from a separate location within C23. The minimum shear strength properties for both materials are ϕ' = 23°, c' = 1kPa. Placement, compaction and testing of the embankment materials is to be done in line with the IDR 208.4 Specification (Document No. 1MC06-CEK-AU-SPE-CS04_000003).
- Where planting requires holes to be made within the geogrid, these will be adequately spaced such that the performance of the geogrid is not compromised. Reference should be made to Drawing No 1MC07-CEK-EV-DDE-C003-000014.
- The reinforcement geogrids shall have the following characteristics:
    - Primary and secondary reinforcement – ULS Design strength according to BS EN ISO 10319 and BS8006-1:2010+A1:2016 of no less than 15kN/m;
    - Basal high strength reinforcement – ULS Design strength according to BS EN ISO 10319 and BS8006-1:2010+A1:2016 of no less than 100kN/m;
    - A creep limited tensile strength over a design life of 120 years;
    - They shall be protected from UV radiation within 24 hours of installation;
    - Account shall be made for installation and chemical damage due to a compacted fill with D50 <10mm and pH range of between 4 and 9;
    - The reinforcement shall have a bond coefficient of at least 0.6 with the embankment fill. The contractor is required to undertake site-specific material and geogrid testing;
    - The ULS design strength should occur at a strain of no more than 10%.
    - The reinforcement geogrids shall be covered by a current and relevant roads and bridges agreement certificate, issued by the British Board of Agreement (BBA);
    - All deliveries of geogrid must carry the accompanying documents which provide the values of all specified parameters. The geogrids must be labelled in accordance with BS EN 10320;
    - All geogrids delivered to site shall be CE marked in accordance with the provisions of BS EN 13251 and other standards appropriate to the application of the product;
    - The installation of the particular geogrid shall be agreed between the manufacturer, designer, and contractor;

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5.2.3 Soil reinforcement design

The following tables detail the layers and lengths of geogrids for each asset.

Table 5-2: Soil reinforcement design for Maintenance Access Road to Pond (South Arm off Calley Farm South)

Road Chainage (m)	Side of embankment	Embankment Height (m)	Reinforcement types	Support Details
0+000 to 0+048	LHS	H > 1.5	Primary + Secondary	For full details see drawings, 1MC06-CEK-GT-DSE-CS04_CL18-000084, 1MC06-CEK-GT-DSE-CS04_CL18-000085 and 1MC06-CEK-GT-DSE-CS04_CL18-000090 to 1MC06-CEK-GT-DSE-CS04_CL18-000093
0+500 to 0+600	LHS	H > 0.5	Basal Layer + Primary + Secondary	
0+000 to 0+048	RHS	H > 1.5	Primary + Secondary	
0+500 to 0+600	RHS	H > 0.5m	Basal Layer + Primary + Secondary	

Notes:

1) Secondary reinforcement is to be provided between the primary reinforcement layers.
2) The design assumes no geogrid will be placed within the top 500mm of the embankment to avoid clashing with utilities, foundations to VRS and signs and highway drainage.

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5.3 IDR 207.2

The design of the soil reinforced slopes for assets as part of IDR 207.2 is summarised below. Further details are included within Appendix E.2 which includes outputs of analysis as well as a section covering Special Geotechnical Measures in accordance with the requirements of the GDR template for HS2. Details of the properties and construction of the soil reinforced slopes is defined in the IDR 207.2 Specification (Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000003) Ref [71].

Table 5-3: Non-Mainline assets with soil reinforcement

HS2 ID	Asset Name	Zone of Soil Reinforcement	
		From	To
HS2-000020RKL	Oat Close SBH/19 Realignment (Bridleway SBH/19 Realignment and Footpath SBH/27 Realignment) LHS	0+085	0+110
		0+000	0+090
	Oat Close SBH/19 Realignment (Bridleway SBH/19 Realignment and Bridleway SBH/19 Link) LHS	0+220	0+240
		0+000	0+080

Notes:

Side of embankment taken as if looking up chainage.

5.3.1 Design approach

Slope stability analyses have been carried out for varying heights of embankments and the varying ground conditions encountered along the asset alignment. The proposed embankment slopes within this section of the GDR have maximum slope gradients of 1V:2.5H along the whole length of embankment.

The main geotechnical risks associated with the reinforced soil slopes are considered to be:

- Global stability and failure of the slope.
- Internal stability of the reinforced soil section
- Excess settlement of the reinforced soil section and differential settlement between reinforced and non-reinforced slopes.

The above risks have been mitigated within the design in accordance with BS8006-1:2010+A1:2016 following the adoption of the soil parameters and ground models as presented within this GDR.

The internal stability of the reinforced soil slopes will be determined in accordance with BS8006-1:2010+A1:2016 'Code of Practice for strengthened / reinforced soils and other fills'.

The limit state design philosophy for reinforced soil walls and abutments involves increasing soil weight and live loading by the appropriate partial load factors and reducing the soil properties and reinforcement base strength by appropriate partial material factors.

The partial factors to be used in the design for both internal and external stability are given in Table 21 of BS8006-1:2010+A1:2016.

For the purposes of this GDR, the soil reinforced embankments are considered as Category 2 (in accordance with Table 9 'Category of Structure Depending Upon Ramification of Failure' within BS8006-1:2010+A1:2016)

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and thus applied with a partial factor of 1.0. The geogrid slope is to be designed following guidance within the Standard Earthwork Drawing (SED) 1MC06-CEK-GT-DSE-C002-000111 Ref [44].

5.3.2 Design Requirements

The following design requirements are applicable:

- The strengthened earthworks will be slopes constructed to a maximum design gradient of 1V:2.5H, soft faced structures with layers of high strength geogrid inserted typically at 1.0m vertical spacing. Geogrid reinforcement is deemed necessary for various slopes along a construction project. On the left-hand side, between Chainages 0+085 and 0+110, and for the right-hand side between Chainages 0+220 and 0+240, geogrid reinforcement is required due to proposed gradients of 1V:3H and earthwork heights ranging from 9.00m to 10.00m and exceeding 3.00m, respectively. Additionally, the left-hand side slope of Footpath SBH/27 realignment (Chainages 0+000 to 0+090) and the right-hand side slope of Bridleway SBH/19 Link (Chainages 0+000 to 0+080) necessitate geogrid reinforcement due to slope gradients ranging from 1V:3H to 1V:3.5H and slope heights surpassing 3.00m.
- The proposed embankments are be constructed as a core of Class 2 materials, with the embankment side slopes constructed of a Class 7C material. Both of these materials are assumed to be recovered Kimmeridge Clay sourced from a separate location within C23. The minimum shear strength properties for both materials is $\phi' = 21°$, $c' = 1kPa$. Placement, compaction and testing of the embankment materials is to be done in line with the IDR 207.2 Specification (Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000003) Ref [71]
- Where planting requires holes to be made within the geogrid, these will be adequately spaced such that the performance of the geogrid is not compromised. Reference should be made to Drawing No 1MC07-CEK-EV-DDE-C003-000014 Ref [46].
- The reinforcement geogrids shall have the following characteristics:
    - Primary and secondary reinforcement – ULS Design strength according to BS EN ISO 10319 and BS8006-1:2010+A1:2016 of no less than 15kN/m;
    - Basal high strength reinforcement – ULS Design strength according to BS EN ISO 10319 and BS8006-1:2010+A1:2016 of no less than 100kN/m;
    - A creep limited tensile strength over a design life of 120 years;
    - They shall be covered from UV radiation within 24 hours of installation;
    - Account shall be made for installation and chemical damage due to a compacted fill with D50 <10mm and pH range of between 4 and 9;
    - The reinforcement shall have a bond coefficient of at least 0.6 with the embankment fill. The contractor is required to undertake site-specific material and geogrid testing;
    - The ULS design strength should occur at a strain of no more than 10%.
    - The reinforcement geogrids shall be covered by a current and relevant roads and bridges agreement certificate, issued by the British Board of Agreement (BBA);
    - All deliveries of geogrid must carry the accompanying documents which provide the values of all specified parameters. The geogrids must be labelled in accordance with BS EN 10320;
    - All geogrids delivered to site shall be CE marked in accordance with the provisions of BS EN 13251 and other standards appropriate to the application of the product;
    - The installation of the particular geogrid shall be agreed between the manufacturer, designer, and contractor;

5.3.3 Soil reinforcement design

The following tables details the layers and lengths of geogrids for each asset.

Table 5-4: Soil reinforcement design for Oat Close SBH/19 Realignment

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Chainage	Side of the Embankment	Embankment Height (m)	Reinforcement types	No. of layers/ Length(m)
0+085 to 0+110	LHS	8.8-10	Primary + Secondary	1 Basal Reinforcement / 21 8 (Primary Reinforcement) / 6 9 (Secondary Reinforcement) / 2
0+000 to 0+090	LHS	4.5-8	Primary + Secondary	7 (Primary Reinforcement) / 7 7 (Secondary Reinforcement) / 2
0+220 to 0+240	RHS	3-3.5	Primary + Secondary	3 (Primary Reinforcement) / 6 3 (Secondary Reinforcement) / 2
0+000 to 0+080	RHS	4-7	Primary + Secondary + Basal	1 Basal Reinforcement / 18 6 (Primary Reinforcement) / 8 7 (Secondary Reinforcement) / 2

Notes:
Secondary reinforcement is to be provided between the primary reinforcement layers.
The design assumes no geogrid will be placed within the top 1 m of the embankment to avoid clashing with utilities, foundations to VRS and signs and highway drainage.

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6 Drainage

The Aylesbury Embankment asset has four drainage catchments associated with it, these belong to two track drainage systems and two land drainage systems:

- Land Drainage C2.L.26 and C2.L.22 Catchment;
- Track Drainage C2.T.21 and C2. T.23.

The 207.1- Aylesbury Embankment IDR covers C2.L.26 and C2.T.23 only. The other two drainage catchments will be discussed in detail under IDR 205.1 - Aylesbury South Cutting.

6.1 Track Drainage C2.T.23

The track drainage catchments are drained using a pre-cast concrete trough drainage system located on the embankment. The drainage networks discharge into a carrier drainage system, where they ultimately discharge to the ground via an infiltration pond.

Track drainage “C2.T.23” is located in Aylesbury Embankment, starts at chainage 60+984 to 61+620, and has a length of 636 m. In general, filter drains are used along the track, except for the portion from km 61+178 to km 61+620 (downline) where there is no special drainage.

There is a central channel along the entire length of the C2.T.23 catchment, and rainwater is discharged from it to the filter drains on the upline side. The highest point of the track is at the 61+120 chainage, 136m after the beginning of the catchment area; therefore, the initial part of the filter drains (both for upline and downline) has a slope opposite to the track.

Track drainage for upline:

From the highest point at km 61+120, the filter drains continue north to km 61+575, where the carrier drain crosses under the tracks and discharges into the ditch, draining into railway drainage pond (C2.T.23-REG-0614). The outlet from the pond is designed into Sedrup watercourse, upstream of the inlet to Sedrup North Culvert.

From km 61+575 to km 61+620, the filter drain is designed with a slope opposite to the track slope, with an outlet to the cross drain.

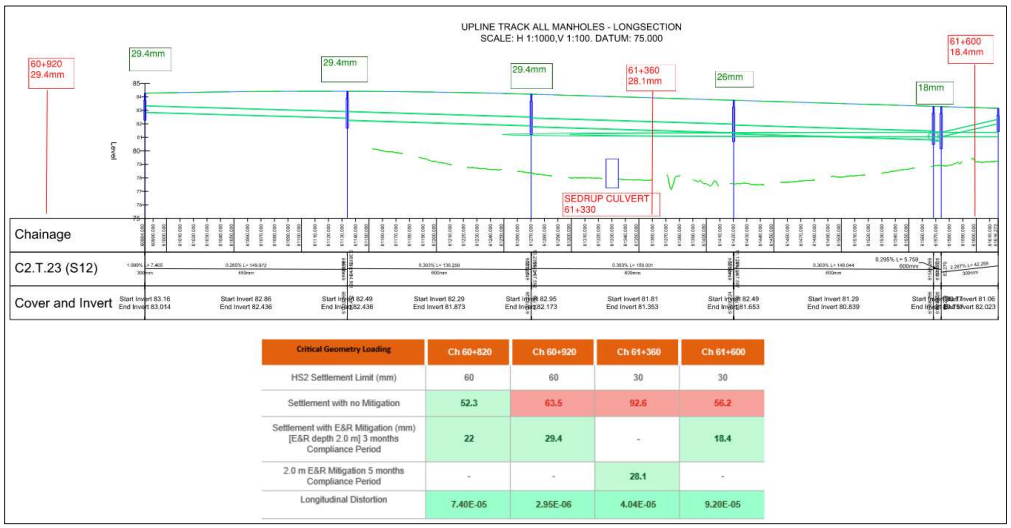
Track drainage for downline:

From km 60+984 to 61+182, there is a filter drain, and at km 61+134, a carrier drain is located under the track and connected to the filter drain on the east.

Based on current settlement values for Aylesbury Embankment shown in Figure 6-1, it was found that settlement values along the entire length of the catchment C2.T.23 are similar in the range between 18 to 30mm.

Drainage slopes along the track are designed for 1:330 - 1:350. Considering maximum settlement between the two chambers, the network levels and slopes along the track may be flattened to a maximum of 1 in 380, and therefore still complying with the minimum slope requirements (1 in 500).

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Critical Geometry Loading	Ch 60+820	Ch 60+920	Ch 61+360	Ch 61+600
HS2 Settlement Limit (mm)	60	60	30	30
Settlement with no Mitigation	52.3	63.5	92.6	56.2
Settlement with E&R Mitigation (mm) [E&R depth 2.0 m] 3 months Compliance Period	22	29.4	-	18.4
2.0 m E&R Mitigation 5 months Compliance Period	-	-	28.1	-
Longitudinal Distortion	7.40E-05	2.95E-06	4.04E-05	9.20E-05

Figure 6-1: Settlement data

The C2.T.23-REG-0614 pond is design at following assumptions:

- Internal pond slope – 1V:6H with 1.35m – 2.25m depth
- External slope of the embankment to the existing ground – 1V:3H with approx. 1.2m depth
- Slope of the embankment to the Access Road – 1V:4H
- Slope of the filled gap between the pond / overflow ditch and the main embankment - 1V:20H
- The C2.T.23-REG-0614 pond was designed as shallow as possible in order to minimize the groundwater interface during construction and post construction. It is also required to maintain the outflow from pond above the Sedrup flood levels. The invert of the pond is 77.80m and the ground levels in the area are 77.80-77.90m.

The stability of the C2.T.23-REG-0614 pond was checked, and the pond design was adapted to the following requirements as shown in Figure 6-2.

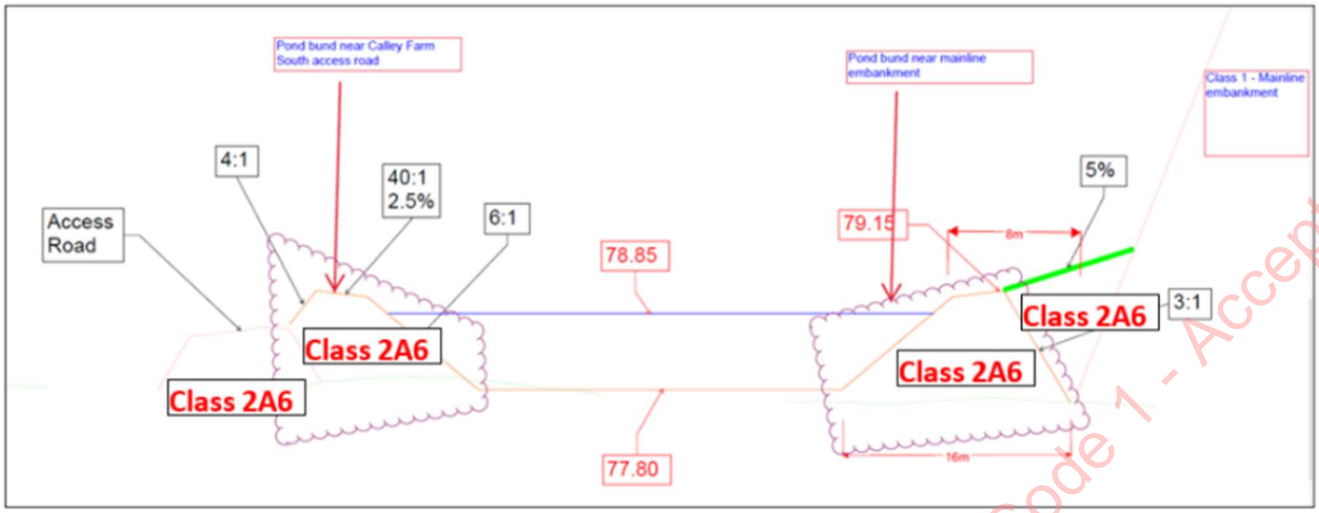


Figure 6-2: Geotechnical requirements for the pond

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The location of pond C2.T.23-REG-0614 is within the alluvium and Portland Sand Formation.

The alluvium and the Portland Sand Formation is designated a Secondary A Aquifer. The pond will be in hydraulic conductivity with groundwater within the alluvium and Portland Sand, therefore, the location is considered hydro-geologically sensitive. The stream located directly south-east of the pond is likely to be in connectivity with the groundwater in the alluvium. To prevent seepage and potential contamination from the pond to the Secondary A aquifers and controlled waters it is recommended that an impermeable liner is constructed across the entire foundation of the pond.

The base of the pond is at approximate ground level which is 77.80m AOD. The maximum flood levels in the adjacent watercourse for the 1 in 1000 year are at a maximum 77.85m AOD, only marginally above the base of the pond.

Maximum long term groundwater level assumed for the design of the access road is 0.5m bgl, which is approximately 77.30m AOD, and is below the proposed base of the pond. In summary, the is a very low risk of uplift to the liner proposed for the pond and no mitigation measures are required to control uplift.

Refer to Aylesbury South Cutting to Oxford Road Embankment: Hydrogeological Appraisal Report (Document number: 1MC06-CEK-EV-REP-CS04_CL18-000022) for further details.

For further details on C2.T.23, reference should be made to the Track Drainage C2.T.23 Catchment – Design and Calculation Report Ref [48].

6.2 Land Drainage C2.L.26

The topography of the Aylesbury Embankment area generally rises towards the southeast, so that the land to the north of the HS2 route gently falls towards it. Surface water run-off from the existing ground and from the proposed side slope of the landscape earthworks discharges into a land cut-off ditch at the toe of the embankment on East side of HS2. The overland flows and the maintenance access (north and South arms of SBH/34) drains to the land ditch on the west side.

For further details on C2.L.26, reference should be made to the Track Drainage C2.L.26 Catchment – Design and Calculation Report Ref [49].

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7 Assessment of Potential Contamination

The contamination risk assessment has been restricted to the HS2 identified Land Quality (LQ) areas only, that were identified in the Environmental Statement Ref [6]

For a full appraisal of the contaminated land assessment, reference should be made to the Supplementary GIR. The Aylesbury Embankment is predominantly located within a rural greenfield area, with agricultural activities evident. Two Land Quality (LQ) constraint areas have been identified within the vicinity of the asset, as follows;

- Hartwell Landfill (LQ 11-04) which is approximately 110m northwest of the LOD of the Embankment (at its nearest point); and,
- Former Hartwell clay, brick/tile works & landfill (LQ 11-03) located 200m north of the LOD of the Embankment.

The Hartwell Landfill Geo-Environmental Assessment Report Ref [50] summarises the available information in relation to contaminated ground and should be referred to for a summary of the existing information with respect to the LQ areas. Based on the distance to the two LQ areas it is unlikely that a plausible contaminant linkage (source-pathway-receptor) exists.

A route wide Made Ground assessment has been completed (Geo-environmental Assessment Report) for Areas of Made Ground Outside Land Quality Areas in C2 and C3 (to be issued – 1MC06-CEK-EV-REP-C002-000225). In summary, six GI locations (ML061-TP438, ML061-TP437A, ML061-WS001, ML061-CT433, ML061-TP435 and ML060-TP427) recorded Made Ground within the Aylesbury Embankment area. The Made Ground encountered typically contained anthropogenic inclusions of rare brick and wood fragments and ranged from 0.15 m to 0.60 m thickness. The presence of Kerogen was noted at one GI location ML061-CR012 at 7.82 mbgl and 13.00 mbgl, however it was identified as a natural occurrence, it was described as potential pockets of kerogen and no lateral migration has been observed.

The potential for site wide ground contamination within the greenfield agricultural area surrounding Aylesbury Embankment is considered to be very low. There remains the potential that isolated areas of unforeseen contamination and/or Made Ground may be present.

There are material management / reuse implications and mitigation measures required for the re-use of natural material and Made Ground. Excavated soils and Made Ground will need to be handled and reused appropriately in accordance with the C2 Design Statement Report for Greenfield Areas [51].

Anthropogenic impacted Made Ground will be segregated and stockpiled separately from the ‘natural’ Made Ground (i.e., contains little to no anthropogenic material), and natural soil, as directed in the Routewide C2 Design Statement Report for Greenfield areas.

All Made Ground material will be tested to identify whether it is chemically suitable for reincorporation within the works, i.e., acceptable, which will include an assessment of geotechnical suitability (including Class U1A or U1B and U2). Unacceptable contaminated earthworks materials shall fall into the Classes U1B and U2 as defined below:

- Unacceptable material Class U1B are contaminated materials excavated within the site, whose level of contamination is above the set reuse criteria noted in the Earthworks Specification Appendix 6/14 and Appendix 6/15 and can be managed by appropriate placement of the materials or treated and processed to meet suitability requirements.
- Unacceptable material Class U2 are contaminated materials excavated from within the site which shall not be used in the permanent works (in terms of chemical use), as they are impacted materials which are unsuitable for remediation or fail acceptance or re-use criteria following treatment; hence may require offsite disposal. Class U2 materials are likely to be radioactive waste (as defined in the Radioactive Substances Act 1993), asbestos unsuitable for incorporation within earthworks materials, heavy end hydrocarbons (compounds >40 carbon atoms) materials unsuitable for remediation or failing to meet the remediation targets.

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Appropriate construction management practices should be adopted during construction works including a watching brief, measures to intercept run-off to prevent contamination of controlled waters, minimise potential favourable conditions that can cause leaching of contamination and dust suppression measures to damp down excavated arisings. Precautionary safety measures are required in relation to the potential ground gas risk to construction workers entering confined spaces.

Intercepted groundwater will be collected and tested prior to discharge. Where this exceeds the relevant screening values, it may require treatment prior to discharge or off-site disposal, in accordance with the appropriate Schedule 33 Environmental Permit.

Unforeseen contamination will be managed in accordance with the EKFB Construction Environmental Management Plan (CEMP) and the C2 Design Statement Report for Greenfield areas.

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8 Ground Movement Impact Assessment

A Damage Risk Assessment of third-party assets is provided in the asset-specific Ground Movement Impact Assessment Report (Doc. No. 1MC06-CEK-GT-REP-CS04_CL18-000042). The assessment was undertaken in accordance with the Ground Engineering Design Basis Statement – Ground Movement Impact Assessment [53].

In summary, one non-contestable High-Pressure gas main (CSO-02-2202) has been identified within the Zone of Influence (ZoI) of the mainline earthwork asset. Refer to the Designer's Monitoring Plan [27] for details on the proposed Instrumentation and Monitoring in this area.

As part of the temporary works during construction, EKFB will have to take appropriate measures to ensure that the buried utility is not affected, and this may require EKFB to carry out monitoring.

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9 Geotechnical Risk Register

The Geotechnical Risk Register (GRR) for IDR 207.2 has been updated to reflect the full or partial mitigations of the geotechnical risks for Aylesbury Embankment undertaken during the detailed design. An extract of the updated GRR along with the geotechnical risk management strategy is provided in Appendix D and presented in

- IDR Package 207.2. – Geotechnical Risk Register Ref.[72].
- IDR Package 207.1. – Geotechnical Risk Register Ref.[54].
- IDR Package 208.4 - Geotechnical Risk Register Ref [35].
- IDR Package 212.5 - Geotechnical Risk Register Ref. [80]

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10 Specification Appendices

The following specifications cover the L2 assets, within the L1 Asset Group Aylesbury Embankment:

- Details of the Series 600 and 1600 Appendices to the Specification for Civil Engineering Works relevant to the scope of the GDR are provided in IDR Package 207.1 – Specification Ref [56].
- Earthworks specification appendices for IDR 208.4 will be provided in the asset specific Series 600 Appendices (Document No. 1MC06-CEK-AU-SPE-CS04-000003) Ref [45].
- Earthworks specification appendices for IDR 207.2 are provided in the asset specific Series 600 Appendices (Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000003) Ref [71]
- Earthworks specification appendices for IDR 212.5 are provided in the asset specific Series 600 Appendices (Document No. 1MC06-CEK-AU-SPE-C002-000058) Ref [77]

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11 Inspection, Testing and Verification Plan

11.1 Mainline Earthworks Control and Verification Requirements

11.1.1 Earthworks Control and Verification Requirements

Earthworks control and verification are included in the following documents:

- HS2 Specification for Civil Engineering Works. Series 0600: Earthworks (Document No. HS2-HS2-CV-SPE-000-010600), Ref. [23].
- Accompanying asset specific Appendix 1/5 and 6/1 to 6/15 for IDR 207.1 are included in Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000002.

11.1.2 Instrumentation and Monitoring

Instrumentation and monitoring details are provided in the following documents:

- Asset specific Designer's Monitoring Plan (Document No. 1MC06-CEK-GT-REP-CS04_CL18-000019), Ref. [27].

In summary, Instrumentation and monitoring has been proposed to manage the following geotechnical risks:

- Differential Settlement due to ground conditions

– 25no Settlement Monuments at 50m intervals and 13No Settlement Plates have been scheduled to monitor settlement along the asset.

11.1.3 Site Testing

Site testing requirements for earthworks are included in the following documents:

- HS2 Specification for Civil Engineering Works. Series 0600: Earthworks (Document No. HS2-HS2-CV-SPE-000-010600), Ref. [23].
- Accompanying asset specific Appendix 1/5 and 6/1 to 6/15 for IDR 207.1 are included in Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000002.

In situ tests will need to be undertaken at the formation level during the construction of the landscape bunds to confirm the minimum undrained shear strength criterion of 50 kPa is achieved and is representative of the underlying material.

11.1.4 Inspections

Requirements for inspections, investigation requirements (including sampling and testing where appropriate) and specific remedial actions are included in the following documents:

- HS2 Specification for Civil Engineering Works. Series 0600: Earthworks (Document No. HS2-HS2-CV-SPE-000-010600), Ref. [23].
- Accompanying asset specific Appendix 1/5 and 6/1 to 6/15 for IDR 207.1 are included in Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000002.

Inspection of the formation for the landscape bunds within the Alluvium extent should be undertaken at the time of construction by a qualified Geotechnical Engineer/Engineering Geologist to identify any soft spots to be removed.

11.2 Non-mainline Earthworks

11.2.1 Earthworks control and verification requirements

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Earthworks control and verification are included in the following documents:

- Accompanying asset specific appendices are included in Document No. 1MC06-CEK-AU-SPE-CS04-000003 for IDR 208.4 Ref [45].
- Accompanying asset specific appendices are included in Document No. 1MC06-CEK-AU-SPE-CS04_CL18-000003 for IDR 207.2 Ref [71].

11.2.2 Site Testing

For ditch slabs foundation specified minimum undrained shear strength of the proposed founding level shall be verified in accordance with the IDR Package 212.5 - Specification (Document No. 1MC06-CEK-AU-SPE-C002-000058)

11.2.3 Instrumentation and Monitoring

Instrumentation and monitoring (I&M) are required for the approach embankments of Oat Close Bridleway (SBH/19) overbridge, according to the typical array for back of highway overbridges (1MC06-CEK-GT-DDE-C002-000038). Instrumentation and monitoring (I&M) for this asset is described in the table below.

No instrumentation or monitoring is required for Maintenance Access Road to Pond (South Arm off Calley Farm South) as part of IDR 208.4

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Table 11-1 – Instrumentation and monitoring Schedule

Array Number	Location	Approximate Chainage (SnakeGrid)	Instrument Type	Total Instrument Length (m)	Quantity of Borehole Instrument	Time of Installation	Comments
1MC06_AEB_LM_60920_A to 1MC06_AEB_LM_60920_C	Oat Close Bridleway (SBH/19) Realignment	Eastern side ML Ch. 60+920 Bridleway SBH/19 Realignment Ch. 0+071 to 0+086	Settlement Monuments (LM)	N/A	3 no. at 0m, 5m and 15m distance from back of abutment	After Construction	Exact locations to be verified depending on construction activities.
1MC06_AEB_LM_60920_D to 1MC06_AEB_LM_60920_F		Western side ML Ch. 60+920 Bridleway SBH/19 Realignment Ch. 0+010 to 0+015 Ch. 0+005 for Footpath SBH/27 Realignment			3 no. at 0m, 5m and 17m distance from back of abutment.		

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11.2.4 Inspection methods and control

Inspection of the formation at time of construction by a qualified Geotechnical Engineer/Engineering Geologist will be required to identify any soft spots to be removed. Identified soft spots will be replaced with Class 1 or 2 material to ensure minimum shear strengths/CBR's as specified in the SCEW.

Inspection for 212.5 ditch slabs shall be carried out in accordance with the requirements of the IDR Package 212.5 Specification 1MC06-CEK-AU-SPE-C002-000058 Ref. [81].

11.2.5 Site testing

Site testing requirements for earthworks is included in the following documents:

- IDR Package 208.4 – Specification – Document No.1MC06-CEK-AU-SPE-CS04-000003 [45]
- IDR Package 207.2 – Specification – Document No.1MC06-CEK-AU-SPE-CS04_CL18-000003 [71]
- IDR Package 212.5 – Specification - Document No.1MC06-CEK-AU-SPE-C002-000058 [81]

In situ tests will need to be undertaken at formation level during construction to confirm the subgrade meets the minimum specified stiffness/undrained shear strength/CBR as per the design requirements.

Pre-construction in situ testing (MEXECone or similar) should be undertaken to confirm the properties of the near surface soils relevant to the design of at-grade pavements, due to the lack of testing data in the upper 2m relevant to pavement design.

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12 References

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[18] OASYS Pdisp Version 20.0 (2019)

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[26] HS2 (2017). Phase One Engineering Geomorphology Assessment Report, Document No. 1D008-EDP-GT-ASM-000-000001, P02

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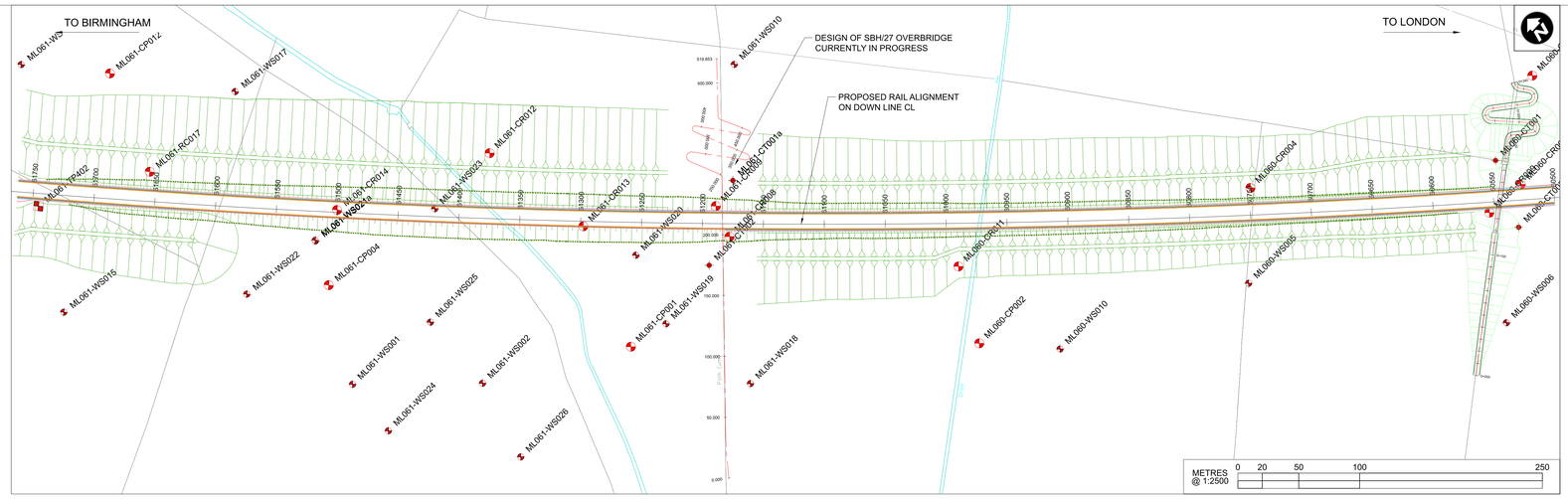
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Appendix A

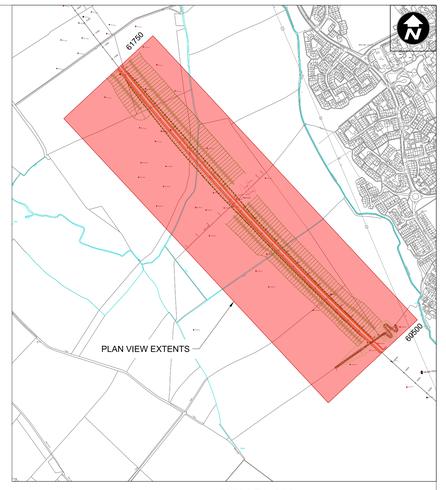
Ground Investigation Plan and Geological Section

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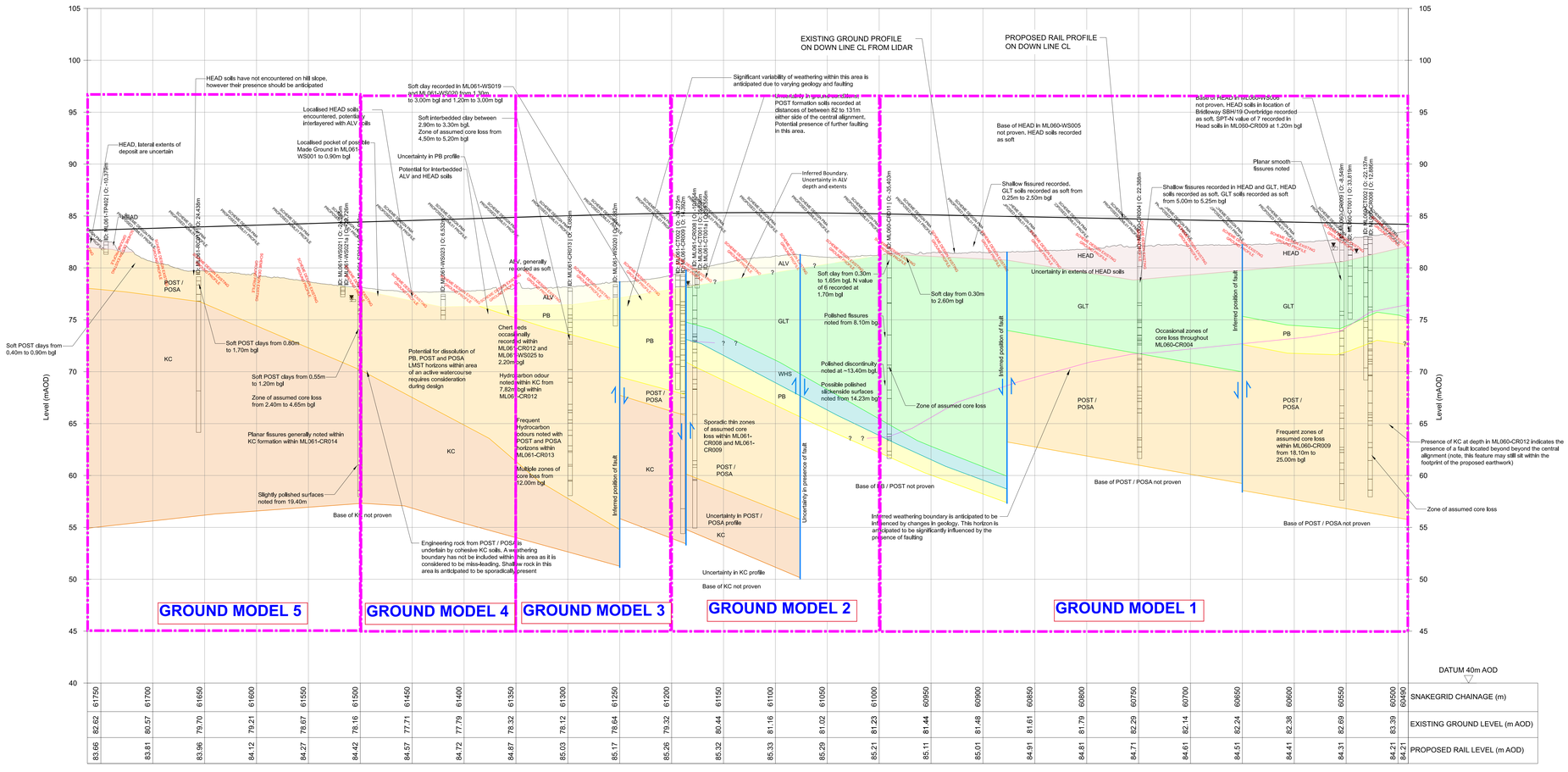
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PLAN
SCALE 1:2500 at A1



KEY PLAN
NOT TO SCALE



GEOLOGICAL LONG SECTION
HORZ 1:2500 & VERT 1:250 at A1

Notes:
1. Do not scale from this drawing.
2. All dimensions are in metres unless indicated otherwise.
3. GI data shown on the long section represents the final status HS2 main GI data as received in May 2018 and ASC supplementary GI data made available at the beginning of the initial ground engineering review during detailed design.Where uncertainties have been identified due to incomplete scope of the supplementary GI, this has been identified within the individual supplementary GIR's and GDR's.
4. Thin surface layers, such as topsoil, have been excluded from the long section. Details of such layers can be viewed on individual exploratory hole logs contained with the associated factual report.
5. The interpreted geological boundaries and inferred weathering boundaries provided on the long section between exploratory holes are indicative only and may be found to vary.
6. Faulting and subsequent displacement of geological units are conjectured, and may be found to vary. Additional faults can be expected.
7. When encountered during drilling, groundwater levels are indicated. However in some instances strikes are likely to have been masked by the use of air-mist or water as a drilling fluid. Groundwater monitoring installations were installed in a number of boreholes. The groundwater data presented represents data made available at the beginning of the initial ground engineering review during detailed design.
8. Existing and proposed ground levels are based on post optimised scheme design levels.
9. Surface LiDAR information is based on a review of Scheme Design BLOM LiDAR (2017) and data received during Detailed Design (1MC12-EKF-GL-SUR-CS03-500013 and 1MC12-EKF-GL-SUR-CS03-500018)

GEOLOGICAL LONG SECTION LEGENDS

STRATIGRAPHY			OTHER SYMBOLS	
	ALV	ALLUVIUM		EXISTING GROUND PROFILE
	HEAD	HEAD		PROPOSED RAIL PROFILE
	GLT	GAULT FORMATION		PROPOSED OVERBRIDGE ALIGNMENT
	WHS	WHITCHURCH SAND FORMATION		POSSIBLE FAULT
	PB	PURBECK GROUP		WATER STRIKE
	POST	PORTLAND SAND FORMATION		WATER LEVEL (20 MINUTE WAIT)
	POSA	PORTLAND STONE FORMATION		
	KC	KIMMERIDGE CLAY FORMATION		

METRES @ 1:250 0 2 5 10 25
Vertical

METRES @ 1:2500 0 20 50 100 250
Horizontal

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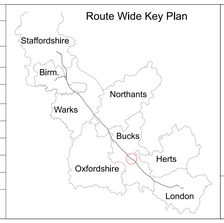
Drawing produced in HS2 SnakeGrid (HS2P1+14) coordinate system

Scale with caution as distortion can occur.

Legends/Notes:

GROUND INVESTIGATION LOCATIONS	GENERAL	DRAINAGE
CABLE PERCUSSION BOREHOLE	FPD DESIGN	CONCRETE DITCH CD
ROTARY CORED BOREHOLE	LIMIT OF DEVIATION(LOD)	EARTH DITCH ED
ROTARY OPEN HOLE BOREHOLE	LIMIT OF LAND TO BE ACQUIRED OR USED(LLAU)	DRAINAGE CHANNEL CH
TRIAL PIT	CUTTING	CUNETTE CUN
OBSERVATION PIT	EMBANKMENT	FILTER DRAIN FDR
CONE PENETRATION TEST	LANDSCAPE BUND	CENTRAL DRAINAGE CHANNEL
WINDOW SAMPLE	RAIL ALIGNMENT ON DOWN LINE CL	UNDER TRACK CROSSING UT
PAVEMENT CORE	NOISE BARRIER	SOLID / SEALED DRAIN CDR
		DRAINAGE POND

Route Wide Key Plan



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Classification
OFFICIAL

Aylesbury Embankment GDR
1MC06-CEK-GT-REP-CS04_CL18-000037 C05
Appendix A Page 1

Appendix B

Summary of Design – Mainline Earthworks Asset Aylesbury Embankment

Number of Pages: 23

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Justification of Characteristic Parameters – Mainline Earthworks

The complete factual assured GI data for Aylesbury Embankment and landscaping was received as part of Data Drop 30.

All data from the Main, Supplementary and the Phase 2 (Detailed Design) GI were included in the updated data plots presented in Figures below. The data were used to derive geotechnical parameters for the in situ material for earthworks Detailed Design.

Characteristic Parameters of In situ Materials:

The characteristic geotechnical parameters of in situ materials have been derived based on the asset specific GI data and supporting route-wide factual GI data available as appropriate. Individual justifications are provided below.

It is important to note that the characteristic geotechnical parameters used for individual design elements other than mainline earthworks design may vary from those stated below. Where parameters adopted for individual design elements vary due to locally variations, justifications and derivations based on the local GI data are provided within the appropriate sections.

Table B-1 Characteristic Geotechnical Parameters of In situ Material

Geological Strata	γ_b (Mg/m^3)	SPT (N)[1]	PI (%)	LL (%)	c_u [13] (kPa)	c_u [14] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	v_u	ν'	E_u (MPa)	E' (MPa)	E_{oed} (MPa)	m_v (m^2/MN)	c_v (m^2/year)
Alluvium	1.8	0-22	48	36	$0 \rightarrow 2.0 = 20$	$0.0 \rightarrow 2.0 = 30$	0-2	21	23	0.5	0.2	6.0	4.8	5.28	0.190	11.05
Head	1.8	$7 + 0.3z$	43	50	$0 \rightarrow 2.0 = 30$ $2.0 += 40$	$2.0 \rightarrow 4.5 = 40$	0-2	21	23	0.5	0.2	$0 \rightarrow 2.0 = 9.0$ $2.0 += 12.0$	$0 \rightarrow 2.0 = 7.2$ $2.0 += 9.6$	$0 \rightarrow 2.0 = 8.0$ $2.0 += 11.0$	$0 \rightarrow 2.0 = 0.125$ $2.0 += 0.090$	5.05
Gault Clay Formation (GLT)	1.9	$6 + 8.6z$	47	70	$50 + 10z \rightarrow 150$	$z > 4.5 = 40 + 8z \rightarrow 150$	8	21	23	0.5	0.2	$15 + 3z \rightarrow 45$	$12 + 2.4z \rightarrow 36$	$13.3 + 2.64z \rightarrow 40$	$\frac{1}{(13.3 + 2.64z \rightarrow 40)}$	1.89
Soft Portland Stone/Sand Formation (POST/POSA)	2.0	$0 - 2$ m $5 + 11z$	35	34	20		0-2	23	25	0.5	0.2	6.0	4.8	5.28	0.189	12.62

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Geological Strata	γ_b (Mg/m^3)	SPT (N)[1]	PI (%)	LL (%)	c_u [13] (kPa)	c_u [14] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	v_u	ν'	E_u (MPa)	E' (MPa)	E_{oed} (MPa)	m_v (m^2/MN)	c_v (m^2/year)
Portland Stone/Sand Formation (POST/POSA)	2.0	$2.0\ m + 9 + 13.5z$	20	34	$50 + 10z \rightarrow 130$		8	26	28	0.5	0.2	$15 + 3z \rightarrow 39$	$12 + 2.4z \rightarrow 31$	$13.3 + 2.64z \rightarrow 34.1$	$\frac{1}{(13.3 + 2.64z \rightarrow 34.1)}$	12.62
Whitchurch Formation (WHS)	2.0	50[2]	20	37	50		10	22	24	0.5	0.2	15	12.0	13.20	0.075	10.10
Purbeck Formation (PB)	2.0	$35 - 50$	No data[3]	No data	80		10	22	24	0.5	0.2	24	19.2	21.12	0.047	0.01[4]
Kimmeridge Clay Formation (KC)	2.0	$8 + 3z$	25	44	$50 + 10z \rightarrow 150$		10	33	35	0.5	0.2	$15 + 3z \rightarrow 45$	$12 + 2.4z \rightarrow 36$	$13.3 + 2.64z \rightarrow 40$	$\frac{1}{(13.3 + 2.64z \rightarrow 40)}$	6.94

Notes and Justifications:

1. z is the depth (m) measured from top of the individual stratum.
2. Only one SPT result was recorded in the Whitchurch Formation, including route wide data.
3. No plasticity data was available for the Purbeck Group, including route wide data.
4. This nominal value has been selected to conservatively model as much settlement as possible occurring in the earthwork, post construction.
5. A f_1 (correlation factor between C_u and SPT 'N') value of 4.5 has been adopted for all cohesive materials (PI values exceed 40%, after Stroud 1989 [55]).
6. In selecting design values for the effective cohesion, the operational conditions which will affect the materials and the consequence of risk have been taken into consideration. A cautious estimate of the design c' value affecting the occurrence of the limit state has been used, taking into account the applicable laboratory test results and comparable UK experience of long-term safe slope inclinations.
7. Values of φ'_{cv} are derived based on the characteristic PI value of a material with reference to Table 2 of BS8004:2015+A1:2020 Ref [56].
8. $\varphi'_{peak} = \varphi'_{cv} + \varphi'_{dil}$, where φ'_{dil} is taken as 2 degrees.
9. Poisson's Ratio for clay is taken from the typical values of saturated and unsaturated clay soil, with reference to Bowles Ref [57], Table 2-7 Foundation Analysis and Design, 5th Edition.

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10. Elastic Modulus values for the clay materials were derived using E_u/c_u = 300, based on the characteristic PI value with reference to CIRIA Report C103. E' was calculated using the relationship $E'/E_u = (1+v')/(1+v_u)$ outlined in CIRIA Report C143 Ref [58].
11. Constrained modulus (E_{oed}) and subsequent coefficient of volume compressibility (mv) has been calculated (where applicable) using the relationships:

    $$E_{oed} = E' (1- v') / \{(1+ v')(1-2v')\} \text{ and } m_v = 1/E_{oed}$$

12. Coefficient of consolidation (c_v) for each lithology have been based on the selected liquid limit with reference to NAVFAC Soil Mechanics Design Manual 7.01 Ref [59].
13. The value of c_u is adopted from chainage 60+500 to 61+200. Refer to Figure B-14 for the undrained shear strength plot.
14. The low strength value of c_u is adopted from chainage 61+200 to 61+763. Refer to Figure B-15 for the undrained shear strength plot.

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Characteristic Parameters of Fill Materials

The properties of fill materials have been selected with reference to the Definition of Scheme Design – Earthworks Material Re-Use and their characteristic parameters are presented in Table B-2.

Table B-2 Characteristic parameters of fill materials

Fill material	Thickness (m)	γ_b (Mg/m^3)	c_u (kPa)	c' (kPa)	ϕ' (°)	ν'	E' (MPa)	Comments
Upper / Lower Embankment Fill	up to 5m / up to 0.5m	2.0	-	0	35	0.2	60	Class 1A1 selected granular material
		2.1	-	1	35	0.2	60	Class 3A1 and Class 3A3 chalk material
		2.1	-	5	35	0.2	60	Class 9M1 stabilised chalk material
Replacement Fill beneath landscape bunds[1]	To be determined at site	1.9	50	1	23 [3]	0.2	-	Class 2A1
Replacement Fill[2]	up to 2.8m	2.0	-	0	35	0.2	60	Class 1A1 selected granular material
		2.1	-	1	35	0.2	60	Class 3A3 chalk material
Landscape Bunds – Class 2A3		1.9	50	1	23 [3]	-	-	Landscape Earthworks Fill Zone 1
Landscape Bunds – Class 2A4		1.9	50	1	23 [3]	-	-	Landscape Earthworks Fill Zone 2
Landscape Bunds – Class 4A1		1.9	40	1	23 [3]	-	-	Landscape Earthworks Fill Zone 3
Foundation material to culverts	2.15m	2.0	-	0	40	0.2	60	Class 1A2 Assumed parameters for the selected granular material.
Enhanced Upper Embankment Fill (UEF*)	Various	-	-	-	-	-	120	Class 6N12 Backfill and cover material to culvert structures in the mainline earthworks' transition zone

Notes and Justifications:

1. *Excavate and Replace within the Alluvium extents (Ch.61+200 to 61+763) beneath landscape bunds.*
2. *Excavate and Replace Fill to mainline earthworks only.*
3. *A peak angle of shearing resistance of 23 degrees is used for material Classes 2A3, 2A4 and 4A1. The material parameters are in-line with field tests undertaken across AA. The angle of shearing resistance has been reduced by 2 degrees for modelling the fill zones as Vadose zone.*

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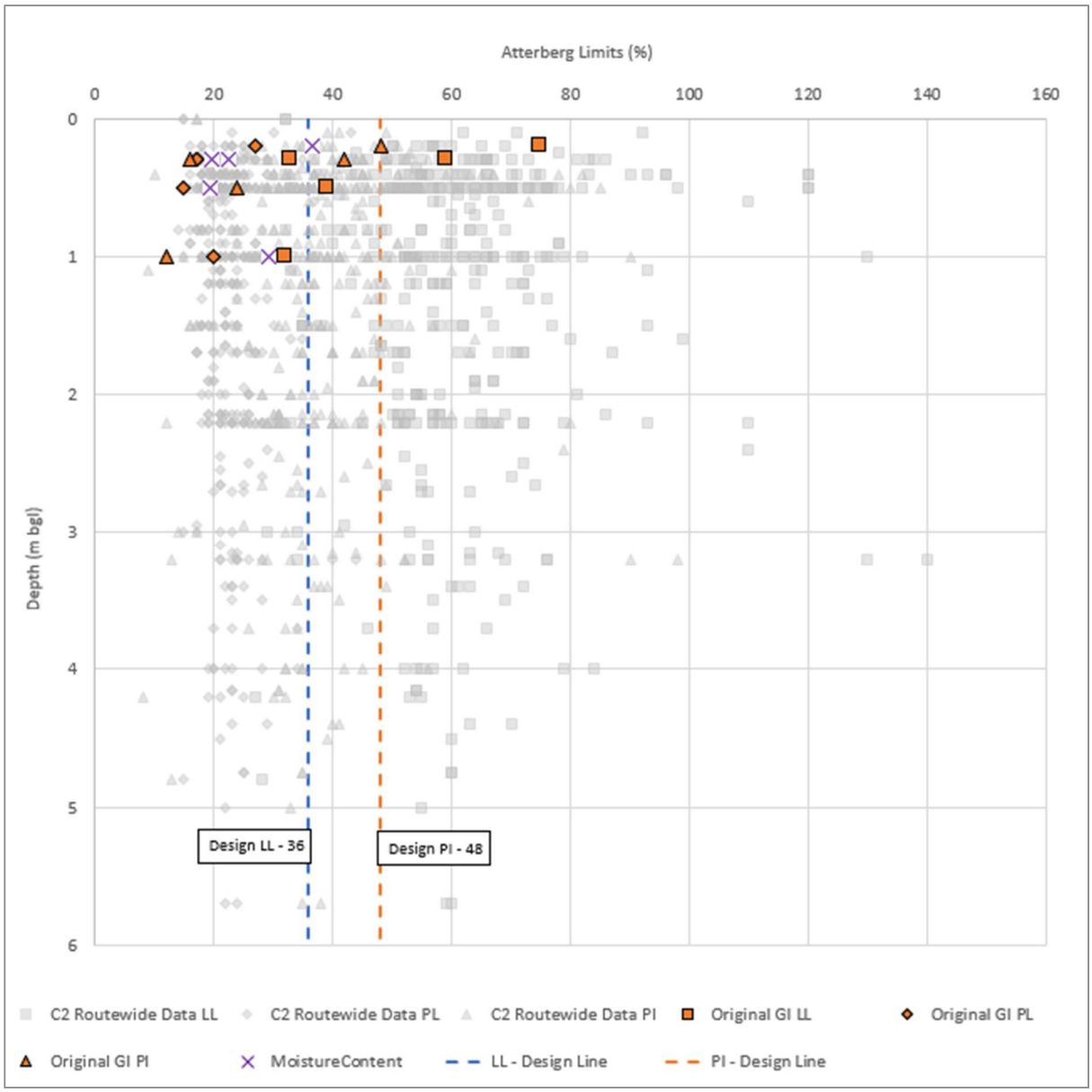


Figure B-1 - Plasticity Index against Depth for Alluvium

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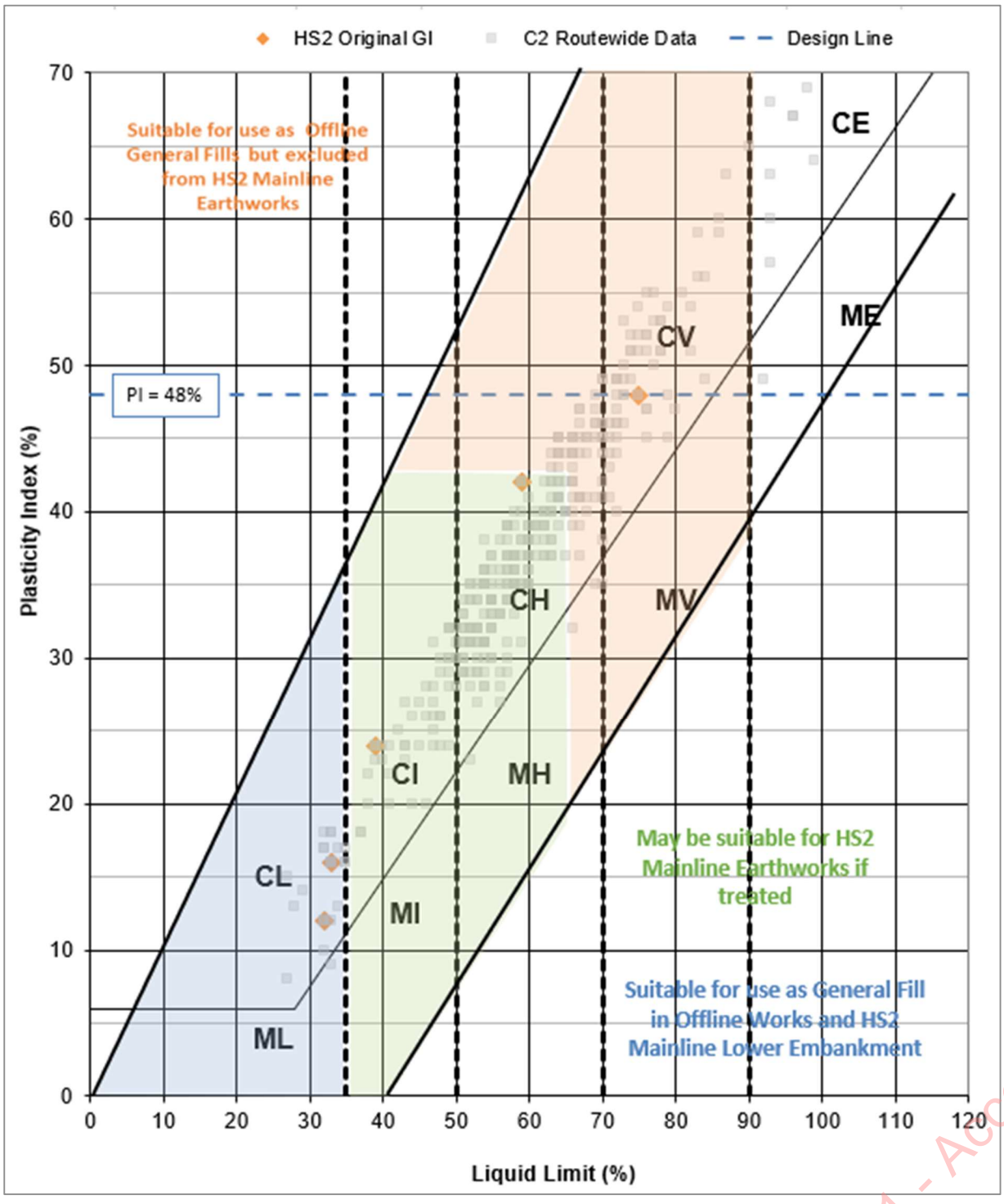


Figure B-2 - A-Line Plot for Alluvium

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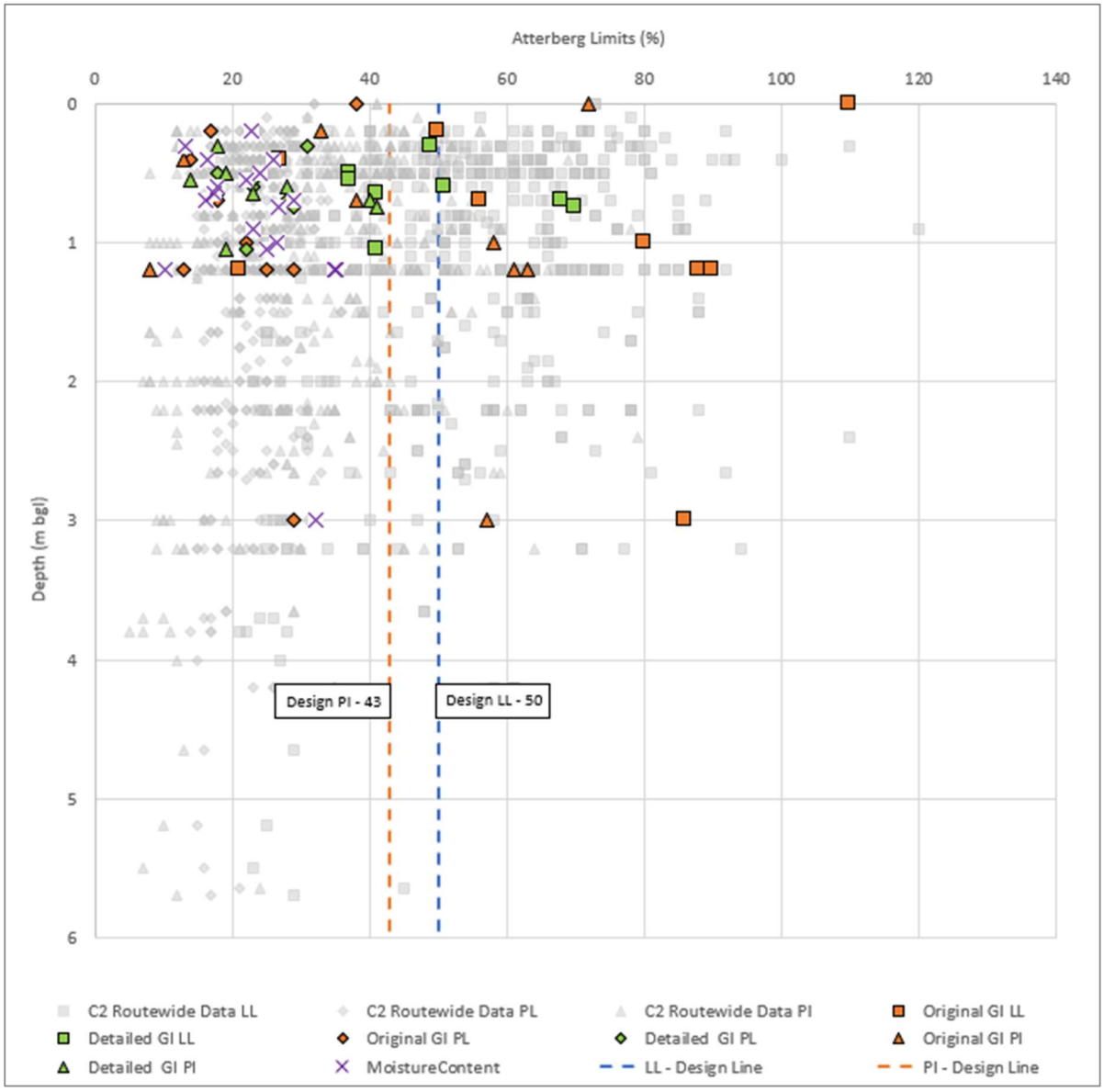


Figure B-3 - Plasticity Index Against Depth for Head Deposits

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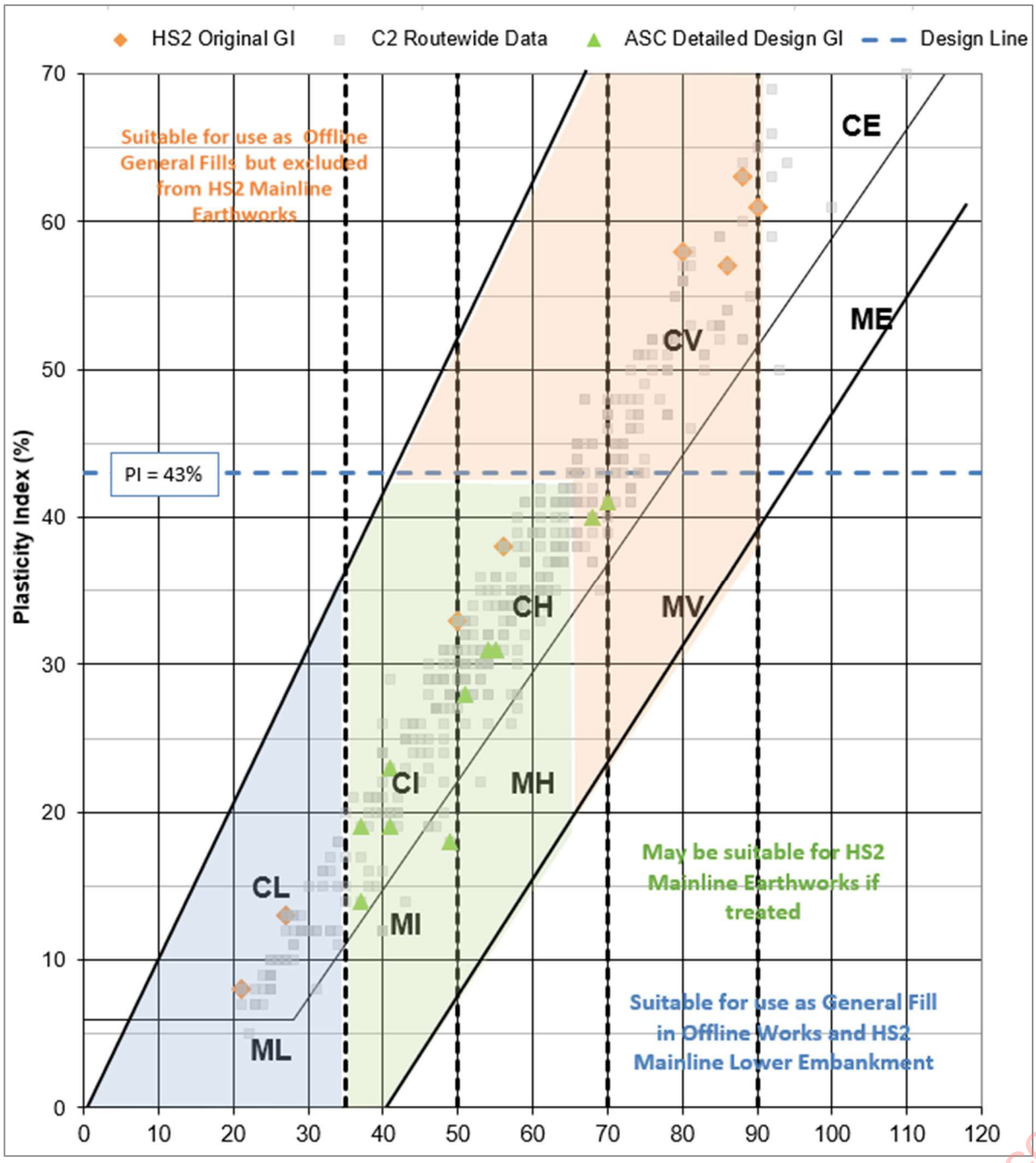


Figure B-4 - A-Line Plot for Head Deposits

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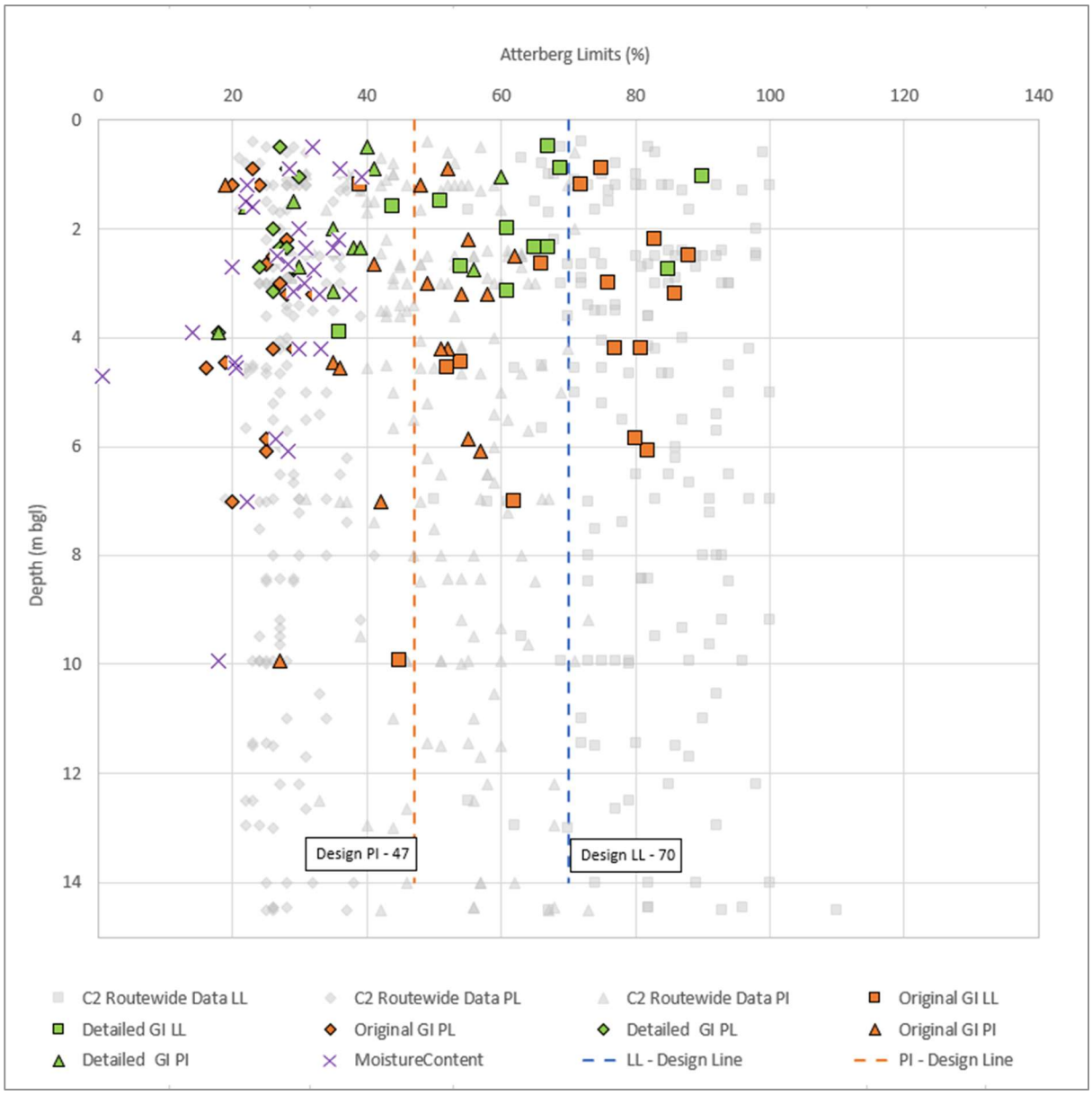


Figure B-5 - Plasticity Index Against Depth for Gault Clay Formation

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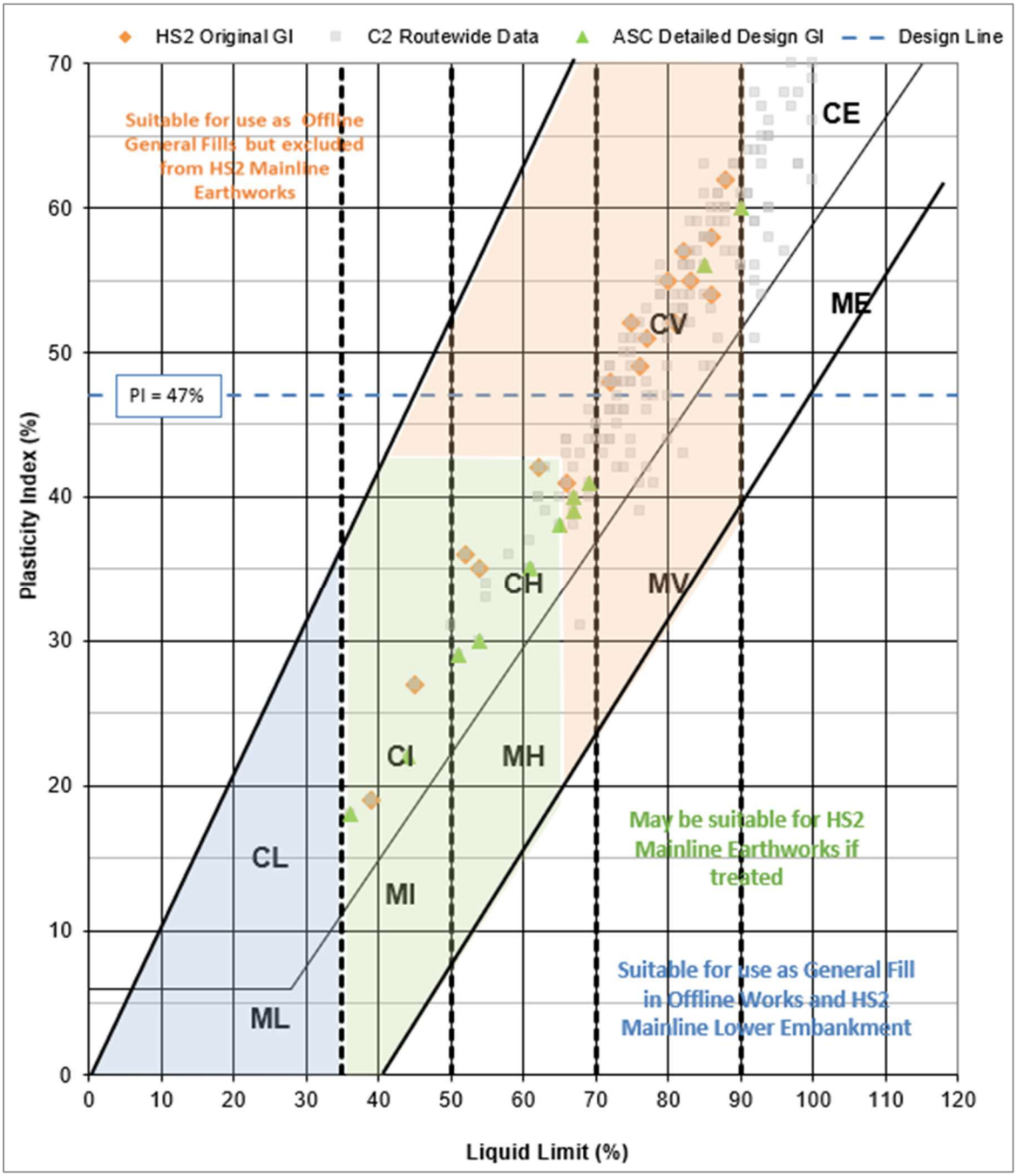


Figure B-6 - A-Line Plot for Gault Clay Formation

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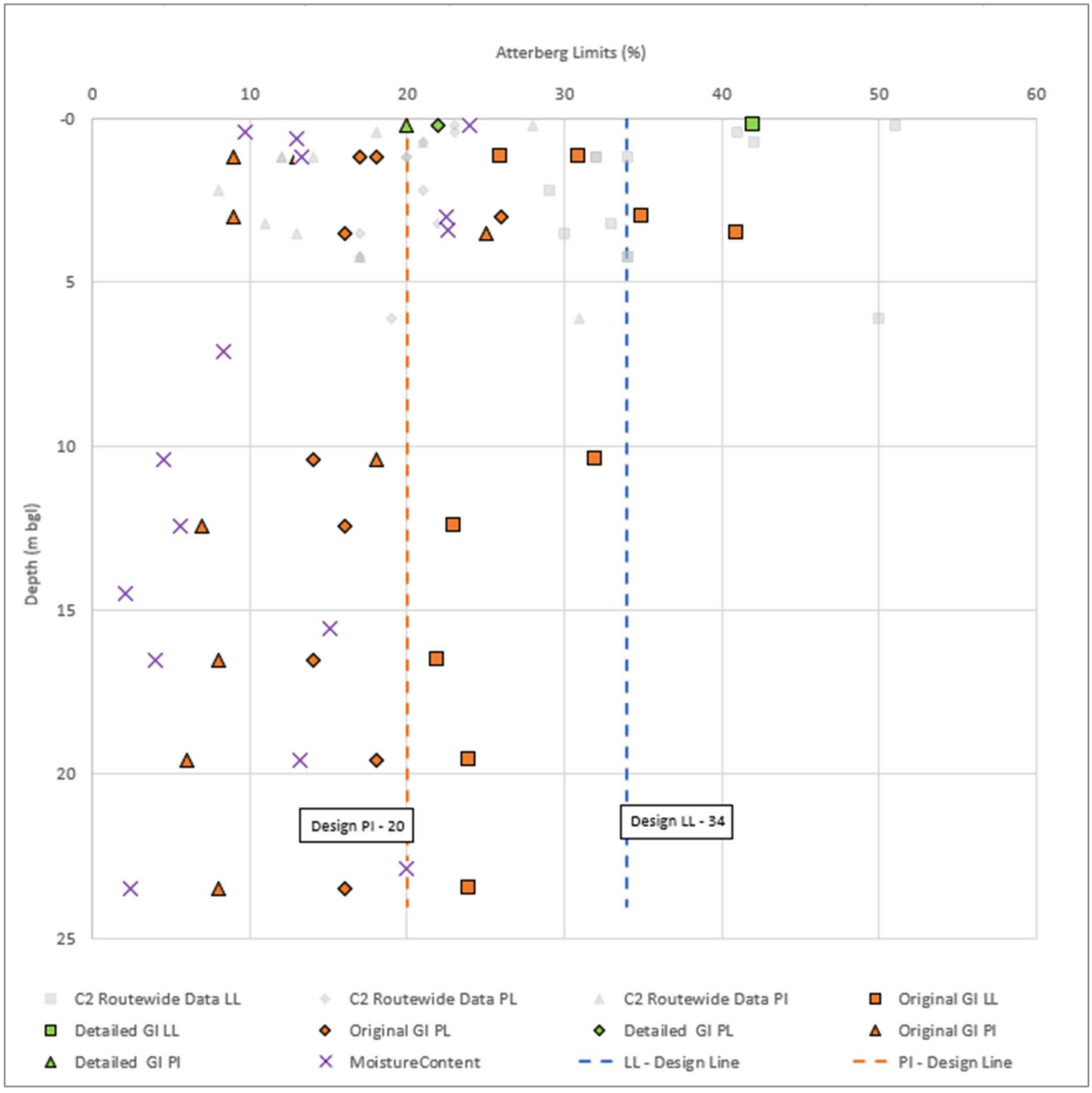


Figure B-7 - Plasticity Index against Depth for Portland Stone/Sand Formation

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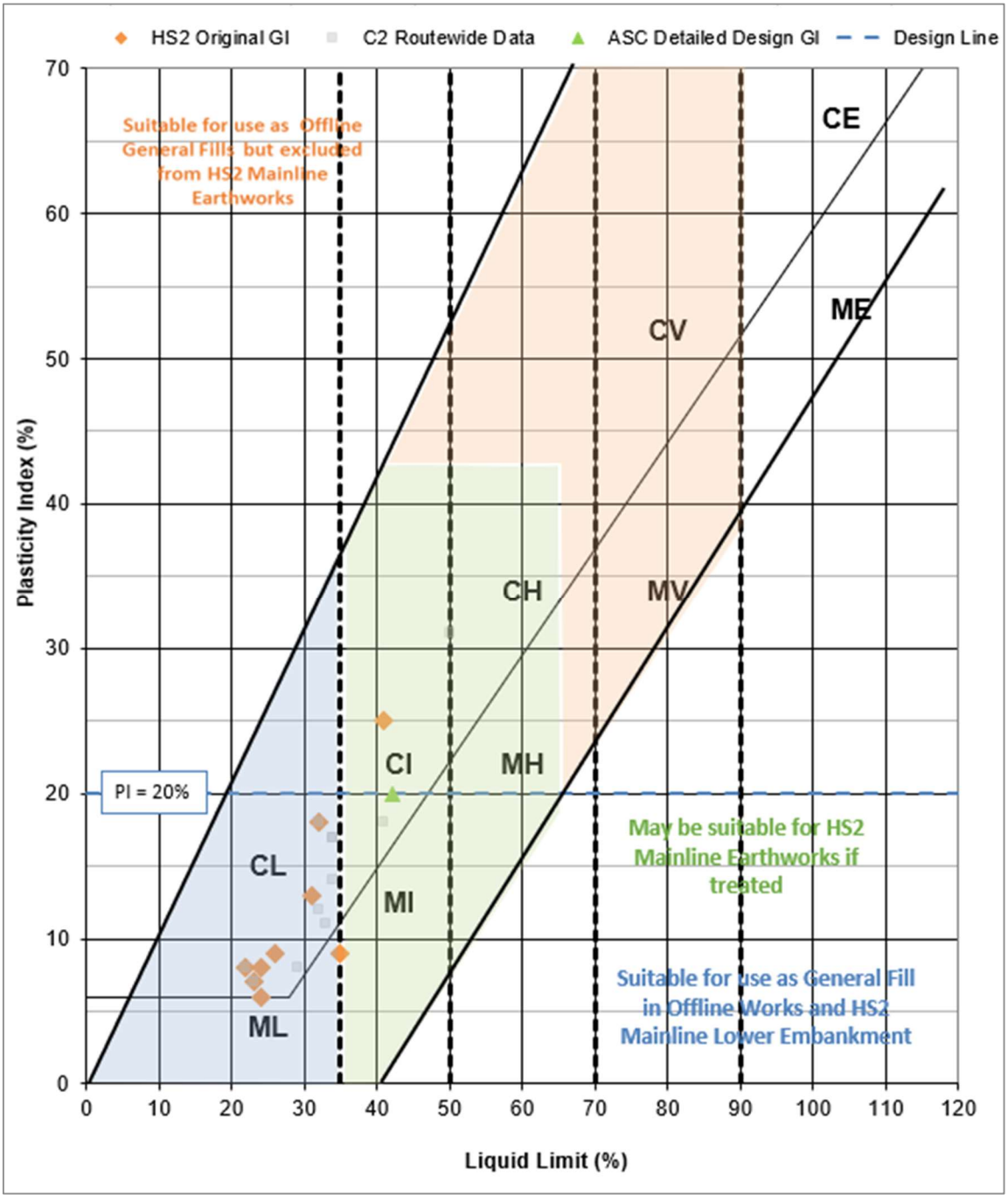


Figure B-8 - A-Line Plot for Portland Stone/Sand Formation

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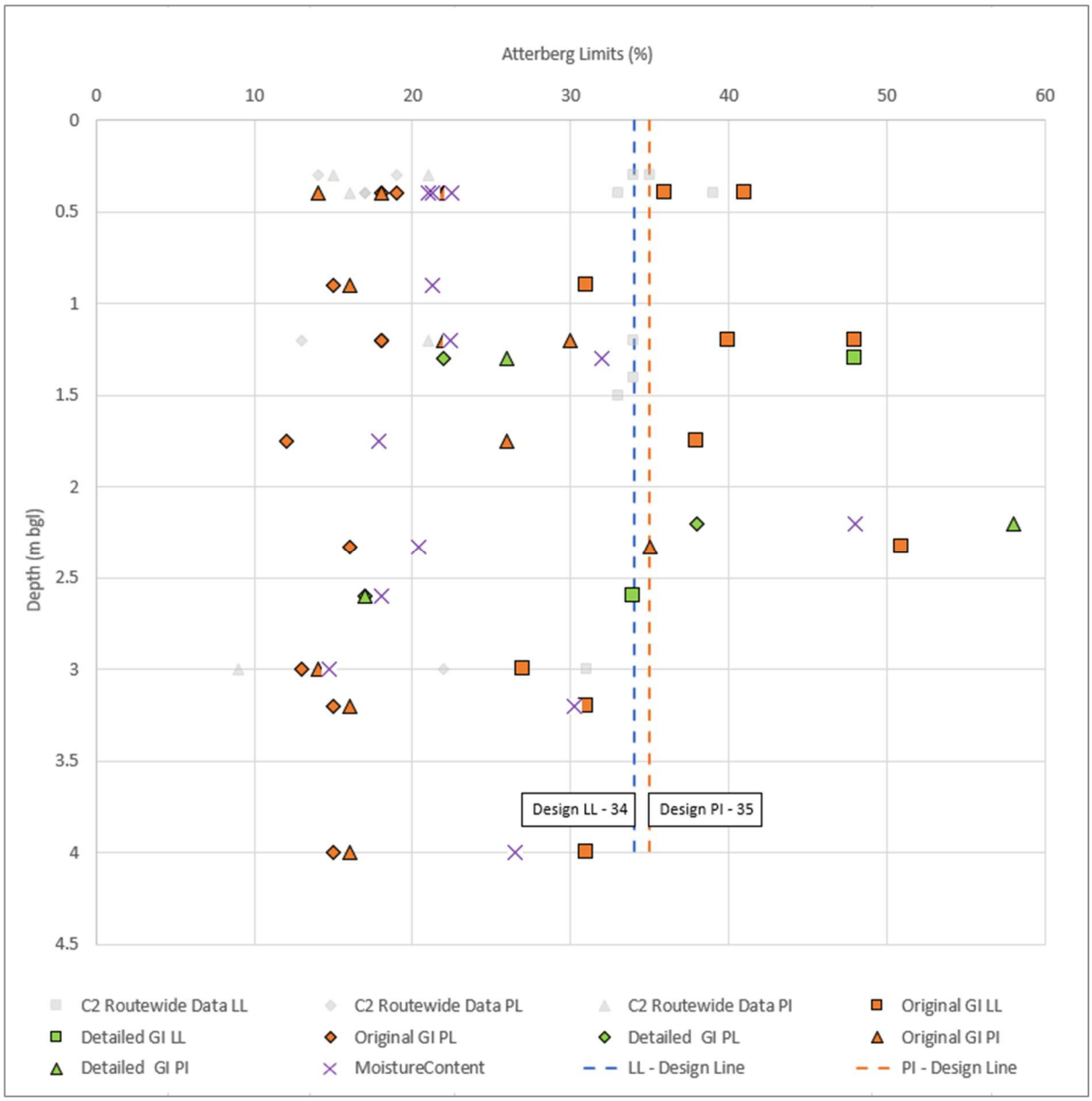


Figure B-9 - Plasticity Index against Depth for Portland Stone/Sand Formation (Soft Soils)

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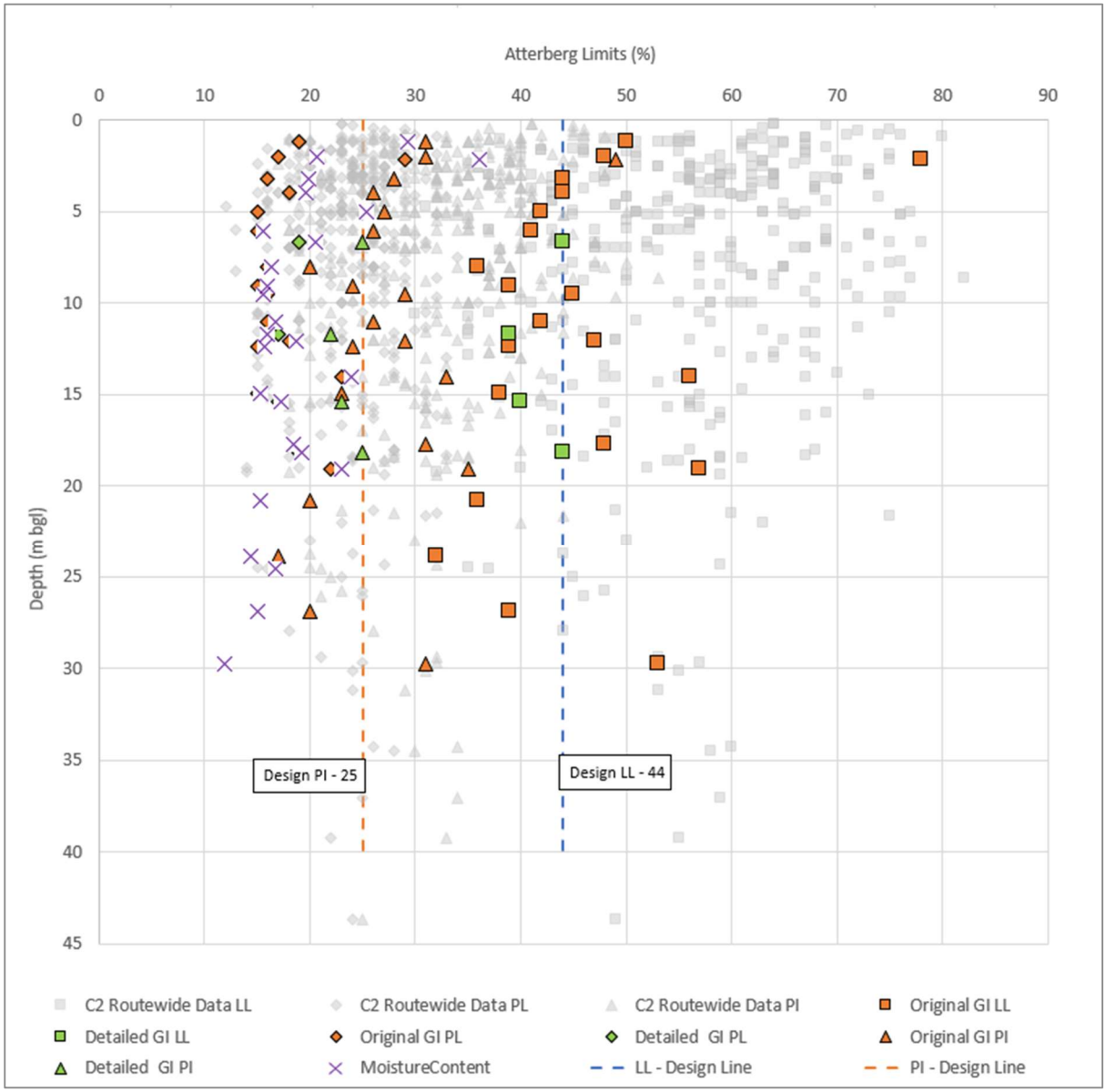


Figure B-10 -Combined Plasticity Against Depth for Kimmeridge Clay Formation and Mudstone

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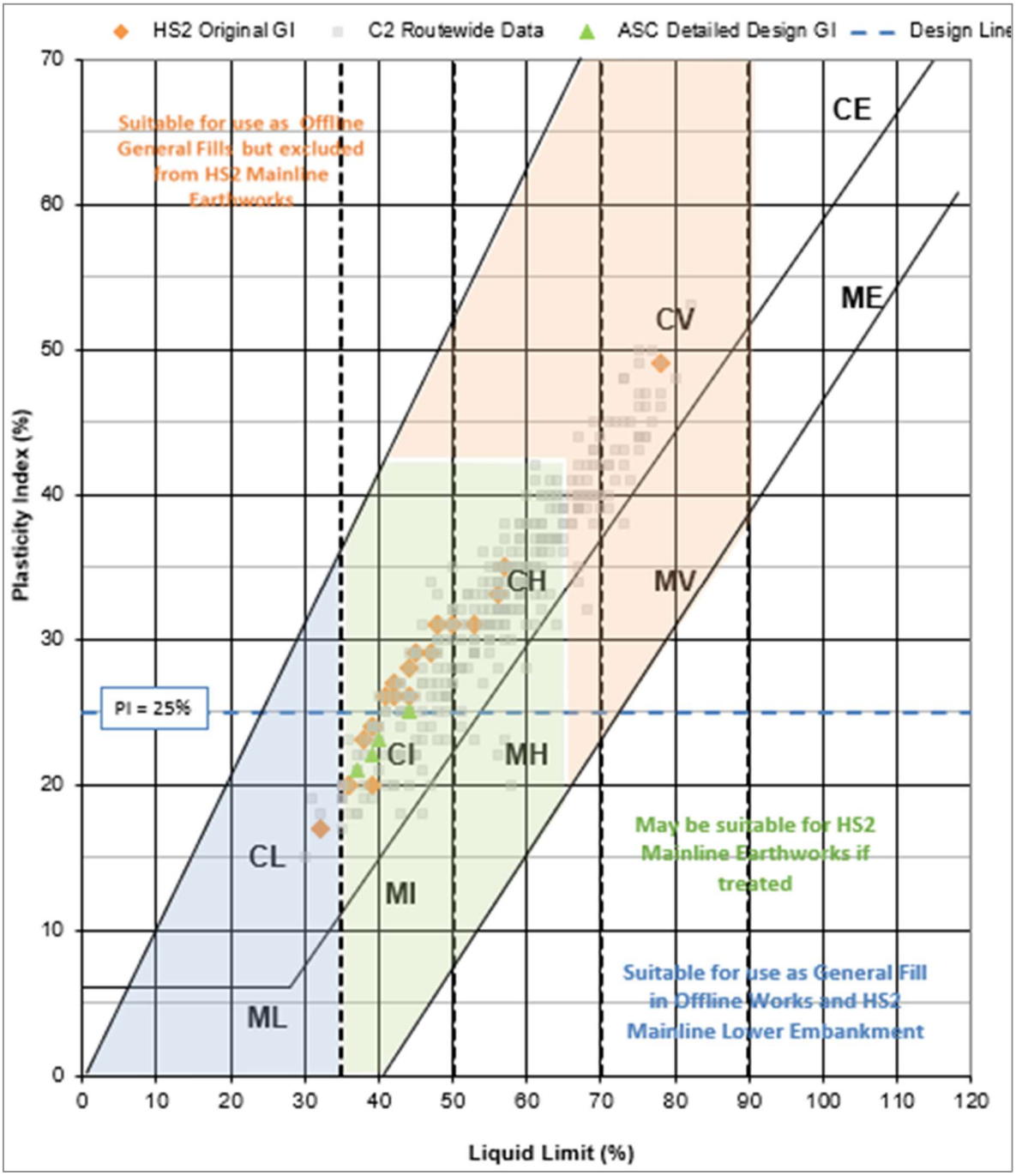


Figure B-11 - Combined A-Line Plot for Kimmeridge Clay Formation and Mudstone

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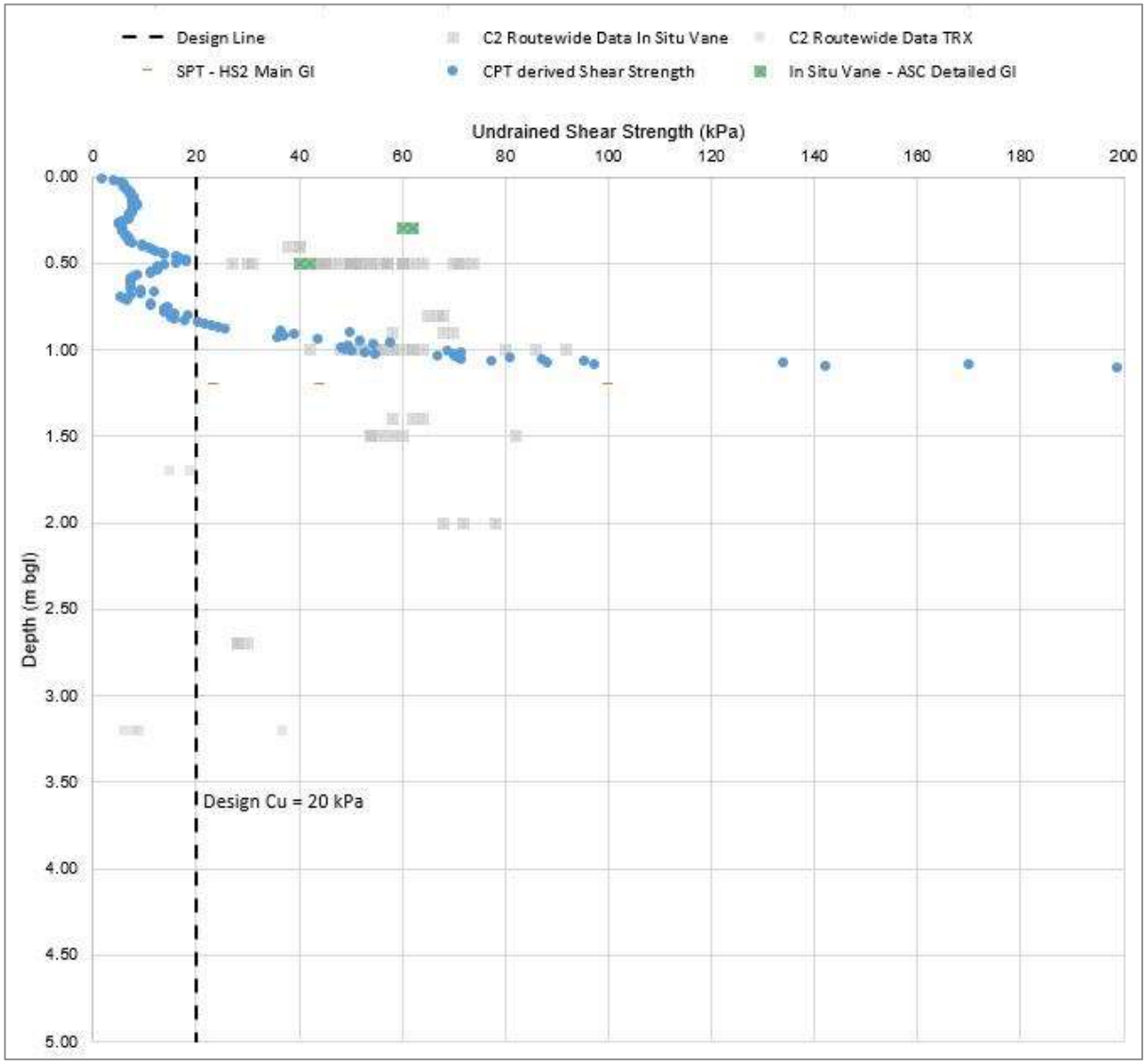


Figure B-12 – Undrained Shear Strength for Alluvium

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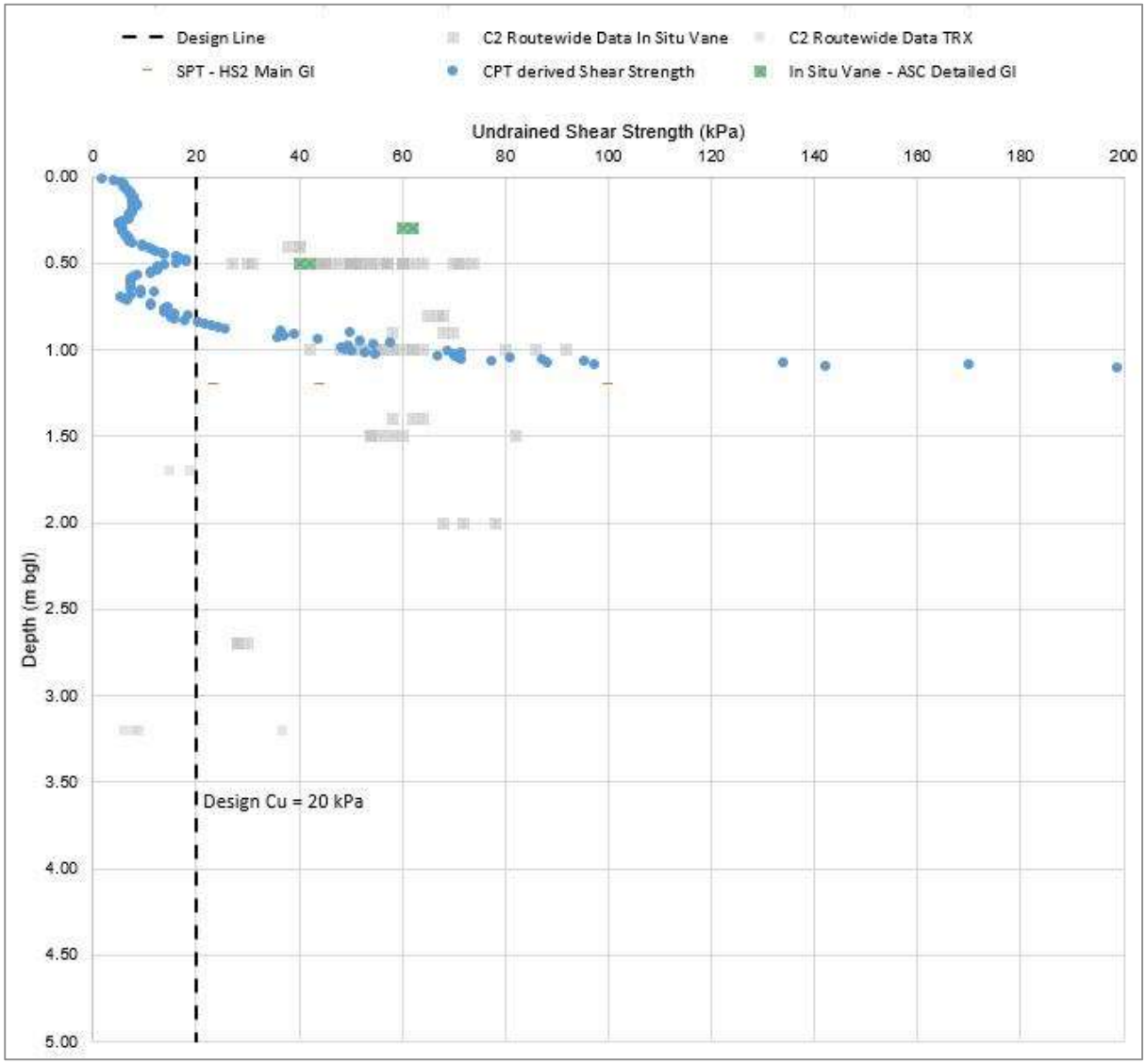


Figure B-13 - Undrained Shear Strength for Head Deposits

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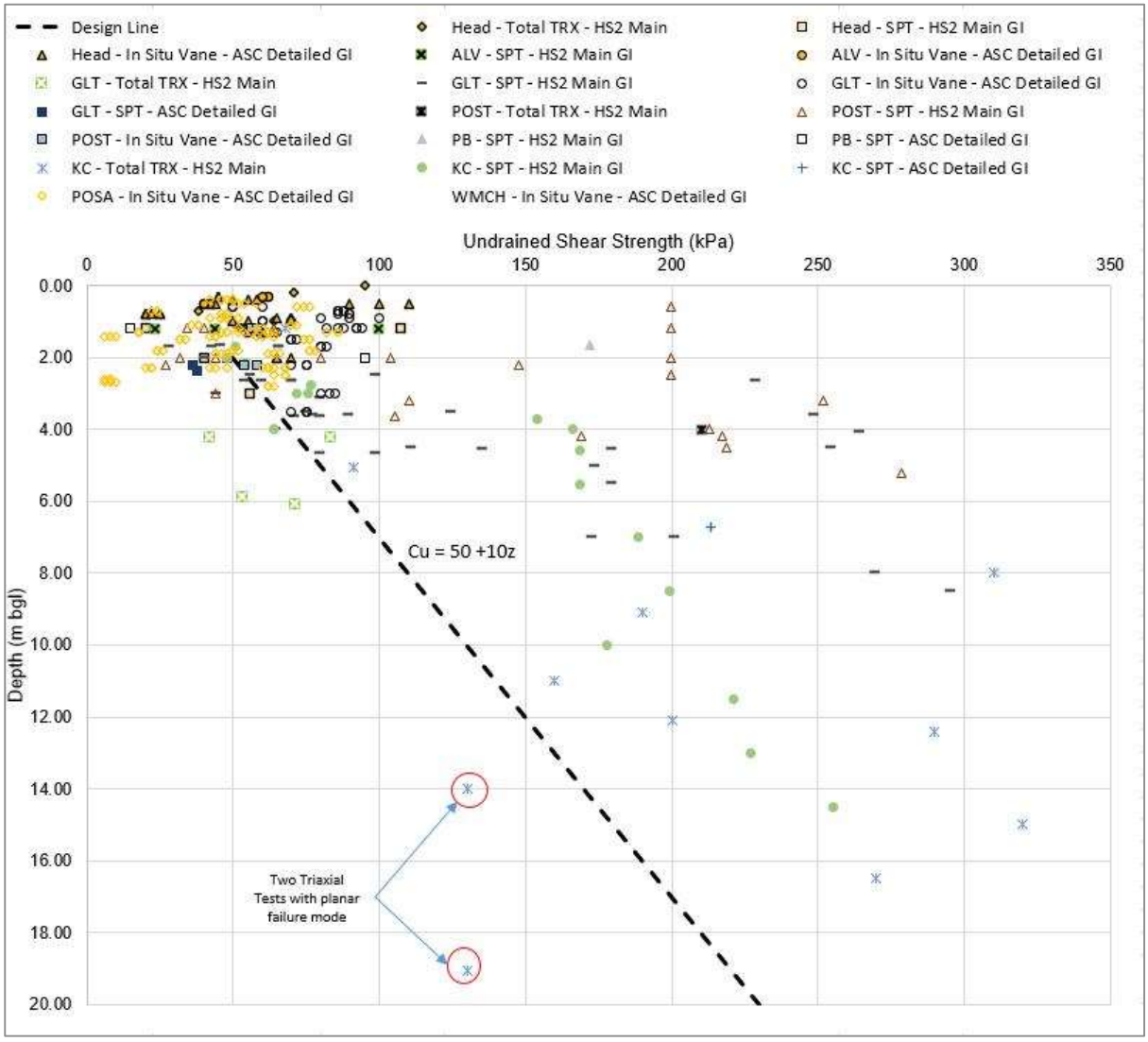


Figure B-14 - Undrained Shear Strength for Kimmeridge Clay, Portland Stone/Sand Formation, Gault Clay Formation from Ch.60+500 to Ch.61+200

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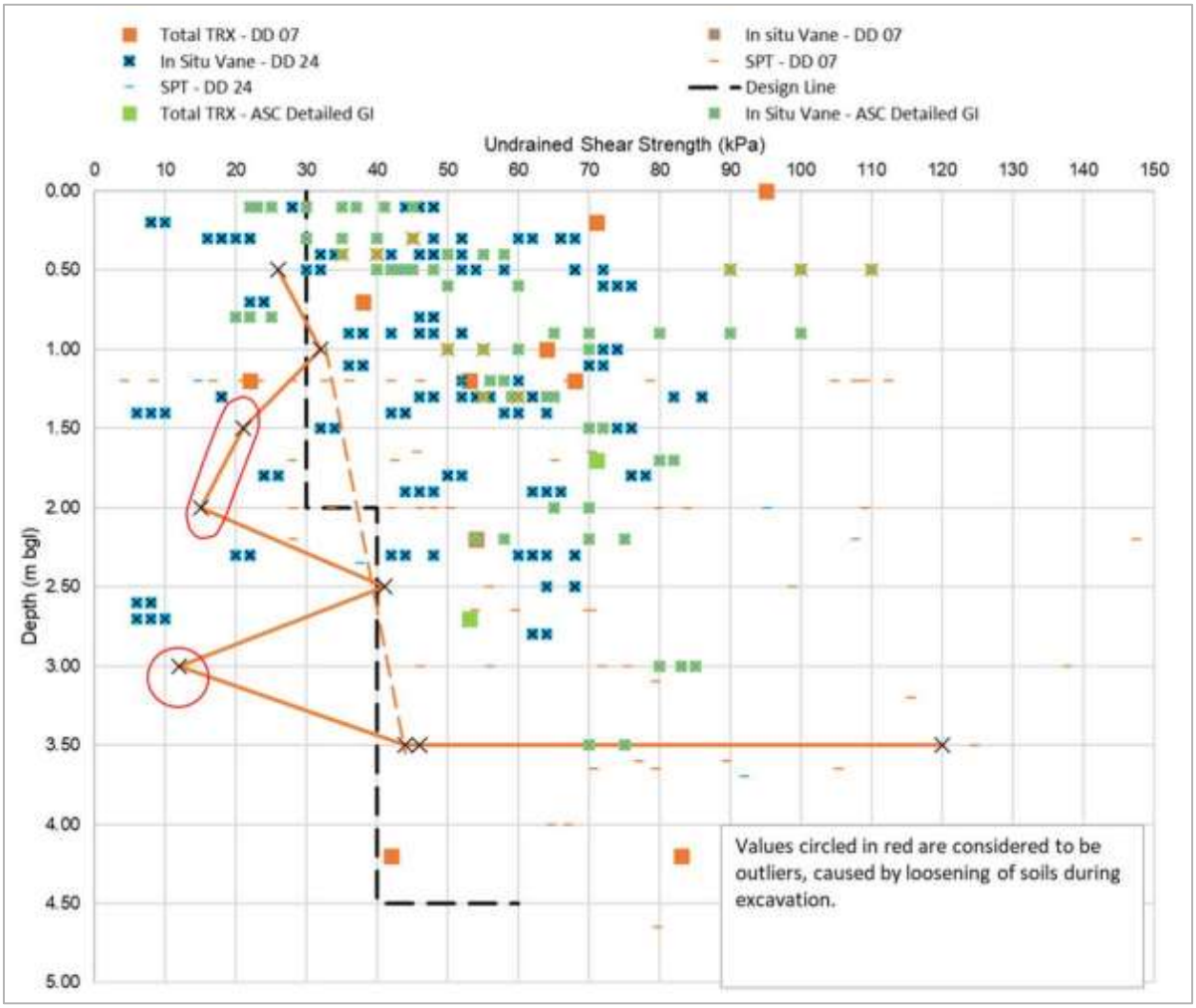


Figure B-15 - Undrained Shear Strength for Kimmeridge Clay, Portland Stone/Sand Formation from Ch.61+200 to Ch.61+763

Conservative strength parameters adopted within Alluvium Extent:

Depth from 0.0 to 2.0 mbgl: Cu = 30 kPa

Depth from 2.0 to 4.5 mbgl: Cu = 40 kPa

Depth > 4.5mbgl: Cu = 40+8z

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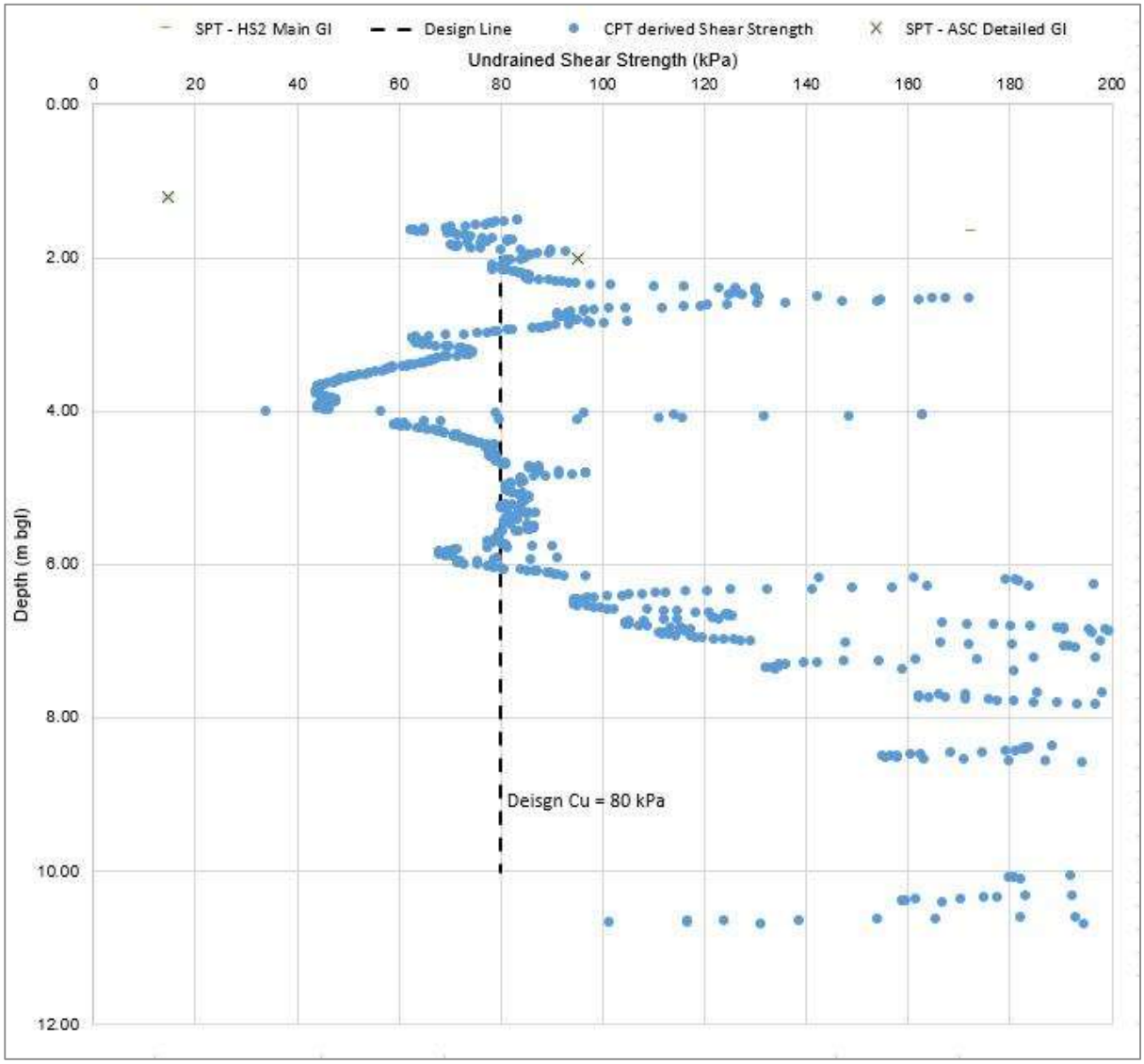


Figure B-16 – Undrained Shear Strength for Purbeck Group

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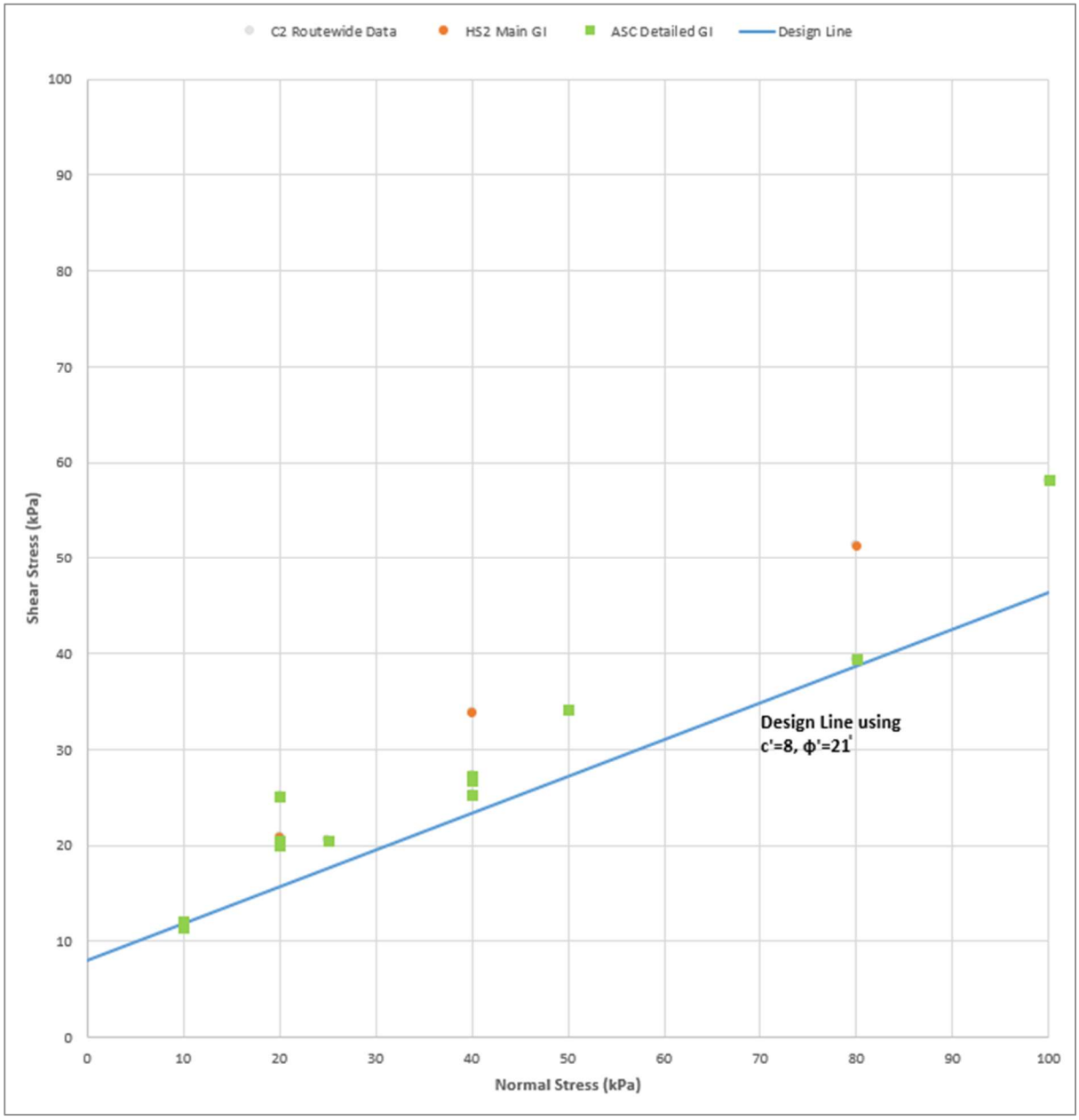


Figure B-17 - Shear Box Testing for Gault Clay Formation

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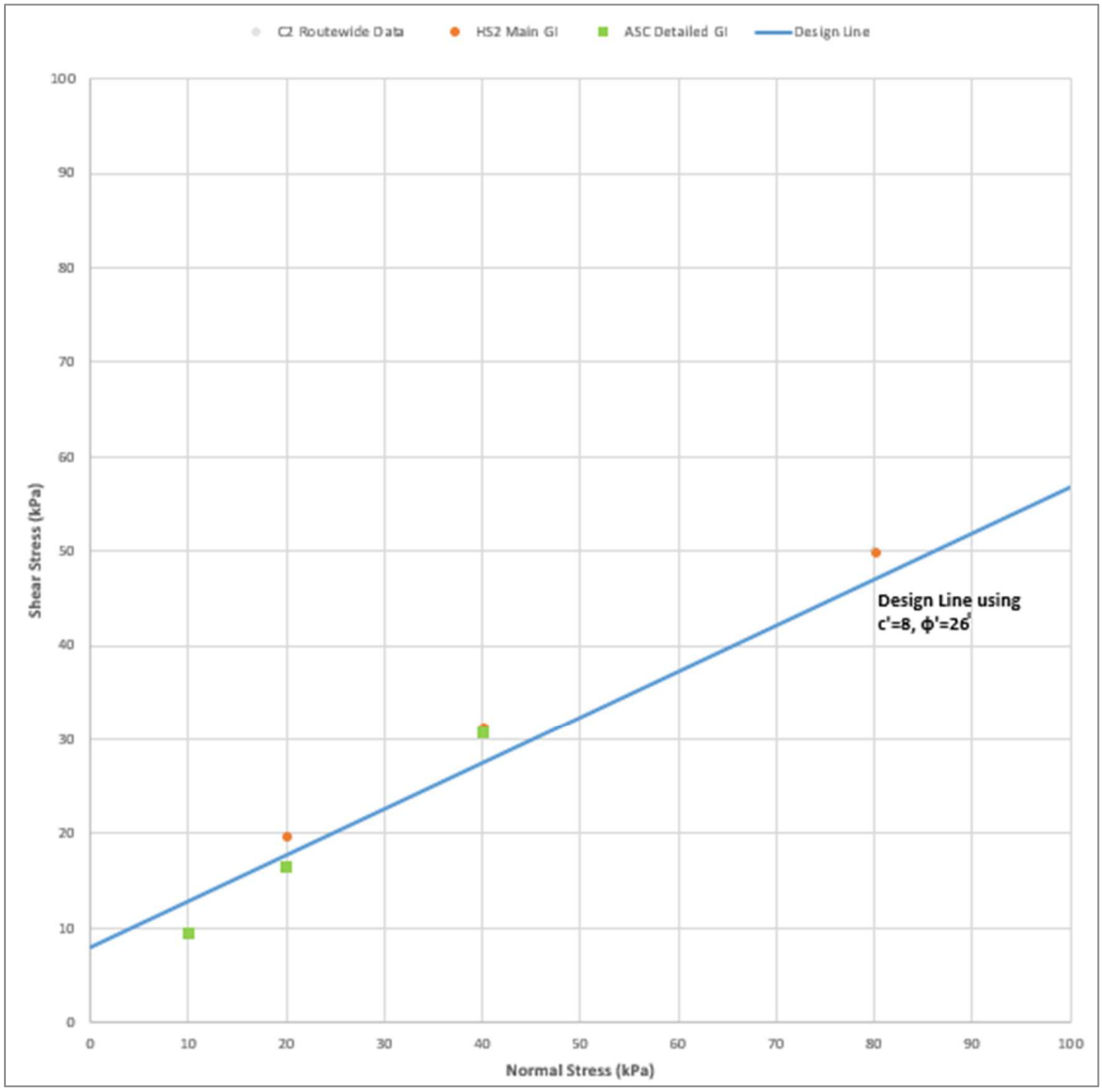


Figure B-18 - Shear Box Testing for Portland Stone/Sand Formation

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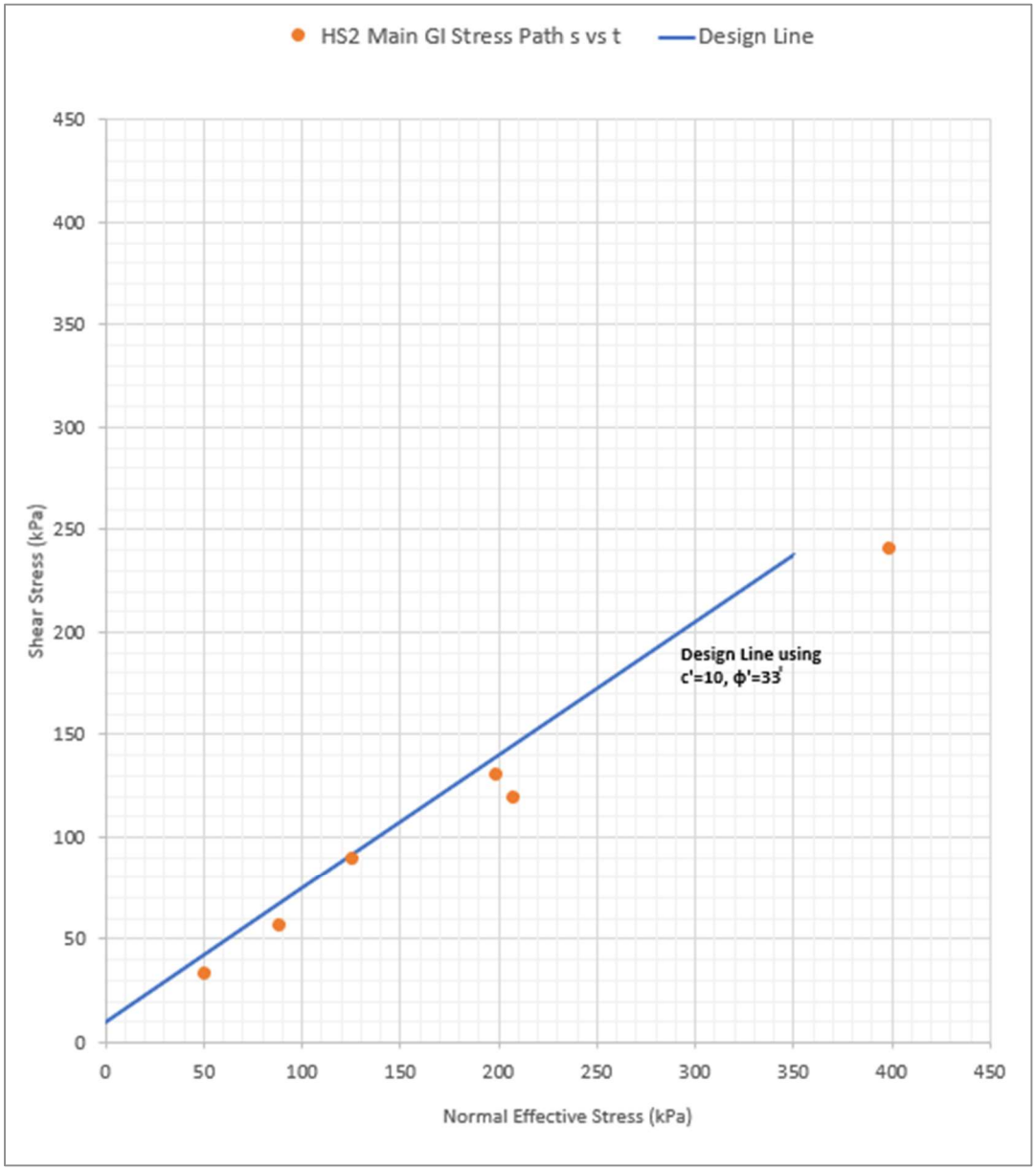


Figure B-19 - Triaxial Data for Kimmeridge Clay Formation

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Appendix C

Calculation Outputs – Mainline and Landscaping Earthworks

Number of Pages: 34

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Mainline Slope Stability Analyses Outputs

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	C-Top of Layer (kPa)	C-Rate of Change (kN/m²)/m)	C-Maximum (kPa)	Piezometric Surface	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)
	Embankment	Mohr-Coulomb	20						0	35	0
	GLT / POST/POSA (Undrained) >4m	S=f(depth)	19		50	10	150	1			
	Landscape Fill (Undrained) Class 2	Undrained (Phi=0)	19	50							
	Landscape Fill (Undrained) Class 4	Undrained (Phi=0)	19	40							
	Lower Head (Undrained)	Undrained (Phi=0)	18	40				1			
	Replacement Fill	Mohr-Coulomb	20					1	0	35	0
	Upper Head (Undrained)	Undrained (Phi=0)	18	30				1			

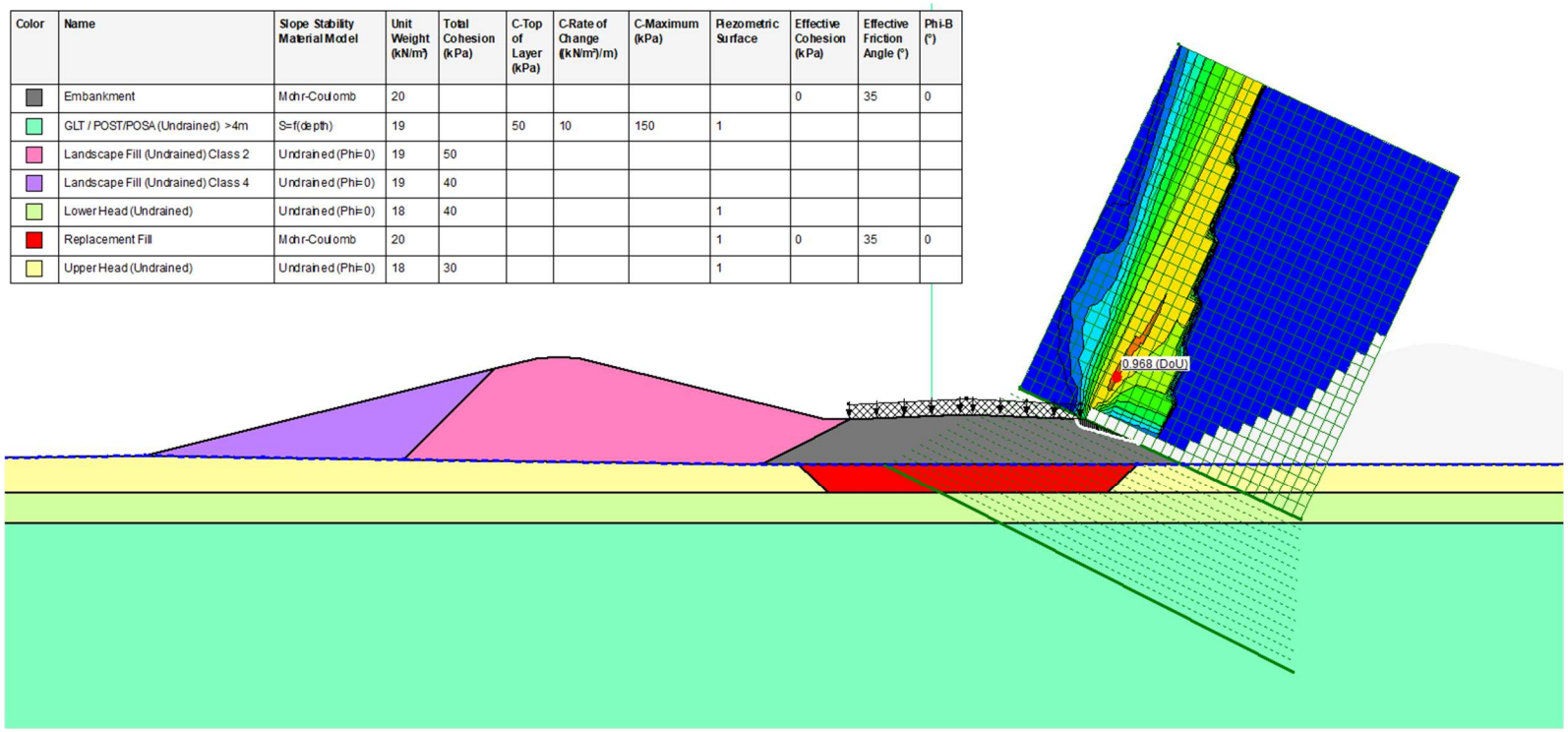
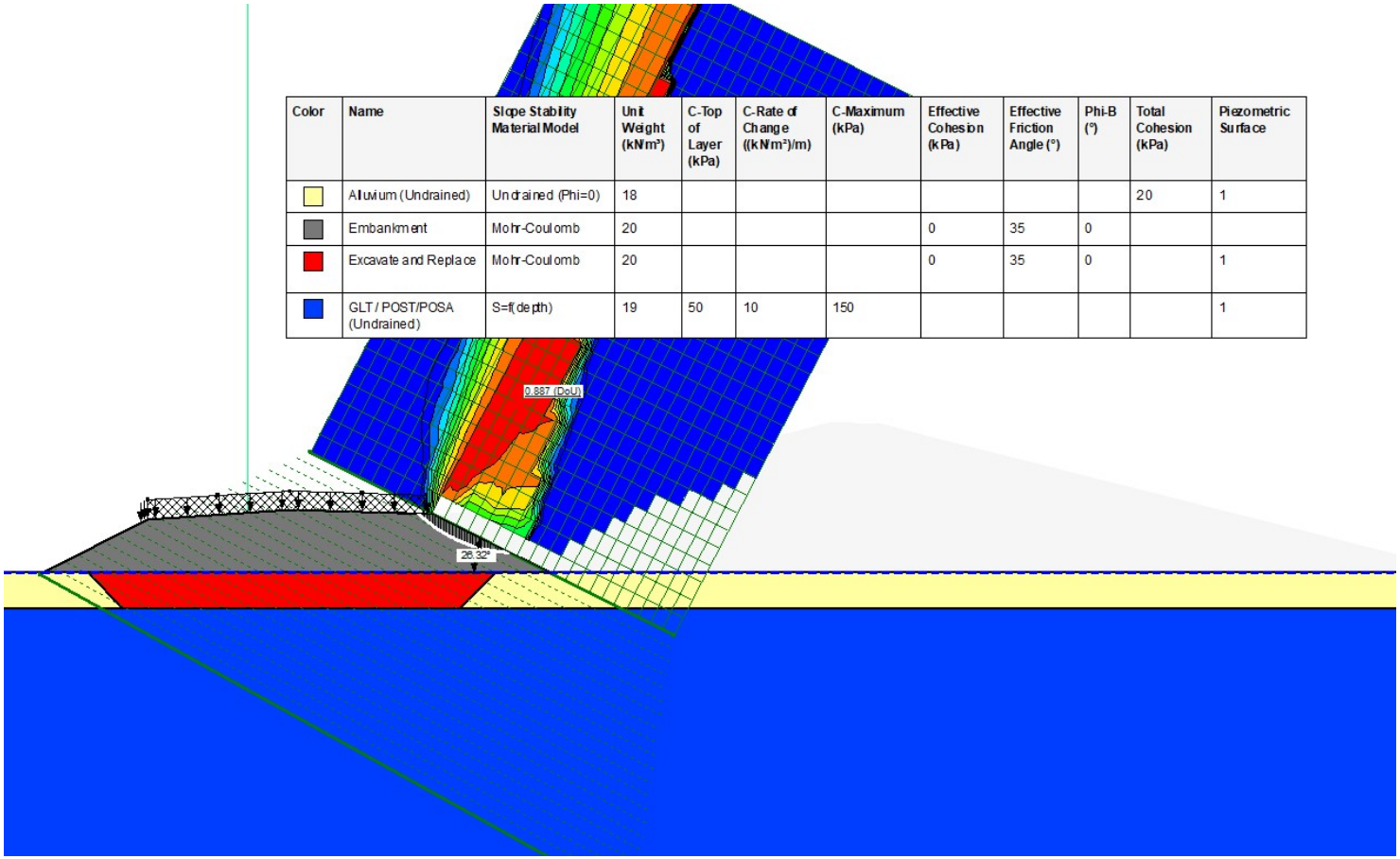


Figure C-1: Ch. 61+000 GM1 – DA1. C2 – Short-term (Undrained) – Optimised Critical Slip Surface – Surcharge applied with 0.5 m offset from the crest

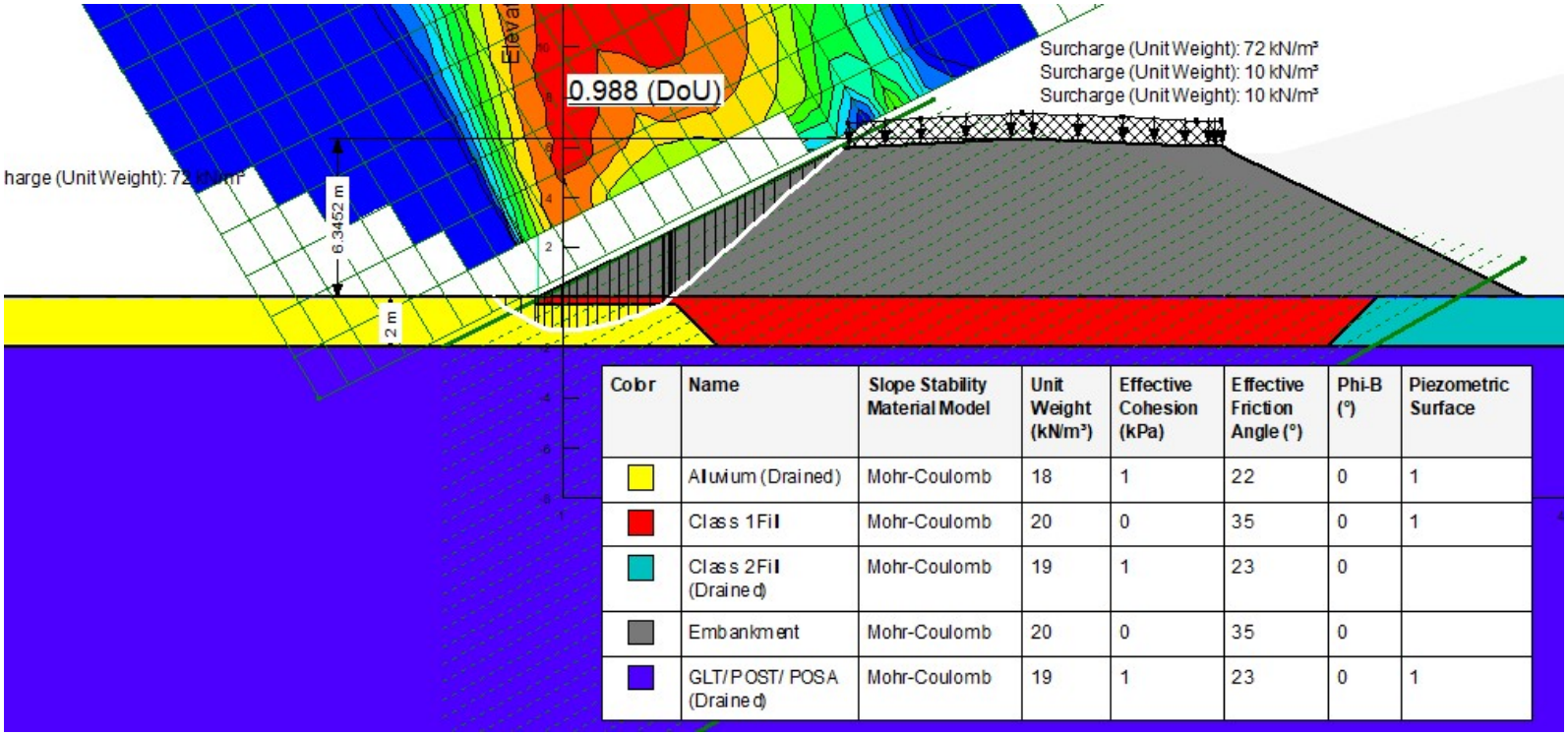
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Total Cohesion (kPa)	Piezometric Surface
	Alluvium (Undrained)	Undrained (Phi=0)	18							20	1
	Embankment	Mohr-Coulomb	20				0	35	0		
	Excavate and Replace	Mohr-Coulomb	20				0	35	0		1
	GLT / POST/POSA (Undrained)	S=f(depth)	19	50	10	150					1

Figure C-2: Ch. 61+000 GM2 – DA1. C2 – Short-term (Undrained) – Optimised Critical Slip Surface – Surcharge applied with 0.5 m offset from the crest

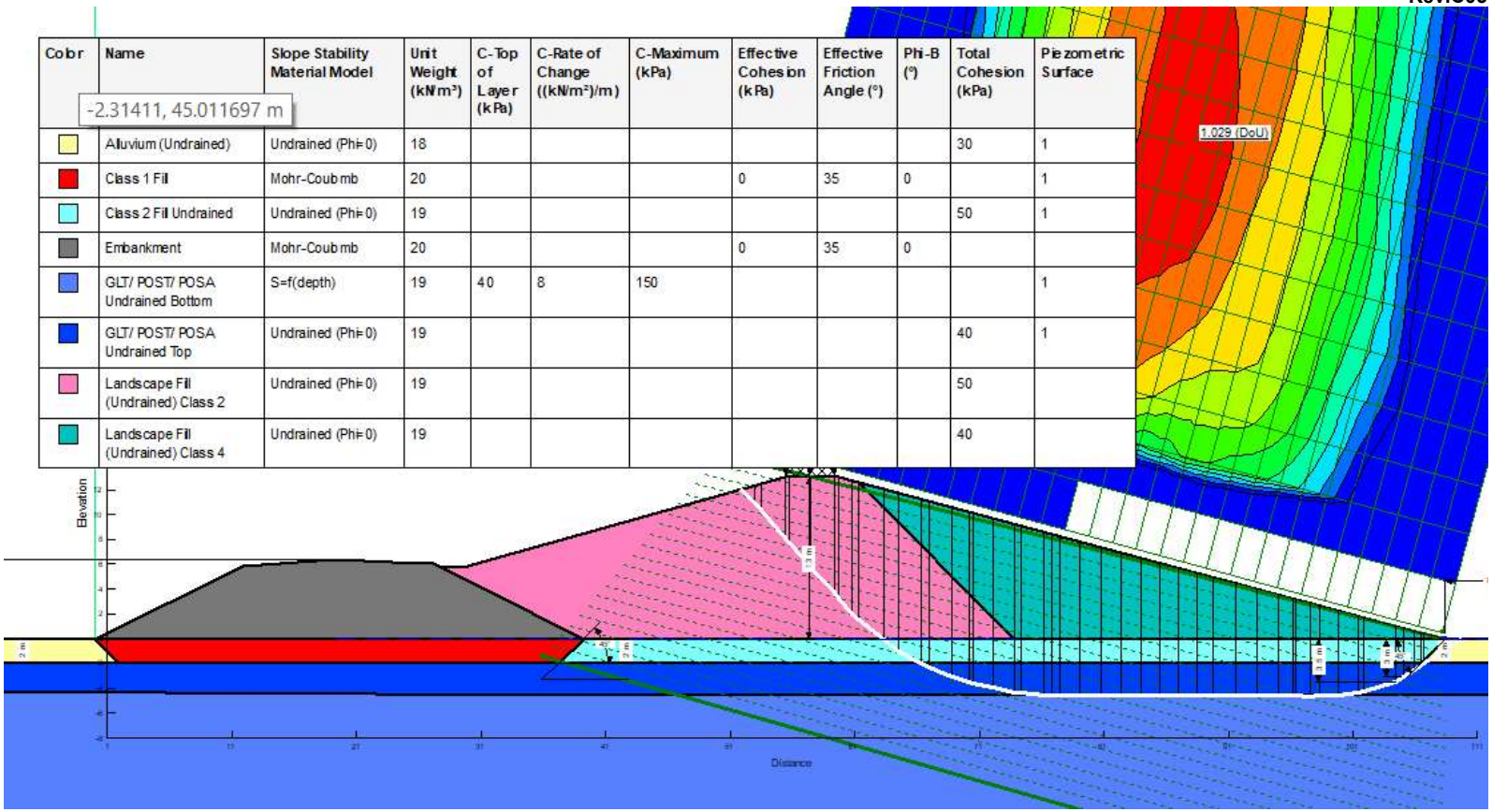
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	1	22	0	1
	Class 1Fil	Mohr-Coulomb	20	0	35	0	1
	Class 2Fil (Drained)	Mohr-Coulomb	19	1	23	0	
	Embankment	Mohr-Coulomb	20	0	35	0	
	GLT/POST/ POSA (Drained)	Mohr-Coulomb	19	1	23	0	1

Figure C-3: CH. 61+420 – DA1. C2 – Long-term (drained) – Optimised Critical Slip Surface – Surcharge applied with 0.5 m offset from the crest

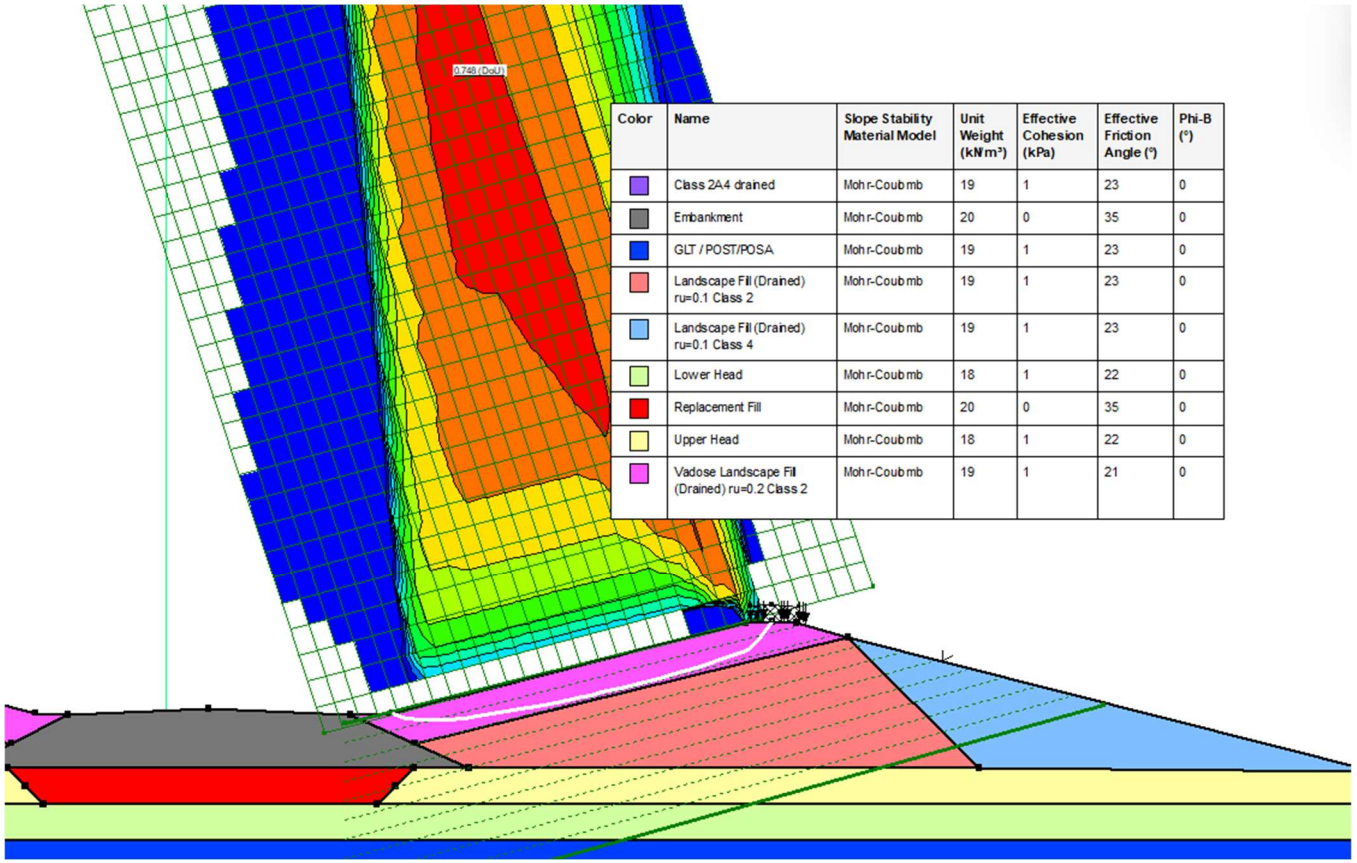
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Total Cohesion (kPa)	Piezometric Surface
	Alluvium (Undrained)	Undrained (Phi=0)	18							30	1
	Class 1 Fil	Mohr-Coulomb	20				0	35	0		1
	Class 2 Fil Undrained	Undrained (Phi=0)	19							50	1
	Embankment	Mohr-Coulomb	20				0	35	0		
	GLT/ POST/ POSA Undrained Bottom	S=f(depth)	19	40	8	150					1
	GLT/ POST/ POSA Undrained Top	Undrained (Phi=0)	19							40	1
	Landscape Fil (Undrained) Class 2	Undrained (Phi=0)	19							50	
	Landscape Fil (Undrained) Class 4	Undrained (Phi=0)	19							40	

Figure C-4: Ch. 61+420 – DA1. C2 – Short-term (Undrained) - Downline External Slope – 1V:4H (Failure with 2m E&R)

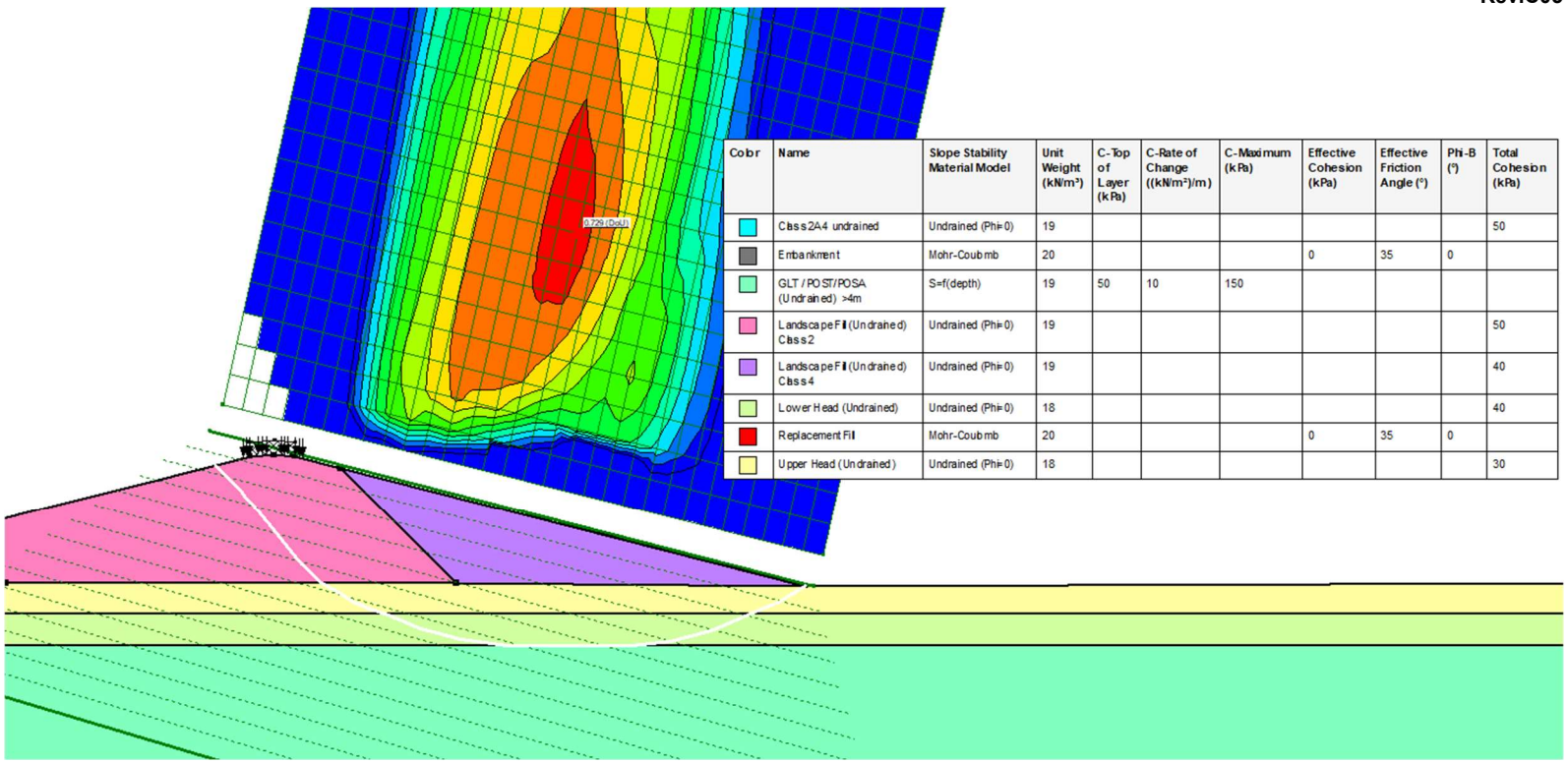
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)
	Class 2A4 drained	Mohr-Coub mb	19	1	23	0
	Embankment	Mohr-Coub mb	20	0	35	0
	GLT / POST/POSA	Mohr-Coub mb	19	1	23	0
	Landscape Fill (Drained) ru=0.1 Class 2	Mohr-Coub mb	19	1	23	0
	Landscape Fill (Drained) ru=0.1 Class 4	Mohr-Coub mb	19	1	23	0
	Lower Head	Mohr-Coub mb	18	1	22	0
	Replacement Fill	Mohr-Coub mb	20	0	35	0
	Upper Head	Mohr-Coub mb	18	1	22	0
	Vadose Landscape Fil (Drained) ru=0.2 Class 2	Mohr-Coub mb	19	1	21	0

Figure C-5: Ch. 60+920 GM1 – DA1. C2 – long-term (drained) – Landscape bund Internal Slope

HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Total Cohesion (kPa)
	Class 2A4 undrained	Undrained (Phi=0)	19							50
	Embankment	Mohr-Coulomb	20				0	35	0	
	GLT / POST/POSA (Undrained) >4m	S=f(depth)	19	50	10	150				
	Landscape Fill (Undrained) Class 2	Undrained (Phi=0)	19							50
	Landscape Fill (Undrained) Class 4	Undrained (Phi=0)	19							40
	Lower Head (Undrained)	Undrained (Phi=0)	18							40
	Replacement Fill	Mohr-Coulomb	20				0	35	0	
	Upper Head (Undrained)	Undrained (Phi=0)	18							30

Figure C-6: Ch. 60+920 GM1 – DA1. C1 – short-term (undrained) – Landscape bund external Slope

HS2 Ltd - Code 1 - Accepted

Mainline Embankment Settlement Analyses Outputs

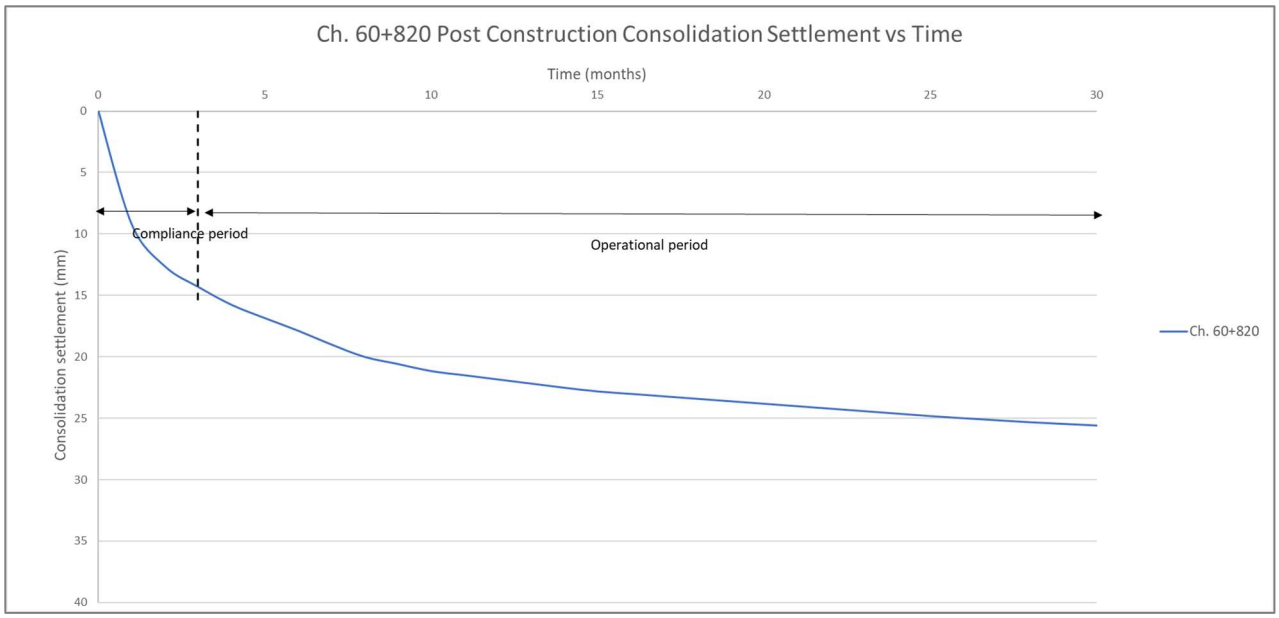


Figure C-7: Ch. 60+820 – Post consolidation settlement profile with time

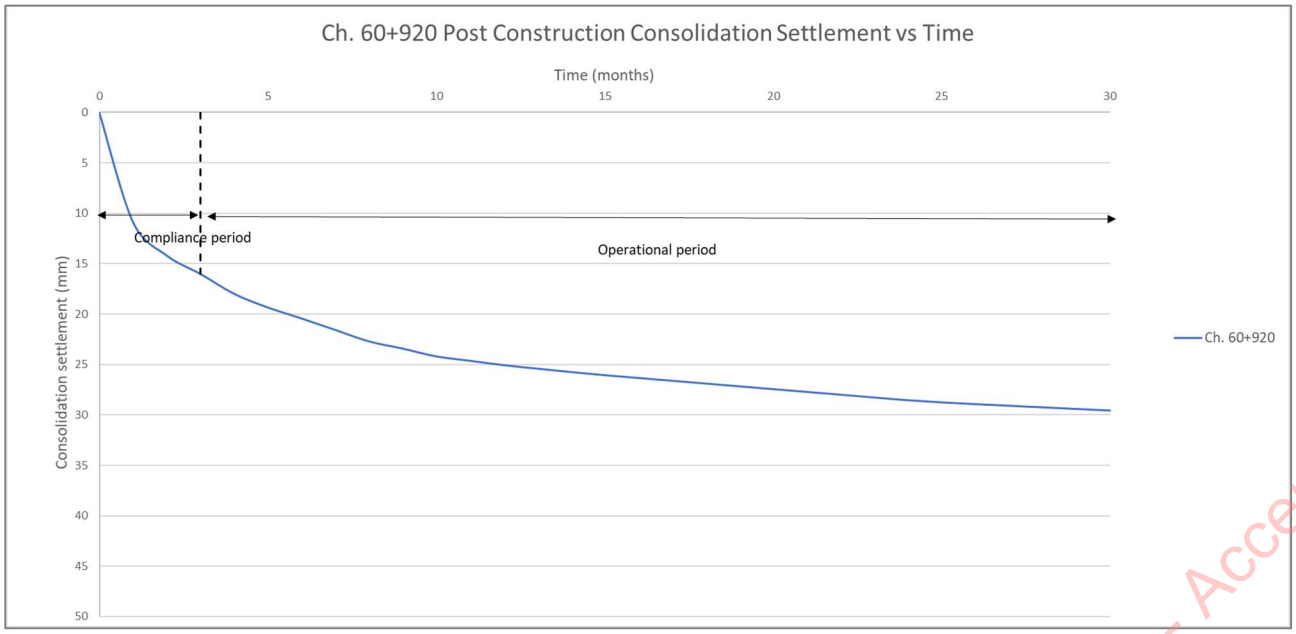


Figure C-8: Ch. 60+920 – Post consolidation settlement profile with time

HS2 Ltd - Code 1 - Accepted

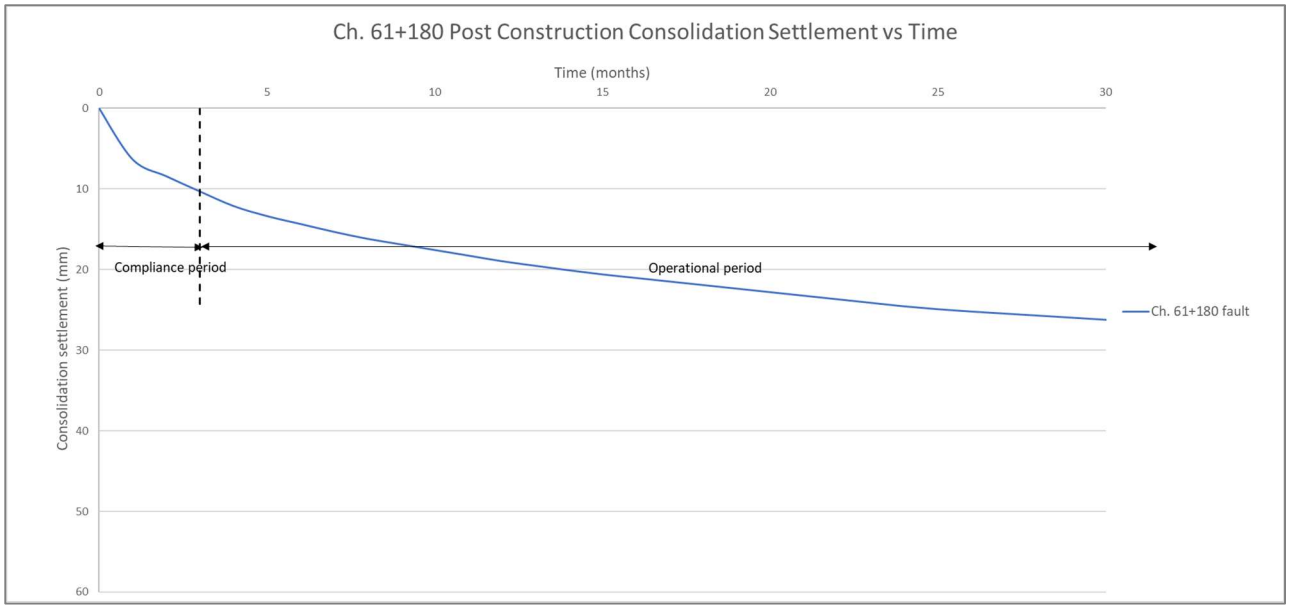


Figure C-9: Ch. 61+180 – Post consolidation settlement profile with time

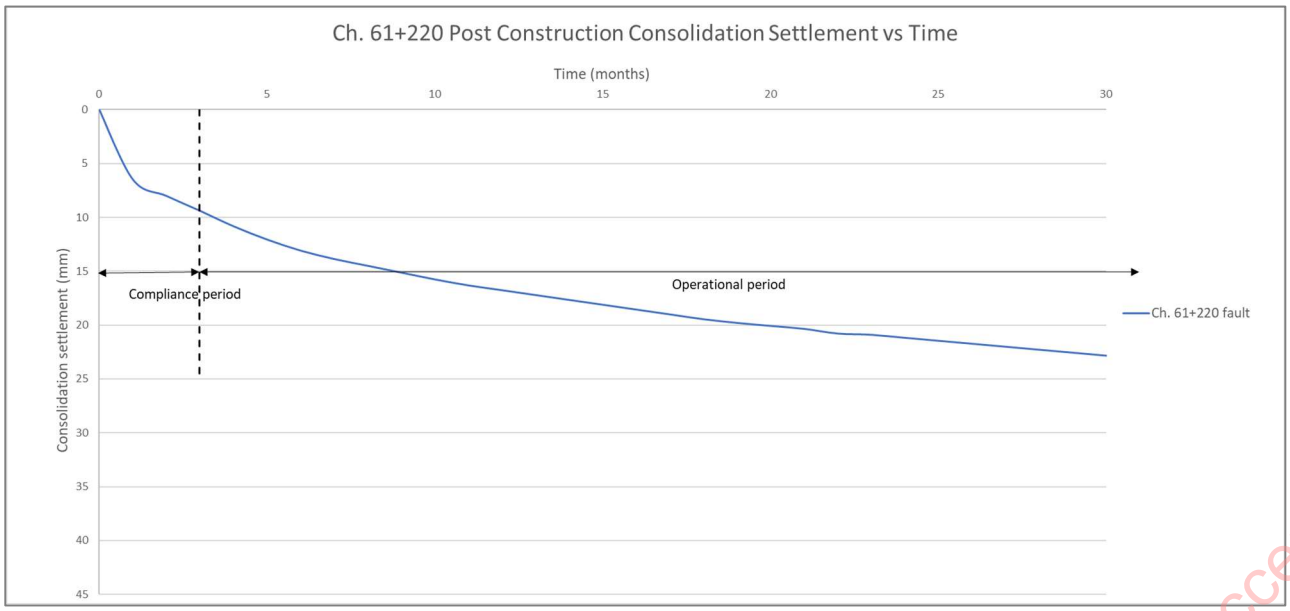


Figure C-10: Ch. 61+220 – Post consolidation settlement profile with time

HS2 Ltd - Code 1 - Accepted

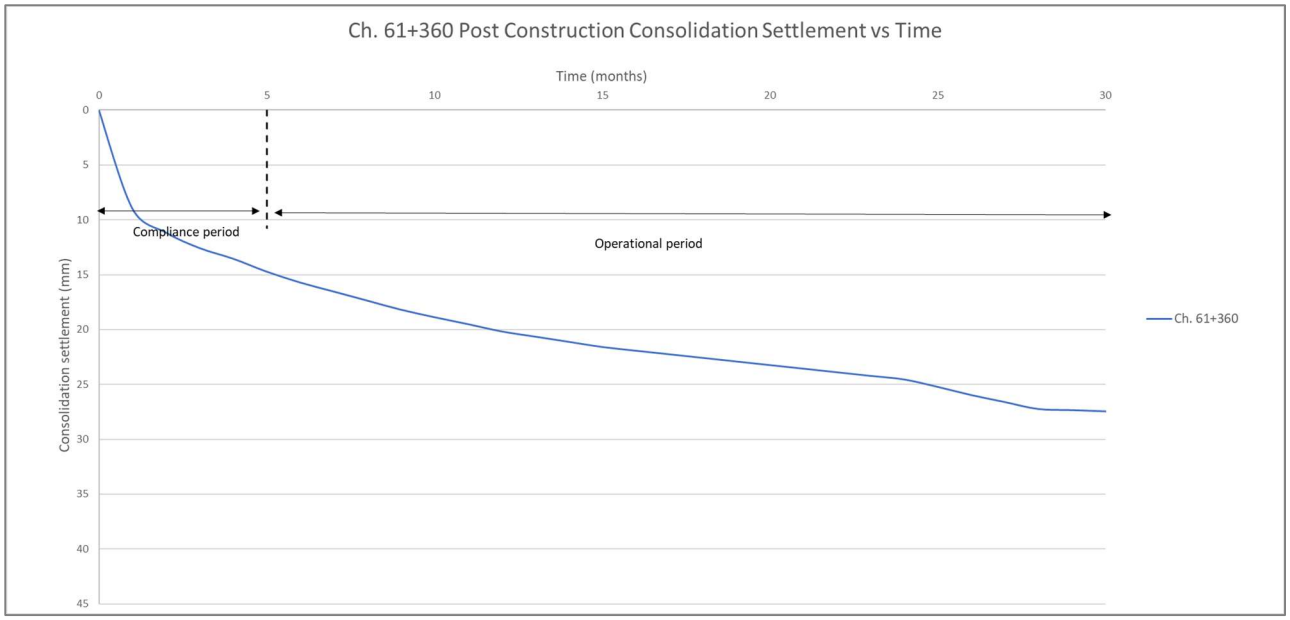


Figure C-11: Ch. 61+360 – Post consolidation settlement profile with time

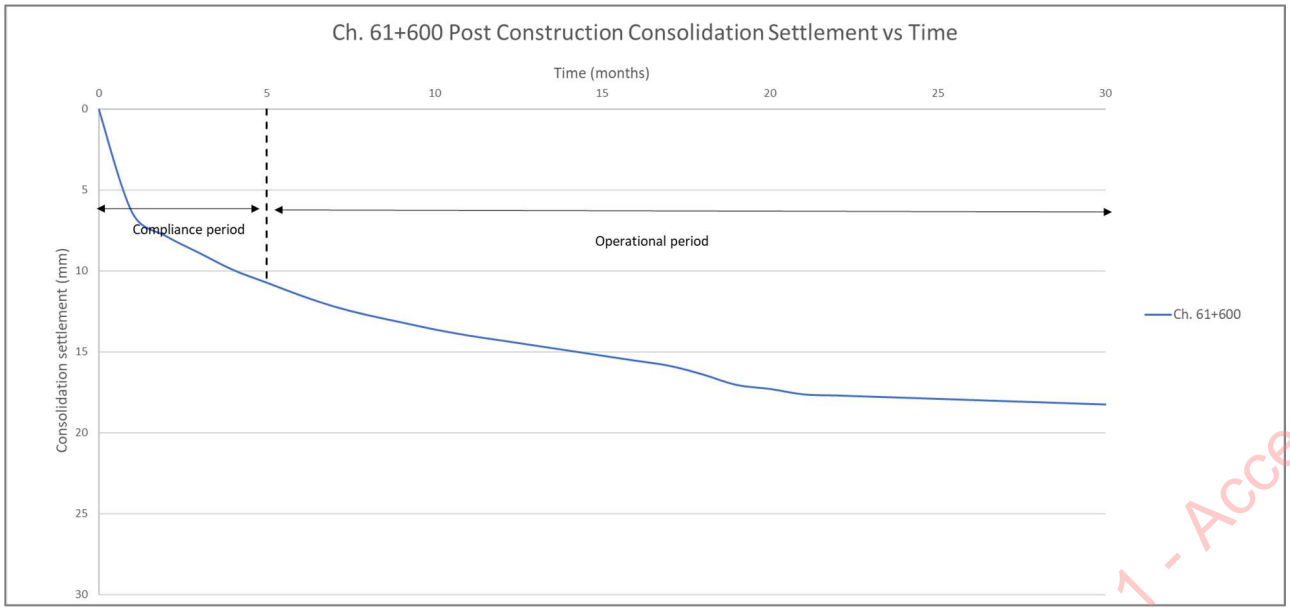


Figure C-12: Ch. 61+600 – Post consolidation settlement profile with time

HS2 Ltd - Code 1 - Accepted

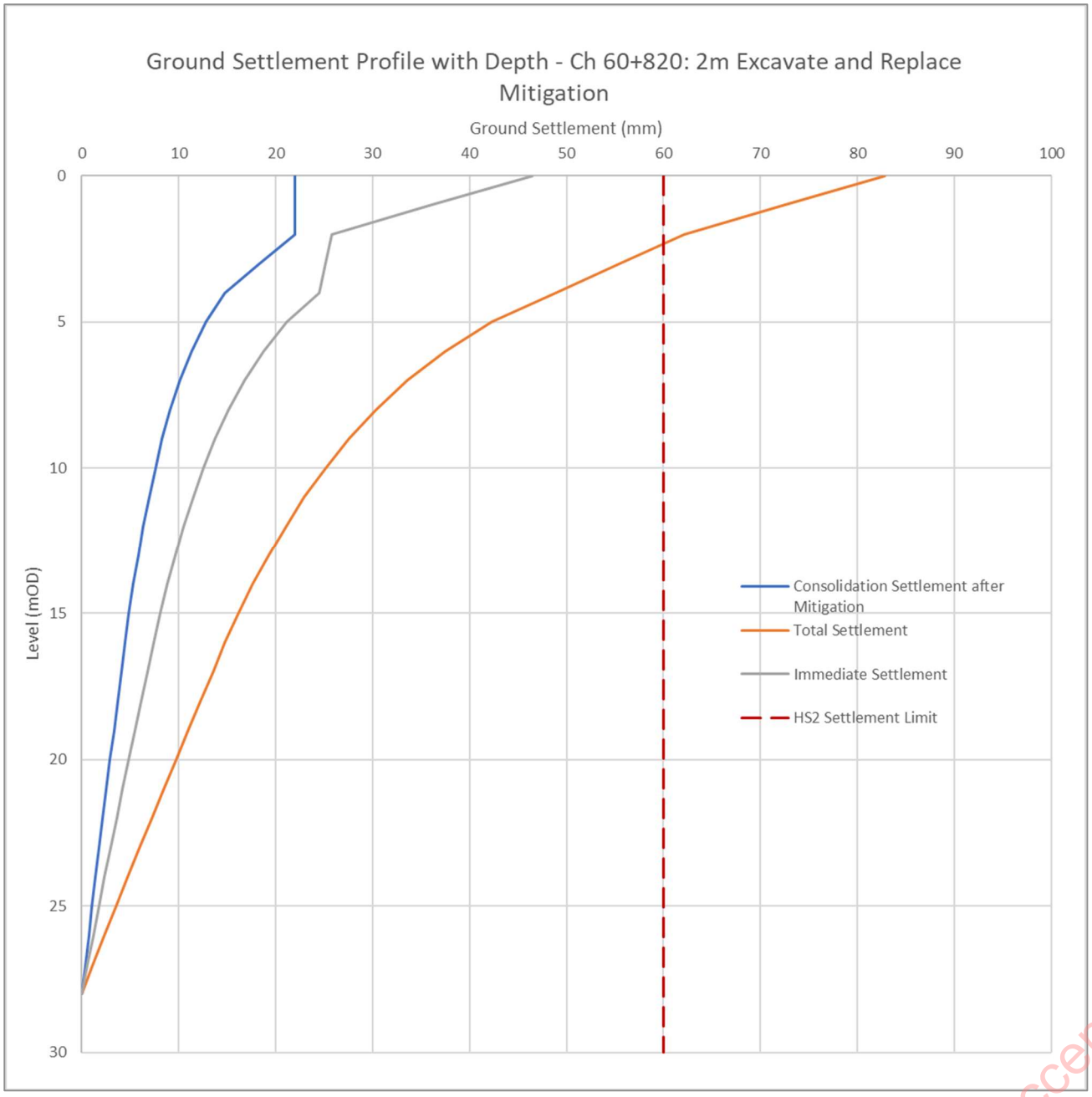


Figure C-13: Ch. 60+820 – Settlement profile with depth with mitigation

HS2 Ltd - Code 1 - Accepted

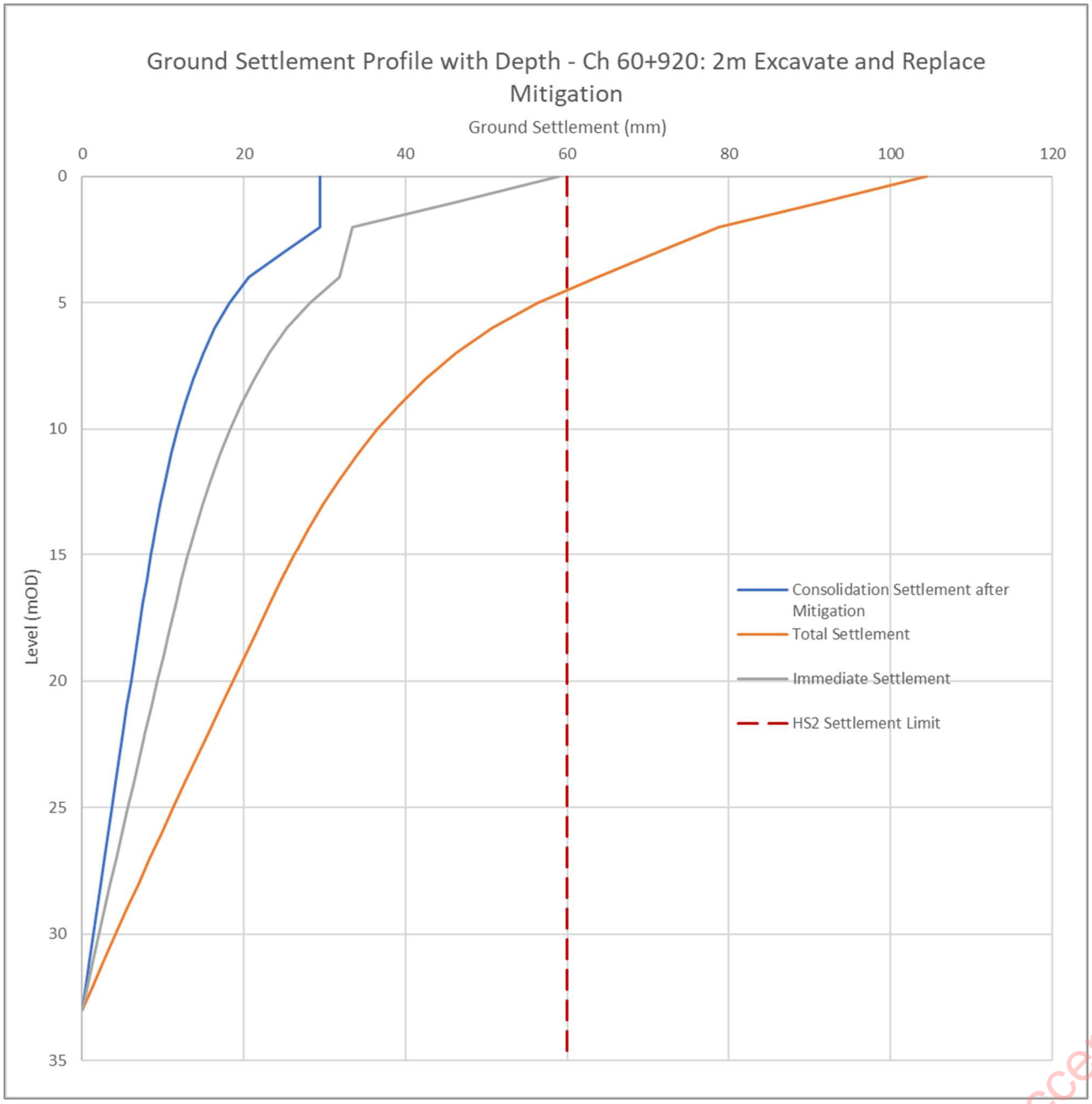


Figure C-14: Ch. 60+920 – Settlement profile with depth with mitigation

HS2 Ltd - Code 1 - Accepted

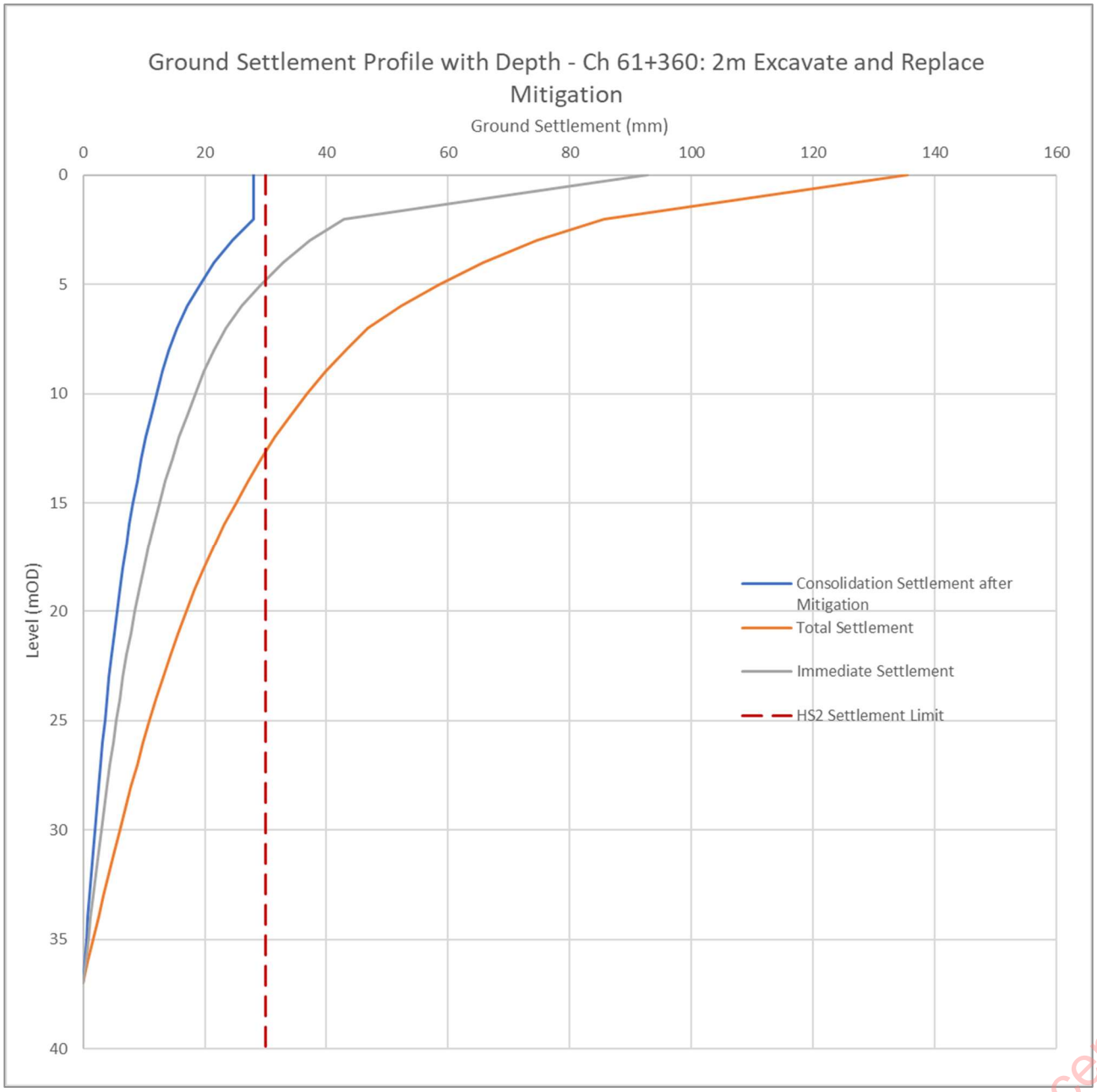


Figure C-15: Ch. 61+360 – Settlement profile with depth with mitigation

HS2 Ltd - Code 1 - Accepted

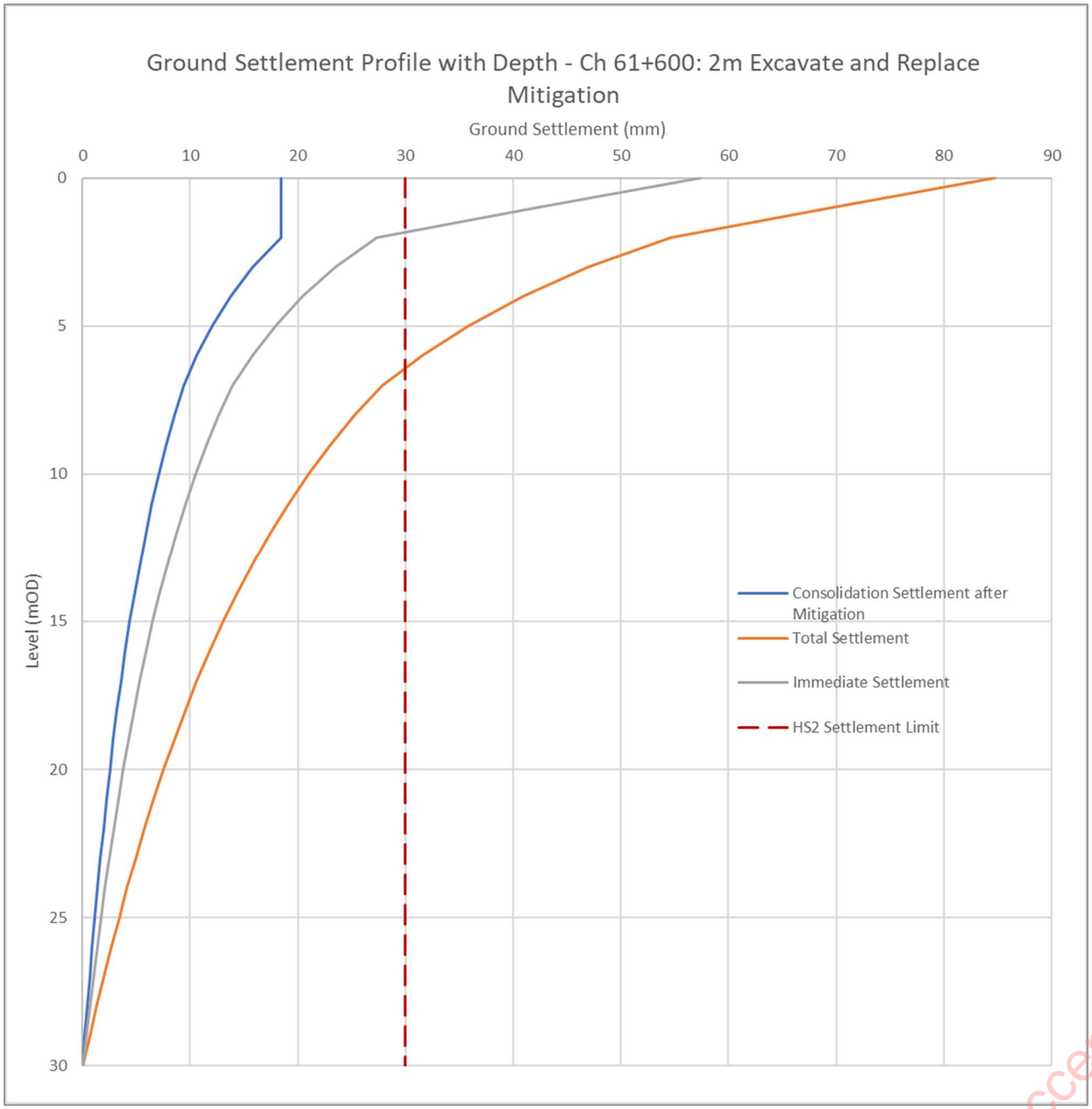


Figure C-16: Ch. 61+600 – Settlement profile with depth with mitigation

HS2 Ltd - Code 1 - Accepted

An assessment has been conducted to ensure the mitigation measures within the extent of transition will meet HS2's longitudinal distortion limit as per Table 4.2/1 from the HS2 Technical Standard – Earthworks, Ref. [3].

The post-construction settlement was calculated in each of the following locations within the transition, and then compared with the distortion limit of 1V:1500H.

- at centre of box culvert (CH. 61+330)
- at the South end of transition (CH. 61+305)
- at the North end of transition (CH. 61+355)

A summary of the assessment results is presented below. It is noted that the predicted differential settlements within the length of the transition are less than 10mm (at both southern and northern extent), which are well below the distortion criteria. In summary, the specified mitigation measures satisfy the requirements.

Table C-1: Transition Details – Ground Settlement of Sedrup North Culvert Transition

Predicted Settlement (mm)	Mitigation Measure Provided		
	North end	Centre of Culvert	South end
Chainage	61+355	61+330	61+305
Post-construction Settlement (mm)	30	24	26
Required Mitigation (E&R + Compliance Period)	2.0 mbgl + 5-month Compliance Period	3.75 mbgl + 5-month Compliance Period	2.0 mbgl + 5-month Compliance Period
Settlement Limit (mm)	30	30	60
Differential Settlement (from centre of the culvert)	5.98	-	1.98
Allowable Differential Settlement (from centre of the culvert)	16.7	-	16.7
Longitudinal Distortion Limit	**1V:1500H**		

Mainline Embankment Dynamic Performance 1D-Tool Results

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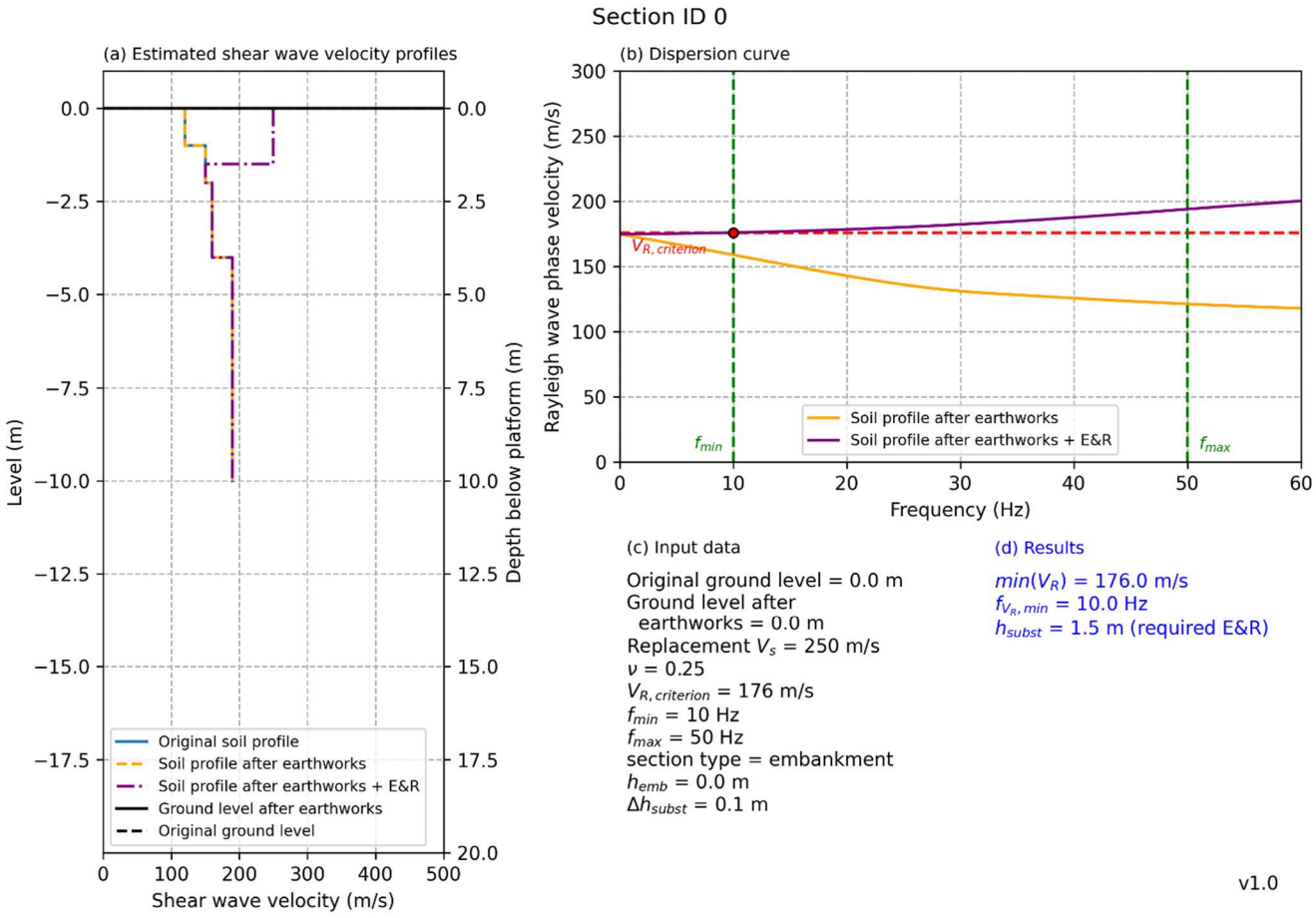


Figure C-17: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

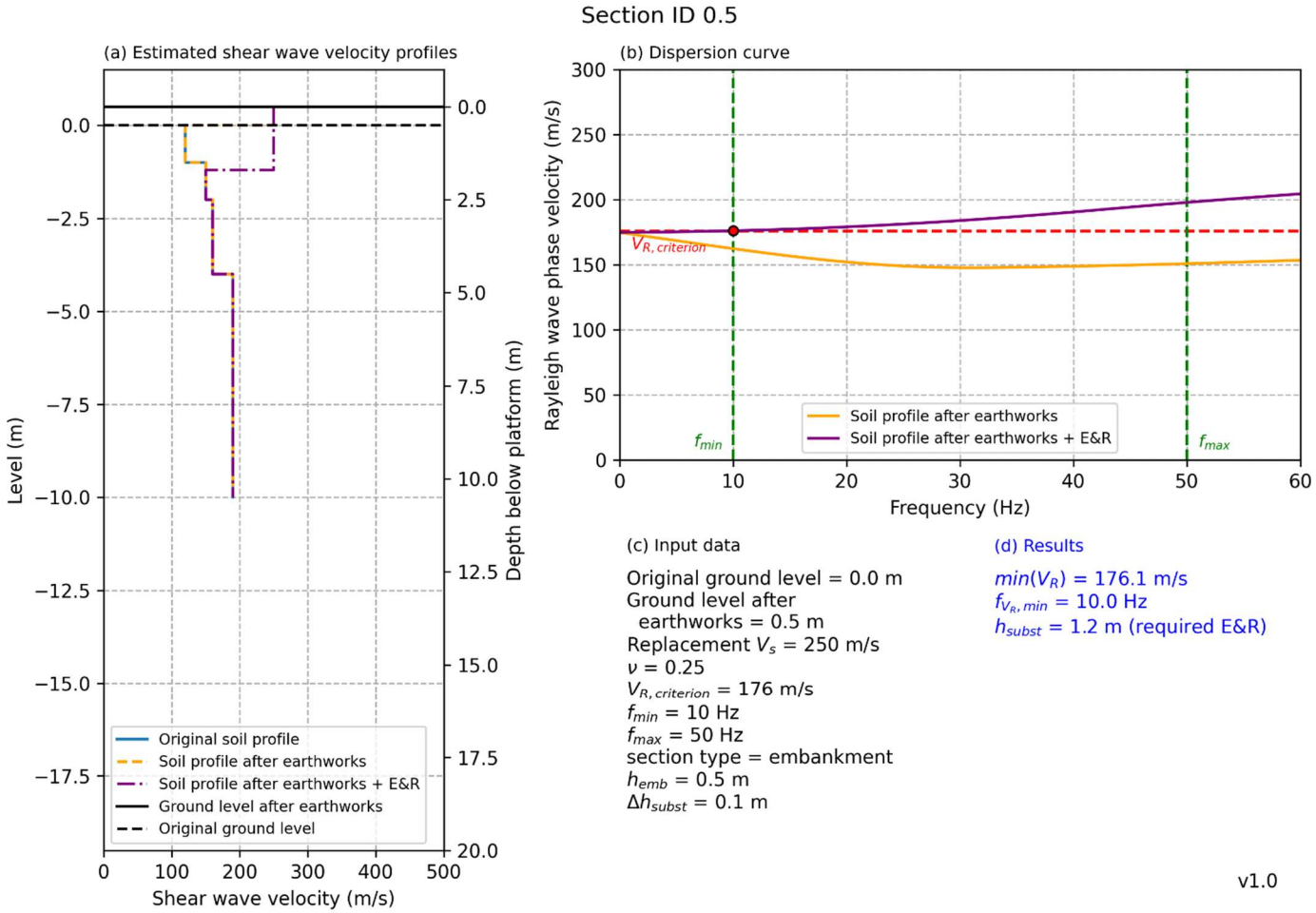


Figure C-18: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

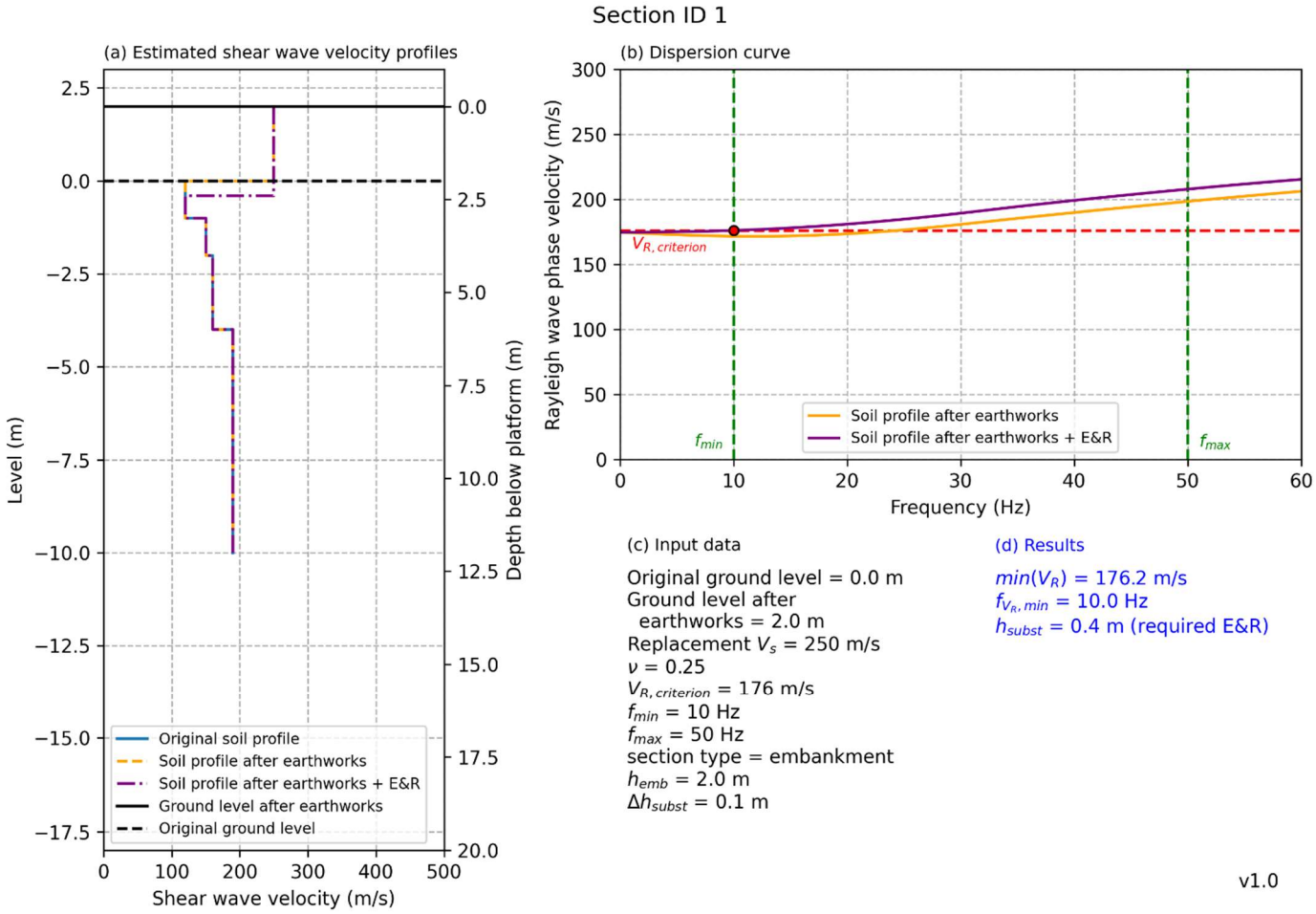


Figure C-19: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

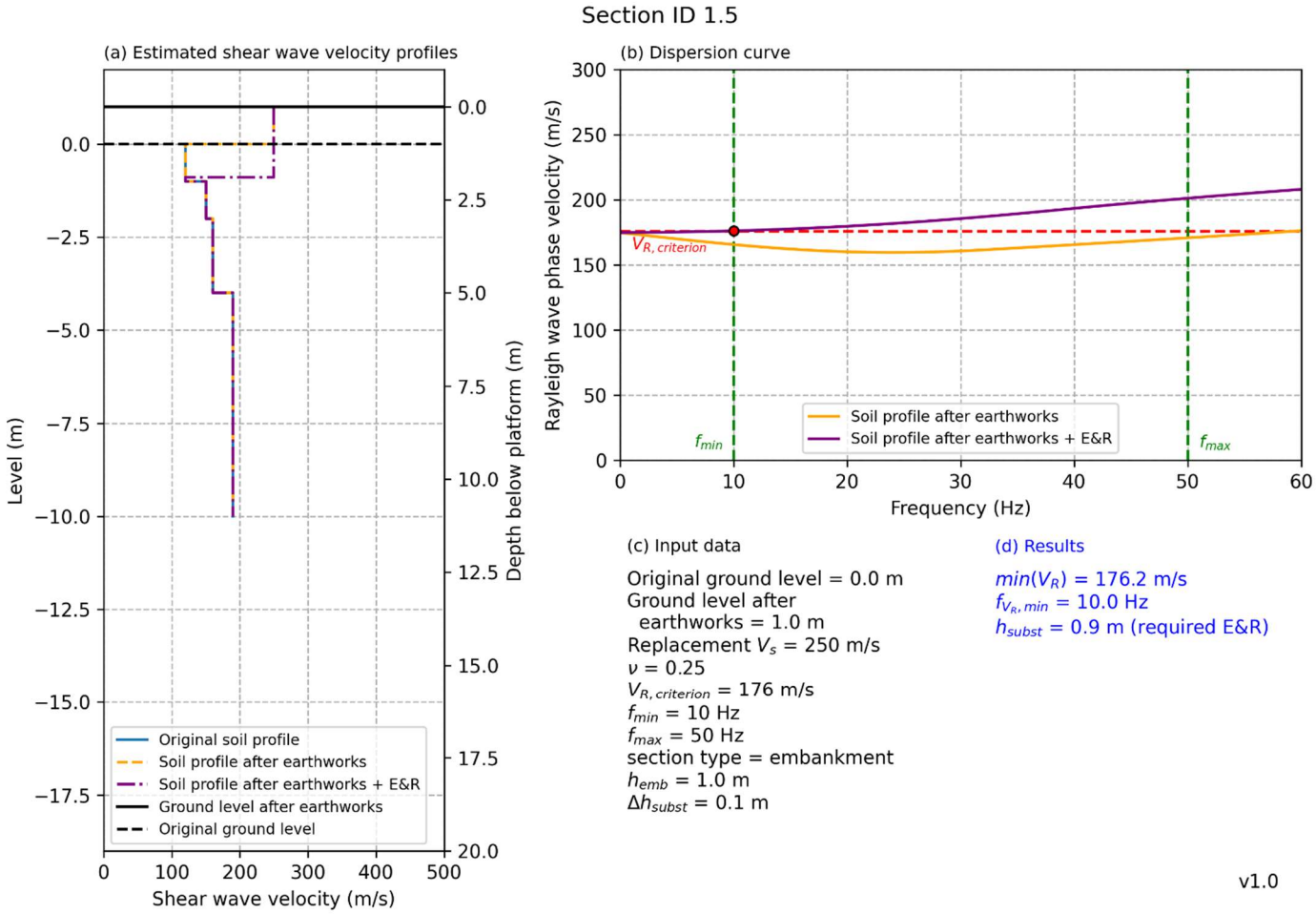


Figure C-20: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

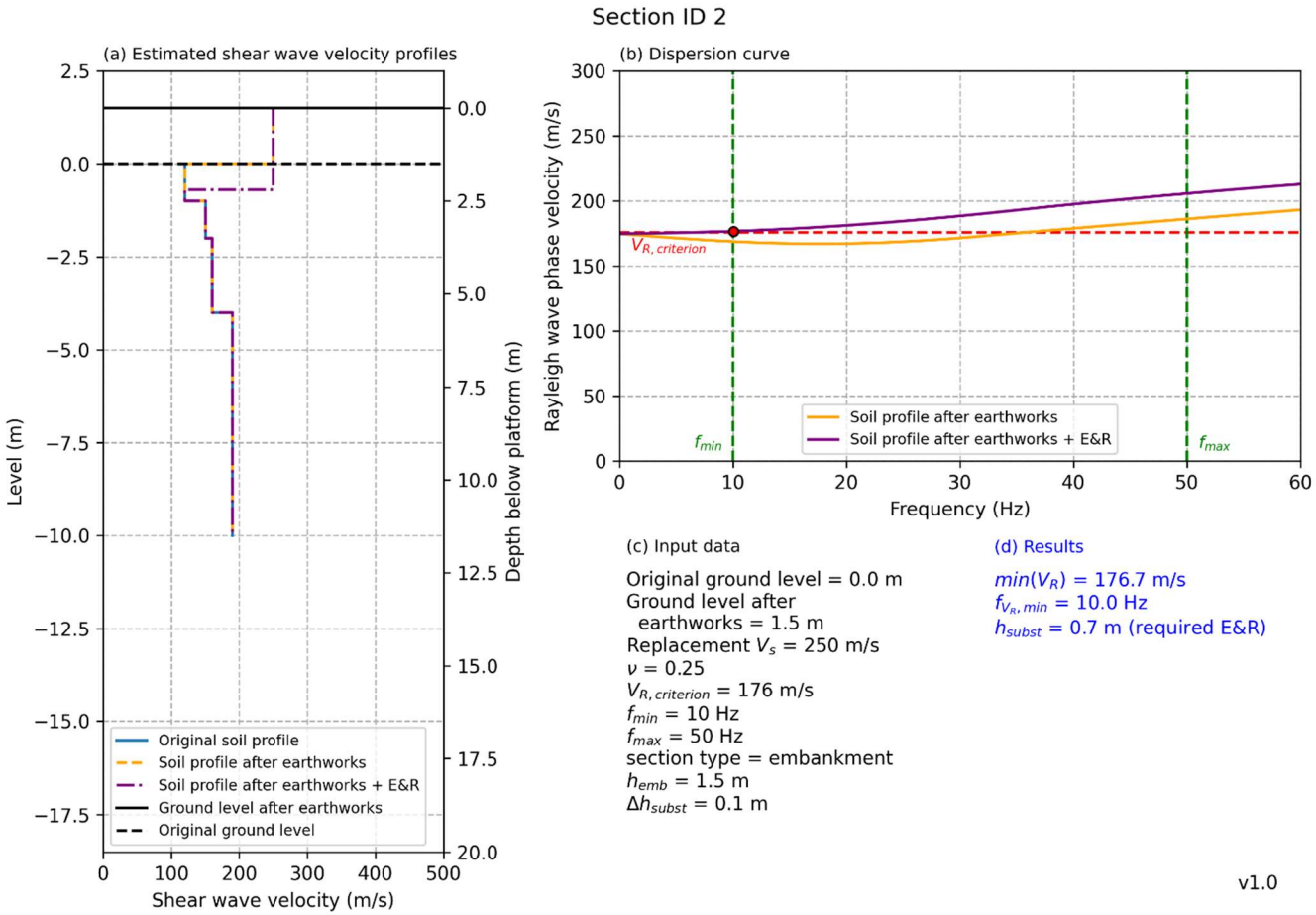


Figure C-21: North Ground Model – Dynamic Performance Mitigation Result (E&R)

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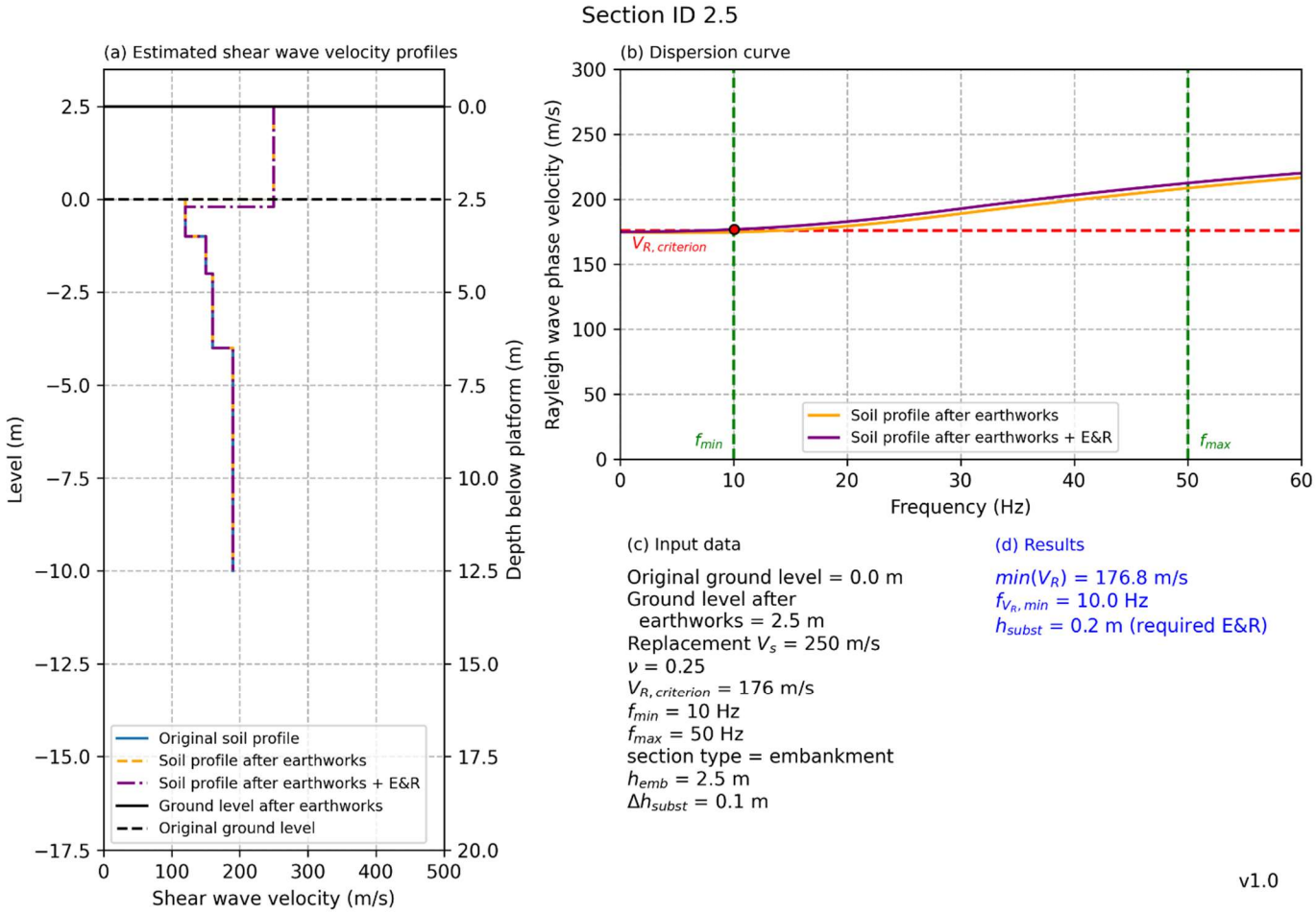


Figure C-22: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

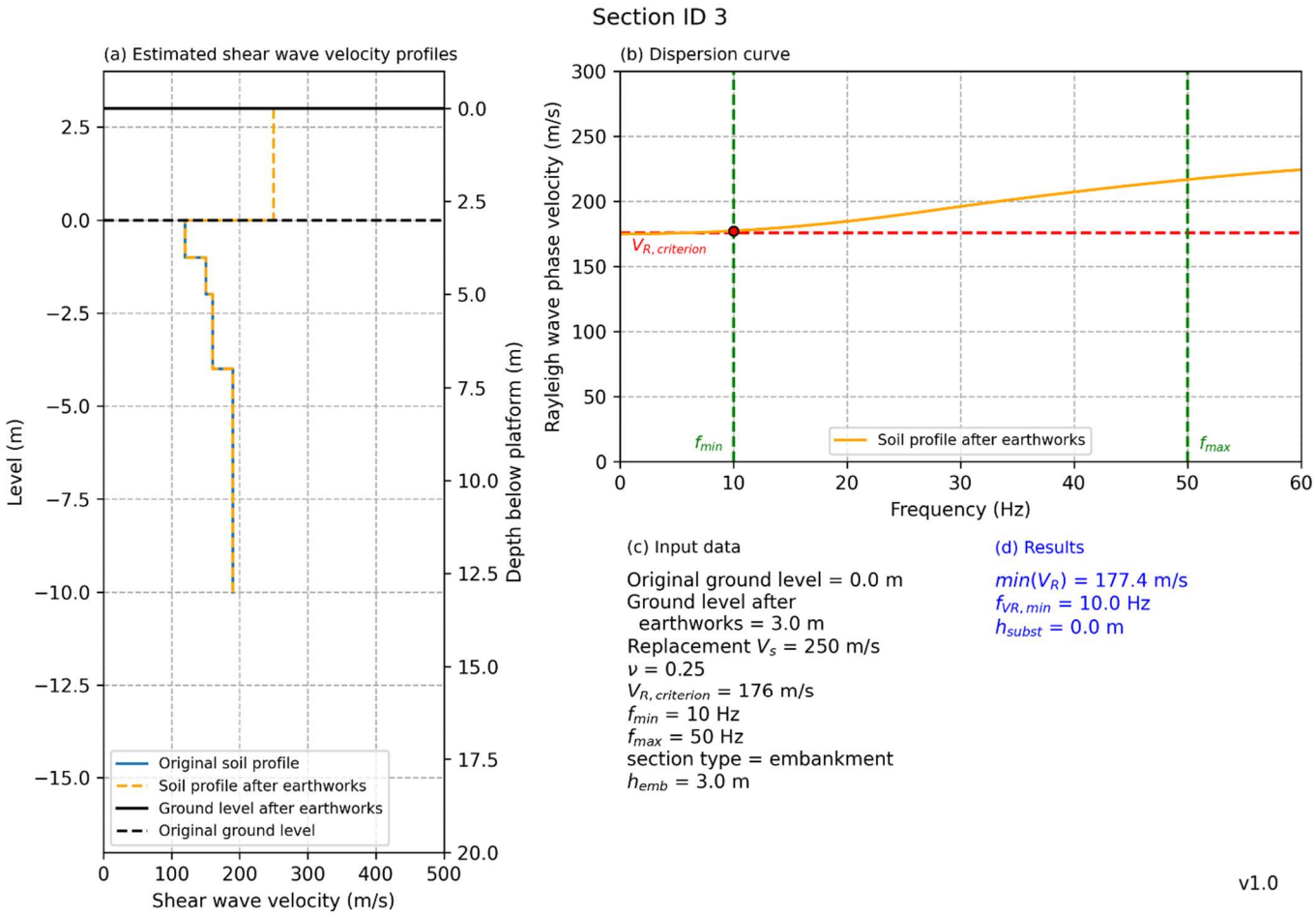


Figure C-23: North Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

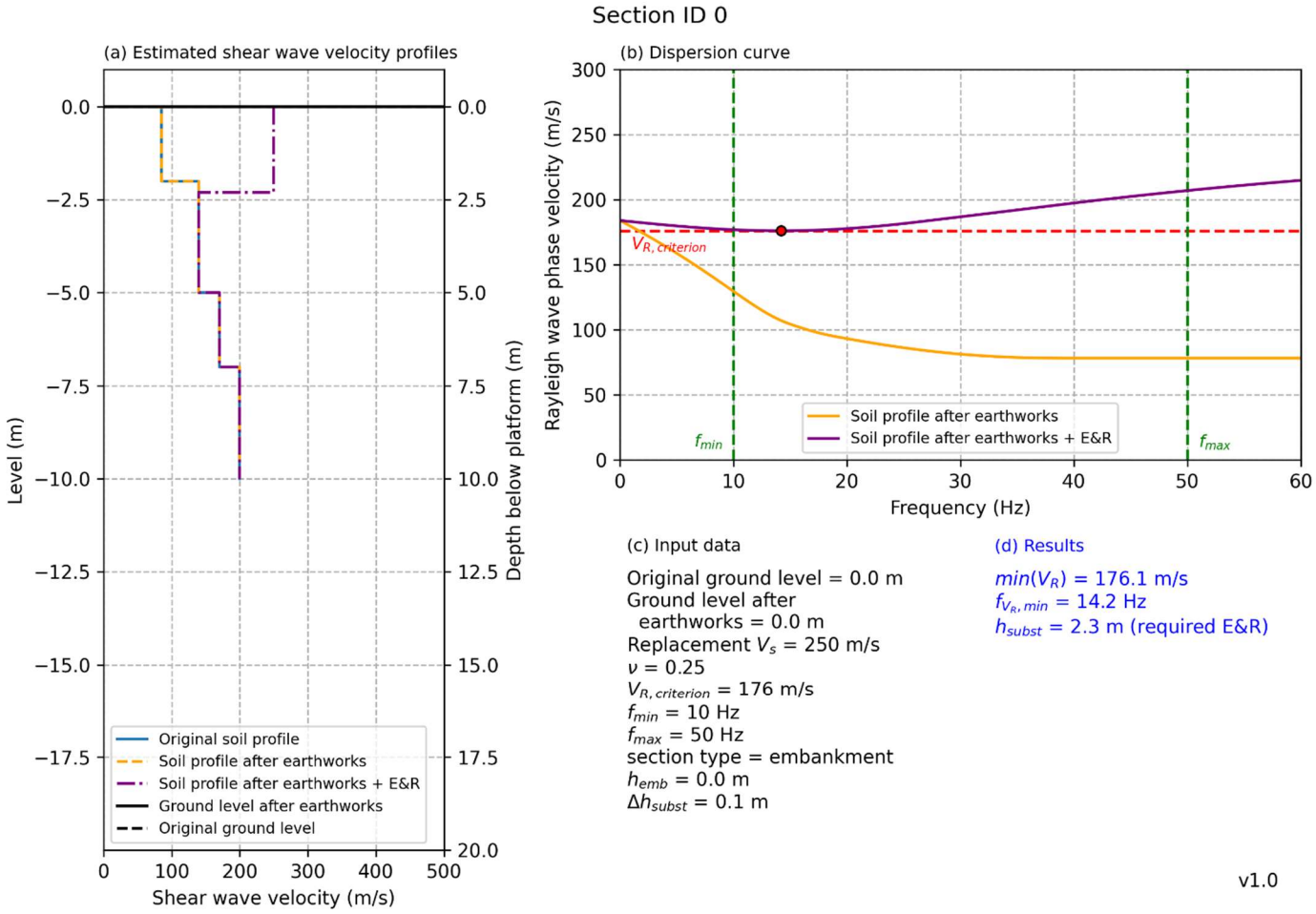


Figure C-24: South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

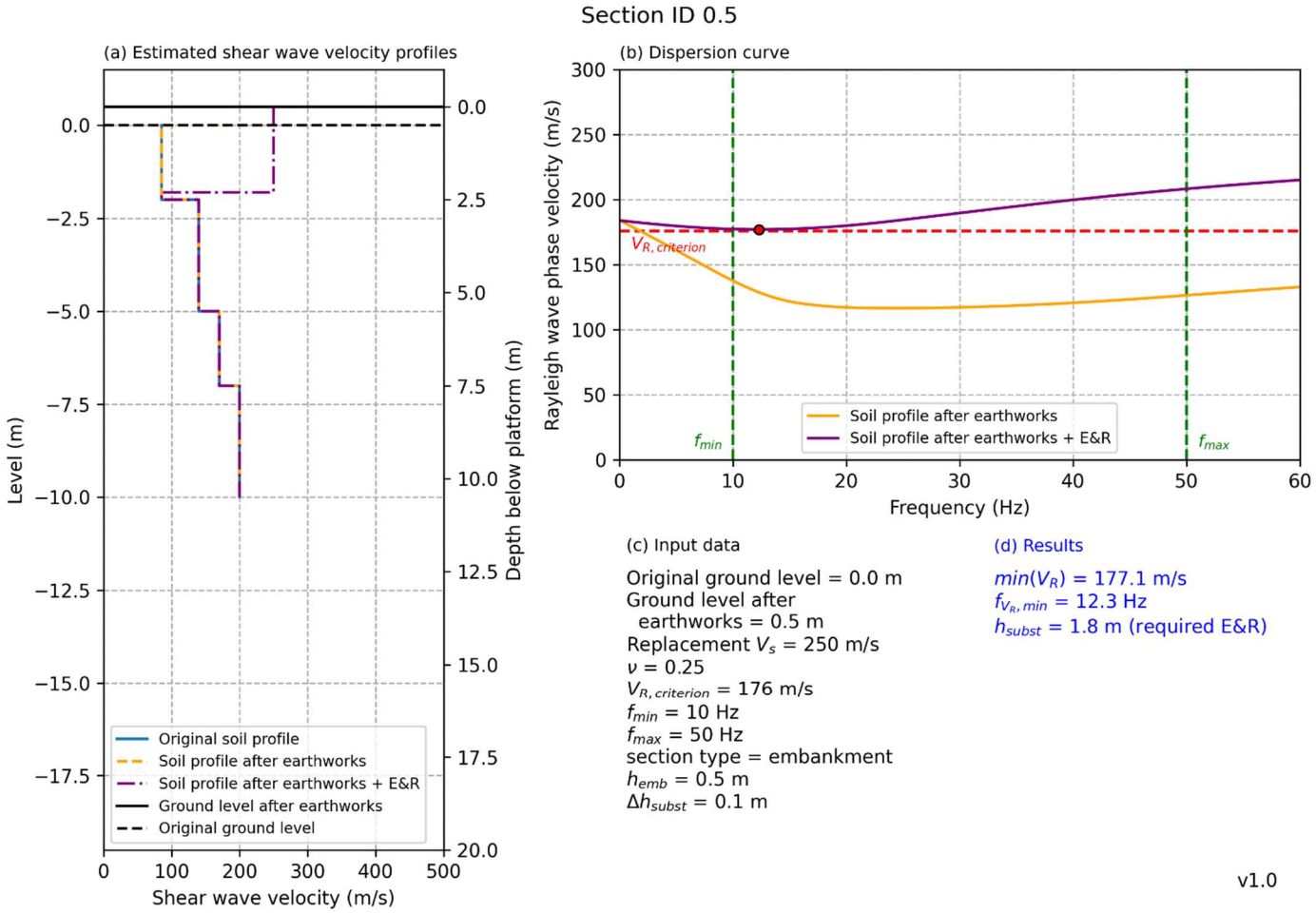


Figure C-25: South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

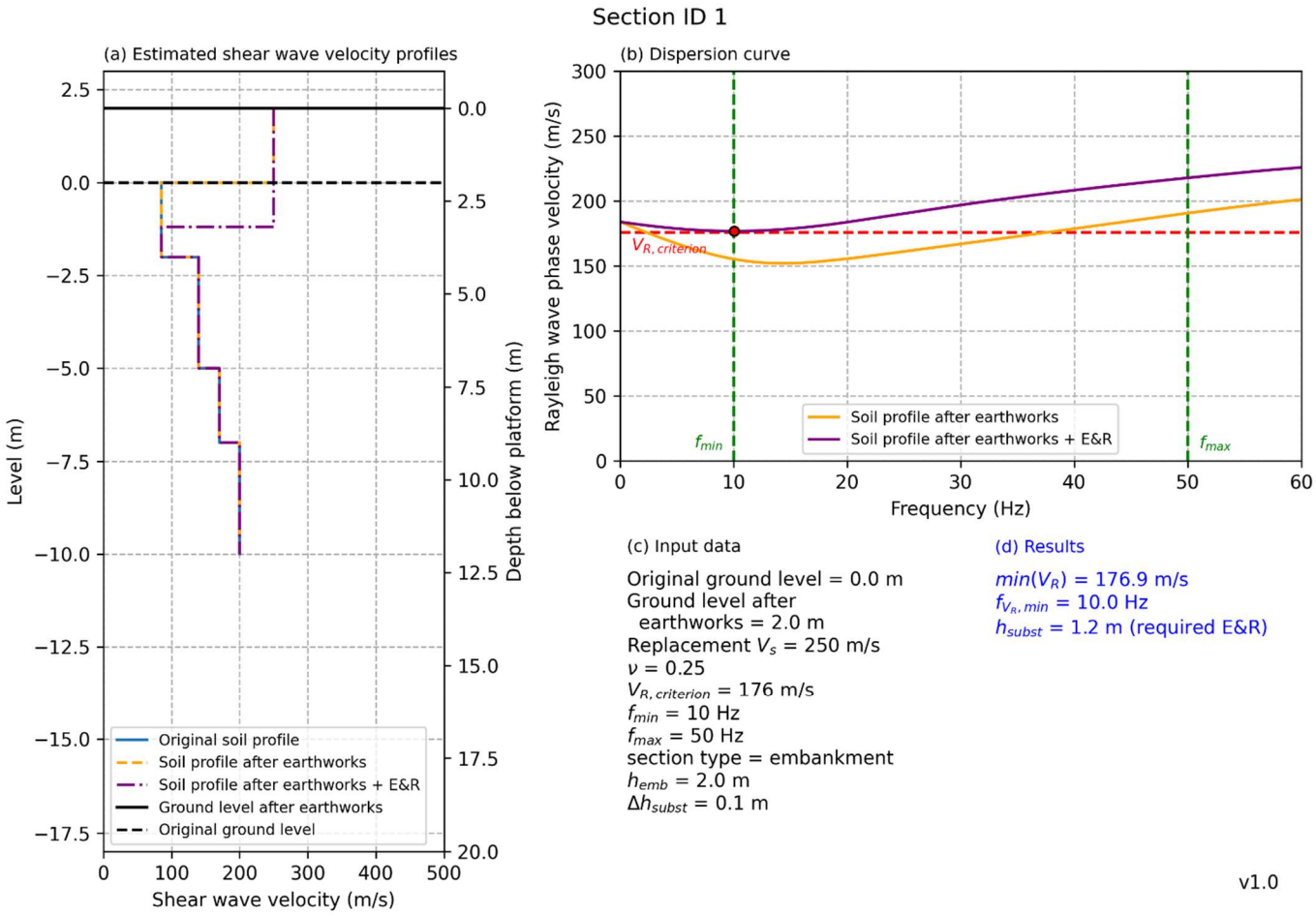


Figure C-26: South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

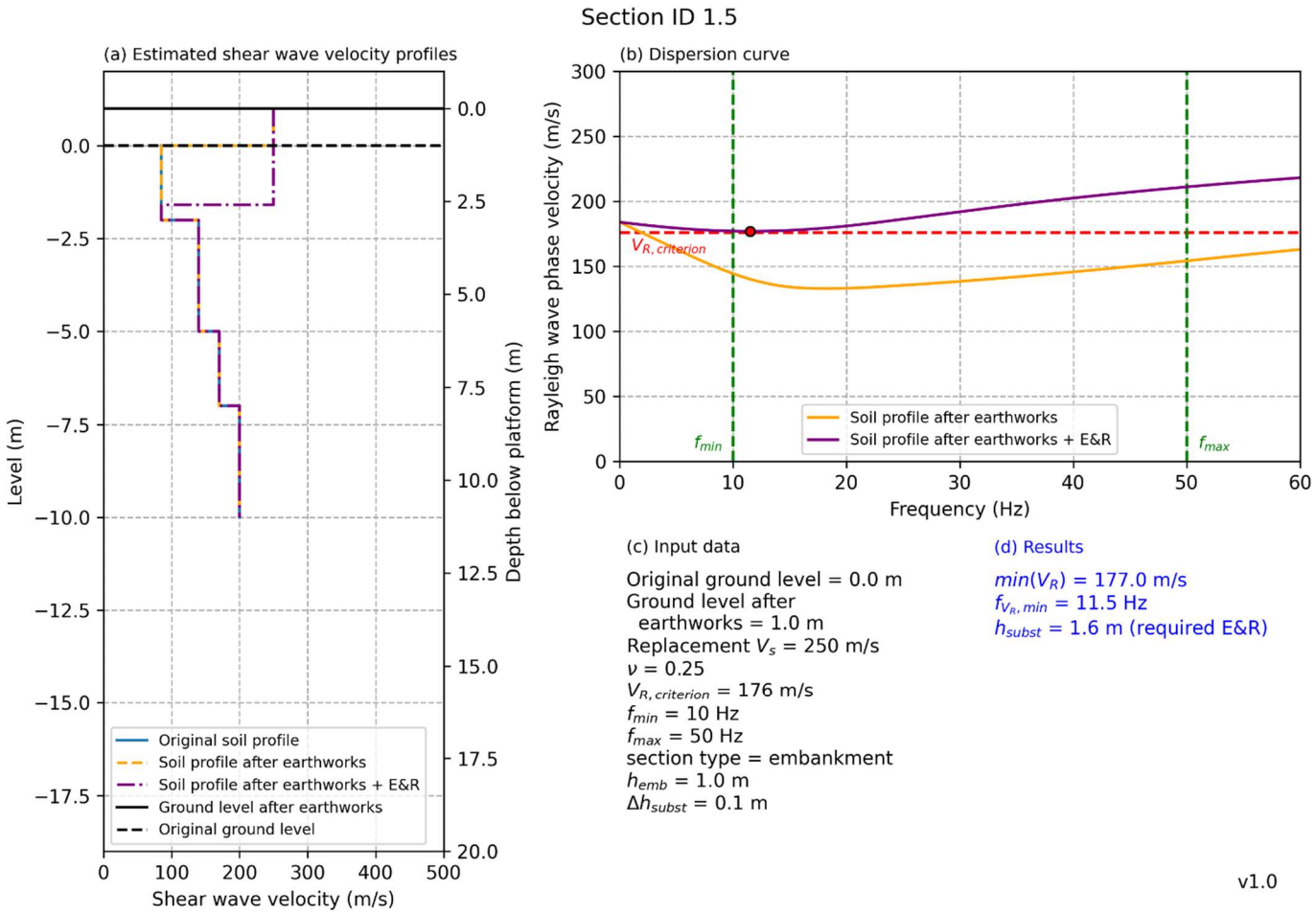


Figure C-27: South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

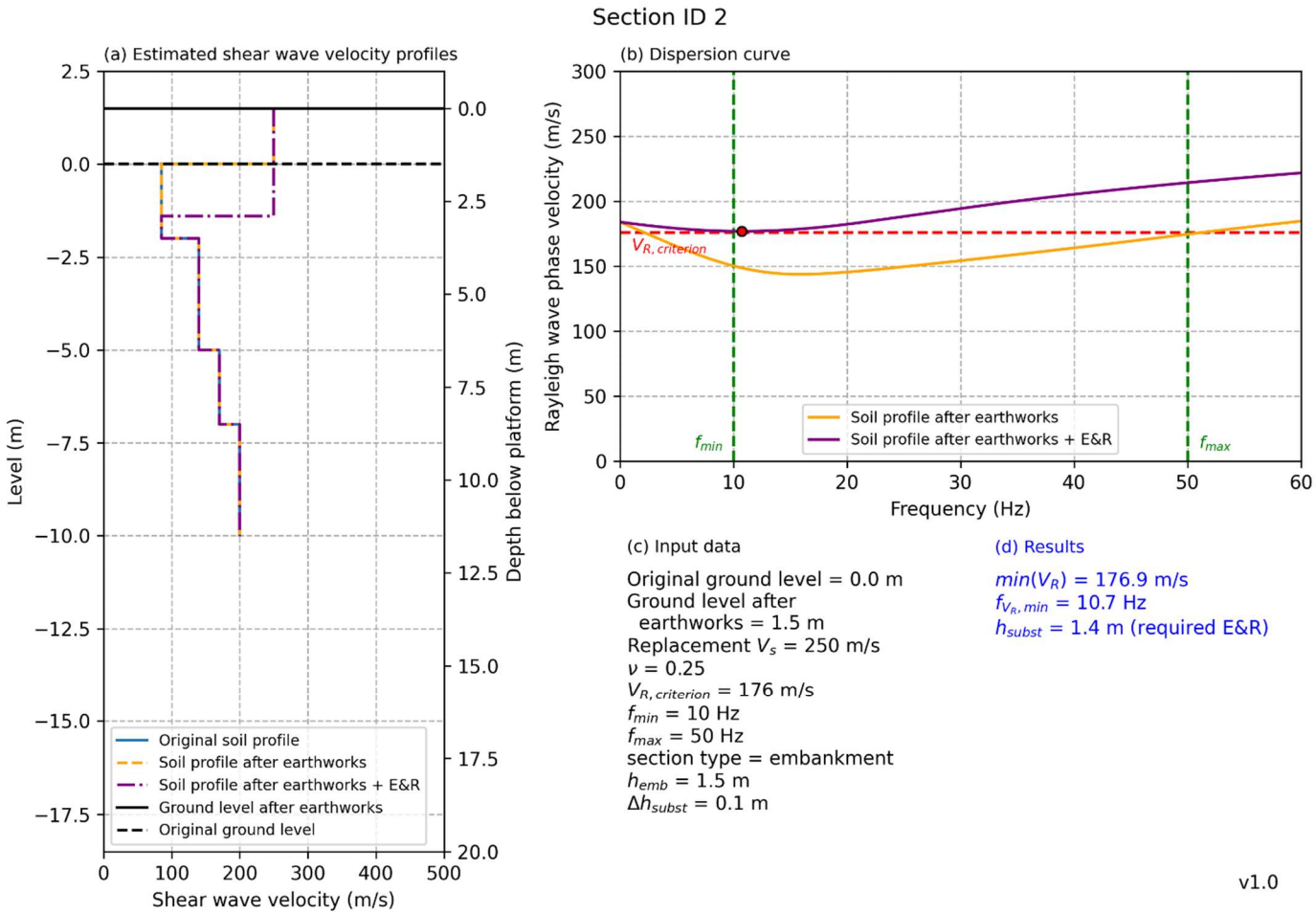


Figure C-28 South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

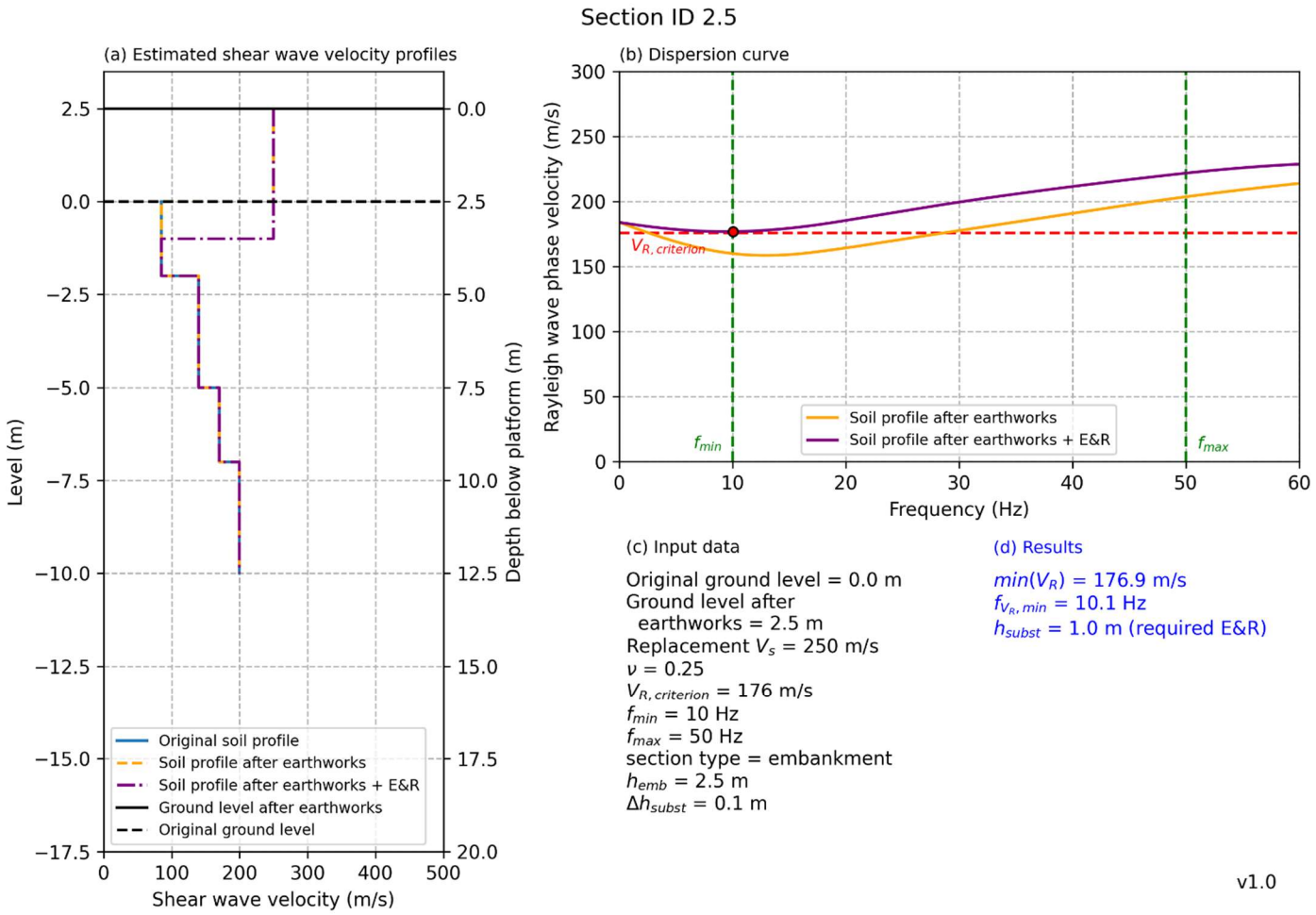


Figure C-29: South Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

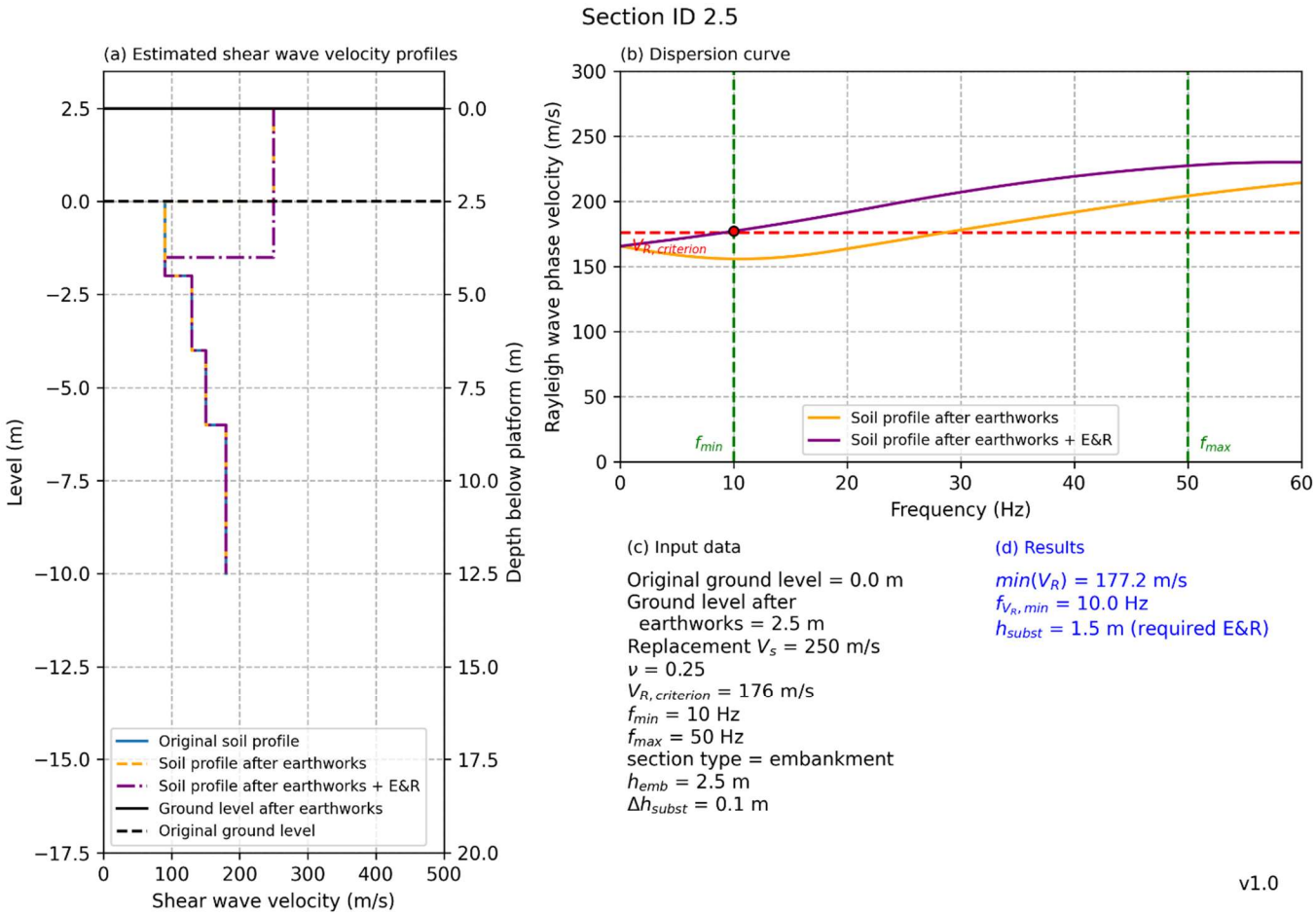


Figure C-30: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

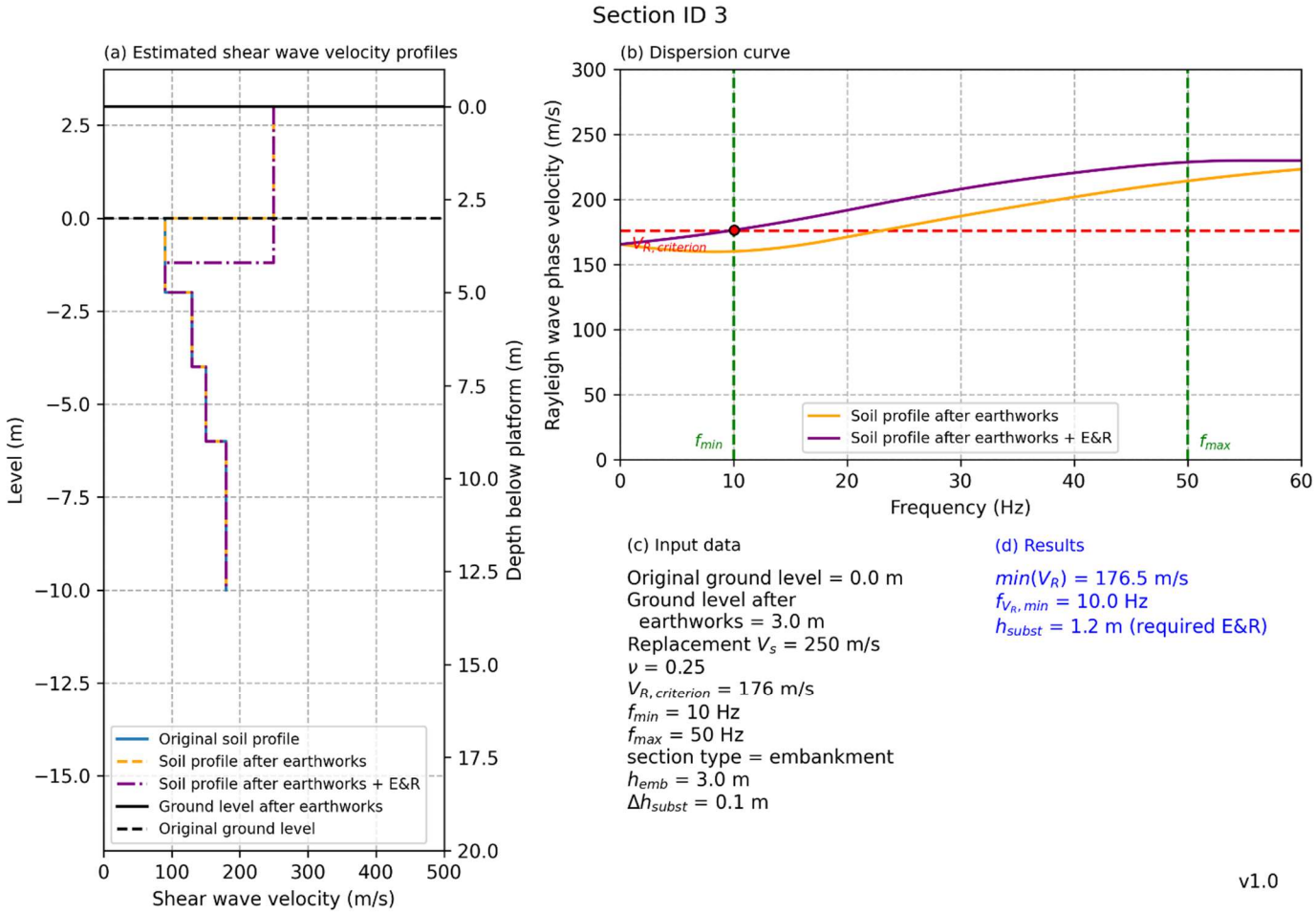


Figure C-31: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

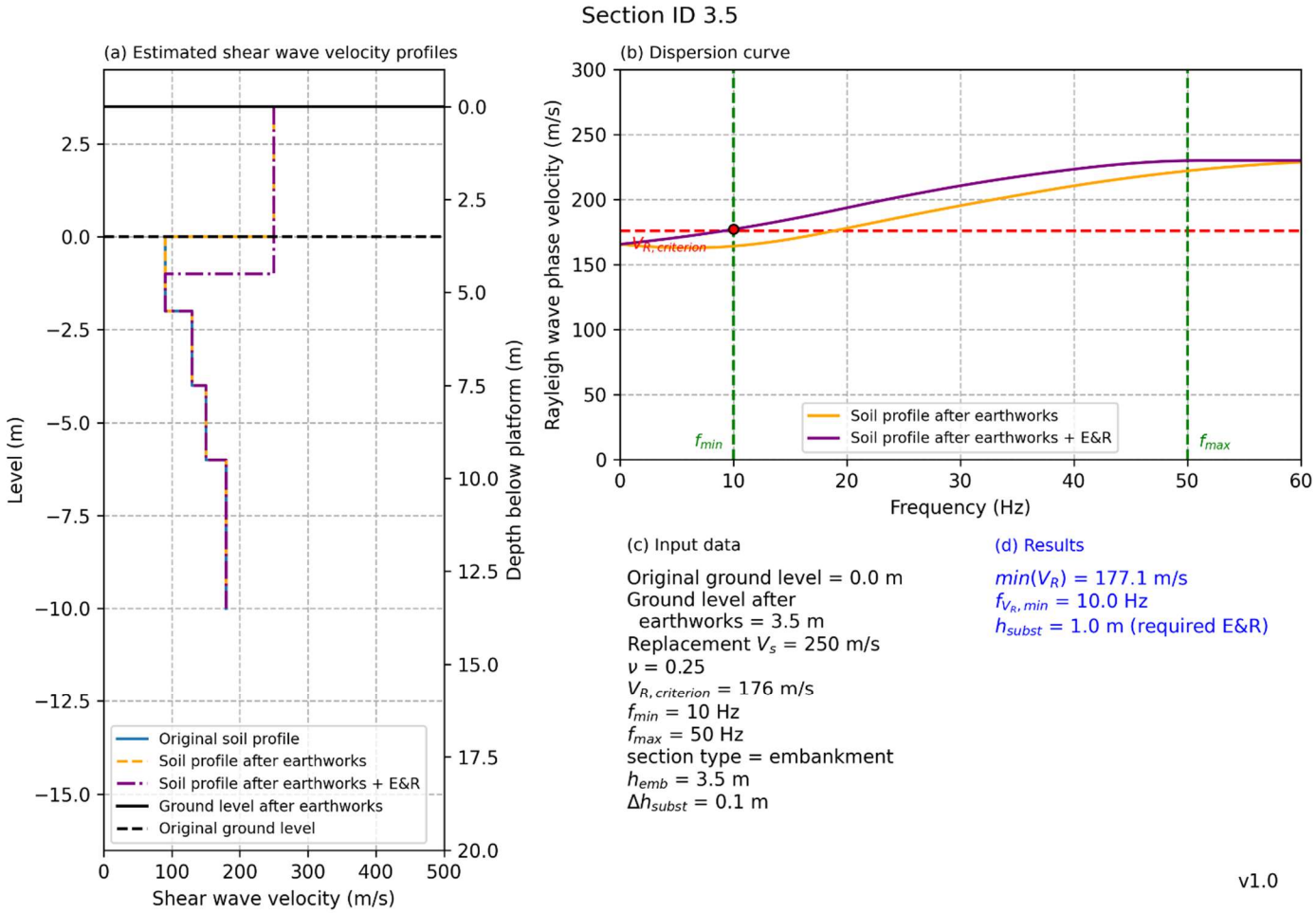


Figure C-32: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

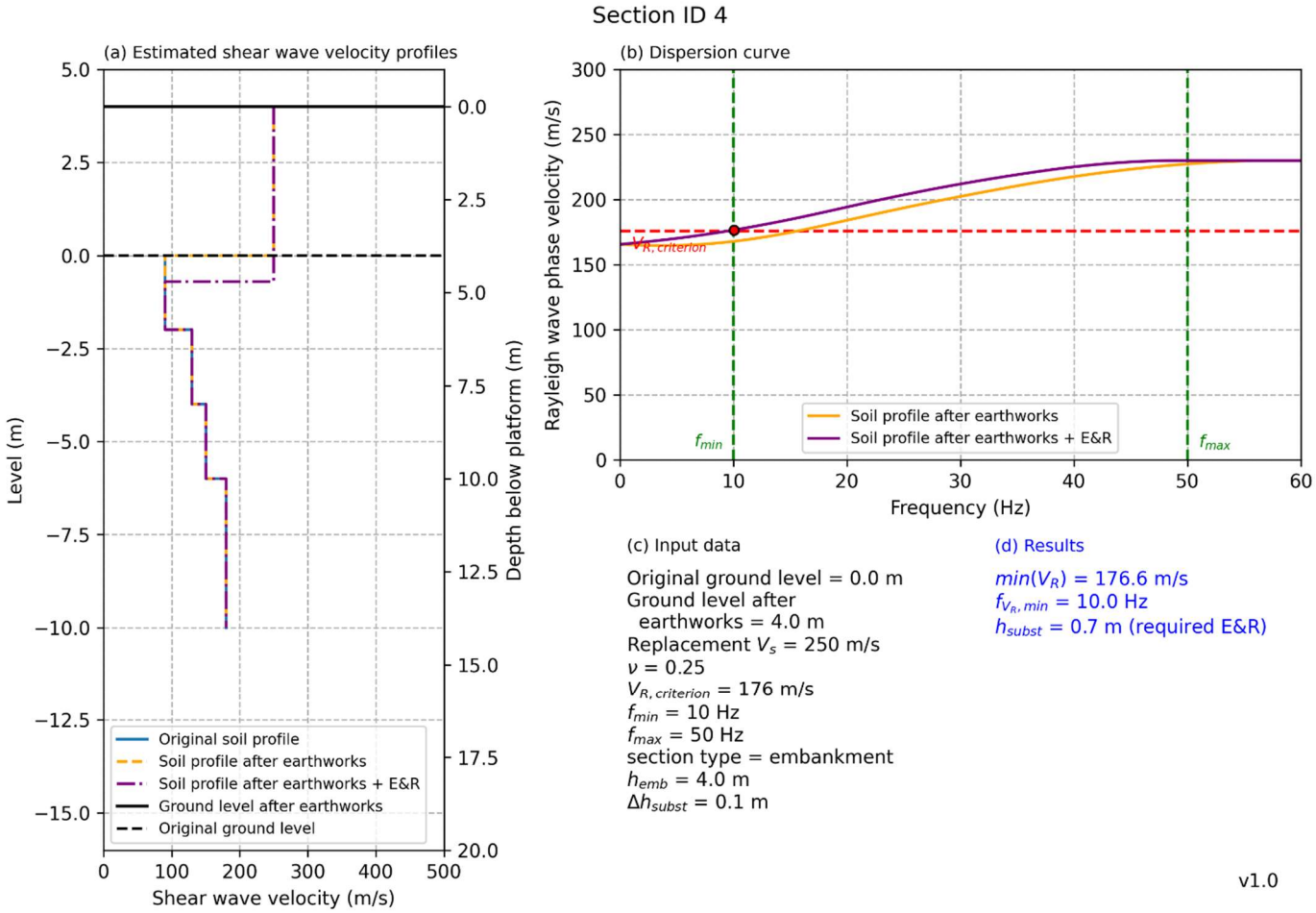


Figure C-33: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

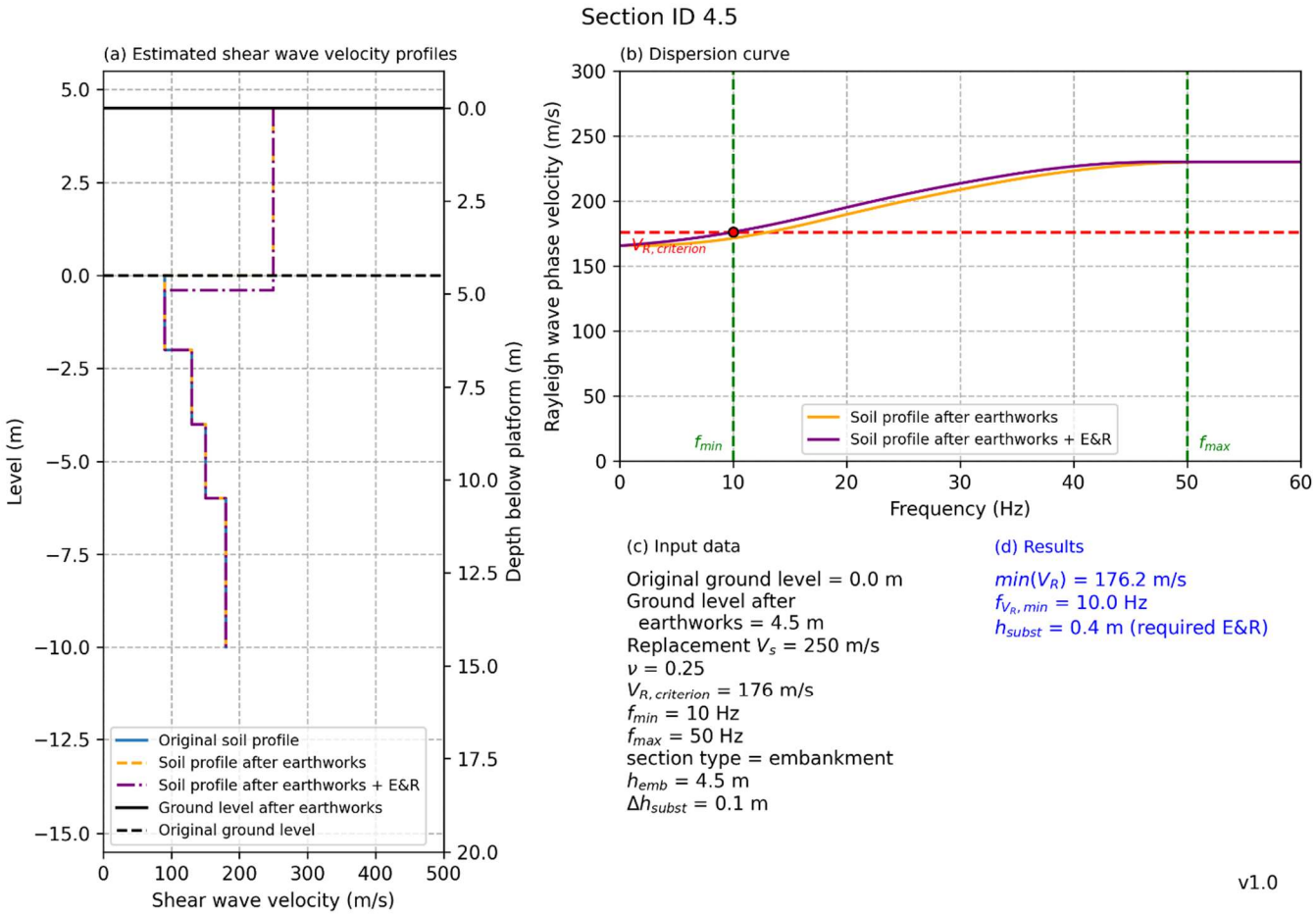


Figure C-34: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

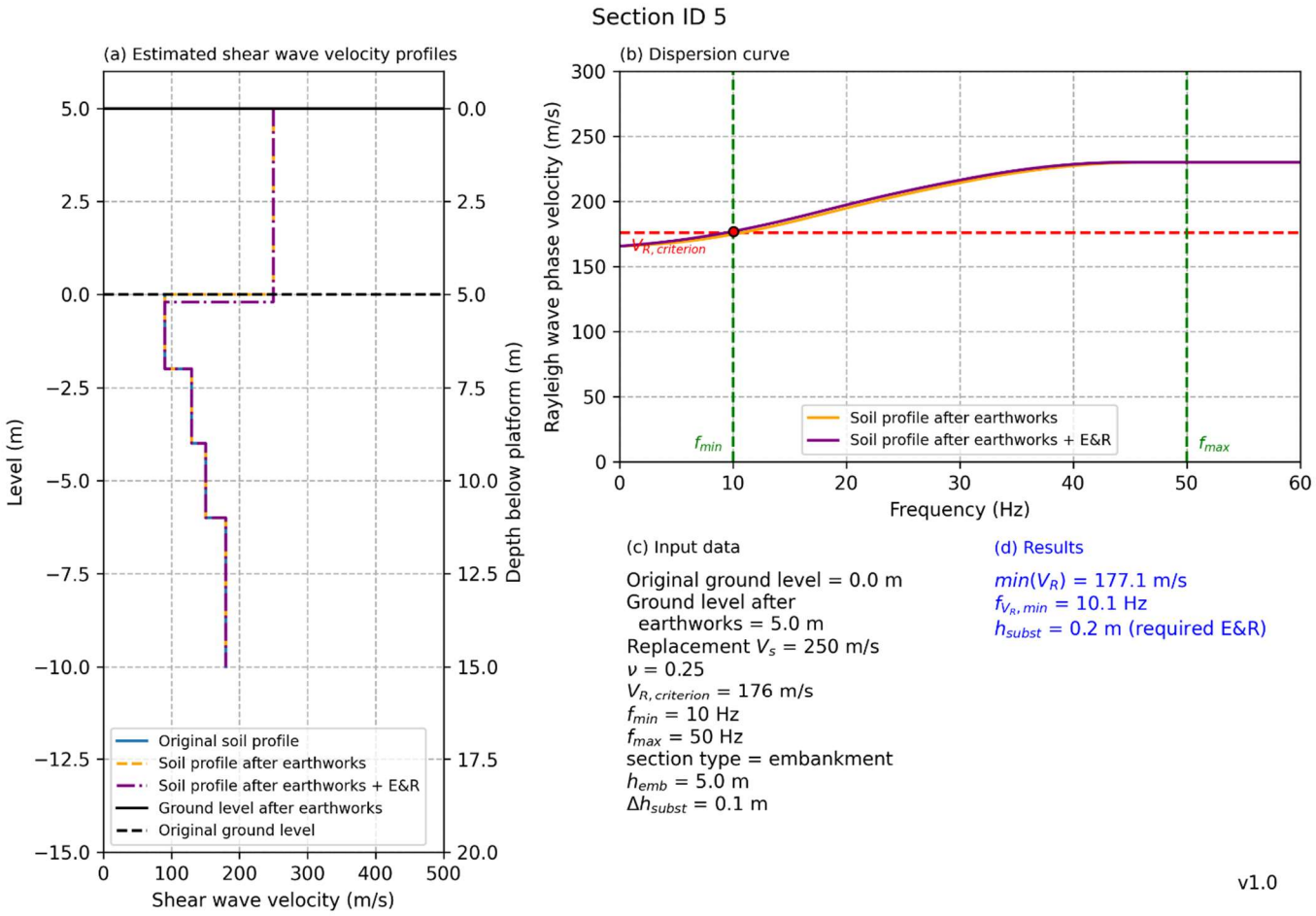


Figure C-35: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

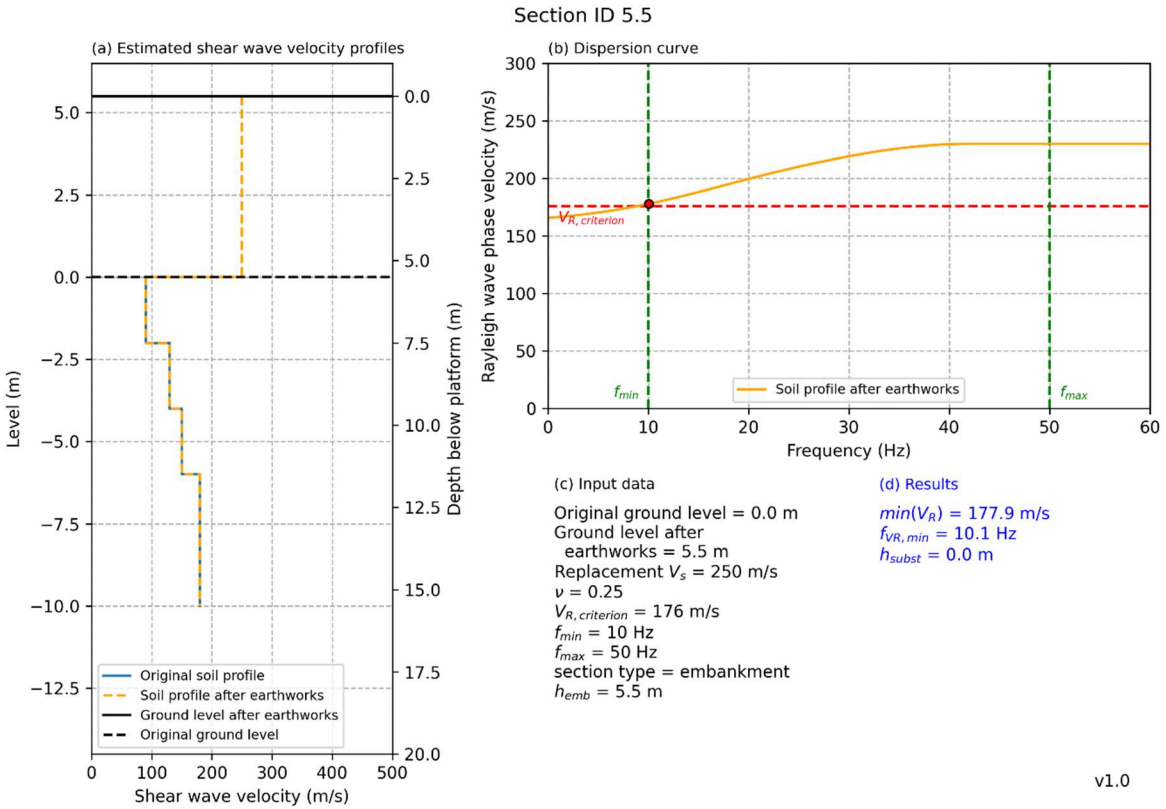


Figure C-36: Central Ground Model – Dynamic Performance Mitigation Result (E&R)

HS2 Ltd - Code 1 - Accepted

Appendix D

Geotechnical Risk Register

Number of Pages: 19

HS2 Ltd - Code 1 - Accepted

Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
GEO_AA_0205	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Eliminated/Avoided	Faulting of unknown displacement being present	Geological process	Resulting in a reduction in stability of earthwork, increased settlement, and lower formation CBR's resulting in an increase in pavement thickness	9	Reduce	Based on results from ground investigation works, no faults are encountered in this area.	Construction and Manufacture	3
GEO_AA_0206	CDM_57076	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Low strength relic shear surfaces may be present within Kimmeridge Clay Formation within ML061-CR014 between 19.40-20.00 mbgl.	Downslope movement or historical slope failures during extreme climatic conditions (periglacial/glacial environment).	Potential slope failures and instability for the earthworks.	9	Reduce	1.Conservative parameters have been adopted for the design. 2.Control by inspection and logging of formation and excavations to identify the presence of these surfaces during construction, and application of mitigation to stabilise these surfaces, refer to specification document for detail	Construction and Manufacture	4
GEO_AA_0207	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Eliminated/Avoided	Locally hard bands / hard ground are identified within strata with widely variable strength and consolidation characteristics due to the juxtaposition of harder and softer ground	Due to presence of more competent limestone bands/ layers within the Portland Group and Ancholme Group	Delays in construction to mobilise correct plant and equipment to form excavations/ cutting.	6	Reduce	1.Inspection and logging of formation and excavations to identify the presence of any hard surfaces during construction. 2. Mobilisation of correct plant and equipment to cope with changing ground conditions. 3. No deep excavations - withdrawn	Construction and Manufacture	3
GEO_AA_0208	CDM_48894	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Soft and compressible ground may be encountered in the formation to earthworks and road pavements	Low strength and variable ground conditions, typically from Alluvium, Head, and low strength clay material.	Resulting in increased total and differential settlements, stability and lower CBR values than designed for.	9	Eliminate	1. Conservative parameters have been adopted for design. 2. Inspection and testing of formation and excavations to identify the presence of any soft ground during construction. 3. Excavate and replace for any identified soft ground during construction in accordance with the Specification. 4. Mitigation in the form of basal geogrid reinforcement and 300mm class 6C1 starter where soft soil has	Construction and Manufacture	4

HS2 Ltd - Code 1 - Accepted

Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
									been identified (Appox. Ch.61+275 to 61+400)		
6GEO_AA_0209	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Shrink-swell clays may be present within the Kimmeridge Clay and Purbeck Group with high potential for volume change.	Changes in volume arising from changes in water content from seasonal variation from the action of vegetation and climate	1. Shrinkage and swelling of formation soils and causing changes in ground stress and the development of earth pressures, ground movement and cracking in soil deposits. 2. Longitudinal and lateral shrinkage / swelling in subgrade formation leading to uneven road and drainage profile and unacceptable total and differential settlements. 3. Development of vadose zone at the surface of soil slopes leading to softening of the near surface soils to critical state condition and premature failure of earthworks	12	Reduce	Provision of appropriate thickness of pavement construction where vulnerable pavement surfaces are proposed. Vadose Zone adopted in accordance with the HS2 Technical Standards - Earthworks in slope stability design incorporate critical state soils and increased pore pressures	Construction and Manufacture	6
GEO_AA_0210	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Chemically aggressive ground is present	The presence of high sulphate minerals within the soil and rock	Chemical attack on and degradation of buried concrete and metallic utilities. Limits on use as backfill and for stabilisation to improve the soils performance as fill material	9	Control	Suitability for use as backfill tested and checked on site as per the SCEW for IDR208.4 (1MC06-CEK-AU-SPE-CS04-000003)	Construction and Manufacture	4

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GEO_AA_0211	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Withdrawn	Groundwater will be encountered at a shallow depth	Due to the possible presence of a high groundwater table	The need for groundwater control measures	12	Control	Duplicate risk	Construction and Manufacture	6
GEO_AA_0212	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Eliminated/Avoided	Flood risk on flood plain.	Flooding from nearby watercourses and groundwater flooding	Flooding causing embankment instability and erosion of materials.	12	Control	There is no risk of flooding to the embankment footprint as the Sedrup North watercourse is moved away at a minimum offset of 8m from the toe.	Construction and Manufacture	4
GEO_AA_0213	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Potential for unforeseen land contamination due to limited amount of information available.	Limited information and nearby LQ areas.	Unforeseen contamination leading to aggressive ground	6	Control	1. All GI and available information has been reviewed for potential contamination. 2. Ensure formation is inspected prior to placement of fill material of highway to ensure no Made Ground is present.	Detailed Design	4
GEO_AA_0214	CDM_48897	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	saturation of the exposed formation to the asset will occur	adverse weather or prolonged wet weather in winter	Softening and degradation of the formation leading to lower shear strengths and CBR values than that assumed for the design	16	Reduce	1. Inspection and testing of formation and excavations to identify the presence of any soft ground due to high groundwater table during construction. 2. Excavate and replace for any identified soft ground during construction in accordance with the Specification.	Construction and Manufacture	4
GEO_AA_0889	N/A	HS2-00002BJSK - Maintenance Access Road to Pond (South Arm off Calley Farm - South)	Risk Reduced ALARP	Drainage trenches being dug for highway drainage will damage the geogrids within the earthwork	Inappropriate digging techniques being used	Geogrid layers being displaced or removed from the highway earthwork resulting in instability / slope failure	9	Reduce	1. Design has conservatively placed all geogrids to reduce the risk of clashing 2. Construction drawings indicate where the potential clashes may occur 3. Where possible trenches should be hand dug and geogrids exposed to be manually trimmed 4. Reinstatement of trimmed or damaged geogrid to comply with the	Construction and Manufacture	4

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									manufactures recommended overlap.		

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GEO_1886	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Reduced ALARP	Incorrect ground model and parameters adopted for design	Lack of GI data available at this asset location	For deep foundations: Longer or shorter pile lengths proposed. Where piles are shorter than required, there is a significant H&S impact due to failure. Where piles are excessively longer than required, there will be an impact on the cost and programme. For shallow footings: excessively large foundations. Inadequate geotechnical parameters could also lead to excessive settlement of the foundations and potential bearing capacity failure of the foundations.	16	Reduce	1) No piles foundations are required for the bridge abutment. 2) Additional detailed GI targeting SBH/19 has been scheduled but results not available yet. Once results are available, design assumptions will be checked and communicated to the client via a RIA. 3) A Rapid impact assessment will be carried out once the scheduled GI becomes available to inform the client the results of the new GI. 4) GI targeting the L1 Asset is available in the vicinity of this SBH19. This GI has been used for the design of SBH19. 5) Detailed design carried out based an efficient design using characteristic values/ground conditions as a cautious estimate of the value governing a limit state based on the SMUGC document. However, the design has been adjusted to be more cautious to address the uncertainty in ground conditions and rely upon the existing limited scope of GI. 6) Mitigation measures such as excavate and replace has been included under the abutment footprints to reduce the risk to uncertainty of ground conditions.	Operations and Maintenance	3
GEO_1887	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Reduced ALARP	Shrink/Swell in clays	High plastic clay may undergo volumetric change as a result of moisture content changes	excessive movements resulting in settlement tolerances being exceeded	9	Control	1. Further GI at abutment location has been proposed. Results from the GI will be reviewed as part of the RIA. 2 Formation of excavated clays to be protected prior to backfilling as per standard earthworks procedures specified in the SCEW. 3 High plasticity clays and/or soft clays will be excavated immediately under the earthworks blister supporting the bridge abutments. The excavated areas will be filled with general fill material (granular). 4 Current design does not include the excavation and exposure of natural ground in the long term. 5. Complete pre construction condition survey to assess condition of third party structures.	Construction and Manufacture	4
GEO_1888	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Reduced ALARP	Chemical attack on concrete	Elevated levels of sulphates are known to occur within the Head deposits and Gault Formation	Deterioration of the foundation of the overbridge over time leading to increased settlement/ differential movement and eventually structural failure of the foundation.	9	Control	1) Bridge abutments will be ground bearing and deep foundations will not be required. Abutments to bear on general fill material. 2) BRE Suite SD1 Concrete Classification testing to be scheduled within the GI targeting SBH19.	Operations and Maintenance	4

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									3) BRE Suite SD1 results for the L1 asset used for design until the supplementary GI becomes available.		
GEO_1889	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Reduced ALARP	Incorrect groundwater level adopted for design	No groundwater monitoring available at asset location or in the vicinity.	1) inaccurate modelling of water for design, which may affect the proposed foundation solution. 2) The foundation solution may not be able to withstand excess pore pressures and uplift forces. 3) Not considering these can lead to failure of the foundations, which will have a significant cost and H&S impact.	16	Control	1) Groundwater monitoring available in the vicinity of the asset. Groundwater level derived from boreholes in the vicinity of the asset 2) Design groundwater level adjusted to adoption of conservative groundwater levels in design for stability due to the lack of monitoring at the abutment location. Design groundwater at 1.0m below existing ground level. This will be approximately the level at which the track drainage will be located. 2) Identification in CDM register and method statements about the requirement for groundwater control measures and need for temporary works to ensure stability of excavations where required.	Construction and Manufacture	6
GEO_1890	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Reduced ALARP	Soft and compressible ground	Presence of Relic Shear Surfaces in Head deposits	1) Local slips which may destabilise shallow foundations. 2) slipped material causing an increased lateral load on the piles 3) Shallow slopes needing to be stabilised.	6	Control	1) Available GI does not show the presence of potential relic surfaces in the vicinity of the asset. However, further GI at abutment location has been proposed. Results from the GI will be reviewed as part of the RIA. 2) Detailed design carried out based an efficient design using characteristic values/ground conditions as a cautious estimate of the value governing a limit state based on the SMUGC document. However, the design has been adjusted to be more cautious to address the uncertainty in ground conditions and rely upon the existing limited scope of GI. 3) Mitigation measures such as excavate and replace of the HEAD deposits has been included under the abutment footprints to reduce the risk to uncertainty of ground conditions and relict surfaces. 4) Contractor to have an experienced geotechnical engineer to inspect excavations during the construction works. 5) If relic surfaces are found, the contractor to treat them as per standard HS2 details for relic surfaces.	Construction and Manufacture	4

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GEO_1891	N/A	HS2-000001293 - Bridleway SBH/19 Overbridge	Risk Eliminated/Avoided	Difficulties in piling	Presence of hard ground within the Portland Group	The use of driven piles and CFA piles may not be possible, resulting in bored piles as the pile solution. This may lead to a higher target cost.	12	Eliminate	1) No piles foundations are required for the bridge abutment.	Operations and Maintenance	3
GEO_1560	CDM-0739, CDM-0751, CDM-0750	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Instability of embankment slopes and/or unacceptable total or differential settlements.	Soft ground is deeper than assumed under the embankment and at transition zones.	Greater settlements than anticipated can cause major issues during construction along the whole line (slab track installation, etc.). Locally it can result in slope stability problems along the embankment and at transition zones. Furthermore, differential settlements in the alignment can disrupt the operation of the future railway.	16	Reduce	1. The ground investigation data was gathered and interpreted into a ground model to better understand the extent of potentially soft material. COMPLETE. 2. Appropriately conservative design parameters were drawn from the available ground investigation data. COMPLETE. 3. Any additional data in subsequent datadrops/ communications from site team were incorporated into the design. COMPLETE. 4. Design of Excavate and Replace including culvert transitions. COMPLETE. 5. Provision of Instrumentation and Monitoring to verify design and ground movements. COMPLETE. 6. Undertake monitoring during construction of earthworks including culvert transitions and mitigate where appropriate during construction. TO BE ACTIONED BY THE CONTRACTOR	Construction and Manufacture	4
GEO_1561	CDM-0743	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Relic shear features may be encountered on sidelong ground and reactivated under embankment loading causing instability.	Reactivation of relic shear surfaces due to embankment loading and excavation during construction.	Reactivated shear surface can cause potential slope failure and instability for the earthwork.	4	Reduce	1. Undertake ground investigation. COMPLETE. 2. Identify the risk of potential relic shear surfaces in the Supplementary GIR. COMPLETE. 3. Undertake detailed design with a reasonably conservative ground model. COMPLETE. 4. The contractor shall maintain a watching brief by a suitably qualified geotechnical engineer/engineering geologist during construction to identify any relic slip surfaces. OUTSTANDING. 5. The contractor shall perform on-site mitigation measures which comprises Excavate and Replace of softened material and across relict shear surfaces (shear keys), and/or provision of basal reinforcement to strengthen / stiffen base of earthworks, if periglacial features had been identified. TO BE ACTIONED BY THE CONTRACTOR.	Construction and Manufacture	2

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GEO_1562	CDM-0745, CDM-0744	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Shrink-swell clays may be present within the Gault Clay, Kimmeridge Clay and Purbeck Group with high potential for volume change.	Seasonal variations in moisture content.	Could cause unacceptable total and differential settlements.	9	Reduce	1. Undertake further ground investigation to confirm the characteristics of potentially swellable soils and to confirm need for mitigation measures. COMPLETE. 2. Provision of Excavate and Replace mitigation. COMPLETE. 3. Design according to HS2 standards. Compliance with the suitability rules of soils for substructure works. COMPLETE. 4. Provision of Instrumentation and Monitoring to verify design and ground movements. COMPLETE. 5. Undertake monitoring during construction of earthworks including culvert transitions. TO BE ACTIONED BY THE CONTRACTOR.	Construction and Manufacture	3
GEO_1564	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Elevated levels of sulphates, sulphides and chlorates or high/low pH will speed the degradation of construction materials (including soil and rock) and / or create an adverse environmental impact	Primarily as a result of the mineralogy of the clay soils and exacerbated by the presence of mobile groundwater and /or the introduction of oxygen from excavations. May also be impacted by anthropogenic contaminants in the soil.	1. Presence of soil and groundwater chemistry that may result in aggressive ground conditions to receptors including construction materials, soil, and groundwater resources 2. Changes in physical properties of soil due to physical or chemical reactions including softening of ground 3. Unstable earthworks features (cuttings / embankments) and/or long term consolidation or heave of ground impacting on railway assets 4. Limiting the use of as-dug or stabilised materials resulting in increased costs due to large haulage distances or importation of selected fill material	12	Reduce	1. Undertake ground investigation, including BRE SD1 suites for sulphate design. COMPLETE. 2. Provide an assessment of the ground chemistry with respect to sulphate bearing strata. COMPLETE. 3. Provide reasonably conservative design sulphate concrete and ACEC classes. COMPLETE. 4. Update the sulphate design to incorporate the results of the detailed design GI at L3. COMPLETE.	Construction and Manufacture	3
GEO_1565	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Shallow groundwater levels. Groundwater strikes were encountered in several investigation locations across the asset. Some of the strikes were found to be sub artesian.	Decreased effective vertical stress can impact on slope stability. Inflow to temporary excavations required for mitigation works.	Reduction in effective strength parameters causing slope failure in long term. Inundation of temporary excavations requiring dewatering measures.	12	Reduce	1. Undertake ground investigation to help determine the local ground conditions and inform detailed design. COMPLETE. 2. Undertake conservative design (adopting cautious groundwater levels). COMPLETE. 3. Temporary works assessments to be undertaken during site works and managed accordingly e.g., RAMS. TO BE ACTIONED BY THE CONTRACTOR AND THE TEMPORARY WORKS DESIGNER.	Construction and Manufacture	2

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GEO_1566	CDM-0766, CDM-0767	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Flood risk on flood plain.	High groundwater from flooding.	Flooding causing embankment instability and erosion of materials.	2	Reduce	1. The Scheme design flood modelling indicates that the risk of flooding is low. COMPLETE. 2. For detailed design, the risk of flooding is negligible as the Sedrup North watercourse is moved away at a minimum offset of 8m from the embankment toe. COMPLETE	Construction and Manufacture	2
GEO_1567	CDM_0770, CDM_0771	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	High ground vibration on proposed embankment and existing natural ground.	Rayleigh waves generated by high speed train on soft ground at formation levels providing inadequate stiffness.	High ground vibration can cause environmental problems and degradation in shear strength of the soil over time.	8	Reduce	1. Additional ground Investigation to determine ground stiffness including direct CSW/MASW, down-hole geophysics etc. COMPLETE. 2. Field Trial to validate method of analysis and design correlations prior to commencement of detailed design. COMPLETE. 3. Undertake conservative design in accordance with the Design Basis Statement. COMPLETE. 4. Construction validation testing to confirm performance criteria has been achieved. TO BE ACTIONED BY THE CONTRACTOR. 5. Account for soft deposits identified by excavate and replacement. TO BE ACTIONED BY THE CONTRACTOR.	Construction and Manufacture	2
GEO_1568	CDM-0762, CDM-0763	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Seismic conditions in the area of the asset	seismic movements	slope instability	8	Reduce	1. Settlement predictions on either side of the fault have been carried out. The differential settlements on either side of the fault have not been considered as a significant issue. COMPLETE 2. A competent Geologist/ Geotechnical Engineer shall inspect the formations and identify any faults/ geohazards and notify the designer. The designer to carry out suitable mitigation, if necessary. ACTIONED BY THE CONTRACTOR.	Construction and Manufacture	4
GEO_1569	CDM-0746	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Running sand conditions in Head deposits, Portland Sand Formation and Purbeck Group. Groundwater strikes were encountered in several investigation locations across the asset. Some of the strikes were found to be sub artesian.	Loose sand is present below groundwater table.	Instability to temporary excavations and excess water ingress.	6	Reduce	1. Undertake ground investigation to help determine the local ground conditions and inform detailed design. COMPLETE. 2. Undertake Excavate and Replace to a depth of 2m bgl based on conservative design adopting shallow groundwater level. COMPLETE. 3. Hydro-geological impacts are considered negligible. COMPLETE.	Construction and Manufacture	2

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GEO_1570	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Groundwater Drawdown in Portland Group which is designated as a principal/secondary aquifer.	temporary excavation	effect on adjacent mature woodlands	6	Reduce	1. Undertake ground investigation to help determine the local ground conditions and inform detailed design. COMPLETE. 2. Undertake Excavate and Replace to a depth of 2m bgl based on conservative design adopting shallow groundwater level. COMPLETE. 3. Hydro-geological impacts are considered negligible. COMPLETE.	Construction and Manufacture	2
GEO_1571	CDM-52988	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Faulting of unknown displacement being present	Geological process	• Variable strata and material properties. • Faulting may form a barrier to, or act as a conduit for groundwater flow. • Reduced bulk strength associated with fault breccias, where present. • Discontinuities of varying and potentially unfavourable orientation, reducing the strength and slope stability significantly. • Differential settlement may occur where assets span faulted materials.	16	Reduce	1. Undertake ground investigation to help to determine the local ground conditions and inform detailed design. COMPLETE. 2. Undertake conservative design (adopting cautious ground models and material parameters). COMPLETE. 3. Observational approach during construction to identify potential signs of further faulting. TO BE ACTIONED BY THE CONTRACTOR 4. Further excavate and replace may be required. TO BE ACTIONED BY THE CONTRACTOR	Construction and Manufacture	4
GEO_3509	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Locally hard bands / hard ground are identified within strata with widely variable strength and consolidation characteristics due to the juxtaposition of harder and softer ground	As a result of the depositional environments and weathering of different materials.	1. Difficulties in installing drainage or service trenches 2. Potential for differential settlement	15	Reduce	1. Undertake ground investigation. COMPLETE. 2. Provide excavation techniques for hard digging. COMPLETE. 3. The designer complete the detailed design of assets sensitive to differential settlement with a moderately conservative ground model which incorporated the risk of hard spots, with appropriate mitigation solutions. COMPLETE. 4. The contractor shall provide a watching brief by a suitably qualified geotechnical engineer/engineering geologist for excavations where hard spots are identified during construction and select an appropriate method for excavation/mitigation where required. Hard spots that are identified which are beyond the scope of the detailed design shall be reported to the asset designer. TO BE COMPLETED.	Construction and Manufacture	1
GEO_3579	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Potential risk of soil and groundwater contamination associated with the Hartwell Landfill (LQ 11.04) (110m northwest of Asset).	Two LQs constraint are present approximately 110m northwest - Hartwell Landfill (LQ 11.04) and at approx. 200m - Former Hartwell clay, old Brick and tile works (LQ 11.03). No soil or leachate exceedances were	Potential risk to human health, controlled waters, and infrastructure. Reduction in water quality (controlled waters) / contaminated groundwater encountered may also result in increased costs and delays to	6	Reduce	1. Further contamination assessment to be undertaken during works. 2. Contaminated GW exceeding screening values may require de-watering, collect and treat/tanker and discharge consents. 3. Management of potentially	Construction and Manufacture	2

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					observed within the nearest LQ of the Hartwell Landfill (LQ 11.04), the LQ 11.03 recorded Made Ground within the area, However, due to the distance to the embankment it is unlikely that a plausible contaminant linkage exists. Six GI locations with Made Ground have been recorded within the LOD and LLAU of the embankment (ML061-TP438, ML061-TP437A, ML061-WS001, ML061-CT433, ML061-TP435 and ML060-TP427) with thickness ranging from 0.40m, 0.50m, 0.60m, 0.30m, 0.60m and 0.30m and containing anthropogenic inclusions of rare brick fragments. Made ground is likely to be removed with the strip and shallow excavation requirements for construction works.	the programme due to obtaining consent and remediation costs.			contaminated soils to be in line with the Remediation Outline Strategy and Verification Plan reports.		
GEO_3924	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Limitations to where material can be re-used in view of their chemical characteristics/re-use criteria and material incorrectly segregated cannot be reused	Two LQs constraint are present approximately 110m northwest - Hartwell Landfill (LQ 11.04) and at appr. 200m - Former Hartwell clay, old Brick and tile works (LQ 11.03). No soil or leachate exceedances were observed within the nearest LQ of the Hartwell Landfill (LQ 11.04), the LQ 11.03 recorded Made Ground within the area, However, due to the distance to the embankment it is unlikely that a plausible contaminant linkage exists. Six GI locations with Made Ground have been recorded within the LOD and LLAU of the embankment (ML061-TP438, ML061-TP437A, ML061-WS001, ML061-CT433, ML061-TP435 and ML060-TP427) with thickness ranging from 0.40m, 0.50m, 0.60m, 0.30m, 0.60m and 0.30m and containing anthropogenic inclusions of rare brick fragments. Made ground is likely to be removed with the strip and shallow excavation	risk of: 1. Failure to comply with regulatory guidance/MMP 2. Increased costs with having to re-do segregation of stockpiled material 3. Increased costs with having to dispose of material 4. Potential Environmental harm to receptors 5. Delay to programme	9	Reduce	1. Correct characterisation of soils 2. Management of potentially contaminated soils to be in line with the Remediation Outline Strategy and Verification Plan reports. 3. Materials Management Plan (MMP) to be adopted	Construction and Manufacture	2

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					requirements for construction works. Potential that Made Ground cannot be re-used within the scheme if heavily contaminated (i.e. if it fails the agreed Material Re-use Criteria)						
GEO_3927	N/A	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Risk of encountering contaminated soils and contaminated water during construction works.	Two LQs constraint are present approximately 110m northwest - Hartwell Landfill (LQ 11.04) and at appr. 200m - Former Hartwell clay, old Brick and tile works (LQ 11.03). No soil or leachate exceedances were observed within the nearest LQ of the Hartwell Landfill (LQ 11.04), the LQ 11.03 recorded Made Ground within the area, However, due to the distance to the embankment it is unlikely that a plausible contaminant linkage exists. Six GI locations with Made Ground have been recorded within the LOD and LLAU of the embankment (ML061-TP438, ML061-TP437A, ML061-WS001, ML061-CT433, ML061-TP435 and ML060-TP427) with thickness ranging from 0.40m, 0.50m, 0.60m, 0.30m, 0.60m and 0.30m and containing anthropogenic inclusions of rare brick fragments. Made ground is likely to be removed with the strip and shallow excavation requirements for construction works. The Culvert is noted to be within an Intermediate Probability Radon Area. Limited ground gas data is available for the site; however, the Hartwell Landfill (LQ 11.04) area was classified as a Characteristic Situation 2 (Low Risk). However, the gas risk assessment may not be representative of the actual ground gas regime of the wider site in view of the limited data.	1. Risk to human health and controlled waters during the works. 2. Risk of delays to the programme - potentially from additional GI to classify material and from remediation 3. Failure to comply with regulatory guidance/MMP 4. Increased costs with having to dispose of material and/ or re-do stockpiling of material. 5. Accumulation of potentially explosive/asphyxiative gases within confined spaces 6. Contaminated groundwater encountered may result in increased costs and delays to the programme due treatment and obtaining appropriate permits	6	Reduce	1. Excavations and confined spaces should be monitored for gas and depleted oxygen prior to man entry in line with recommendations of HSE guidance. 2. De-watering could be required during shallow excavations. Groundwater will need to be tested prior to discharge and contaminated GW exceeding screening values may require collect and treat/tanker and discharge consents. 3. Management of potentially contaminated soils to be in line with the Remediation Outline Strategy and Verification Plan reports.	Construction and Manufacture	2

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GEO_4310	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Radon gas risk associated with possible confined spaces.	The embankment is noted to be within a Low and Intermediate Probability Radon Area. However, it is understood that confined spaces in which Radon could accumulate are limited within the Aylesbury Embankment (Sedrup north Culvert only). Therefore, the risk is considered to be low.	1. Potential risk to construction workers entering confined spaces 2. Creation of preferential pathway into nearby buildings/confined spaces.	3	Eliminate	Precautionary health and safety measures for workers particularly when entering confined spaces.	Construction and Manufacture	2
GEO_4327	N/A	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	Risk of ground gases (methane, carbon dioxide, depleted oxygen, carbon monoxide and hydrogen sulphide - Radon assessed separately).	Limited ground gas data is available for the site; however, the Hartwell Landfill (LQ 11.04) and Former Hartwell clay, old Brick and tile works (LQ 11.03) areas were classified as a Characteristic Situation 2 (Low Risk). However, the gas risk assessment may not be representative of the actual ground gas regime of the wider site in view of the limited data.	1. Potential risk to construction workers entering confined spaces if elevated gas concentrations/depleted oxygen concentrations are encountered. 2. Creation of preferential pathway into nearby buildings/confined spaces. 3. Accumulation of potentially explosive/asphyxiate gases in confined spaces	6	Reduce	1. Further gas assessment to be undertaken once GI information is available. 2. Excavations and confined spaces should be monitored for gas and depleted oxygen prior to man entry in line with recommendations of HSE guidance.	Construction and Manufacture	2
GEO_AA_0524	CDM_52993	HS2-000001027 - Aylesbury Embankment	Risk Reduced ALARP	The proposed SGN gas main (non-contestable asset - CSO-02-2202) may settle more than envisaged by SGN.	Excessive deformation of the pipeline during the construction/operational phase of the project as consolidation of the ground under the landscape loading is completed.	Potential explosion of the gas main affecting the infrastructure surrounding it.	20	Reduce	1. HS2 to ensure that the designs of the pipeline and the landscape bunds are adequately coordinated and that the risks associated with the gas main are reduced to a tolerable degree. - COMPLETE 2. HS2 to confirm that the predicted bending profile and settlements of the pipeline are suitable for the loading from the mainline embankment and landscape bund. - COMPLETE 3. ASC to confirm that the permanent works are consistent with the information previously provided to the utility diversion team. COMPLETE 4. EKFB to take appropriate measures to locate and protect buried utility, in line with SGN requirements. EKFB to ensure utility is not impacted during construction and monitor the utility in line with SGN requirements.	Construction and Manufacture	5

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Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
GEO_3048	CDM_45042	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	High groundwater at this site causing inundation of excavations, soft soils at the ground surface	1) Groundwater model derived for the indicates that groundwater level is likely to be high. 2) Shallow Head deposits present at the site are anticipated to be permeable. This will allow seepage of groundwater relatively fast. It is anticipated that about 2.0m below the base of the culvert will be required to remove soft deposits, which allow groundwater to potentially enter the excavation.	Excavation collapse, inundation, and drowning	15	Control	1) Contractor to have a de-watering plan for the site and ensure that all site operatives are aware of such plan. Additional site controls including drainage and watching brief may be required during construction. 2) Communicate as CDM risk to contractor.	Construction and Manufacture	3
GEO_3049	N/A	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Unsuitable ground conditions as a founding stratum	1) Shallow materials that are not likely to provide appropriate founding material for the culvert. 2) Made Ground has been identified during the ground investigations at near surface, impacting the proposed founding of the culvert. 3) Alluvial and Heap deposit were also identified at shallow depth during the ground investigations.	Greater than anticipated differential settlement which may damage culvert joints permanently and make culvert capacity reduced or completely redundant	16	Control	1) The existing Made Ground and potentially shallow soft deposits (alluvium/Head) will be removed and replaced with suitable founding granular material. It is anticipated that 2.0m from the base of the culvert will be excavated and replaced with suitable founding material. 2) In situ testing of the proposed formation of the culvert will be required to provide assurance of the soil properties at formation level.	Construction and Manufacture	2
GEO_AA_0179	N/A	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Excessive long term differential settlement variance between hard/soft ground along the culvert causes excessive rotation at culvert joints and thus, pull out of culvert segments and permanent damage to the culvert.	1) Asset being located in the vicinity of an interpreted fault with relatively different ground models either side of the interpreted fault. 2) Insufficient ground investigation data at the asset location to map interpreted fault.	Greater than anticipated differential settlement which may damage culvert joints permanently and make culvert capacity reduced or completely redundant	16	Reduce	1) Additional GI (ML061-CR462) is available and has been considered in the current design. This borehole has also been used to identify the potential fault. As a consequence of this, two different ground models have been derived under the culvert footprint. These two ground models represent the different ground conditions under the side of the fault line, which will have an impact on the predicted movements of the culvert concrete sections. 2) Current design has been carried using out using both ground models and the fault line under the culver and maximum horizontal and vertical movements were used to design the culvert concrete sections. 3) To mitigate the risk of excessive	Construction and Manufacture	4

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									horizontal separation of the concrete sections of the culvert, an internal lining will be installed inside the culvert after both the culvert and embankment over the culvert are constructed. This has been discussed and agreed with EKFB and documented in EW-3114 dated 14th Feb 2023 and RRM on 28th Feb and 15th March 2023. 4) During the excavation to install the culvert, the potential fault line will need to be inspected and its location recorded and mapped as per the SCEW spec for the culvert. Backfilling of the excavation and construction of the culvert cannot be continued until the mapping of the fault is shared with ASC.		
GEO_AA_0180	N/A	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Excessive total long term settlement of the culvert due to compressible ground below the culvert.	1) Asset being located in the vicinity of an interpreted fault. 2) Compressible ground below the culvert around the fault combined with high loading from the landscape embankment.	Greater than anticipated total long term settlement of the culvert, loss of freeboard and leaving culvert capacity reduced.	6	Reduce	1) Additional GI (ML061-CR462) is available and has been considered in the current design. This borehole has also been used to identify the potential fault. As a consequence of this, two different ground models have been derived under the culvert footprint. These two ground models represent the different ground conditions under the side of the fault line, which will have an impact on the predicted movements of the culvert concrete sections. 2) Current design has been carried out using both ground models and the fault line under the culvert and maximum horizontal and vertical movements were used to design the culvert concrete sections. 3) the current estimated maximum total settlement considering the presence of the potential fault line is within the permissible limits for the concrete segments of the culvert. 4) Qualified and competent Geotechnical Engineer shall inspect the excavation below footprint of culvert to confirm the stratigraphy & location of the potential fault line. 5) The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level.	Construction and Manufacture	3

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GEO_AA_0181	N/A	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Overconservative or incorrect design parameters and ground model adopted for design	Inadequate or insufficient ground investigation and laboratory and in situ testing to derive ground and groundwater models	1) Inadequate solution proposed, which potentially can lead to failure of the culvert. 2) Greater than anticipated total long term settlement of the culvert. 3) H&S risk during construction.	12	Reduce	1) Initial HS2 Gi available and used in detailed design. 2) Additional GI (ML061-CR462) was available at the time of the detailed design and was also used to derive the ground model and the geotechnical parameters. 3) Both the initial HS2 GI and additional GI provide sufficient information to mitigate uncertainty in the ground model and geotechnical parameters adopted for the design.	Operations and Maintenance	3
GEO_AA_0183	CDM_26105	HS2-000001403 - Sedrup North Culvert	Risk Reduced ALARP	Running Sands or unstable excavations	Loose sand and groundwater or soft silts present in the excavation and replace under the culvert	1) Inundation/collapse of excavation resulting from loss of lateral support. 2) H&S risk - loss of life of construction workers.	15	Control	1) Contractor to ensure suitable temporary works if required during construction. 2) Excavation and Replacement (E&R) of Topsoil, head deposit and alluvial formation has been proposed with 2m of E&R below the culvert founding level. 3) Communicate as CDM risk to the contractor.	Construction and Manufacture	4
GEO_AA_0698	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	Elevated levels of sulphates, sulphides and chlorates or high/low pH will speed the degradation of construction materials such as concrete and metallic objects.	Primarily as a result of elevated Sulphates and Chlorides in the Gault Clay Formation and high pH values, exacerbated by the presence of mobile groundwater and /or the introduction of oxygen from excavations. May also be impacted by anthropogenic contaminants in the soil.	1. Presence of soil and groundwater chemistry that may result in aggressive ground conditions causing cracking of concrete, change in volume and deterioration of the structure over time leading to failure. 2. Limiting the use of as-dug or stabilised materials resulting in increased costs due to large haulage distances or importation of selected fill material.	12	Eliminate	1. Design has been carried out to include for potential aggressive ground due to presence of sulphates and Chlorides. 2. Further GI has been received and considered to identify risk of aggressive conditions in the Gault Clay Formation. 3. Design complies with BRE 363 Concrete testing and classification as set out in the HS2 and ASC design guidance to determine the concrete class to ensure adequate concrete is used. 4. Design is in accordance with BRE Suite SD1 testing to determine the concrete class to ensure adequate concrete is used.	Construction and Manufacture	4
GEO_AA_0699	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	Soft and compressible ground is present below formation levels.	1. Local variation of geology and weathering grades. 2. Weakening and softening of the Head Deposits and/or the Gault Clay Formation due to impacts of climate change. 3. The presence of normally consolidated cohesive soils or disturbed ground as a	1. A greater risk of instability and/or unacceptable total or differential settlements from loading/unloading from embankment bridleway requiring mitigation. 2. CBR values lower than that designed for at-grade pavement design.	9	Reduce	1. Temporary supports to be used where loose/soft/unstable materials are observed during excavation, 2. Observational approach to remove any loose/soft/unstable material encountered during excavation and replace with selected fill material prior to construction of embankment access road, 3. Design on long term parameters as opposed to short term parameters	Construction and Manufacture	6

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					result of water ingress, or disturbance of natural strata and associated with man's activity.				4. Construct pavements in dry weather or outside winter and spring months.		
GEO_AA_0700	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	Soil volume may change significantly in the Gault Clay Formation (shrink-swell) with water content (separate to heave) independent of loading.	Susceptibility of highly plastic fine grained soils to changes in volume arising from changes in water content from seasonal variation from the action of vegetation and climate.	1. Shrinkage and swelling of formation soils and causing changes in ground stress and the development of earth pressures, ground movement and cracking in soil deposits. 2. Longitudinal and lateral shrinkage / swelling in subgrade formation leading to uneven highway profile and unacceptable total and differential settlements causing damage to the highways 3. Differential ground movements causing damage to highway support and material degradation 4. Cracking of formation leading to deformation of highway drainage 5. Strain softening of soils in cutting slopes or excavations 6. Reduction in design CBR values	15	Reduce	1. All Ground investigation reviewed to confirm the characteristics of potentially swellable soils 2. Soil slopes designed according to HS2 Technical Standards to take account of long term degradation in shrink/swell prone clays 3. CBR values designed for long term performance, allowing for degradation due to shrink/swell 4. Keep formations dry (provide appropriate groundwater control measures and protection from inclement weather) from inclement weather)	Construction and Manufacture	6
GEO_AA_0701	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	Potential variable weathering profiles may be present below formation level.	Due to the varying effects of climate and weather on the foundation soil.	Resulting in variations in strength and compressibility in the foundation soils, leading to a higher risk of excessive settlement and instability.	9	Reduce	1. The geological conditions will be followed up in this area during excavation works to assess the encountered ground conditions and validate design. Undertake re-design as necessary. 2. Geotechnical parameters with an appropriate degree of conservatism have been used for design.	Construction and Manufacture	4
GEO_AA_0702	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	The presence of faulting may lead to an incorrect ground model adopted for Oat Close Bridleway (SBH/19) Realignment.	Historical geological tectonic process leading to the presence of unidentified faults Presence of and interpreted fault in close proximity to the SBH/19 Overbridge. Geological faulting has been identified and can be seen in the geological long section in Appendix B of the GIR.	Faulting may result in highly variable ground conditions, increased risks associated with ground stability, settlement and high groundwater flows in the area which could affect the foundation design for Oat Close Bridleway (SBH/19) Realignment. Differential movements are more likely in a faulted zone.	9	Reduce	1. Observational approach during construction to record the presence of soft and wet areas associated with faulting. 2. A conservative design approach has been undertaken and has adopted mitigation measures as necessary. 3. Geological conditions will be followed up in this area during excavation works to refine the position of the faults on the long section and assess the encountered ground conditions. 4. Further GI has been received and considered in the assessment of fault presence.	Construction and Manufacture	4

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									5. Design validated to confirm embankment stability and settlement are still within the HS2 technical requirements and adopt mitigation measures where required.		
GEO_AA_0703	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	Low strength relic shear surfaces may be present in the superficial Head Deposits and the Gault Clay beneath Oat Close Bridleway (SBH/19) Realignment embankment.	Reactivation of relic shear surfaces due to embankment loading and excavation during construction. Aa a result of historical down slope movement and instability during historical extreme climatic conditions (glacial and periglacial periods).	Reactivation of these low strength shear surfaces may result in local slips which may destabilise the embankment.	9	Reduce	Possible slickenside surfaces were encountered in the Gault Clay in borehole ML060-CR011 at depths of between 14.23m bgl and 16.65m bgl. If relic slip surfaces are encountered during excavation, then local stabilisation may need to be carried out to protect the earthworks. 1. Further GI received and used to identify the potential presence of these features. 2. Inspection/observational approach during construction and 3. Design mitigation incorporated where these features have been identified or are suspected to be present.	Construction and Manufacture	6
GEO_AA_0704	N/A	HS2-000020RKL - Oat Close Bridleway (SBH/19) Realignment	Risk Reduced ALARP	High or perched groundwater levels may be encountered in excavations during construction.	Due to the presence of a groundwater level close to surface as identified by the GI.	1. Reduction of embankment stability 2. Temporary works implications (instability of excavations, groundwater control etc.) 3. Increased pavement thicknesses to deal with soft and saturated soils during construction	12	Reduce	1. Provision of temporary works to control groundwater inflow and associated excavation instability 2. Construct pavements on Class 2 fill material and roads and access tracks at grade and in shallow cut on highly plastic soils in summer months or dry weather conditions where possible 3. Continue groundwater monitoring and provision for testing of groundwater for contaminants to manage off-site disposal 4. Provision for embankment drainage to be deployed during construction (observational approach), if necessary 5. All GI data will be passed to the Contractor to assess temporary works requirements for provision of temporary works to control groundwater inflow and associated excavation instability.	Construction and Manufacture	8
GEO_AA_0720	N/A	HS2-00002N7VQ - Aylesbury Embankment - Ancillary Culvert 061-AC1	Risk Reduced ALARP	Potential for localised Head, Alluvium (ALV) deposits, and soft material associated with existing drainage ditches under the foundation of the proposed culvert.	1. Presence of Soft deposits 2. Existing drainage ditches and possible attenuation ponds with softened material surrounding them.	1. Culvert and headwalls instability 2. Excessive total and differential settlements 3. Delay to works 4. Higher maintenance	8	Reduce	1. Undertake further ground investigation to assess risk and confirm the need for and design of ground treatment measures. COMPLETE 2. Design of Excavate and Replace. COMPLETE 3. All localised areas of soft deposits	Construction and Manufacture	2

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									below the foundation, encountered during construction, are to be completely removed and replaced prior to any culvert construction as detailed in the Specifications. 4. Verify ground conditions on site prior to construction		
GEO_AA_0721	N/A	HS2-00002N7VQ - Aylesbury Embankment - Ancillary Culvert 061-AC1	Risk Reduced ALARP	Possible fault(s) associated with a difference in level of Portland Sand Formation; position is speculative. Faulting and subsequent displacement of geological units are conjectured and vary.	Discontinuity in the stratigraphy	1. Unexpected ground conditions 2. Unacceptable total and differential settlements 3. Faulting may form a barrier to, or act as a conduit for groundwater flow 4. Damage to structural capacity of the culvert resulting in leakages	6	Control	1. Conservative design parameters adopted for design. COMPLETE 2. Additional GI at asset location is required to inform design. COMPLETE 3. Observational method to be undertaken during construction to identify presence of potential faults. EKFB to instigate mitigation works comprising for example the excavation of susceptible material and replacement with suitable engineered fill, as directed by ASC.	Construction and Manufacture	4
GEO_AA_0722	CDM_60079	HS2-00002N7VQ - Aylesbury Embankment - Ancillary Culvert 061-AC1	Risk Reduced ALARP	High/Shallow groundwater level	Shallow groundwater level in superficial deposits and presence of various drainage/ watercourses/ sources of water'	1. Groundwater ingress into excavations during construction 2. Culvert instability during construction 3. Unacceptable total or differential settlements due to ground softening during construction	12	Reduce	1. Undertake ground investigation to help determine the local ground conditions and inform detailed design. COMPLETE 2. Undertake conservative design (adopting cautious groundwater and flood levels). COMPLETE 3. Temporary works assessments to be undertaken during site works and managed accordingly e.g., RAMS.	Construction and Manufacture	2
GEO_AA_0723	N/A	HS2-00002N7VQ - Aylesbury Embankment - Ancillary Culvert 061-AC1	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete)	Elevated Sulphates and Chlorides in Gault Clay Formation	Deterioration and change in volume of concrete structure over time leading to structural failure	9	Reduce	1. Undertake BRE Special Digest 1 assessment based on asset-wide data. COMPLETE 2. Appropriate concrete class to be specified on drawings. COMPLETE	Construction and Manufacture	2
GEO_AA_0741	NA	HS2-00002N7VP - Aylesbury Embankment - Ancillary Culvert 060-AC4	Risk Reduced ALARP	Potential for localised Head deposits, and soft/weathered Gault Formation, and soft material associated with existing drainage ditches under the foundation of the proposed culvert.	1. Presence of Soft deposits 2. Existing drainage ditches and possible attenuation ponds with softened material surrounding them.	1. Culvert and headwalls instability 2. Excessive total and differential settlements 3. Delay to works 4. Higher maintenance	8	Reduce	1. Undertake further ground investigation to assess risk and confirm the need for and design of ground treatment measures. COMPLETE 2. Design of Excavate and Replace. COMPLETE 3. All localised areas of soft deposits below the foundation, encountered during construction, are to be completely removed and replaced prior to any culvert construction as detailed in the Specifications. 4. Verify ground conditions on site prior to construction.	Construction and Manufacture	4

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GEO_AA_0742	CDM_59864	HS2-00002N7VP - Aylesbury Embankment - Ancillary Culvert 060-AC4	Risk Reduced ALARP	High / Shallow groundwater level	Shallow groundwater level in superficial deposits and presence of various drainage/ watercourses/ sources of water.	1. Groundwater ingress into excavations during construction 2. Culvert instability during construction 3. Unacceptable total or differential settlements due to ground softening during construction	12	Reduce	1. Undertake ground investigation to help determine the local ground conditions and inform detailed design. COMPLETE 2. Undertake conservative design (adopting cautious groundwater and flood levels). COMPLETE 3. Temporary works assessments to be undertaken during site works and managed accordingly e.g., RAMS.	Construction and Manufacture	2
GEO_AA_0743	CDM_59841	HS2-00002N7VP - Aylesbury Embankment - Ancillary Culvert 060-AC4	Risk Reduced ALARP	Relic shear features may be encountered on soil under the foundation and reactivated under culvert & headwall loading causing instability.	Reactivation of relic shear surfaces on Gault Formation due to culvert construction and loading	Local slips which may destabilise temporary slopes formed for culvert installation.	3	Reduce	1. If relic shear surfaces are identified the need for any additional measures shall be agreed with the designer prior to placing of fill materials. 2. A qualified and competent geotechnical engineer or geologist shall be on site supervising during excavation and construction of culvert and Headwalls.	Construction and Manufacture	2
GEO_AA_0744	NA	HS2-00002N7VP - Aylesbury Embankment - Ancillary Culvert 060-AC4	Risk Reduced ALARP	Shrink-swell clays may be present within the Gault Formation with high potential for volume change.	Seasonal variations in moisture content.	Could cause unacceptable total and differential settlements.	9	Reduce	1. Design of Excavate and Replace. COMPLETE 2. Design according to HS2 standards. Compliance with the suitability rules of soils for substructure works. COMPLETE 3. A qualified and competent geotechnical engineer or geologist shall be on site supervising during excavation and construction of culvert and Headwalls.	Construction and Manufacture	3
GEO_AA_0751	N/A	HS2-00002N7VP - Aylesbury Embankment - Ancillary Culvert 060-AC4	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete)	Elevated Sulphates and Chlorides in Gault Clay Formation	Deterioration and change in volume of concrete structure over time leading to structural failure	9	Reduce	1. Undertake BRE Special Digest 1 assessment based on asset-wide data. COMPLETE 2. Appropriate concrete class to be specified on drawings. COMPLETE	Construction and Manufacture	2
GEO_AA_0958	CDM_61121	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	As per the ground model potentially soft and/or compressible strata is present beneath the proposed ditch slab.	The Presence of soft cohesive Alluvium / Head / or highly weathered and/or saturated Portland Sand/Stone Formation in the vicinity of the asset.	Soft of low strength and high compressibility. 1. Machinery may log down. 2. Significant settlement expected, there may be a chance of forward tilt or bearing capacity failure, of the ditch slab shallow foundations. 3. Instability in temporary works 4. Instability of excavation during construction	9	Reduce	Permanent works design is in accordance with HS2 and published codes (e.g., EC7). Temporary works designer to assess and ensure the stability of all excavations during construction, using suitable ground and groundwater level assumptions.	Construction and Manufacture	6

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GEO_AA_0959	CDM_61118	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	High groundwater table may be present in the ground.	The proximity of river floodplains and watercourses and poor drainage within the near-surface soils	1. Poor construction conditions 2. Erosion of excavation slope faces. 3. Inundation of excavations. 4. Instability of temporary works. 5. Need to adopt dewatering techniques	6	Reduce	1. Design of permanent structures are in line with HS2 standards to account for appropriate groundwater levels and climate change. 2. A conservative groundwater level was considered for the permanent design of the foundations of the Ditch Slab. 3. Contractor to assess temporary works required and adopt a safe system of works for construction e.g. consideration to groundwater control during works.	Construction and Manufacture	3
GEO_AA_0960	N/A	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete).	The presence of high sulphates in underlying materials	Cracking of concrete, change in volume and deterioration of the structure over the period of time leading to failure.	16	Reduce	Based on the data available near the asset and based on BRE Suite SD1, the concrete in Head deposits has been conservatively classified as ACEC-4 and DS-4.	Construction and Manufacture	2
GEO_AA_0961	CDM_61157	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	As per the ground model potentially soft and/or compressible strata is present beneath the proposed ditch slab.	The Presence of soft cohesive Alluvium/Head / or highly weathered and/or saturated Portland Sand/Stone Formation in the vicinity of the asset.	Soil of low strength and high compressibility. 1. Machinery may log down. 2. Significant settlement expected, there may be a chances of forward tilt or bearing capacity failure, of the ditch slab shallow foundations. 3. Instability in temporary works 4. Instability of excavation during construction	9	Reduce	Permanent works design is in accordance with HS2 and published codes (e.g., EC7). Temporary works designer to assess and ensure the stability of all excavations during construction, using suitable ground and groundwater level assumptions.	Construction and Manufacture	6
GEO_AA_0962	CDM_61146	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	High groundwater table may be present in the ground.	The proximity of river floodplains and watercourses and poor drainage within the near-surface soils	1. Poor Construction conditions 2. Erosion of excavation slope faces. 3. Inundation of excavations. 4. Instability of temporary works. 5. Need to adopt dewatering techniques	6	Reduce	1. Design of permanent structures are in line with HS2 standards to account for appropriate groundwater levels and climate change. 2. A conservative groundwater level was considered for the permanent design of the foundations of the Ditch Slab. 3. Contractor to assess temporary works required and adopt a safe system of works for construction e.g. consideration to groundwater control during works.	Construction and Manufacture	3

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GEO_AA_0963	N/A	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete).	The presence of high sulphates in underlying materials	Cracking of concrete, change in volume and deterioration of the structure over the period of time leading to failure.	16	Reduce	Based on the data available near the asset and based on BRE Suite SD1, the concrete in Head deposits has been conservatively classified as ACEC-4 and DS-4.	Construction and Manufacture	2
GEO_AA_0964	CDM_61158	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Obstructions	Incorrect or overconservative ground model and parameters adopted for design	Limited geotechnical information available – The details of the existing ditch on site are unknown	9	Reduce	1. Undertake conservative design adopting worst-case ground models and cautious estimate of parameters. 2. Excavate and Replace mitigation during construction, if needed.	Reduce	6
GEO_AA_0965	N/A	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Traffic loading exceeds that designed for and bearing capacity of the founding stratum is exceeded.	Heavier than designed traffic load driving over the ditch slab.	Exceedance of the founding stratum bearing capacity and instability/failure of the ditch slab foundation.	4	Control	1. Design has been undertaken using the drawings and traffic loading provided by the ancillary structures team. Accidental loading allowed for in accordance with the AiP. 2. Contractor to implement the requirements of the design and follow the drawings. 3. Temporary works designer to ensure appropriate traffic loading taken into account at the top of temporary excavations 4. Operator to ensure that traffic using the ditch slabs does not exceed those designed for.	Operations and Maintenance	2
GEO_AA_0966	N/A	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	Traffic loading exceeds that designed for and bearing capacity of the founding stratum is exceeded.	Heavier than designed traffic load driving over the ditch slab.	Exceedance of the founding stratum bearing capacity and instability/failure of the ditch slab foundation.	4	Control	1. Design has been undertaken using the drawings and traffic loading provided by the ancillary structures team. Accidental loading allowed for in accordance with the AiP. 2. Contractor to implement the requirements of the design and follow the drawings. 3. Temporary works designer to ensure appropriate traffic loading taken into account at the top of temporary excavations 4. Operator to ensure that traffic using the ditch slabs does not exceed those designed for.	Operations and Maintenance	2

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IDR 212.5 – Aylesbury Embankment Ditch Slabs. October 2024 [80]

Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
GEO_AA_0958	CDM_61121	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	As per the ground model potentially soft and/or compressible strata is present beneath the proposed ditch slab.	The Presence of soft cohesive Alluvium / Head / or highly weathered and/or saturated Portland Sand/Stone Formation in the vicinity of the asset.	Soil of low strength and high compressibility. 1. Machinery may log down. 2. Significant settlement expected, there may be a chance of forward tilt or bearing capacity failure, of the ditch slab shallow foundations. 3. Instability in temporary works 4. Instability of excavation during construction	9	Reduce	Permanent works design is in accordance with HS2 and published codes (e.g., EC7). Temporary works designer to assess and ensure the stability of all excavations during construction, using suitable ground and groundwater level assumptions.	Construction and Manufacture	6
GEO_AA_0959	CDM_61118	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	High groundwater table may be present in the ground.	The proximity of river floodplains and watercourses and poor drainage within the near-surface soils	1. Poor construction conditions 2. Erosion of excavation slope faces. 3. Inundation of excavations. 4. Instability of temporary works. 5. Need to adopt dewatering techniques	6	Reduce	1. Design of permanent structures are in line with HS2 standards to account for appropriate groundwater levels and climate change. 2. A conservative groundwater level was considered for the permanent design of the foundations of the Ditch Slab. 3. Contractor to assess temporary works required and adopt a safe system of works for construction e.g. consideration to groundwater control during works.	Construction and Manufacture	3
GEO_AA_0960	N/A	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete).	The presence of high sulphates in underlying materials	Cracking of concrete, change in volume and deterioration of the structure over the period of time leading to failure.	16	Reduce	Based on the data available near the asset and based on BRE Suite SD1, the concrete in Head deposits has been conservatively classified as ACEC-4 and DS-4.	Construction and Manufacture	2
GEO_AA_0961	CDM_61157	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	As per the ground model potentially soft and/or compressible strata is present beneath the proposed ditch slab.	The Presence of soft cohesive Alluvium/Head / or highly weathered and/or saturated Portland Sand/Stone Formation in the vicinity of the asset.	Soil of low strength and high compressibility. 1. Machinery may log down. 2. Significant settlement expected, there may be a chances of forward tilt or bearing capacity failure, of the ditch slab shallow foundations. 3. Instability in temporary works 4. Instability of excavation during construction	9	Reduce	Permanent works design is in accordance with HS2 and published codes (e.g., EC7). Temporary works designer to assess and ensure the stability of all excavations during construction, using suitable ground and groundwater level assumptions.	Construction and Manufacture	6

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Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
GEO_AA_0962	CDM_61146	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	High groundwater table may be present in the ground.	The proximity of river floodplains and watercourses and poor drainage within the near-surface soils	1. Poor Construction conditions 2. Erosion of excavation slope faces. 3. Inundation of excavations. 4. Instability of temporary works. 5. Need to adopt dewatering techniques	6	Reduce	1. Design of permanent structures are in line with HS2 standards to account for appropriate groundwater levels and climate change. 2. A conservative groundwater level was considered for the permanent design of the foundations of the Ditch Slab. 3. Contractor to assess temporary works required and adopt a safe system of works for construction e.g. consideration to groundwater control during works.	Construction and Manufacture	3
GEO_AA_0963	N/A	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Potential chemical attack and degradation of construction materials (e.g. concrete).	The presence of high sulphates in underlying materials	Cracking of concrete, change in volume and deterioration of the structure over the period of time leading to failure.	16	Reduce	Based on the data available near the asset and based on BRE Suite SD1, the concrete in Head deposits has been conservatively classified as ACEC-4 and DS-4.	Construction and Manufacture	2
GEO_AA_0964	CDM_61158	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Obstructions	Incorrect or overconservative ground model and parameters adopted for design	Limited geotechnical information available – The details of the existing ditch on site are unknown	9	Reduce	1. Undertake conservative design adopting worst-case ground models and cautious estimate of parameters. 2. Excavate and Replace mitigation during construction, if needed.	Reduce	6
GEO_AA_0965	N/A	HS2-00002YD79 - Aylesbury Embankment Pedestrian Ditch Slab No.1	Risk Reduced ALARP	Traffic loading exceeds that designed for and bearing capacity of the founding stratum is exceeded.	Heavier than designed traffic load driving over the ditch slab.	Exceedance of the founding stratum bearing capacity and instability/failure of the ditch slab foundation.	4	Control	1. Design has been undertaken using the drawings and traffic loading provided by the ancillary structures team. Accidental loading allowed for in accordance with the AiP. 2. Contractor to implement the requirements of the design and follow the drawings. 3. Temporary works designer to ensure appropriate traffic loading taken into account at the top of temporary excavations 4. Operator to ensure that traffic using the ditch slabs does not exceed those designed for.	Operations and Maintenance	2

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Geotechnical Risk ID	Associated CDM Register Risk ID	Level 2 Asset	Risk Status	Risk Description (There is a threat that…)	Risk Cause (Due to….)	Risk Impact (Resulting in….)	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Next Action Due	Residual Risk Rating
GEO_AA_0966	N/A	HS2-00002YD7F - Aylesbury Embankment Vehicular Ditch Slab No.1	Risk Reduced ALARP	Traffic loading exceeds that designed for and bearing capacity of the founding stratum is exceeded.	Heavier than designed traffic load driving over the ditch slab.	Exceedance of the founding stratum bearing capacity and instability/failure of the ditch slab foundation.	4	Control	1. Design has been undertaken using the drawings and traffic loading provided by the ancillary structures team. Accidental loading allowed for in accordance with the AiP. 2. Contractor to implement the requirements of the design and follow the drawings. 3. Temporary works designer to ensure appropriate traffic loading taken into account at the top of temporary excavations 4. Operator to ensure that traffic using the ditch slabs does not exceed those designed for.	Operations and Maintenance	2

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Appendix E

E.1 Geotechnical Calculation Report – Maintenance Access Road to Pond (South Arm off Calley Farm - South)

Number of Pages: 77

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1 Introduction

1.1 Scope and Objective of Calculation Pack

The purpose of this document is to present the geotechnical design calculations for the Maintenance Access Road to Pond (South Arm off Calley Farm – South) to be submitted as part of IDR 208.4

1.2 Summary of Asset

The Maintenance Access Road to Pond (South Arm off Calley Farm – South) is located between mainline Ch. 61+300 and 61+875m (SnakeGrid) within Section C2 (Aylesbury Area), centred at approximate British National Grid Reference E 480731 N 211511 (OSGB). The asset is part of the Aylesbury Embankment Level 1 Asset Group. Mainline assets to the north and the south are Aylesbury North Cutting and Aylesbury South Cutting, respectively.

Figure 1-1 and Table 1-1 identify the location and attributes of the Maintenance Access Road, respectively.

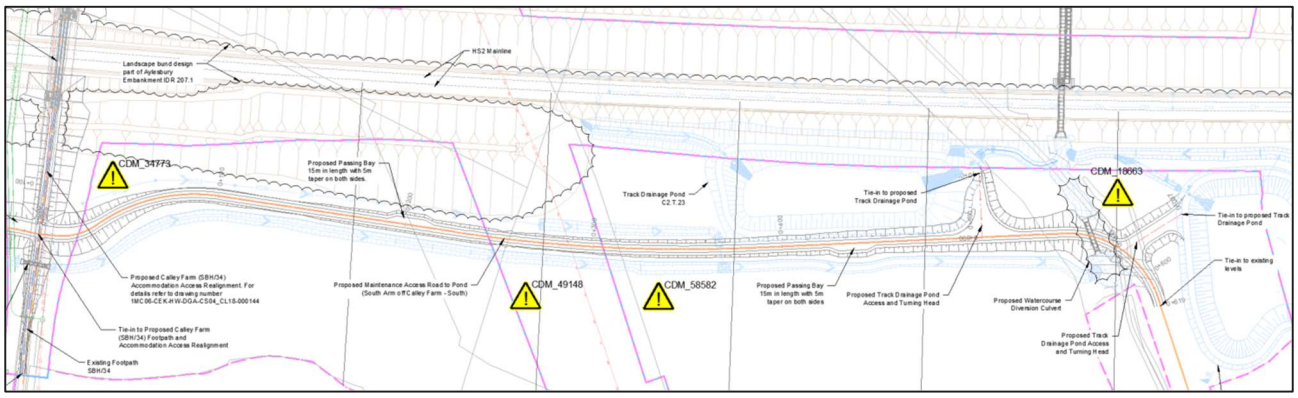


Figure 1-1- General Arrangement Plan

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Table 1-1: Asset Details

Asset Description	Detail
Name of design element	**Maintenance Access Road to Pond (South Arm off Calley Farm – South)**
Contract/section/lot	AA
HS2 Asset ID	HS2-00002BJSK
Type	Carriageway
Highway Start - End chainage (approx.)	0+61+300 to 61+875m
Length	619m
Maximum height/depth	Embankment = 2.5m
Design slope angle	1V:3H
Landscape bunds	Yes, left hand side of highway from Ch.61+580 to 61+850
Geotechnical Category	GC1

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1.2.1 Sources of Information

The following documents should be consulted to provide supporting information for the geotechnical assessment and design of the assets:

Table 1-2: Source of Information

Report / Drawing Titles	Document No
Aylesbury Embankment - Supplementary GIR	1MC06-CEK-GT-REP-CS04_CL18-000011
Aylesbury Embankment GDR	1MC06-CEK-GT-REP-CS04_CL18-000037
Aylesbury Embankment - 2D Plan Model	1MC06-CEK-GT-DMA-CS04_CL18-000005
Calley Farm (SBH-34) Footpath and Accommodation Access Realignment - Asset Group 2D XREF DM2	1MC06-CEK-HW-DMA-CS04_CL18-000003
IDR 208.4 – Specification	1MC06-CEK-AU-SPE-CS04-000003
IDR 208.4 – Geotechnical Risk Register	1MC06-CEK-GT-REG-CS04-000003
IDR 208.4 - Assumptions Register	1MC06-CEK-AU-REG-CS04-000023
IDR 208.4 - CDM Risk Register	1MC06-CEK-AU-REG-CS04-000009
IDR 208.4 – Departures Register	1MC06-CEK-AU-REG-CS04-000018
Maintenance Access Road to Pond (South Arm off Calley Farm - South) - Highway General Arrangement Sheet	1MC06-CEK-HW-DGA-CS04_CL18-000171
Maintenance Access Road to Pond (South Arm off Calley Farm - South - Highway Earthworks General Arrangement	1MC06-CEK-HW-DGA-CS04_CL18-000172
Maintenance Access Road to Pond (South Arm off Calley Farm - South – Highway Cross-Sections Sheet 1	1MC06-CEK-HW-DSE-CS04_CL18-000035
Maintenance Access Road to Pond (South Arm off Calley Farm - South) – Highway Earthwork Cross-Sections Sheet 2	1MC06-CEK-HW-DSE-CS04_CL18-000036
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 1	1MC06-CEK-GT-DSE-CS04_CL18-000090
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 2	1MC06-CEK-GT-DSE-CS04_CL18-000091
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 3	1MC06-CEK-GT-DSE-CS04_CL18-000092

The following other sources of information have also been used within the assessment:

- Ground Investigation (GI) data and monitoring data from the HS2 Main Ground Investigation (GI);

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- GI data and monitoring data from the Supplementary GI;
- Data available from Stage 2 GI, where available (Data Drop 25); and
- ASC GIS Viewer.

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2 Summary of Scheme Design

The scheme design for the asset was captured within Calley Farm SBH34 Accommodation Footpath Highway Asset Design Statement and was limited to the asset geometry and alignment shown in the drawings 1MC06-CEK-HW-DGA-CS04_CL18-000026 and 1MC06-CEK-HW-DGA-CS04_CL18-000070.

No significant changes have occurred since Scheme Design and no specific analysis was undertaken for the Maintenance Access Road to the Pond (South Arm off Calley Farm – South).

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3 Detailed Design Proposals

The detailed design proposals for the asset can be found in the table below.

Table 3-1: Asset Details, Detailed Design

Asset Description	Maintenance Access Road to Pond (South Arm off Calley Farm – South)
Type	Carriageway
Start - End Mainline Chainage (approx.)	61+300 to 61+875m
Length (m) (approx.)	619m
Carriageway/ Verge Width	3.5 / 1.5m
Pavement Design	Unbound (AT2)
Max. Structure Height at Asset Chainage	2.5m @ ch.0+020
Proposed Detailed Design Slope Angle	1V:3H
Landscape Bunds	Yes, left hand side of highway from Ch.0+000 to 0+290

3.1 Relevant Boreholes and Other Information

A list of the exploratory holes under consideration when deriving the ground model are included in the table in Appendix B.

A total of 30 no exploratory holes have been considered in the design of this asset, the locations of which are shown in the plan included within Appendix B.

The Uncertainty in Ground Conditions Classification for the highway asset group is Class 1, considering the available GI data and Geotechnical Category of the earthworks, and has been undertaken in line with the Ground Engineering Design Basis Statement – Management of Uncertainty in Ground Conditions (1MC06-CEK-GT-REP-C002-000127). This classification confirms sufficient GI data is available for the Detailed Design.

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3.2 Site Constraints

3.2.1 Landscaping

The landscape earthworks are adjacent to the left hand side of the highway from approx. Ch. 0+000 to 0+290m and no adverse impact on the highway earthwork is anticipated.

3.2.2 Drainage

Drainage ditches are present on to the south of the highway from Ch. 0+000 to 0+570 and then either side of the highway earthwork from approx. Ch. 0+270 to 0+365. In places these form small cuttings (1V:2H up to 1.15m deep) approx. 2m away at the toe of the highway earthwork. These are not anticipated to adversely impact the highway earthworks but will be included within the analysis to ensure they do not affect the stability of the highway earthworks.

There are two track drainage attenuation ponds with associated highway access from Ch. 0+380 to 0+500 (Pond C2.T.23-REG-0614), 0+585 (Pond C2.T.21-PND-0611) which lie adjacent to the highway earthworks. The pond geometry has been included within the stability analyses to ensure any effect of it on the highway is accounted for.

Culvert 061-AC3 is proposed beneath the highway at Ch 0+565m, linking into the Sedrup Northern watercourse. Structure specific earthworks will be defined as part of the culvert design (1MC06-CEK-CV-DSE-CS04_CL18-000050 and 1MC06-CEK-CV-DGA-CS04_CL18-000071).

3.2.3 Utilities

There is an existing high pressure gas main that passes beneath the Maintenance Access Road at approximate Ch 0+280m. At this location the highway is proposed to be at grade or on shallow embankments up to 0.75m in height. A 16m slab is proposed on top of the gas main under the proposed carriageway and drainage. For further details refer to the Aylesbury - FDR Pack (CSO-02-2202) drawings.

Aylesbury Embankment - Phase 2 Ground Movement Impact Assessment Report (1MC06-CEK-GT-REP-CS04_CL18-000042) provides further information regarding the ground movements within this asset.

3.2.4 Contaminated Ground Risk Assessment

The following summary has been extracted from the Supplementary GIR.

The Aylesbury Embankment is predominantly located within rural green fields, with agricultural activities.

The following Land Quality (LQ) constraint areas have been identified as potentially “contaminated” land that may impact on or be impacted by the proposed HS2 scheme. Their identification was based on an initial desk study review carried out by the Atkins Desk Study, where baseline information was obtained from a combination of desktop sources, site visits and previous ground investigations:

- Hartwell Landfill (LQ 11.04) which is approximately 110m northwest of the LOD of the Embankment (at its nearest point); and
- Former Hartwell clay, brick/tile works & landfill (LQ 11.03) located 200m north of the LOD of the Embankment.

At the time of production of the Geo-Environmental Reports, limited exploratory hole and chemical data was available within the footprint of Aylesbury Embankment. Specifically, there were no chemical testing (soil, leachate, groundwater, or surface water) being undertaken within the chainage limits of the Aylesbury Embankment.

Made Ground has thus far not been encountered within the LOD or LLAU of the Embankment and no visual/ olfactory evidence of contamination was identified.

In the absence of chemical testing targeting the LQ with respect to the Aylesbury Embankment, the potential risk to human health is still considered to be Low for this asset given the greenfield setting. However, there

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are LQ areas which are outside of the LOD & LLAU of the Embankment and there is a potential for elevated risk for the HS2 earthworks in close proximity to these LQ constraint areas.

There also remains a potential risk from site-wide unforeseen contamination, and where encountered, further testing and assessment may be required.

Please refer to the GIR for a full account of the contaminated ground risk assessment,

3.3 Construction Sequence

The construction sequence is yet to be confirmed by the Client, however, the construction sequence is not anticipated to affect the design of the proposed access road earthworks.

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4 Ground Conditions

4.1 General

Detailed geological descriptions, test results of the encountered materials and recorded groundwater levels, are presented in the Supplementary GIR.

4.2 Ground Model and Characteristic Parameter values

This section describes the proposed specified earthwork materials and natural stratigraphy along with the characteristic parameter values used within the design as presented in the Table 4-1 to Table 4-3 below.

Table 4-1 - Summary of Earthworks and General Natural Stratigraphy

Stratigraphy	Depth range (mbgl)	Summary Description
Engineering Fill (Class2A4/7C3)	Variable	Recovered material from the Kimmeridge Clay Formation for use in Highway Earthworks
Engineering Fill (Class 1/6N/6P)	Variable	To be used for structural fill around the wingwalls and abutments and low height embankments nominally <0.75m in height
Alluvium	0.00 - 2.20	Soft to firm, occasionally stiff yellowish brown and light brown sandy silty clay or clayey silt. Occasionally recovered as loose light brown clayey gravelly sand or sandy gravel.
Purbeck Group	2.00 - 8.10	Medium strong to very strong grey limestone/calcareous mudstone and siltstone. Occasionally recovered as soft to very stiff greenish grey and grey laminated clay.
Portland Stone Formation	4.50 - 20.00+	Strong to very strong thinly bedded grey limestone and extremely weak to weak calcareous siltstone/mudstone. At shallower depths POST is typically recovered as soft to firm bluish grey/ orangish grey sandy gravelly clay.
Portland Sand Formation	4.50 - 20.00+	Strong to very strong grey limestone and occasionally firm to very stiff dark grey calcareous silt.
Kimmeridge Clay Formation	7.40 - 20.00+	Firm to very stiff dark grey/light bluish grey sandy calcareous clay and occasionally very weak to medium strong grey silty mudstone.

The ground models to be used in the highway design are summarised within Table 4-2. The Portland Stone and Portland Sand Formations have been combined within the ground model as “Portland Group” as presented within the Supplementary GIR Long Section.

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Table 4-2 - Summary of Ground Models to be used

Ground Model	Strata	Depth Range (mbgl)
Ground Model 1 **Ch. 0+000 to 0+500**	Portland Group	0.00 – 7.50
	Kimmeridge Clay Formation	7.50 – 20.00+
Ground Model 2 **Ch. 0+500 to 0+619**	Alluvium	0.00 – 2.00
	Purbeck Group	2.00 – 4.50
	Portland Group	4.50 – 15.00
	Kimmeridge Clay Formation	15.00 – 20.00+

Notes:

Thin localised occurrences of Made Ground, Head and Alluvium deposits are encountered but is assumed will be excavated and replaced during construction

Table 4-3 - Summary of Characteristic Parameter values for Slope Stability and Settlement Assessment

Strata/ Fill	PI (%)	Unit Weight γ (kN/m³)	c_u (kN/m²)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)
Alluvium	48[2]	18[3]	20[1 / 11]	21[5]	22[6]	0[7]
Purbeck Group	-	20[3]	80[1]	22[1]	24[1]	1[7]
Portland Group	25[2]	20[3]	45[4]	25[5]	27[6]	1[7]
Kimmeridge Clay Formation	35[2]	20[3]	40+8z (up to 150kN/m²)[10]	23[5]	25[6]	1[7]
Class 2A4 / 7C3	-	20[8]	50 [8]	21[8]	23[8]	1
Class 1A4	-	20[3]	-	-	35[9]	0

Notes:

1. Parameter values taken from Aylesbury Embankment due to limited available local data.
2. Plasticity Index derived using asset specific data, see Appendix C, Figure C1.
3. Unit weights (γ) of the in-situ materials are determined based on the asset specific GI factual data and available supporting factual GI data, as appropriate. Reference should be made to the supplementary GIR / GDR.
4. Characteristic line from asset specific Cu data plot, see Appendix C, Figure C4. An f_1 (correlation factor between Cu and SPT 'N') value of 4.5 has been adopted for all cohesive materials, with reference to Stroud 1974.
5. ϕ'_{cv} derived from BS8004:2015, Table 2, using the above-mentioned PI values.
6. ϕ'_{pk} values derived from ϕ'_{cv}, using a dilation value of 1° for Alluvium and 2° for Portland Group Kimmeridge Clay as per BS8004:2015.
7. Cohesion is considered to be 1kPa for cohesive soils and 0kPa for granular.
8. Class 2A4 / 7C3 Fill material consists of re-used Kimmeridge Clay Formation, parameter values based on L1 wide asset characteristic lines.

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Strata/ Fill	PI (%)	Unit Weight γ (kN/m³)	c_u (kN/m²)	ϕ'_{cv} (°)	ϕ'_{pk} (°)	c' (kPa)

9. Class 1A4 value are as per the IDR 208.4 Specification (1MC06-CEK-AU-SPE-CS04-000003).
10. Z = depth measured from top of stratum.
11. soft soil which has a cu<20kPa is to be removed during construction.

4.3 Groundwater

There are 7 no. of monitoring locations within the vicinity of this Level 2 asset and the results are presented within the table below. A conservative groundwater level of 0.1mbgl has been adopted for the purposes of design.

Table 4-4 – Summary of Groundwater Readings

Borehole Reference	No. of Readings	Min Reading (mbgl)	Average Reading (mbgl)	Max Reading (mbgl)
ML061-CP001	13	0.1	0.83	1.3
ML061-CR009	13	1.47	2.49	3.1
ML061-CR012	13	0.54	1.01	1.52
ML061-CR013	14	0.68	1.36	2.07
ML061-CR014	3	0.75	1.09	1.27
ML061-CR419	14	0.54	1.61	2.5
ML061-RC017	3	0.38	0.76	0.99

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4.4 Geohazards

A summary of the key potential geotechnical geohazards that may affect the assets are provided in Table 4-5.

Table 4-5: Summary of Geohazards

Risks	Description
Faulting	Potential Faulting identified at mainline chainages Ch 060+650, 060+875, 061+075, 061+180, and 061+250, resulting in locally weaker ground conditions. Conservative ground model and parameters have been considered in the slope stability design. Ground conditions to be assessed on site prior to construction of earthworks and soft areas excavated and replaced in accordance with IDR Specification 208.4.
Soft ground	Occurrences of very soft or loose material can lead to instability of embankments and excessive ground movement and low CBR values affecting pavement design. Any soft ground occurrences are considered to be localised and should be excavated and replaced in accordance with IDR Specification 208.4
Aggressive ground conditions	Occurrences of gypsum crystals can be associated with high sulphate content which can lead to chemical attacks on concrete structures. All concrete and fill that interface with the natural ground is to be designed adequately to BRE 363 considering available chemical testing.
Shallow groundwater	Potentially shallow groundwater present below embankments, leading to instability. Conservative groundwater level considered in the design.
Shrink / swell	Changes in soil volume by shrinking or swelling from changes in moisture content independent of loading leading to ground movements. Conservative ground parameters have been considered in the slope stability and settlement design including the adoption of a Vadose zone as set out in the HS2 Technical Standard – Earthworks (HS2-HS2-GT-STD-000-000001) for assessment of long-term slope stability.
Relic Shear Surfaces	Polished discontinuity surfaces were encountered within Kimmeridge Clay Formation within ML061-CR014 between 19.40-20.00 mbgl.
Contaminated Ground	Limited data available has been reviewed for potential contamination, however, this remains a potential risk from site-wide unforeseen contamination. The formation is to be inspected prior to placement of fill material of highway to ensure no Made Ground is present. Undertake a watching brief during construction, and isolate and test any materials that have the potential to pose a risk to human health and the environment.

These key geotechnical hazards and their locations within Aylesbury Embankment Group and associated assets are detailed in the Supplementary GIR. Reference should also be made to IDR208.4 geotechnical risk register (1MC06-CEK-GT-REG-CS04-000003).

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5 Slope Stability Assessment

5.1 General

Slope stability analyses for static performance is outlined in this section.

5.2 Basis of Geotechnical Design

5.2.1 Introduction

The static global stability of earthworks has been calculated adopting the principles of limit state equilibrium in accordance with Eurocode 7. Specifically, Ultimate Limit State (ULS) Design Approach 1 Combination 1 (DA1 C1) and Combination 2 (DA1 C2) have been assessed adopting partial factors in accordance with Eurocode 7 with the requirement to achieve a utilisation less than or equal to 1. Assessments have been undertaken to satisfy the requirements of the HS2 Technical Standard - Earthworks. Serviceability Limit State (SLS) assessments are discussed in subsequent sections.

5.2.2 Design Approach for Static Analysis:

1. Slope stability was modelled using the commercial geotechnical software SLOPE/W.
2. Morgenstern-Price method of slope stability analysis was used.
3. Circular slips were assessed adopting the ‘Entry and Exit’ methods of slip surface determination within the software SLOPE/W.
4. A vadose zone (2.0m thick in accordance with HS2 standard HS2-HS2-GT-STD-000-000001) has been modelled for embankment slopes comprising cohesive materials with critical state parameters, subject to surface water infiltration, to represent disturbance/softening and long-term degradation at the slope surface.
5. An r_u coefficient of 0.25 has been used for modelling the vadose zone in embankments both below and over 3m in height where over the edge drainage is present.
6. Slope stability has assessed shallow (confined to embankment slope) and deeper (global) potential slip surfaces. In addition, bund slopes have been assessed where applicable. The highest utilisation or critical slip surface is reported.
7. A depth of 0.3m was used as minimum slip surface depth in order to rule out insignificant shallow slip surfaces.
8. Both the short-term (undrained) and the long-term (drained) conditions were analysed for the static condition.
9. The offline highway embankment slopes are proposed to be constructed at a gradient of 1V:3H.
10. A variety of embankment heights (1.0m to 3.0m) were analysed, which represent the height range of embankments encountered for the assets.

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5.3 Actions/ Loading

The surcharge loadings applied in the embankment slope stability analyses are summarised below. Partial factors in accordance with Eurocode 7 were applied.

Load Case A (static – short-term condition):

- A permanent surcharge of 20kPa of construction load over the carriageway and verge (from crest to crest) was adopted to check stability of embankment under construction and in the short-term conditions using undrained parameter values.

Load Case B (static – long-term condition):

- A permanent surcharge of 10kPa over the verge and a permanent 20kP load over carriageway was adopted to check the stability of embankment under long term conditions with drained parameter values adopted.
- A 1m exclusion zone was applied back from the crest of the embankments with no surcharging adopted within this zone.
- Any favourable surcharging associated to variable action is ignored.

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5.4 Calculation Results

The output of the slope stability calculations for the unreinforced and reinforced embankments are provided in the tables below and a selection of the calculation outputs is presented in Appendix D:

Table 5-1: Slope Stability Analyses

Analysis No.	Embankment Height (m)	Fill Class	Ground Model	Analysis
1	1.0	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)
2	1.5	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)
3	2.0	2A4	1	Unreinforced Embankment (MAR - South Arm off Calley Farm South)
4	0.5	2A4	2	Unreinforced Embankment (MAR - South Arm off Calley Farm South) – non ditch interface
5	1.0	2A4	2	Unreinforced Embankment (MAR - South Arm off Calley Farm South) – ditch interface
6	3.0	2A4 / 7C3	1	Reinforced Embankment (MAR - South Arm off Calley Farm South)
7	2.0	2A4 / 7C3	2	Reinforced Embankment (MAR - South Arm off Calley Farm South)
8	1.35	2A4	1	Pond C2.T.23-PND-0614 cutting adjacent to access road
9	1.35	2A4	2	Pond C2.T.23-PND-0614 cutting adjacent to access road
10	2.3 (1.2 Emb / 1.1 Cutting)	2A4	2	Pond C2.T.21-PND-0611 cutting adjacent to access road
11	2.3 (Cutting)	2A4	2	Pond C2.T.21-PND-0611 cutting adjacent to access road

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5.4.1 Analysis of unreinforced slopes with Class 2A4 fill

Table 5-2 - Summary of Slope Stability Results – Unreinforced Embankments Ground Model 1- Short / Long Term Conditions

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
1	1.0	1V:3H	Class 2A4	0.874	0.315
2	1.5	1V:3H	Class 2A4	0.968	0.360
3	2.0	1V:3H	Class 2A4	1.035	0.404

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

3) A 2m deep vadose zone has been modelled within the embankment with an Ru of 0.25 with the remaining embankment fill modelled with an Ru of 0.1.

4) For approximate chainages of embankment heights please refer to Table 9-1 within Section 9.

5) High utilisation shown above in red highlights the need for mitigation in the form of geogrid reinforcement. Additional analysis is shown in Table 5-4 where the embankments have been designed to be reinforced.

Table 5-3 – Summary of Slope Stability Results – Unreinforced Embankments Ground Model 2 – Short / Long Term Conditions

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 – DA1C2)	
				Drained	Undrained
4	0.5	1V:3H	Class 2A4	0.897	0.341
5	1.0	1V:3H	Class 2A4	1.079	0.483

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

3) A vadose zone has been modelled within the embankment with an Ru of 0.25.

4) For approximate chainages of embankment heights please refer to Table 9-1 within Section 9.

5) High utilisation shown above in red highlights the need for mitigation in the form of geogrid reinforcement. Additional analysis is shown in Table 5-4 where the embankments have been designed to be reinforced.

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In Summary

- Maintenance Access Road to Pond (South Arm off Calley Farm – South) Class 2A4 Fill Highway Embankments greater than 1.5m in height from ch.0+000 to 0+500 at a proposed side-slope gradient of 1V:3H will require geogrid reinforcement to the side slopes to meet the required HS2 performance.
- Maintenance Access Road to Pond (South Arm off Calley Farm – South) Class 2A4 Fill Highway Embankments greater than 0.5m in height from ch.0+500 to 0+600 at a proposed side-slope gradient of 1V:3H will require geogrid reinforcement to the side slopes to meet the required HS2 performance.

5.4.2 Analysis of soil reinforced slopes with Class 2A4 / 7C3 Fill

Table 5-4 - Summary of Slope Stability Results – Reinforced Embankments (Class 7C3) – Ground Model 1- Long term drained conditions

Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	Location (m above base)	Length (m)	DoU (As Per BS8006	DoU (As Per DA1 C2)
6	3.0	1V:3H	6	0	6.0	0.853	0.861
				0.5	2.0		
				1	6.0		
				1.5	2.0		
				2	6.0		
				2.5	2.0		

Notes:

1) All results are presented as degree of utilisation

2) All drained analyses include a 10kPa permanent surcharge across a 2m verge from the crest of the slope and 20kPa permanent surcharge across the carriageway

3) A 2m deep vadose zone has been modelled within the embankment with an Ru of 0.25 with remaining embankment fill modelled with an Ru of 0.1.

4) For approximate chainages of embankment heights please refer to Table 9-1 within Section 9.

Table 5-5 - Summary of Slope Stability Results - Reinforced Embankments (Class 7C3) – Ground Model 2 – Long term drained conditions

Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	Location (m above base)	Length (m)	DoU (As Per BS8006	DoU (As Per DA1 C2)
7	2.0	1V:3H	4	0	6.0	0.824	0.980
				0.5	2.0		
				1	6.0		
				1.5	2.0		

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Notes:

1) All results are presented as degree of utilisation

2) All drained analyses include a 10kPa permanent surcharge across a 2m verge from the crest of the slope and 20kPa permanent surcharge across the carriageway

3) A 2m deep vadose zone has been modelled within the embankment with an Ru of 0.25 with remaining embankment fill modelled with an Ru of 0.1.

4) For approximate chainages of embankment heights please refer to Table 9-1 within Section 9.

In Summary, Maintenance Access Road to Pond (South Arm off Calley Farm – South) from Ch.0+000 to 0+500 (GM1):

- Highway embankments with 1V:3H side slopes that require reinforced embankments, up to 3.0m in height, can be constructed using Class 2A4 / 7C3 material for meeting the required HS2 performance acceptability criteria.

In Summary, Maintenance Access Road to Pond (South Arm off Calley Farm – South) from Ch.0+500 to 0+600 (GM2):

- Highway embankments with 1V:3H side slopes that require reinforced embankments, up to 3.0m in height, can be constructed using Class 2A4 / 7C3 material for meeting the required HS2 performance acceptability criteria.
- Geogrid with 100kN/m tensile capacity required at base of the highway embankment. A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

5.4.3 Analysis of Pond Slopes

The stability of the highway where it ties into the attenuation ponds was analysed and the results are summarised below:

Table 5-6 - Summary of Slope Stability Results – C2.T.23-REG-0614 - Short / Long Term Conditions

Analysis No.	Pond Depth (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
8	1.35	1V:6H	2A4	0.662	0.226
9	1.35	1V:6H	2A4	0.701	0.321

Table 5-7 - Summary of Slope Stability Results – C2.T.21-REG-0611 – Ground Model 2 - Short / Long Term Conditions

Analysis No.	Pond Depth (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
10	2.3 (1.2 Emb / 1.1 Cutting)	1V:5H	2A4	0.874	0.429

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Analysis No.	Pond Depth (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
11	2.3 Cutting	1V:5V	2A4	0.954	0.394

In summary, the analysis shows that where the pond sides slopes interface with the highway earthworks, the embankments / side slopes will stand without requiring reinforcement.

5.5 Summary of Reinforcement Design

The following sections detail the proposed design solution(s) along the alignment for both sides of the embankment.

Table 5-8 - Summary of design solution along alignment

Left Hand Side		Right Hand Side	
Ch.	Solution	Ch.	Solution
0+000 to 0+048	Geogrid Reinforcement	0+000 to 0+048	Geogrid Reinforcement
0+048 to 0+500	Unreinforced	0+048 to 0+500	Unreinforced
0+500 to 0+600	Geogrid Reinforcement[2]	0+500 to 0+600	Geogrid Reinforcement[2]
0+600 to 0+619	Unreinforced	0+600 to 0+619	Unreinforced
0+000 to 0+034 (C2.T.23-PND-0614) Access	Unreinforced	0+000 to 0+034 (C2.T.23-PND-0614) Access	Reinforced
0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced	0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced

Notes:

1. Side of embankment taken as if looking south along chainage
2. Geogrid with 100kN/m tensile capacity is required at base of the highway embankment, as per Appendix 6/9 of the IDR208.4 Specification(1MC06-CEK-AU-SPE-CS04-000003). A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

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6 Settlement Analysis

Due the gentle change in gradient of highway embankments proposed, the rate of change of post construction ground movements are expected to be gradual and within tolerable limits for the proposed maintenance access road. No specific settlement assessment has been carried out for the highway design. Furthermore, no specific excavate and replace or hold periods will be required to manage the ground movements other than inspection by a qualified Geotechnical Engineer/ Engineering Geologist .

A specific settlement analysis will be carried out for the Culvert (061-AC3) earthworks and reported in section 4 of the Aylesbury Embankment GDR (1MC06-CEK-GT-REP-CS04_CL18-000037).

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7 Pavement Design

7.1 Introduction

The pavement design has been undertaken in accordance with the DMRB, Volume 7: Pavement Design and Construction as well as the HS2 Technical Standard and the HS2 Highway Standard Details.

The road pavements are designed in accordance with CD225, whereas the footpaths/ bridleways are designed in accordance with CD239. Design CBR values are derived for embankments and the existing ground based on the available in-situ and laboratory testing as well as correlations with other derived parameters.

With reference to CD225, for road pavement, the lowest of the short-term CBR (for construction traffic loadings) and long-term equilibrium CBR (for long-term traffic loadings) should be adopted for design. The short-term CBR is based on either plate bearing tests or CBR testing, or by correlation with undrained strength testing within the near surface soils. The long-term equilibrium CBR value is based on the Plasticity Index (PI) of the natural subgrade or fill material, based on Table C1 of LR 1132.

7.2 Assessment of Asset CBR Values

The pavement founding strata for Maintenance Access Road to Pond (South Arm off Calley Farm - South) may consist of Alluvium, Purbeck Group, Portland Group, or Class 2A4 / 7C3 fill. The design CBR for each formation stratum is presented in Table 7-1.

Table 7-1: Design CBR values

Stratum	Short Term CBR (%)	Long Term Equilibrium CBR (%)	Design CBR (%)	Comments
Class 2A4 / 7C3	2.0	2.0	2.0	CBR assumes a minimum of 50kPa undrained shear strength and plasticity index of ≤50% as specified in IDR 208.4 Specification.
Alluvium	1.0	2.0	1.0	Short-term CBR based on the undrained shear strength (using correlation of CBR = c_u/23) with design c_u value of 20kPa. Long-term CBR based upon design plasticity index of 48% and correlation with Table C1 of LR1132. Refer to Appendix C for data plots.
Portland Group	2.0	3.5	2.0	Short-term CBR based on the undrained shear strength (using correlation of CBR = c_u/23) with design c_u value of 45kPa. Long-term CBR based upon design plasticity index of 25% and correlation with Table C1 of LR1132. Refer to Appendix C for data plots.
Purbeck Group	3.5	-	3.5	Short-term CBR based on the undrained shear strength (using correlation of CBR = c_u/23) with design c_u value of 80kPa.

During construction (Ch.0+000 to 0+500), verification testing is required to confirm the in-situ CBR value is in accordance with the design. Soft spots identified as cohesive soils with a shear strength of < 45 kPa / CBR < 2.0% should be dug out and replaced as set out in the IDR 208.4 Specification.

However, during construction (Ch.0+500 to 0+619,), verification testing is also required to confirm the in-situ CBR value is in accordance with the design. Soft spots identified as cohesive soils (Alluvium) with a shear

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strength of < 20 kPa / CBR < 1.0% (Alluvium) should be dug out and replaced as set out in the IDR 208.4 Specification. Construction of at-grade access roads should be avoided in winter and wet conditions to reduce the possibility of degradation and softening of the near surface soils during construction.

7.3 Summary of Pavement Design

Considering the design CBR and the usage of the highway assets, further geotechnical input is required to ensure a suitable pavement thickness/ foundation is considered. The following sections describe the geotechnical input, where deemed appropriate, with a summary presented in Table 7-2.

7.3.1 Unbound Pavements

The design for unpaved access roads is undertaken with reference to figure C3 of LR 1132, see Figure 7-1 below. The thickness of the unpaved access road is dependent on the CBR value of the subgrade and the cumulative standard axles over its design life.

For unbound pavement type AT2 (250mm thick), considering a design CBR of 2.0% and 2 nos. Heavy Goods Vehicle movement per annum (corresponding to 392 equivalent standard axles per year), a subgrade improvement/ thickening of subgrade, with an additional 100mm of Class 6F1/2, is required to be added onto the pavement thickness (i.e., a total thickness of pavement of 350mm).

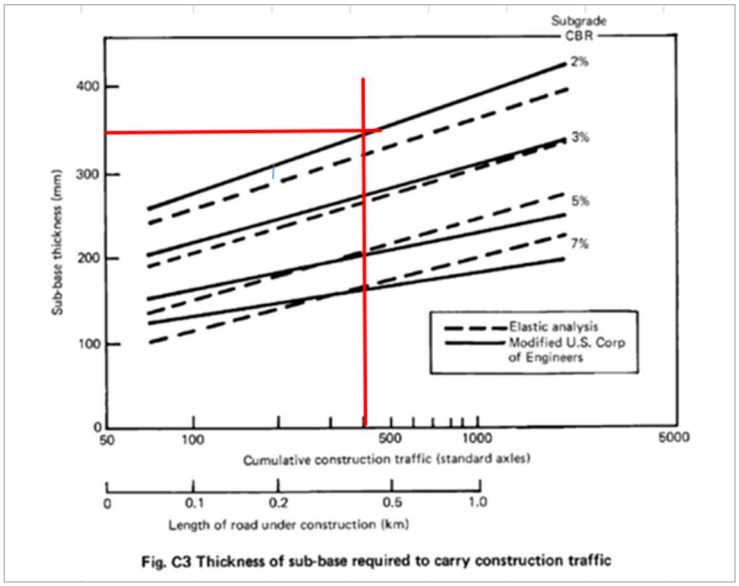


Figure 7-1: Thickness of Sub-base required for Unbound Access Tracks (CBR =2.0%)

For unbound pavement type AT2 (250mm thick), considering a design CBR of 1.0% and 2 nos. Heavy Goods Vehicle movement per annum (corresponding to 392 equivalent standard axles per year), a subgrade improvement/ thickening of subgrade, with an additional 200mm of Class 6F1/2, is required to be added onto the pavement thickness (i.e., a total thickness of pavement of 450mm).

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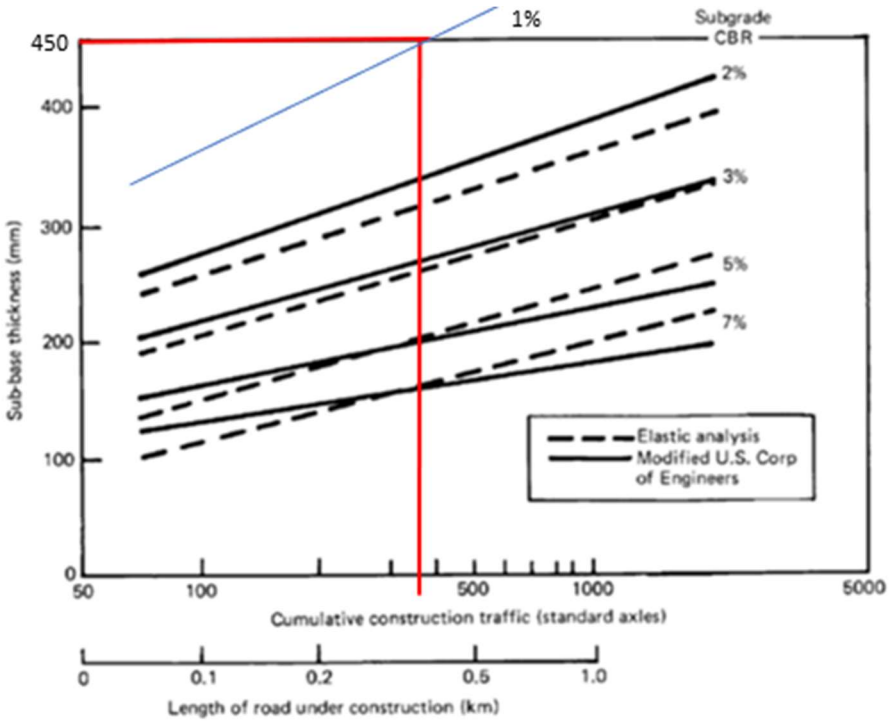


Fig. C3 Thickness of sub-base required to carry construction traffic

Figure 7-2: Thickness of Sub-base required for Unbound Access Tracks (CBR = 1.0%)

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Table 7-2: Summary of Pavement Design

Asset	Chainage		Permitted Pavement Option	Pavement Type	CBR (%)	Standard Pavement Thickness	GEO Input
	From	To					
Maintenance Access Road to Pond (South Arm off Calley Farm – South)	0	600	AT2	Unbound	2.0	250	Additional 100mm added to subbase with Type 1 (Total thickness = 350mm) [1 / 2]
	600	619			1.0		Additional 200mm added to subbase with Type 1 (Total thickness = 450mm) [1 /2]
Maintenance Access Road to Pond (South Arm off Calley Farm – South). C2.T.23-REG-0614 Pond Access	0	34			2.0		Additional 100mm added to subbase with Type 1 (Total Thickness = 350mm) [1 / 2]
Maintenance Access Road to Pond (South Arm off Calley Farm – South). Pond C2.T.21-REG-0611 Pond Access	0	37					

Notes:

[1] Assumes trafficking of 2 nos. HGV movements per annum.

[2] Subgrade mitigation has been specified to achieve a minimum subgrade modulus of 31MPa / CBR of 2.5% at the formation of the pavement (i.e., below subbase or capping).

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8 Ground Treatment

Excavate and Replace (E&R) will be carried out below earthworks where soft spots have been identified, in accordance with the Specification and SED G106 (Drawing No. 1MC06-CEK-GT-DSE-C002-000106). It is noted that the contractor may substitute Class 1 material with Class 2A4 / 2B4 for replacement material unless specified as LWA.

For earthworks at grade or up to 0.75m in height ((Ch.0+000 to 0+500), the formation level (below the top soil strip) is to be exposed and ground conditions inspected/tested such that encountered soils with an undrained shear strength of <45kPa, equivalent to a short term CBR of <2.0%, shall be excavated and replaced accordingly.

For earthworks at grade or up to 0.5m in height ((Ch.0+500 to 0+619), the formation level (below the top soil strip) is to be exposed and ground conditions inspected/tested such that encountered soils with an undrained shear strength of <20kPa, equivalent to a short term CBR of <1.0%, shall be excavated and replaced accordingly.

A high strength basal reinforcement layer, with 100kN/m tensile capacity, has been specified between chainages 0+500 to 0+600, as per Appendix 6/9 of the IDR 208.4 Specification(1MC06-CEK-AU-SPE-CS04-000003). The purpose of the inclusion of a high strength geotextile basal reinforcement is to prevent potential failure surfaces into an existing drainage ditch from extending beneath the proposed low height highway embankment and will control any potential differential settlements. A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

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9 Summary of Highway Design

The following table summarises the geotechnical design along the alignment.

Table 9-1 - Summary of Geotechnical Input – Maintenance Access Road to Pond (South Arm off Calley Farm – South)

Element	Left Hand Side									Right Hand Side								
	0+000 to 0+048	0+048 to 0+062	0+062 to 0+438	0+0438 to 0+500	0+500 to 0+545	0+545 to 0+600	0+600 to 0+619	0+000 to 0+034 (Pond C2.T.23-PND-0614	0+000 to 0+037 (Pond C2.T.21-PND-0611	0+000 to 0+048	0+048 to 0+062	0+062 to 0+438	0+438 to 0+500	0+500 to 0+545	0+545 to 0+600	0+600 to 0+619	0+000 to 0+034 (Pond C2.T.23-PND-0614	0+000 to 0+037 (Pond C2.T.21-PND-0611
Embankment Heights (m)	2.5 – 1.5	1.5 – 0.75	<0.75	0.75 – 1.5	1.5 - 2.0	2.0 - 0.75	<0.50	<2.0	<1.5	2.5 – 2.0	2.0 – 0.75	<0.75	0.75 – 1.50	1.50 – 2.0	2.0 – 0.75	<0.5	<2.0	<1.5
Side Slope Angle	1V:3H									1V:3H								
Side Slope Design	Reinforced	Soil Slope			Reinforced[5]		Soil Slope	Soil Slope	Reinforced[5]	Reinforced	Soil Slope			Reinforced[5]		Soil Slope	Reinforced[5]	
Embankment Fill Class	Class 2A4 / 7C3	Class 2A4	Class 1A4 / 2A4	Class 2A4	Class 2A4 / 7C3		Class 1A4 / 2A4	Class 2A4	Class 2A4 / 7C3	Class 2A4 / 7C3	Class 2A4		Class 1A4 / 2A4	Class 2A4 / 7C3		Class 1A4 / 2A4	Class 2A4 / 7C3	
Subgrade DSG Assumed	Class 2A4		Portland Group	Class 2A4			Alluvium	Class 2A4		Class 2A4	Portland Group		Class 2A4			Alluvium	Class 2A4	
Subgrade DSG CBR Value (%)	2.0						1.0	2.0		2.0						1.0	2.0	
Subbase Thickening Layer	Yes (100mm)						Yes (200mm)	Yes (100mm)		Yes (100mm)						Yes (200mm)	Yes (100mm)	
Pavement thickness (mm)	250									250								
Mitigation SED	G103 G114	G103	G102	G103	G103 G114		G102	G103	G103 G114	G104 G114	G103	G102	G103	G103 G114		G102	G103 G114	

Notes:
1. Details of pavement design are provided in Section 7.
2. Geotextile separation layers are to be placed below all embankments in accordance with SED's, G103, G104 (1MC06-CEK-GT-DSE-CS002-000104), G106 (1MC06-CEK-GT-DSE-CS002-000106) and G114 (1MC06-CEK-GT-DSE-CS002-000111).
3. Topsoil strip required where embankments < 3m in height.
4. Starter layer required (300mm Class 6C1) – Ch. 0+500 to 0+600
5. Geogrid with 100kN/m tensile capacity required at base of the highway embankment. A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

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10 Reuse of Material

A summary of the material likely to be produced on site:

- A nominal 300mm topsoil strip is anticipated for the entire highway alignment, to be reused as Class 5.
- In areas of minor cutting, Portland Stone Formation and Portland Sand Formation may be recovered and are assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface material. In areas of minor cutting, Alluvium Formation may be recovered and is assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface materials

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11 Specification

Please refer to document No. 1MC06-CEK-AU-SPE-CS04-000003 for IDR 208.4 Specification.

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12 Proposed Testing and Verification Plan

12.1.1 Instrumentation and monitoring

No instrumentation or monitoring is required for the asset.

12.1.2 Inspection Methods / Controls

Inspection of the formation at time of construction should be undertaken by a qualified Geotechnical Engineer/ Engineering Geologist to identify any soft spots (i.e., of an undrained shear strength < 45kPa, equivalent to a short-term CBR of 2.0%) between chainages 0+000 to 0+500.

Inspection of the formation (Ch.0+500 to 0+619) at time of construction should also be undertaken by a qualified Geotechnical Engineer/ Engineering Geologist to identify any soft spots (i.e., of an undrained shear strength < 20kPa, equivalent to a short-term CBR of 1.0%).

Identified soft spots shall be excavated and replaced (E&R), in accordance with the SED G106 (1MC06-CEK-GT-DSE-C002-000106) and the IDR 208.4 Specification, with Class 1A4 or 2A6 material to achieve the minimum shear strength/ CBR.

12.1.3 Site testing

In-situ tests will need to be undertaken at formation level during construction to confirm the CBR meets the minimum specified within the design.

Pre-construction trial pits and in-situ testing (MEXECone or similar) should be undertaken to confirm the properties of the near surface soils relevant to the design of at-grade pavements, due to the lack of testing data in the upper 2m relevant to pavement design.

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13 Risks

13.1.1 Geotechnical Risk Register

This calculation pack should be read in conjunction with Geotechnical Risk Register (GRR). A snapshot of the GRR at the time of producing this document is included in Appendix C of the Aylesbury Embankment GDR (1MC06-CEK-GT-REP-CS04_CL18-000037). The following key risks have been identified:

- Soft Ground
- High Groundwater

For a full account of the geotechnical risks refer to the IDR Specific GRR (1MC06-CEK-GT-REG-CS04-000003)

13.1.2 CDM Risk Register

This calculation pack should be read in conjunction with CDM Risk Register. This register is a live GIS database that is under ongoing review and update throughout the life of the project.

The following key risks have been identified:

- Presence of soft/loose ground.
- Presence of high groundwater

For a full account of the CDM risks refer to the IDR Specific CDM Risk Register (1MC06-CEK-AU-REG-CS04-000009).

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14 Assurance Input

14.1.1 Assumptions

Key assumptions used for the design:

- Recovered Kimmeridge Clay Formation will be used for the highway earthworks as Class 24 / 7C3 material with an upper Plasticity Index limit of 40% corresponding to Class 2A4/2B4 fill material.
- Special vehicle traffic (Special Types General Order (STGO or SO) maximum surcharge loading of 20 kPa has been assumed for stability calculations
- No surcharge or vehicular loadings will be applied in the long term to within 1m of the crest of any embankment. This is in part to manage risks associated with maintenance, to avoid vehicles running too close to the edge of the earthworks with the risk of toppling down the embankment side-slopes;
- Average conditions during construction has been assumed for the design of pavements. This means that the exposure and construction on road formations should exclude working in adverse weather or wet conditions and preferably be undertaken in the summer months;
- The presence of an impermeable lining within selected ponds will restrict the development of a Vadose Zone as defined by the HS2 Technical Standard for Earthworks.

For a full account of the assumptions used refer to the IDR Specific Assumptions Register (1MC06-CEK-AU-REG-CS04-000023).

14.1.2 Departures

The following departures are applicable to the assets listed above in section 1.2:

- DJV-DEP-000543 – Departure to clarify the requirements for checking category for highway assets. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000006 which has been accepted by HS2.
- DJV-DEP-000499 – Relaxation of Upper Limits on Liquid Limits and Plasticity Index for 7C selected wet cohesive material for use as fill to reinforced soil as set out in the HS2 SCEW. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000003 which has been accepted by HS2.
- DJV-DEP-000466 – Relaxation of Grading Limits for 7C selected wet cohesive material for use as fill to reinforced soil as set out in the HS2 SCEW. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000004 which has been accepted by HS2.

For a full account of the departures refer to the C23 Route wide Master Departures Register (1MC12-EKF-AU-REG-C000-000019).

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APPENDIX A : Maps and Drawings

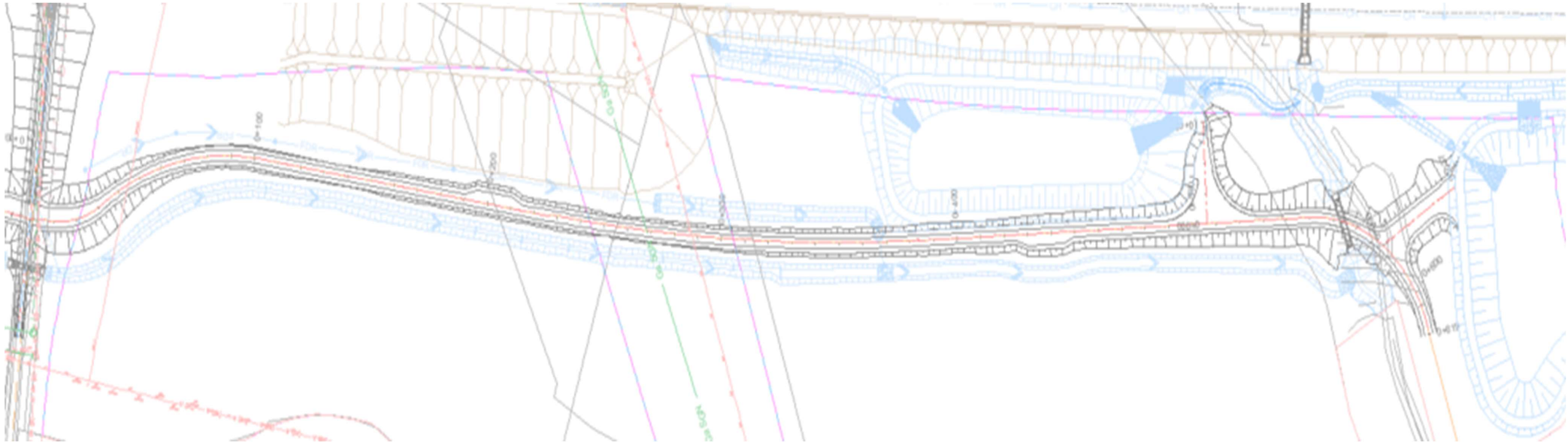
No of pages: 8

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General Site Location Plan

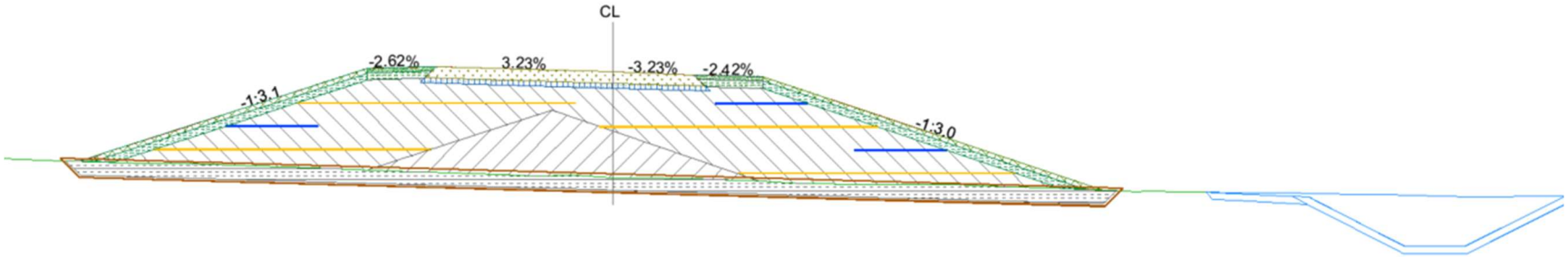
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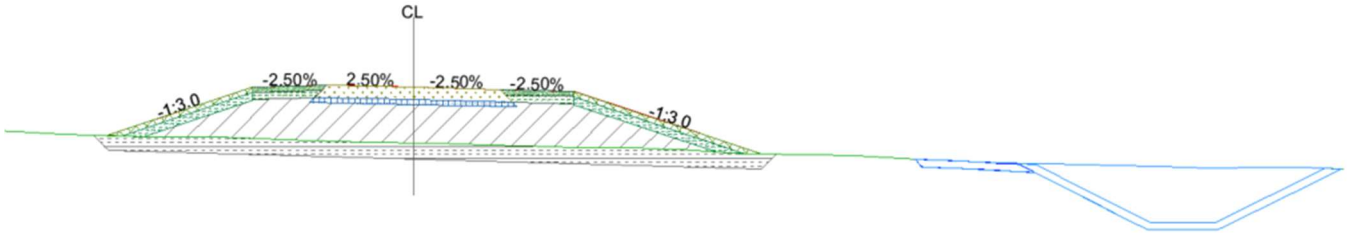
Highway Earthworks General Arrangement Plan

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Highway Earthworks Cross Sections

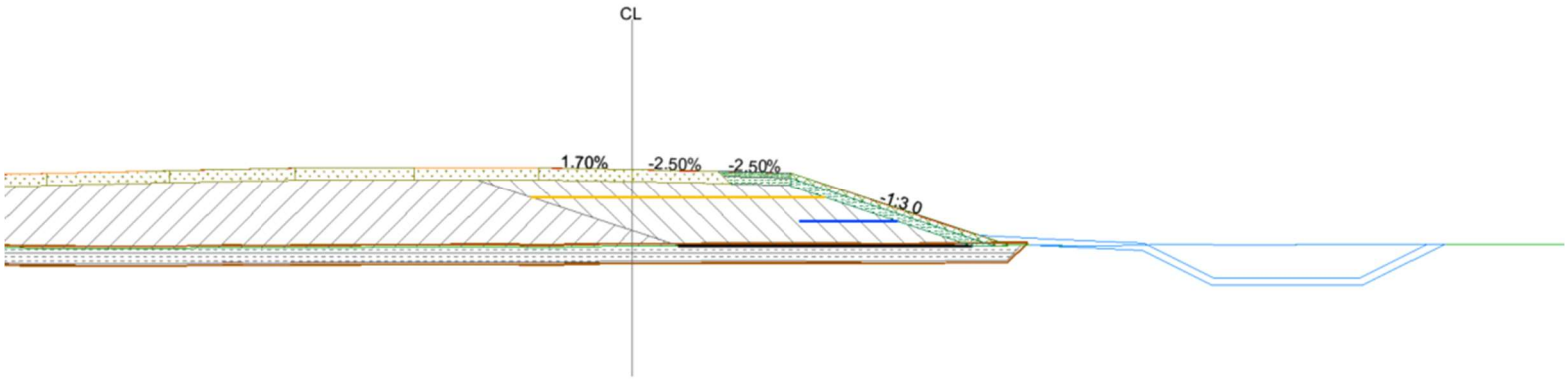


Highway Earthworks Cross Sections – Maintenance Access Road to Pond (South Arm off Calley Farm – South) Ch.0+023



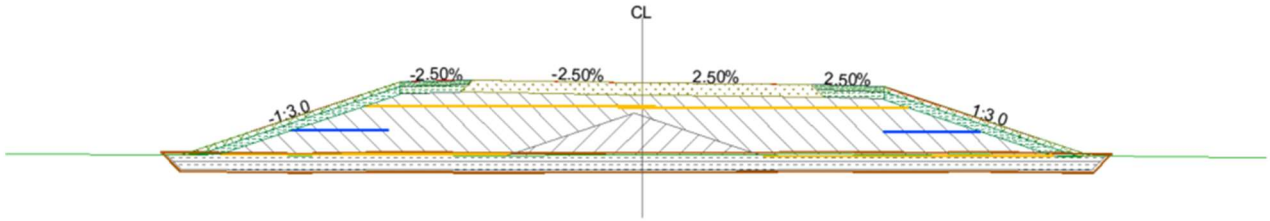
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Highway Earthworks Cross Sections – Maintenance Access Road to Pond (South Arm off Calley Farm – South) Ch.0+051



Highway Earthworks Cross Sections – Maintenance Access Road to Pond (South Arm off Calley Farm – South) Ch. 0+511

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Highway Earthworks Cross Sections – Maintenance Access Road to Pond (South Arm off Calley Farm – South) Access to Drainage Pond C2.T.21 Ch. 0+018

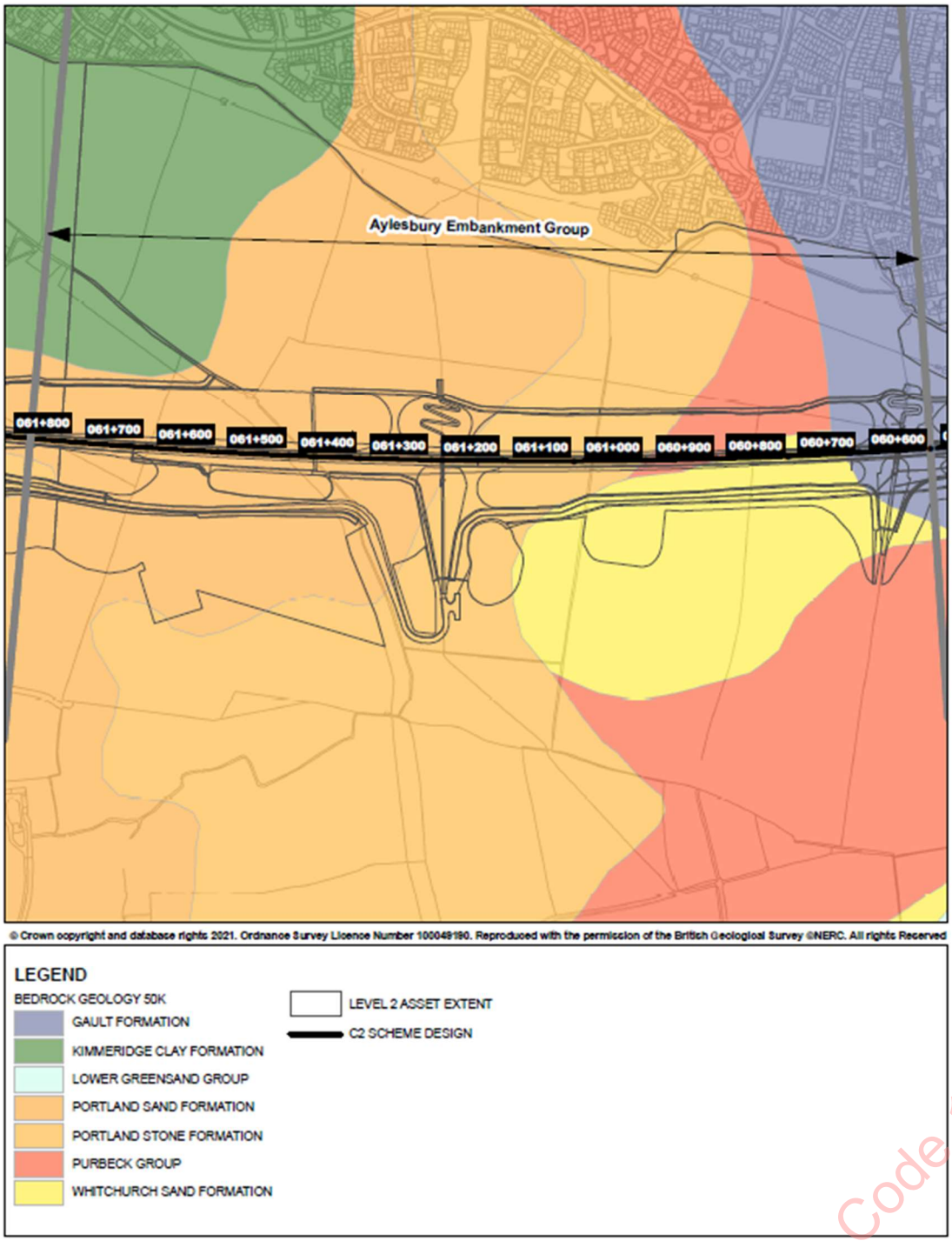
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Aylesbury Embankment Group

Bedrock Geology

ARCADIS setec COWI

0 50 100 150 200
Metres
Scale at A4: 1:7,000



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LEGEND

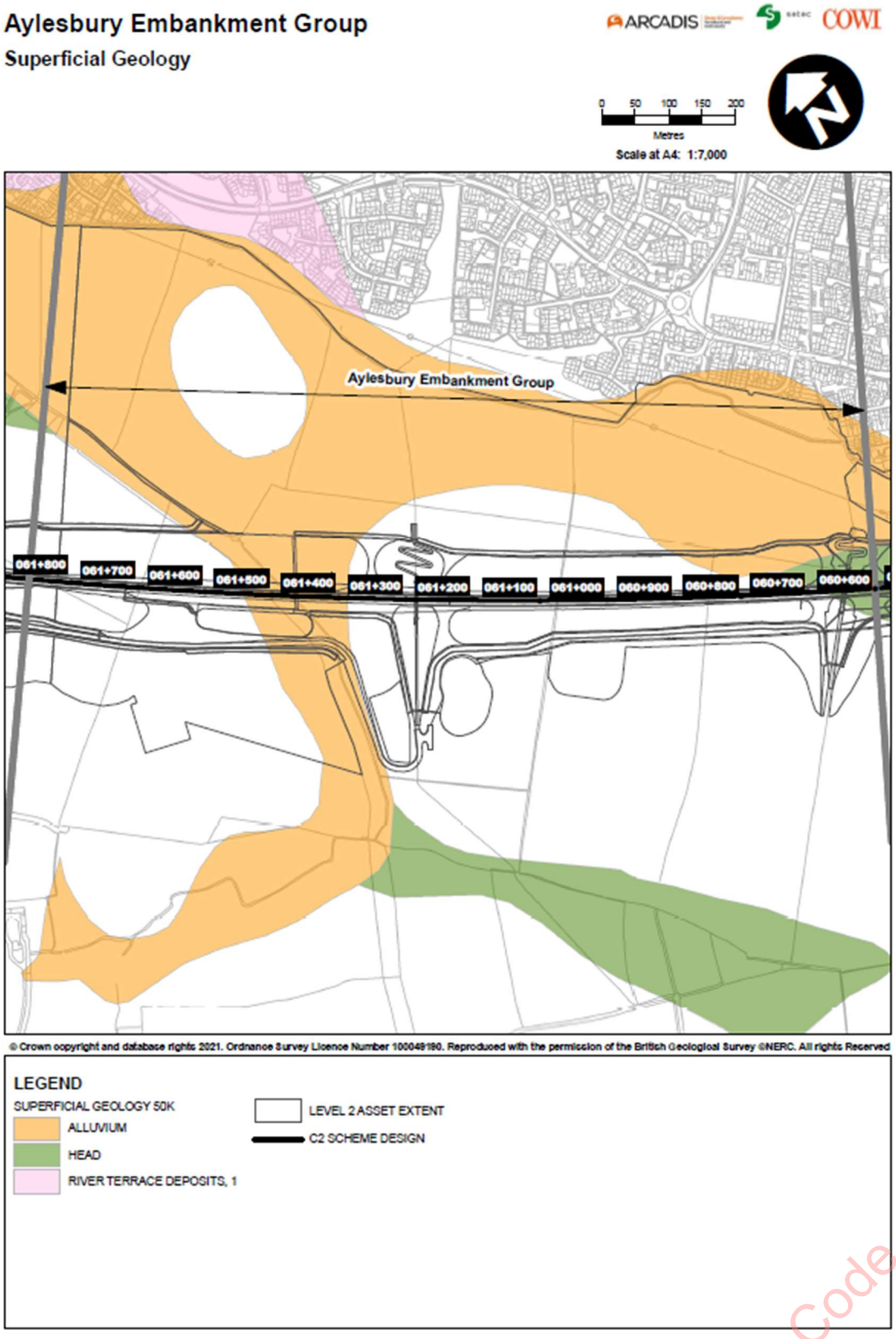
BEDROCK GEOLOGY 50K

- GAULT FORMATION
- KIMMERIDGE CLAY FORMATION
- LOWER GREENSAND GROUP
- PORTLAND SAND FORMATION
- PORTLAND STONE FORMATION
- PURBECK GROUP
- WHITCHURCH SAND FORMATION

- LEVEL 2 ASSET EXTENT
- C2 SCHEME DESIGN

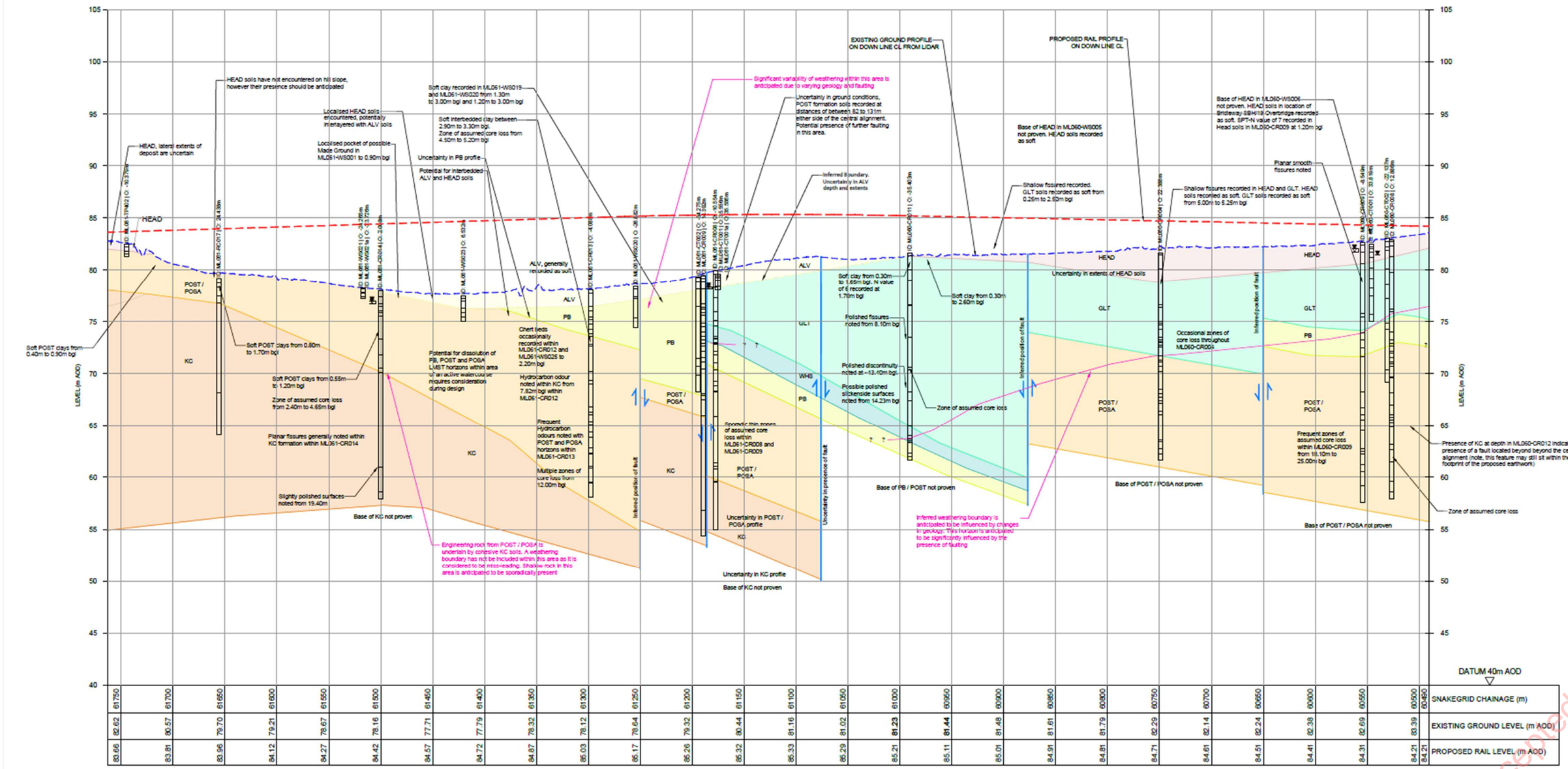
Bedrock Geology Map

HS2 Ltd - Code 1 - Accepted



Superficial geology Map

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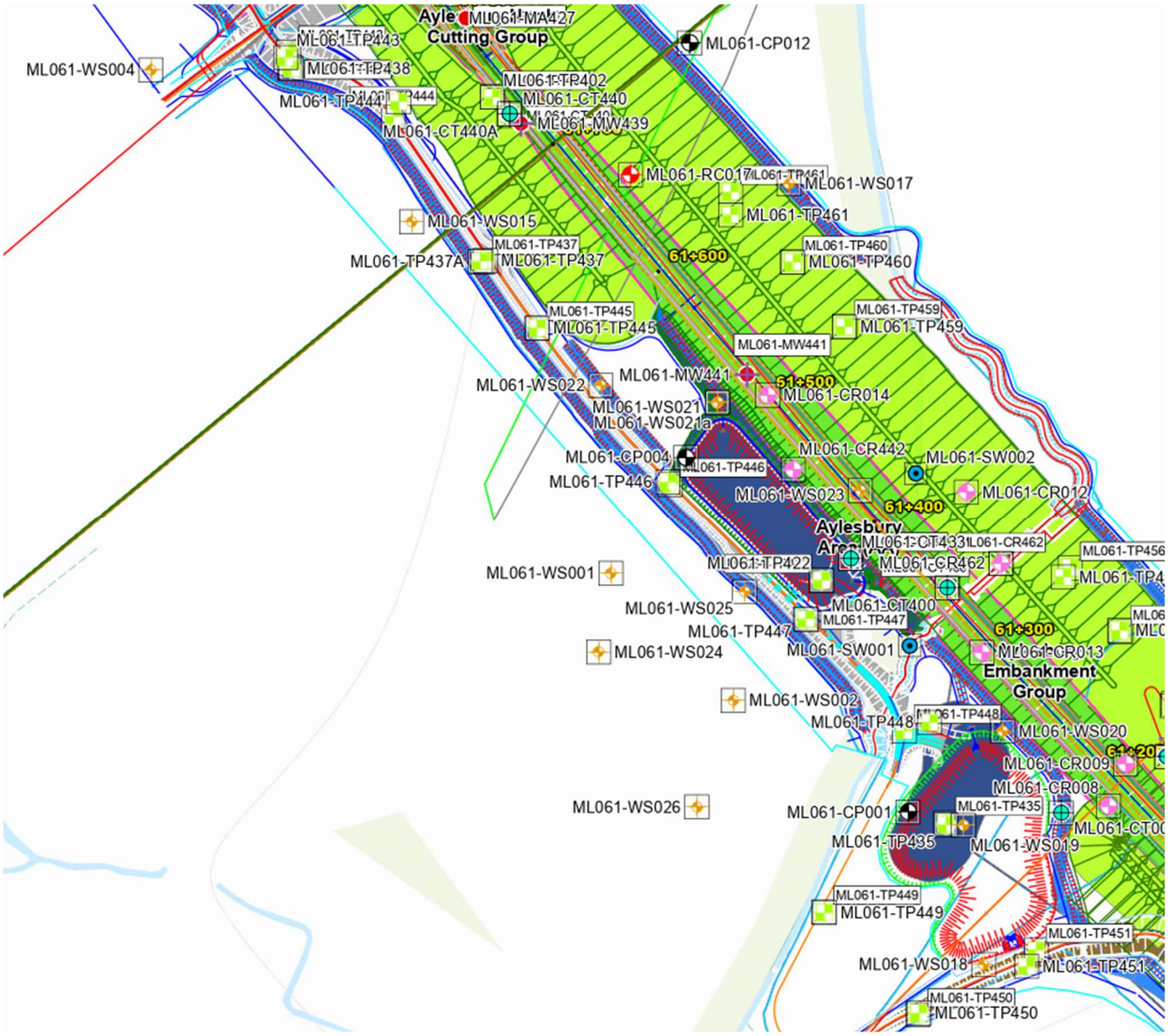
Long Section extracted from GIR

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APPENDIX B : Ground Investigation Information

Number of Pages: 3

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GI Location Plan

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Completed Exploratory Holes

Exploratory Hole ID	Type	Depth (mBGL)	Ground Level AGS (mOD)	Approx. Distance from Asset (m)	Easting	Northing
ML061-CP001	CP	4.70	78.2	10	480684	211603
ML061-CP004	CP	5.00	77.89	25	480547	211815
ML061-CR008	CP+RC	25.00	79.93	125	480806	211609
ML061-CR009	CP+RC	25.00	79.4	130	480815	211634
ML061-CR012	CP+RC	20.00	77.68	110	480716	211797
ML061-CR013	CP+RC	20.00	78.06	50	480727	211700
ML061-CR014	CP+RC	20.00	77.96	80	480595	211854
ML061-CR419	CP+RC	30.00	85.12	60	480313	212121
ML061-RC017	RC	15.00	79.15	100	480510	211985
ML061-TP402	TP	1.20	82.51	45	480426	212031
ML061-TP422	TP	3.80	77.82	20	480630	211742
ML061-TP435	TP	4.00	78.44	50	480707	211596
ML061-TP437	TP	0.50	80.32	0	480423	211932
ML061-TP437A	TP	2.20	80.32	0	480419	211931
ML061-TP438	TP	3.2	83.12	0	480304	212046
ML061-WS001	WLS	4.45	78.25	60	480502	211744
ML061-WS002	WLS	50.00	78.2	60	480578	211669
ML061-WS004	WLS	2.45	84.33	65	480219	212044
ML061-WS011	WLS	2.95	84.36	25	480286	212096
ML061-WS015	WLS	1.65	81.22	25	480378	211955
ML061-WS018	WLS	4.00	79.78	0	480731	211512
ML061-WS019	WLS	3.00	78.53	45	480718	211596

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Exploratory Hole ID	Type	Depth (mBGL)	Ground Level AGS (mOD)	Approx. Distance from Asset (m)	Easting	Northing
ML061-WS020	WLS	4.00	78.4	55	480741	211653
ML061-WS021	WLS	0.70	78.14	50	480565	211849
ML061-WS021a	WLS	1.05	78.26	50	480565	211849
ML061-WS022	WLS	2.45	78.70	5	480494	211858
ML061-WS023	WLS	2.45	77.46	75	480652	211797
ML061-WS024	WLS	3.95	79.08	100	480495	211697
ML061-WS025	WLS	5.00	77.87	10	480583	211735
ML061-WS026	WLS	2.65	78.91	85	480556	211604

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APPENDIX C Selected Data Plots

Number of Pages: 5

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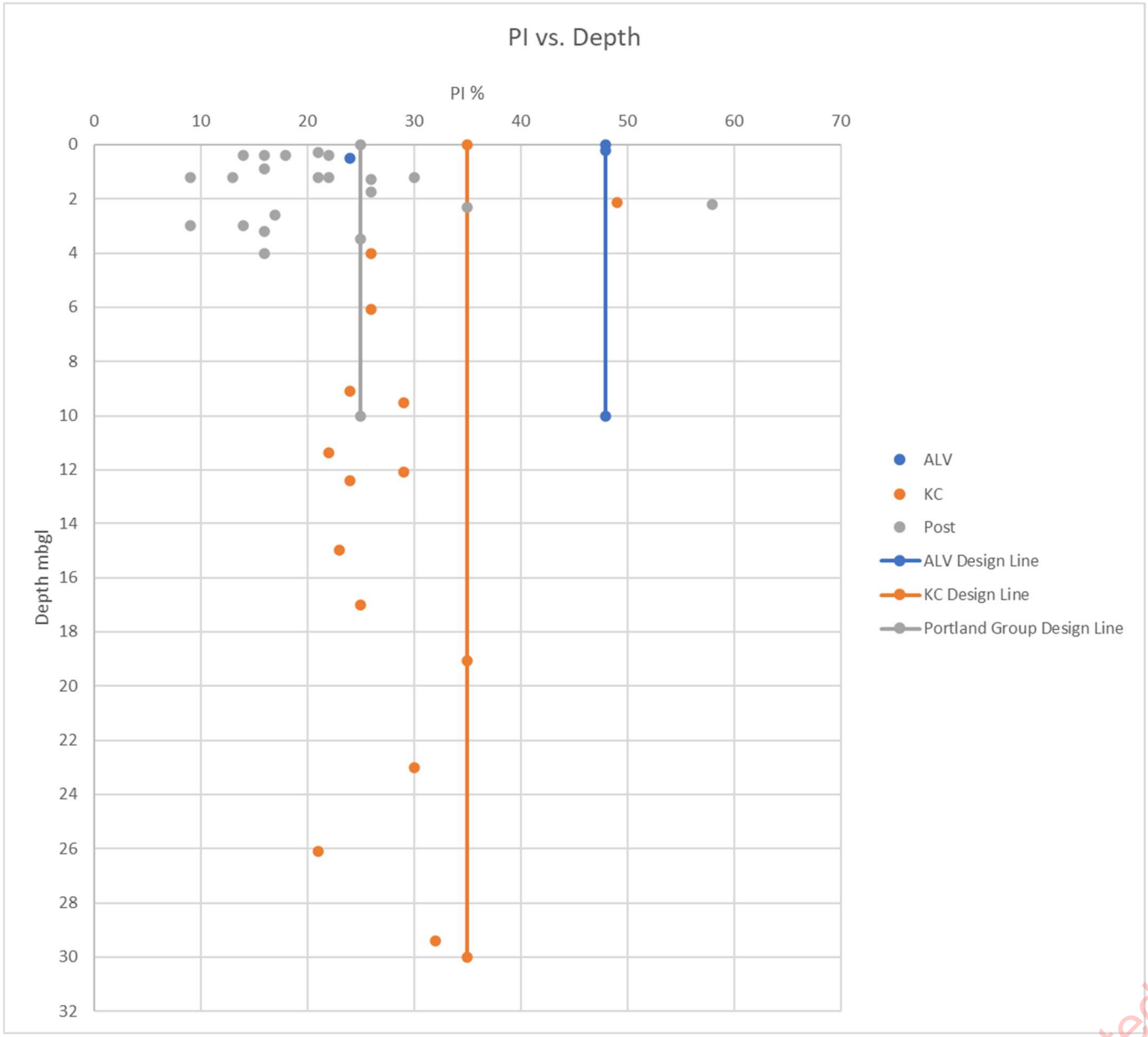


Figure C1: Plasticity Index vs Depth

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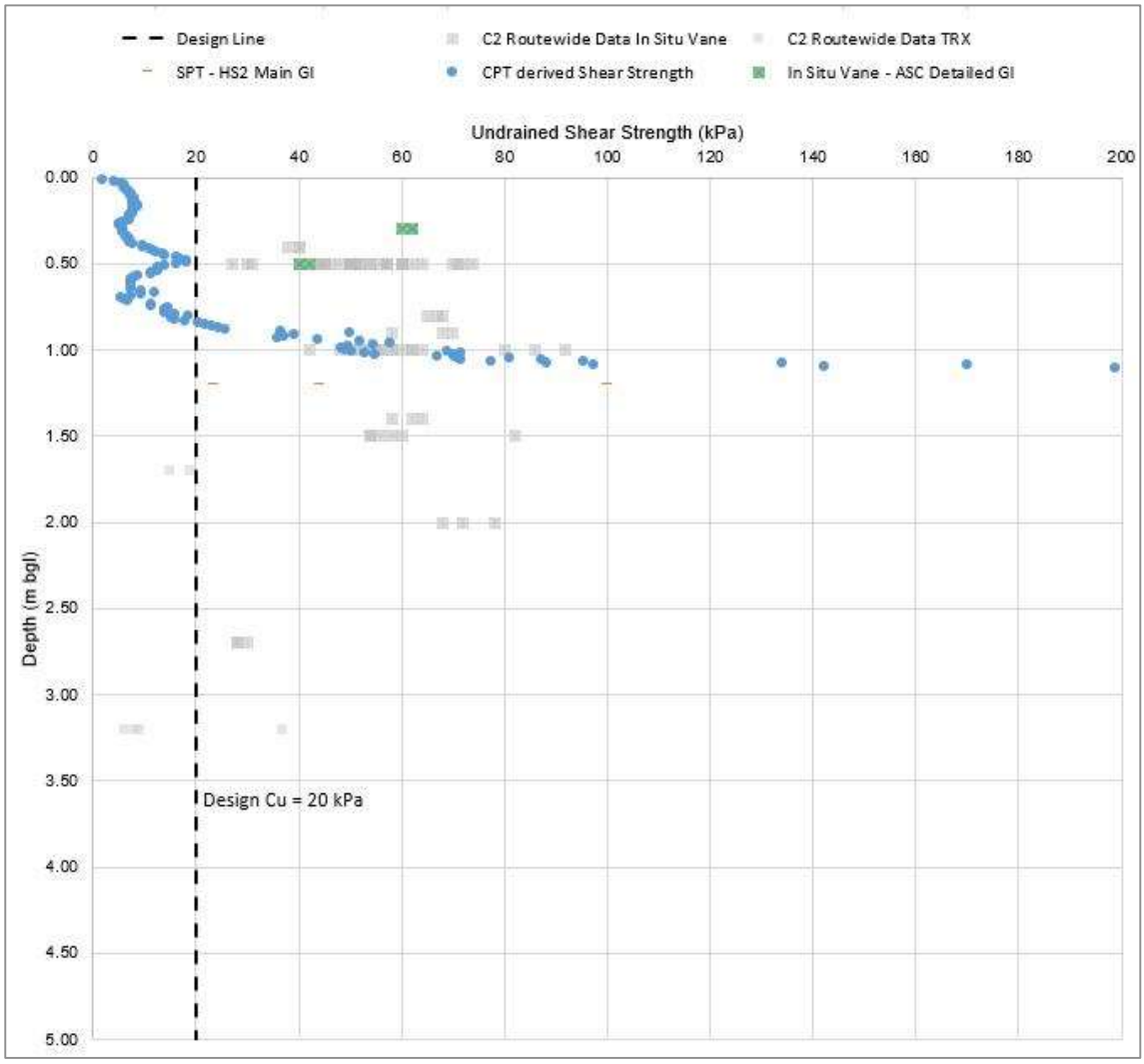


Figure C-2 - Undrained Shear Strength for Alluvium

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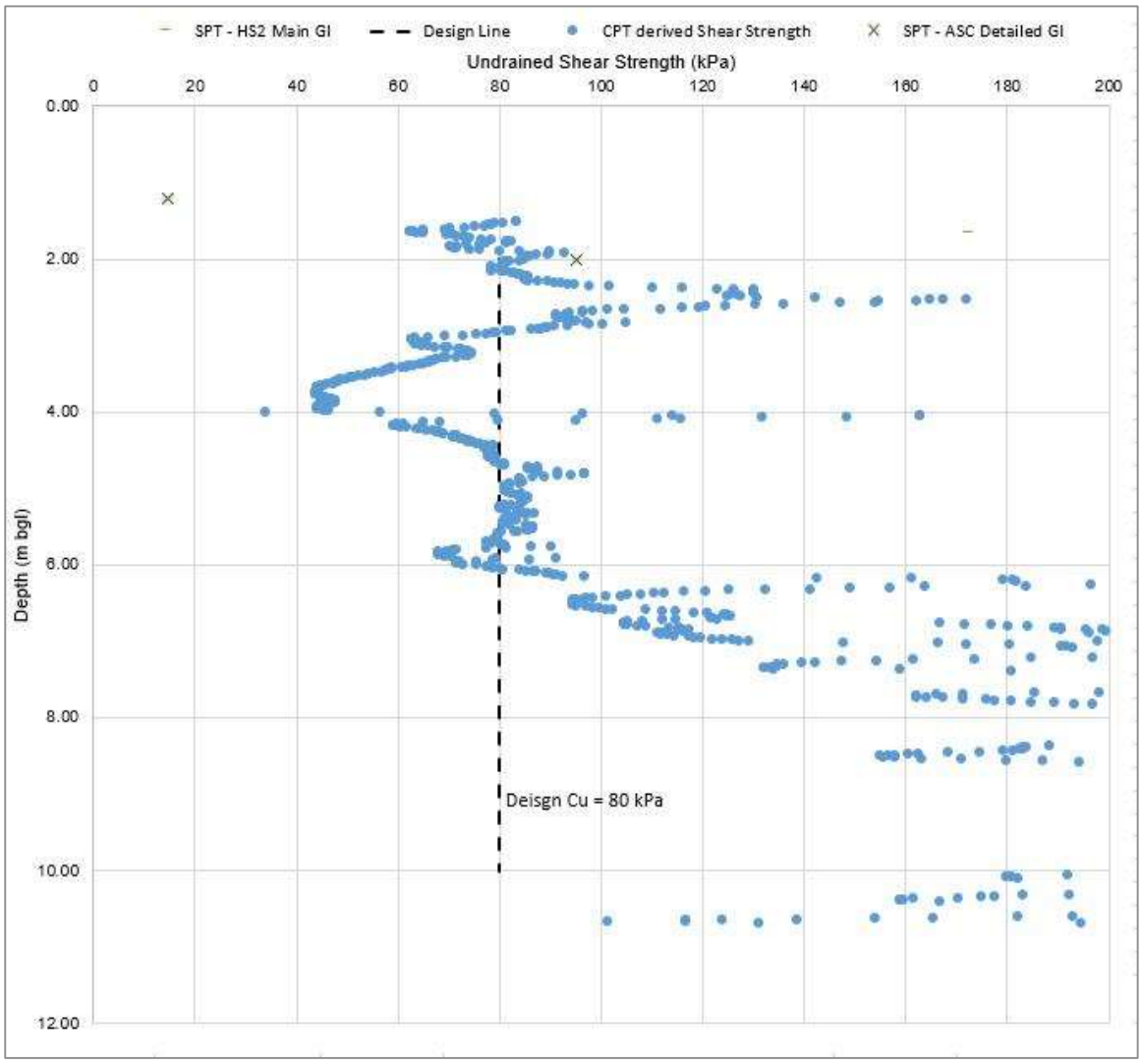


Figure C-3 - Undrained Shear Strength for Purbeck Group

HS2 Ltd - Code 1 - Accepted

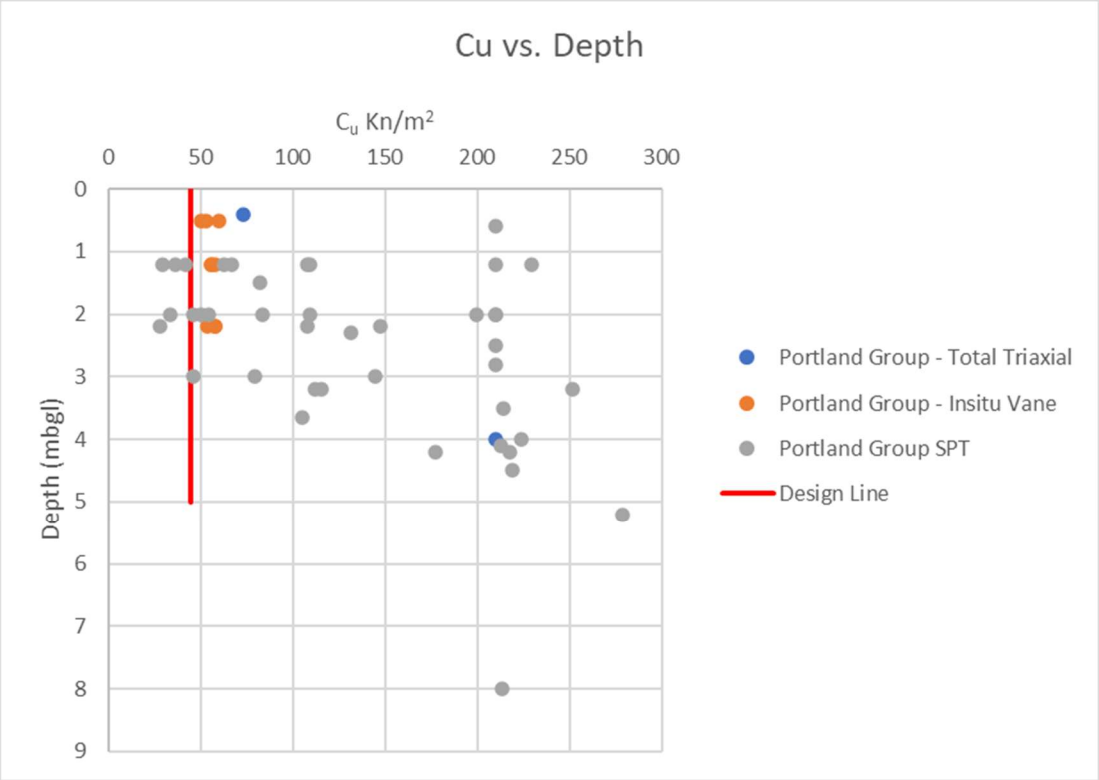


Figure C-4 - Undrained Shear Strength for Portland Group

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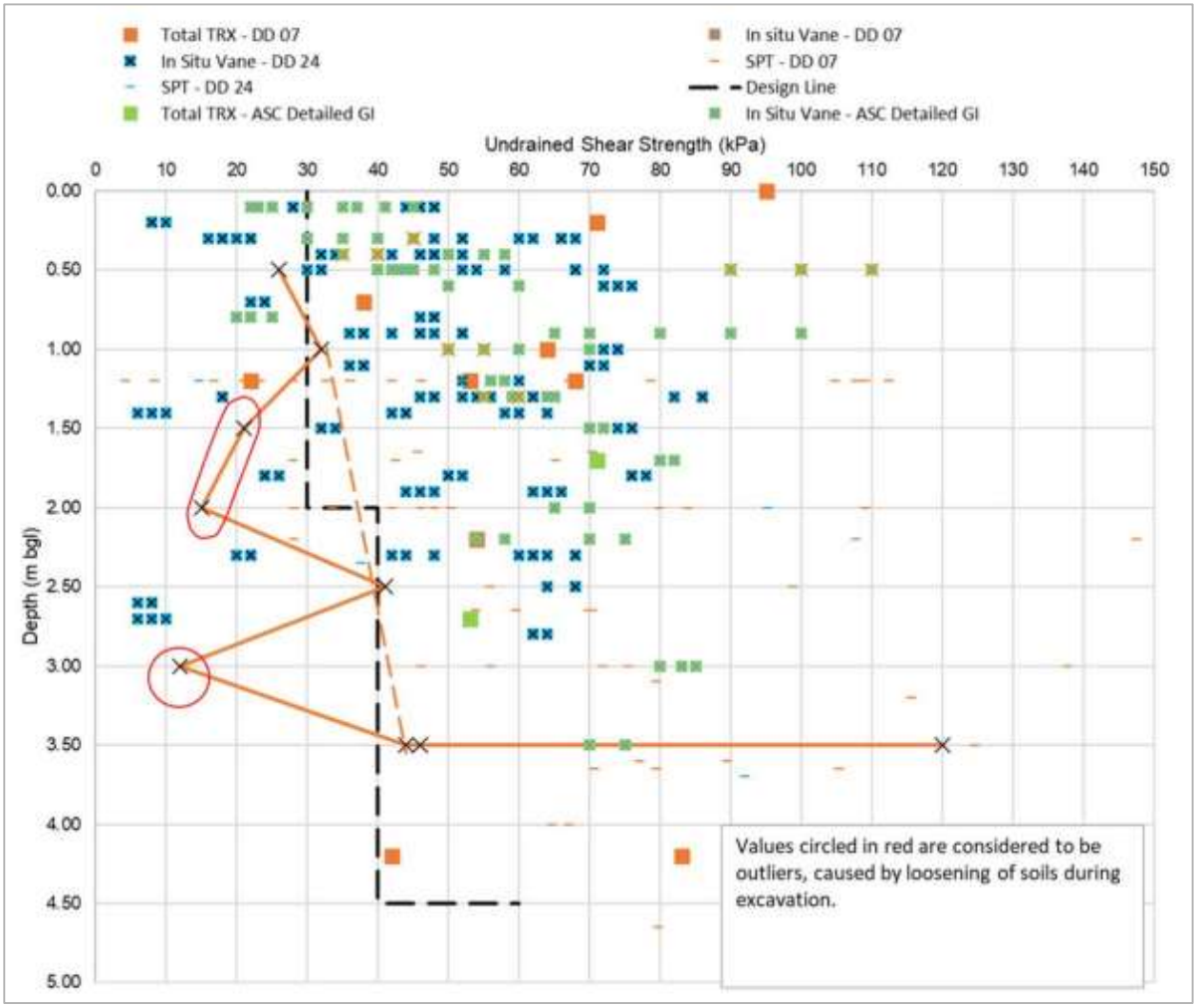


Figure C-5 - Undrained Shear Strength for Kimmeridge Clay

Conservative strength parameters adopted within Alluvium Extent:

Depth from 0.0 to 2.0 mbgl: Cu = 30 kPa

Depth from 2.0 to 4.5 mbgl: Cu = 40 kPa

Depth > 4.5mbgl: Cu = 40+8z

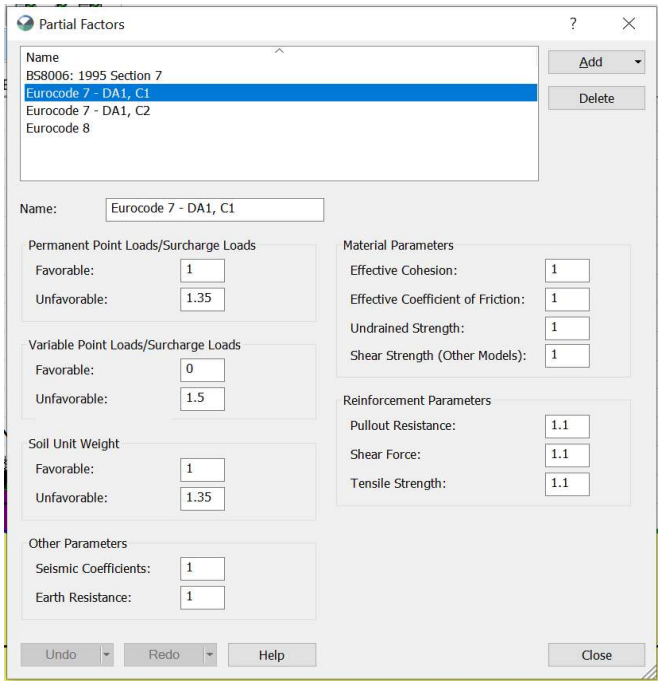
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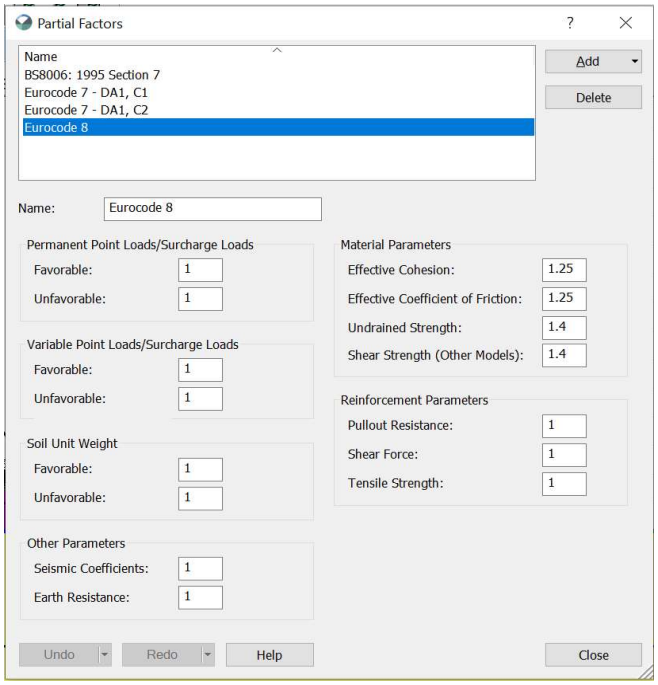
APPENDIX D : Design Summary Outputs

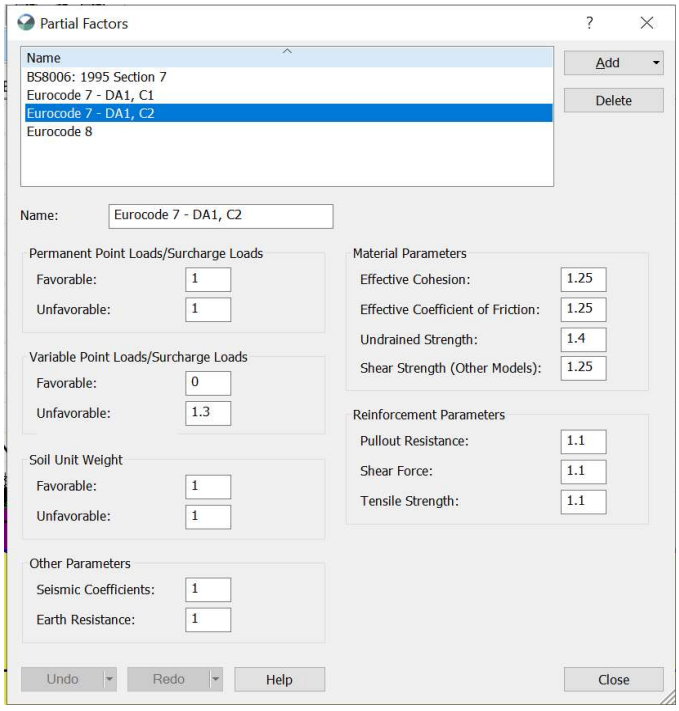
Number of Pages: 14

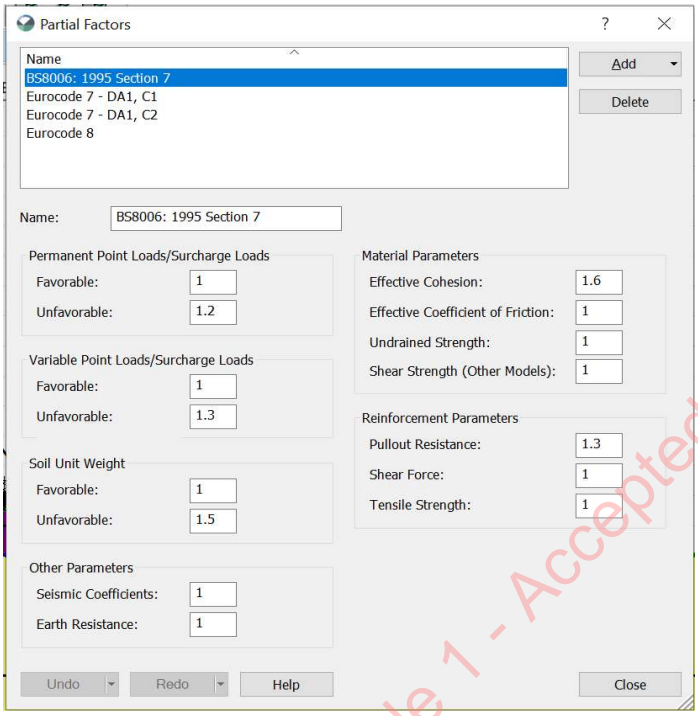
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Slope Stability Partial Factors

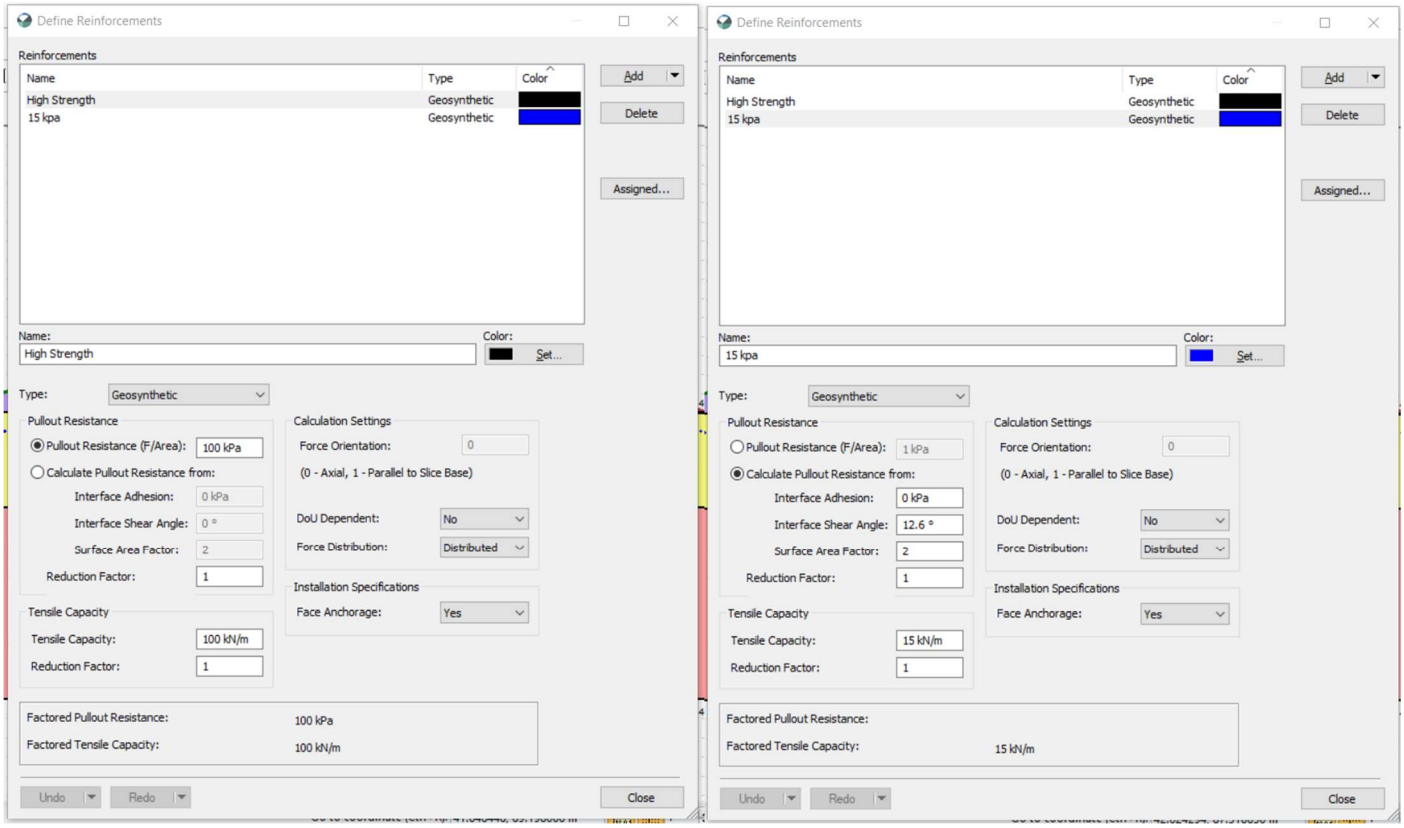






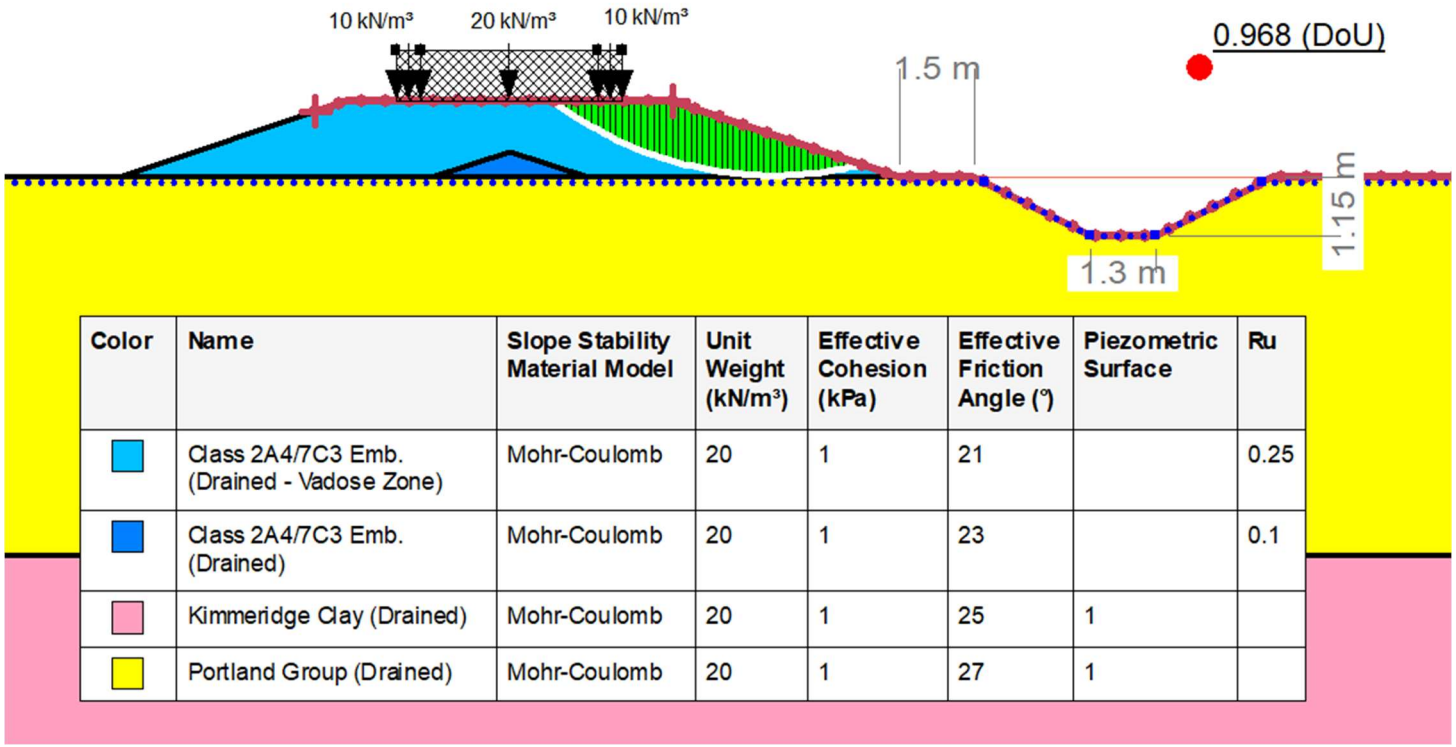


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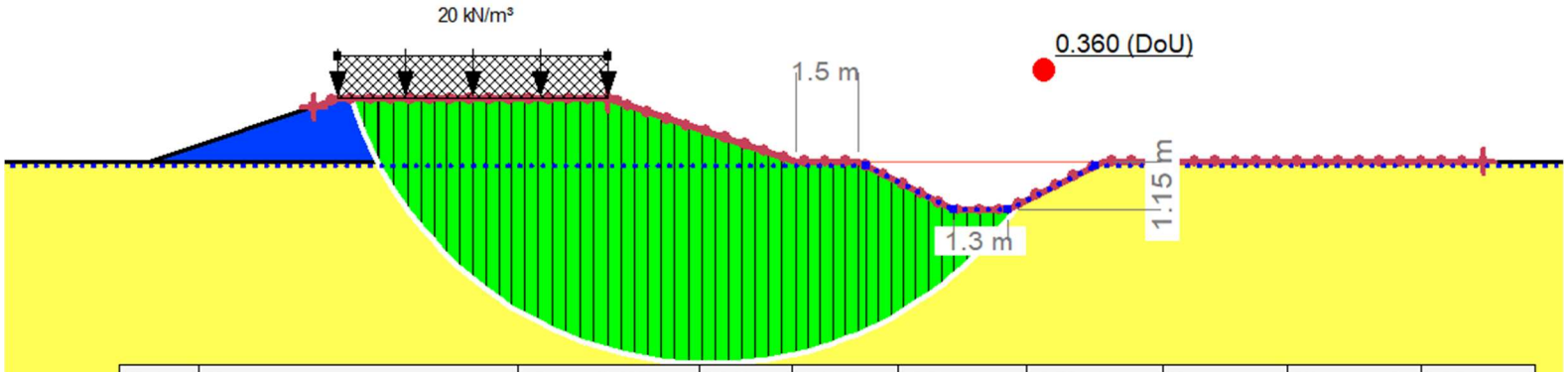
Selected Results



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface	Ru
	Class 2A4/7C3 Emb. (Drained - Vadose Zone)	Mohr-Coulomb	20	1	21		0.25
	Class 2A4/7C3 Emb. (Drained)	Mohr-Coulomb	20	1	23		0.1
	Kimmeridge Clay (Drained)	Mohr-Coulomb	20	1	25	1	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27	1	

Analysis No.2.: Embankment Height 1.5m (Ground Model 1) Drained conditions EC7 - DA1C2 Unreinforced DoU= 0.968

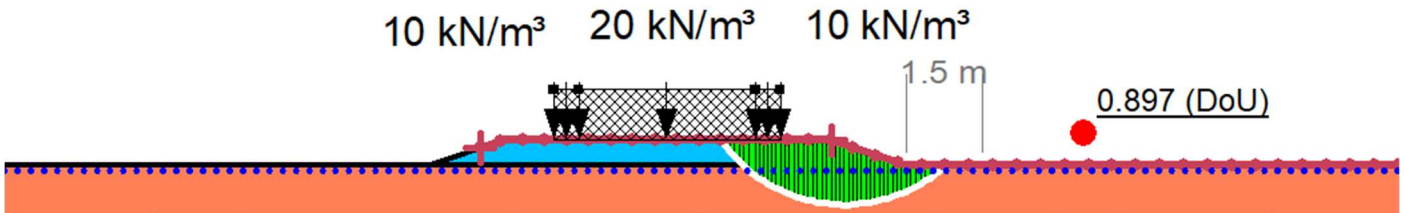
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Piezometric Surface	Total Cohesion (kPa)
	Class 2A4/7C3 Emb. (Undrained)	Undrained (Phi=0)	20						50
	Kimmeridge Clay (Undrained)	S=f(datum)	20	64	8	150	-7.5	1	
	Portland Group (Undrained)	Undrained (Phi=0)	20					1	45

Analysis No.2.: Embankment Height 1.5m (Ground Model 1) Undrained conditions EC7 - DA1C2 Unreinforced DoU= 0.360

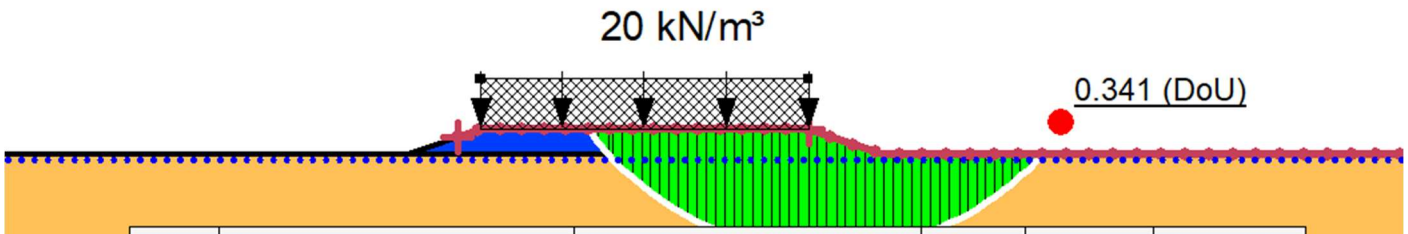
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	0	22		1
	Class 2A4/7C3 Emb. (Drained - Vadose Zone)	Mohr-Coulomb	20	1	21	0.25	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27		1
	Purbeck Group (Drained)	Mohr-Coulomb	20	1	24		1

Analysis No.4.: Embankment Height 0.5m (Ground Model 2) Drained conditions EC7 - DA1C2 Unreinforced DoU= 0.897

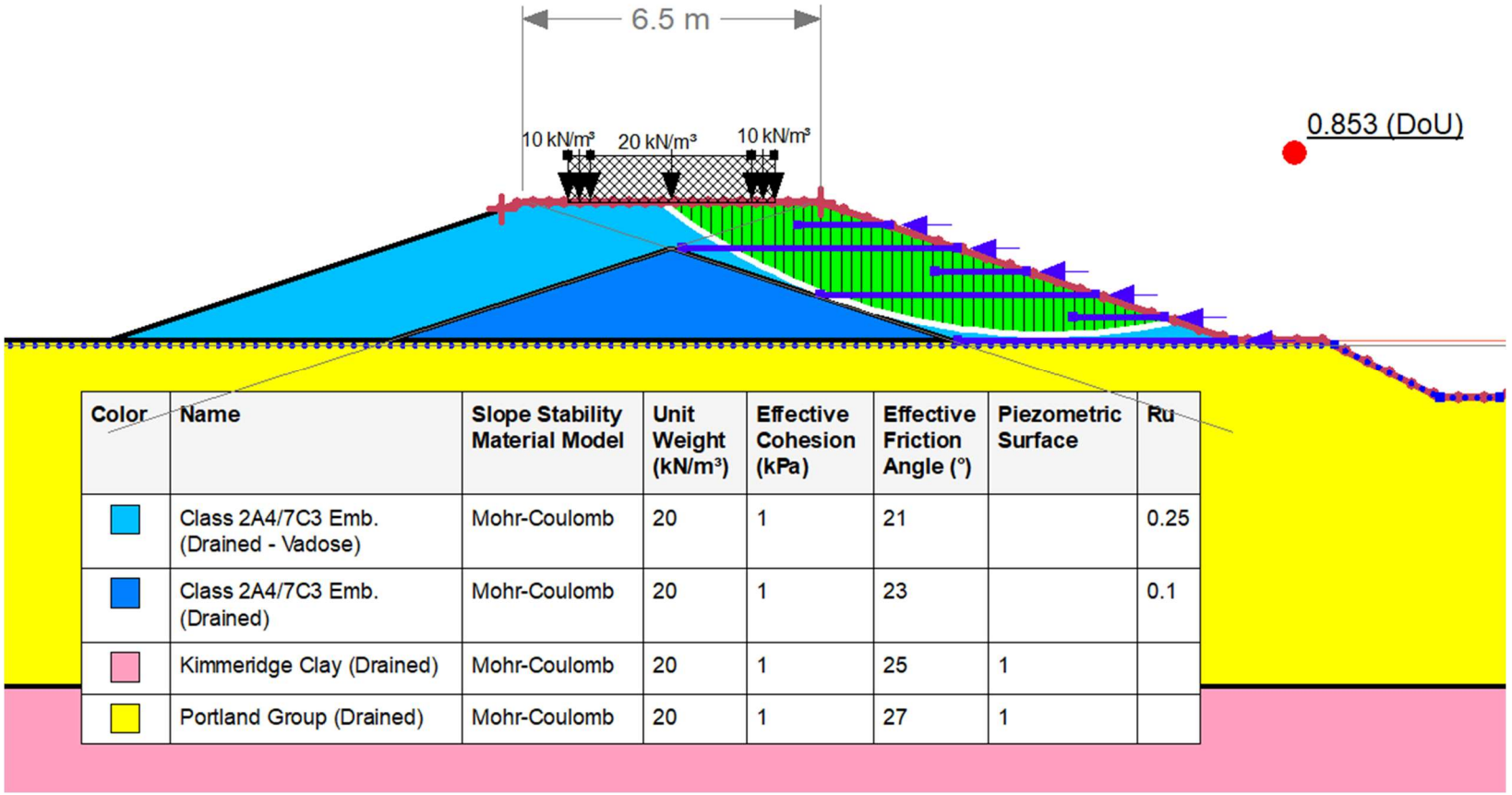
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Piezometric Surface
	Alluvium (Undrained)	Undrained (Phi=0)	18	20	1
	Class 2A4/7C3 Emb. (Undrained)	Undrained (Phi=0)	20	50	
	Portland Group (Undrained)	Undrained (Phi=0)	20	45	1
	Purbeck Group (Undrained)	Undrained (Phi=0)	20	80	1

Analysis No.4.: Embankment Height 0.5m (Ground Model 2) Undrained conditions EC7 - DA1C2 Unreinforced DoU= 0.341

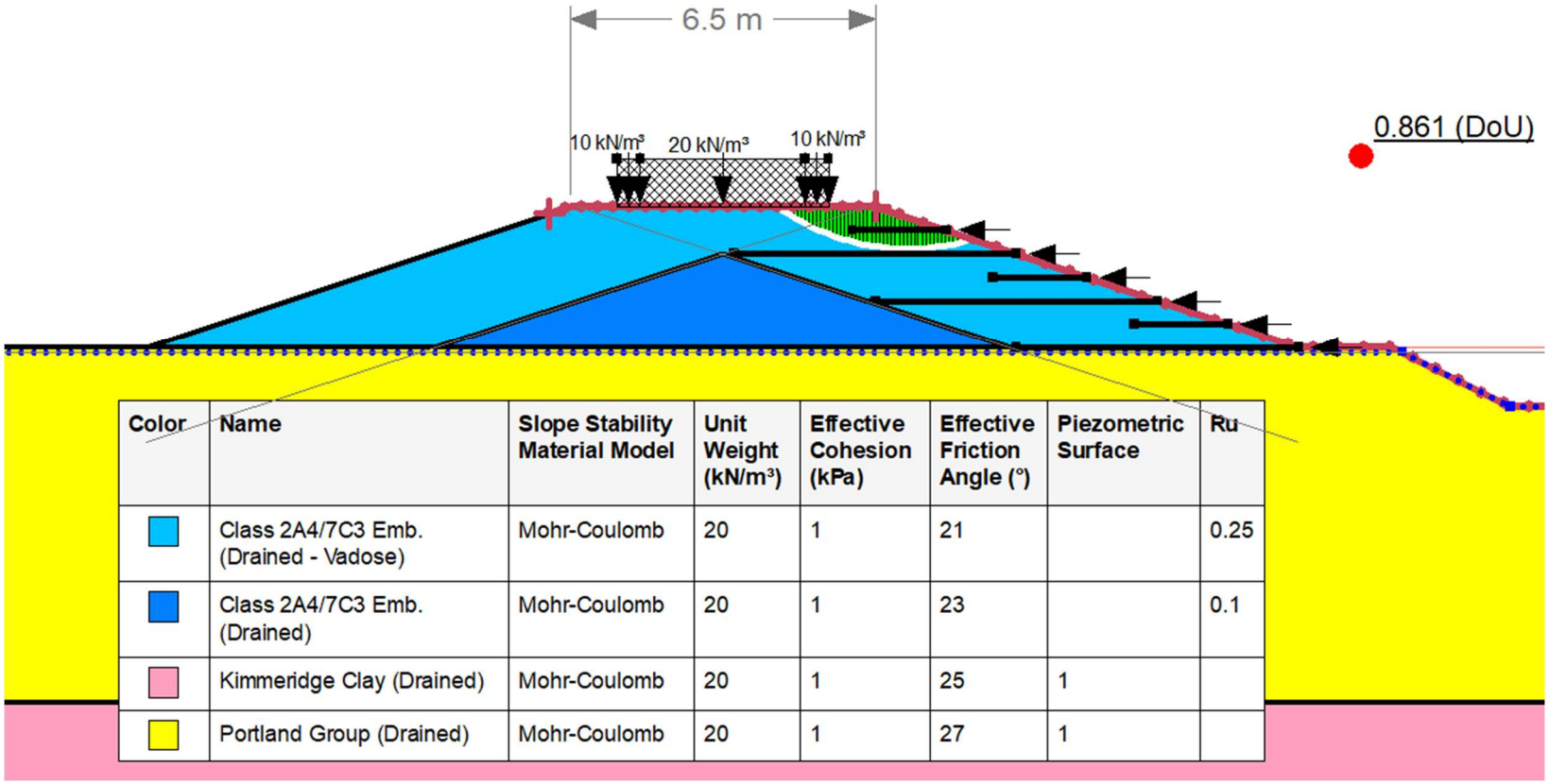
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface	Ru
	Class 2A4/7C3 Emb. (Drained - Vadose)	Mohr-Coulomb	20	1	21		0.25
	Class 2A4/7C3 Emb. (Drained)	Mohr-Coulomb	20	1	23		0.1
	Kimmeridge Clay (Drained)	Mohr-Coulomb	20	1	25	1	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27	1	

Analysis No.5.: Embankment Height 3.0m (Ground Model 1) Drained conditions BS8006 Reinforced DoU=0.853

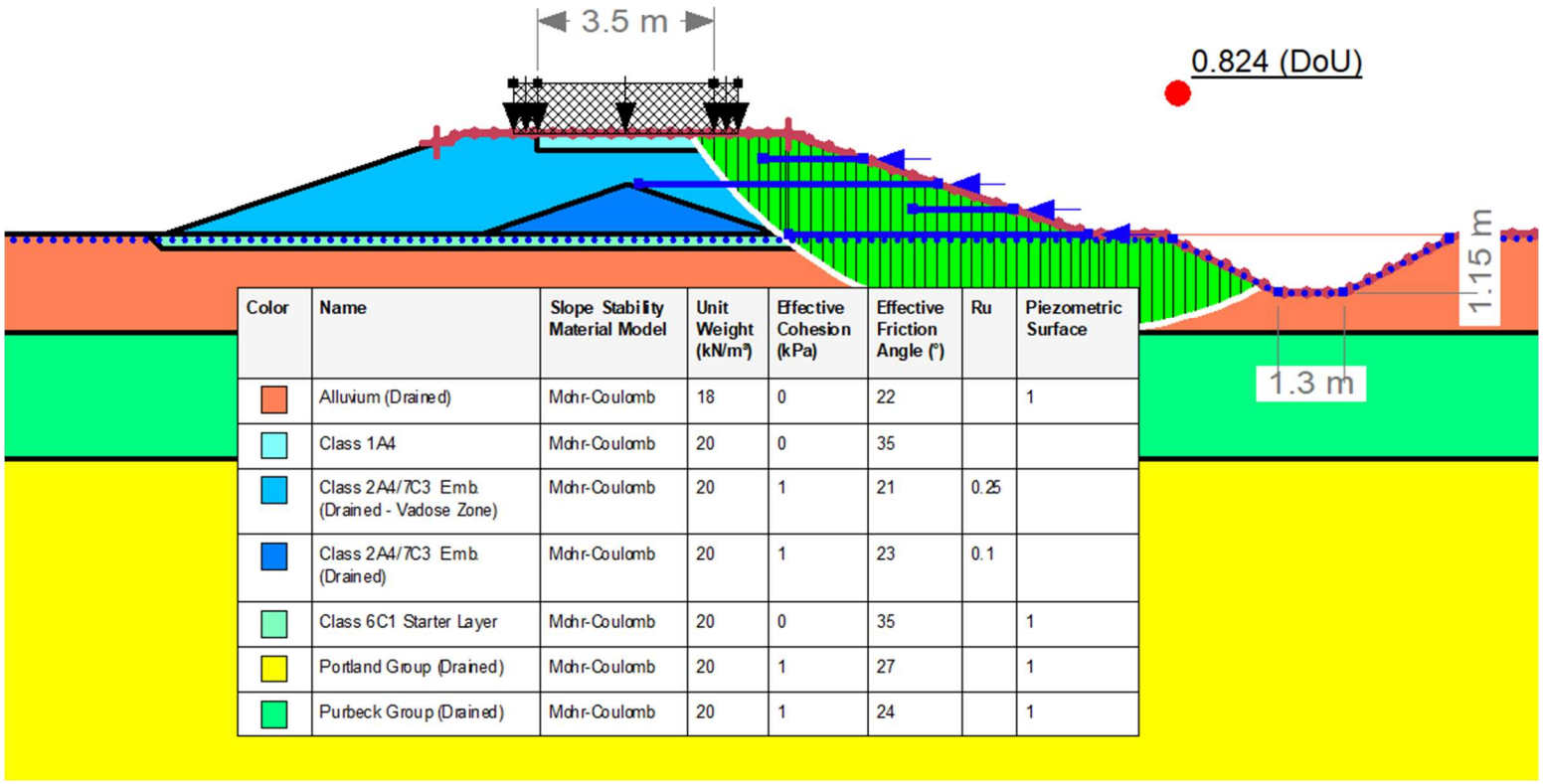
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface	Ru
	Class 2A4/7C3 Emb. (Drained - Vadose)	Mohr-Coulomb	20	1	21		0.25
	Class 2A4/7C3 Emb. (Drained)	Mohr-Coulomb	20	1	23		0.1
	Kimmeridge Clay (Drained)	Mohr-Coulomb	20	1	25	1	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27	1	

Analysis No.5.: Embankment Height 3.0m (Ground Model 1) Drained conditions EC7- DA1C2 Reinforced DoU=0.861

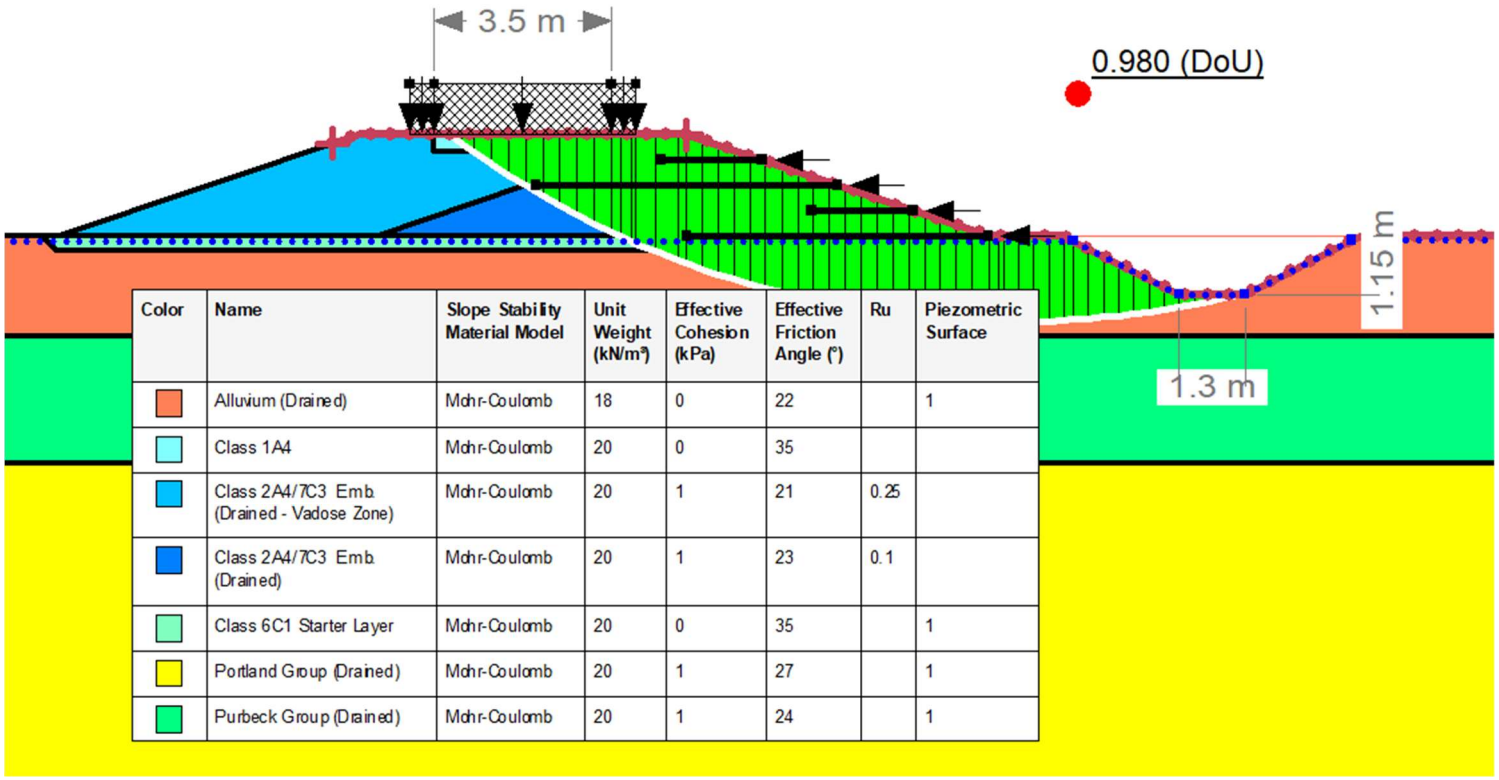
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	0	22		1
	Class 1A4	Mohr-Coulomb	20	0	35		
	Class 2A4/7C3 Emb. (Drained - Vadose Zone)	Mohr-Coulomb	20	1	21	0.25	
	Class 2A4/7C3 Emb. (Drained)	Mohr-Coulomb	20	1	23	0.1	
	Class 6C1 Starter Layer	Mohr-Coulomb	20	0	35		1
	Portland Group (Drained)	Mohr-Coulomb	20	1	27		1
	Purbeck Group (Drained)	Mohr-Coulomb	20	1	24		1

Analysis No.6.: Embankment Height 2.0m (Ground Model 2) Drained conditions BS8006 Reinforced DoU=0.824

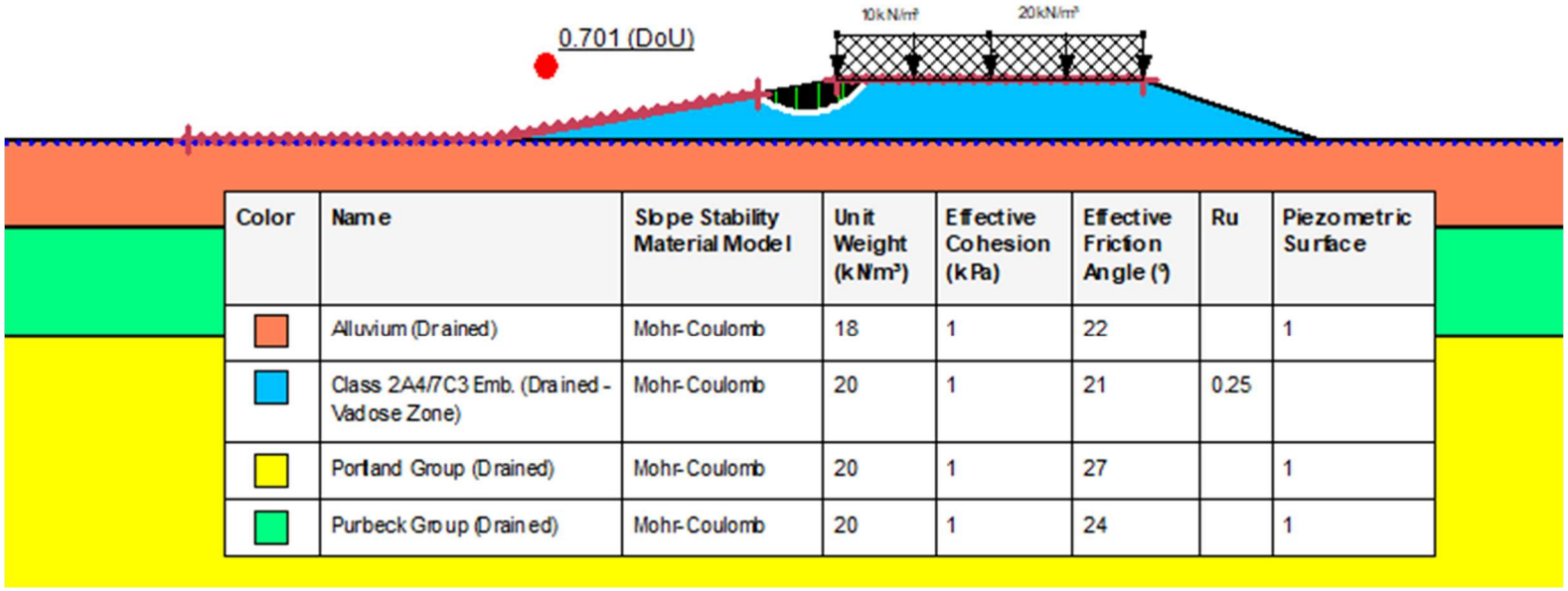
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	0	22		1
	Class 1A4	Mohr-Coulomb	20	0	35		
	Class 2A4/7C3 Emb (Drained - Vadose Zone)	Mohr-Coulomb	20	1	21	0.25	
	Class 2A4/7C3 Emb (Drained)	Mohr-Coulomb	20	1	23	0.1	
	Class 6C1 Starter Layer	Mohr-Coulomb	20	0	35		1
	Portland Group (Drained)	Mohr-Coulomb	20	1	27		1
	Purbeck Group (Drained)	Mohr-Coulomb	20	1	24		1

Analysis No.6.: Embankment Height 2.0m (Ground Model 2) Drained conditions EC7- DA1C2 Reinforced DoU=0.980

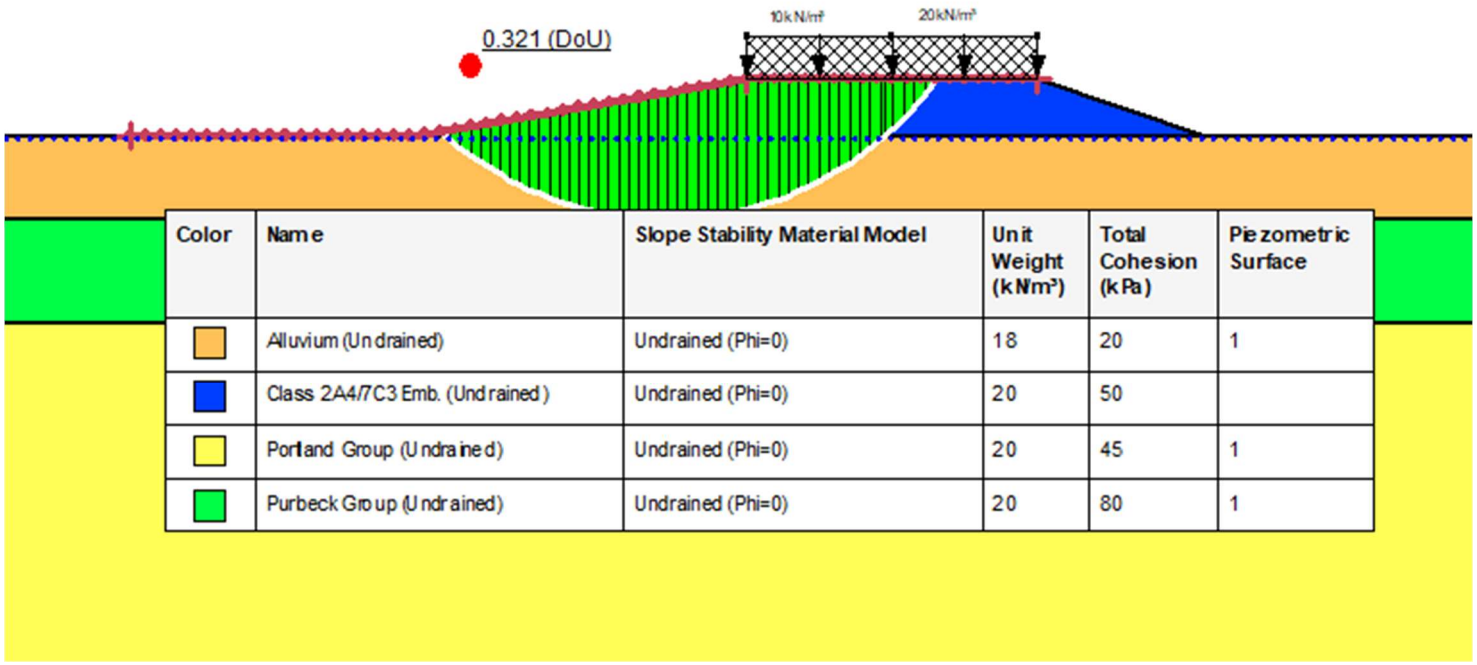
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	1	22		1
	Class 2A4/7C3 Emb. (Drained - Vadose Zone)	Mohr-Coulomb	20	1	21	0.25	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27		1
	Purbeck Group (Drained)	Mohr-Coulomb	20	1	24		1

Analysis No 8.: Pond C2.T.23-PND-0614 cutting adjacent to access road (Ground Model 2) Drained conditions EC7 - DA1C2 Unreinforced DoU=0.701

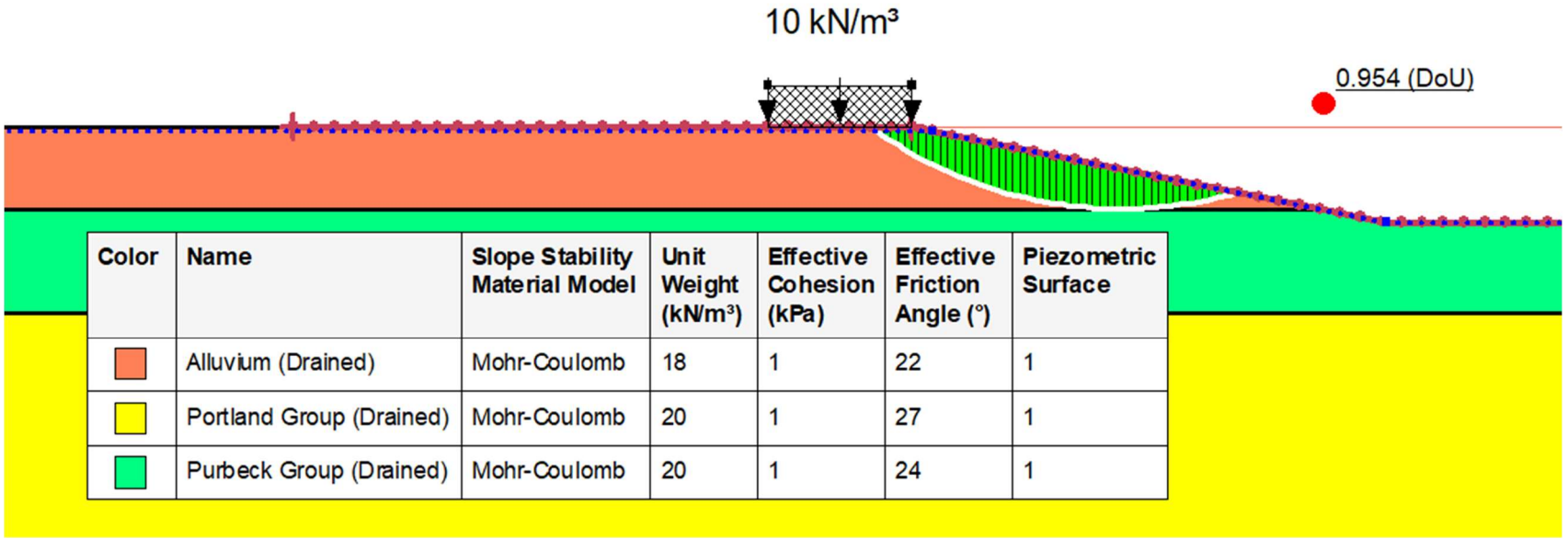
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Piezometric Surface
	Alluvium (Undrained)	Undrained (Phi=0)	18	20	1
	Class 2A4/7C3 Emb. (Undrained)	Undrained (Phi=0)	20	50	
	Portland Group (Undrained)	Undrained (Phi=0)	20	45	1
	Purbeck Group (Undrained)	Undrained (Phi=0)	20	80	1

Analysis No 8.: Pond C2.T.23-PND-0614 cutting adjacent to access road (Ground Model 2) Undrained conditions EC7 - DA1C2 Unreinforced DoU=0.321

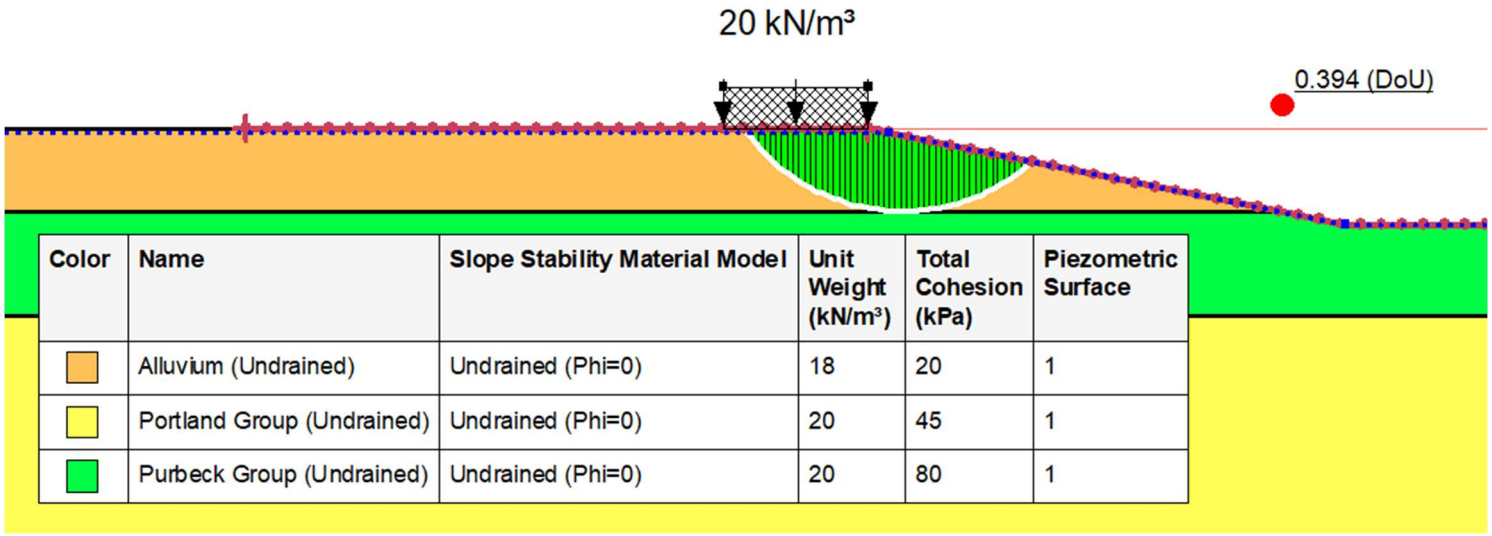
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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium (Drained)	Mohr-Coulomb	18	1	22	1
	Portland Group (Drained)	Mohr-Coulomb	20	1	27	1
	Purbeck Group (Drained)	Mohr-Coulomb	20	1	24	1

Analysis No 10.: Pond C2.T.21-PND-0611 cutting adjacent to access road (Ground Model 2) Drained conditions EC7 - DA1C2 Unreinforced DoU=0.954

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Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Piezometric Surface
	Alluvium (Undrained)	Undrained (Phi=0)	18	20	1
	Portland Group (Undrained)	Undrained (Phi=0)	20	45	1
	Purbeck Group (Undrained)	Undrained (Phi=0)	20	80	1

Analysis No 10.: Pond C2.T.21-PND-0611 cutting adjacent to access road (Ground Model 2) Undrained conditions EC7 - DA1C2 Unreinforced DoU=0.394

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Appendix E: Strengthened Earthworks Appraisal Form

E1 Introduction

This section presents the Strengthened Earthworks Appraisal Form (SEAF) for the Highways Embankments associated with Maintenance Access Road to Pond (South Arm off Calley Farm – South) located in Section C2 Central.

The proposed scheme includes embankments that are to be constructed using Strengthened Earthworks to support embankments at heights and gradients that do not comply with the stability requirements of EC7 and the HS2 Technical Standard - Earthworks

E2 Scheme Details

E2.1 Name of Scheme

HS2 Aylesbury Area – Maintenance Access Road to Pond (South Arm off Calley Farm – South) Embankments – HS2 Asset ID HS2-00002BJSK

E2.2 Type of Highway

Single carriageway road/ unpaved maintenance access road.

E2.3 Permitted Traffic Speed

30kph.

E2.4 Nature of Scheme Element

New highway construction to provide access to 2 No attenuation ponds to be constructed as part of the HS2 Scheme. Pond References C2.T.23-REG-0614, C2.T.21-PND-0611.

E3 Strengthened Earthwork Type, Purpose, and Location

E3.1 Generic Type of Strengthened Earthwork

Strengthened Soil Slopes

E3.2 Purpose of Strengthened Earthwork

The embankment slopes are to be constructed at an inclination greater than the natural safe slope angle due to restricted land take.

E3.3 Intended Locations for Use

Highway embankments which are not able to stand up unsupported within the Maintenance Access Road to Pond (South Arm off Calley Farm – South). Specific Locations for the use of reinforced soils slopes are detailed in the Table Below, as per Table E1 below.

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Table E1 – Summary of Embankment Design

Left Hand Side		Right Hand Side	
Ch.	Solution	Ch.	Solution
0+000 to 0+048	Reinforced	0+000 to 0+048	Reinforced
0+048 to 0+500	Unreinforced	0+048 to 0+500	Unreinforced
0+500 to 0+600	Reinforced[2]	0+500 to 0+600	Reinforced
0+600 to 0+619	Unreinforced	0+600 to 0+619	Unreinforced
0+000 to 0+034 (C2.T.23-PND-0614) Access	Unreinforced	0+000 to 0+034 (C2.T.23-PND-0614) Access	Reinforced
0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced	0+000 to 0+037 (C2.T.21-PND-0611) Access	Reinforced

Notes:

1. Side of embankment taken as if looking south along chainage
2. Geogrid with 100kN/m tensile capacity required at base of the highway embankment as per Appendix 6/9 of the IDR2084. Specification(1MC06-CEK-AU-SPE-CS04-000003). A 300mm thick starter layer of Class 6C1 fill is required beneath the geogrid.

E4 Outline of Existing Ground Conditions and Groundwater Conditions

E4.1 Ground Investigation Data

The ground investigation data available for the Maintenance Access Road to Pond (South Arm off Calley Farm – South) is set out in Appendix B and the corresponding Supplementary Ground Investigation Report (GIR) 1MC06-CEK-GT-REP-CS04_CL18-000011.

A total of 30 no Exploratory Holes have been considered in the design of this asset, the locations of which are shown the plan included within Appendix A

The Uncertainty in Ground Conditions Classification for the highway asset group is Class 1, considering the available GI data and Geotechnical Category of the earthworks, and has been undertaken in line with the Ground Engineering Design Basis Statement – Management of Uncertainty in Ground Conditions (1MC06-CEK-GT-REP-C002-000127). This classification confirms sufficient GI data is available for Detailed Design.

E4.2 Existing Ground Conditions

The ground conditions are presented in in Section 4.2 of this document. In summary, the ground models are as follows:

Ch 0+000 to 0+500 – Portland Stone/Portland Sand Formation over Kimmeridge Clay Formation.

Ch 0+500 to 0+619 –Alluvium over Purbeck Group over Portland Stone/Portland Sand Formation. Thickness of Head/Alluvium varies between 0.00m and 2.20m bgl

E4.3 Existing Groundwater Conditions

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The groundwater conditions are summarised in Section 4.3 of this document. A Design Groundwater Level of 0.15m has been adopted for this asset.

E4.4 Soil and Groundwater Chemistry

Section 5 of the Supplementary Ground Investigation Report (GIR) 1MC06-CEK-GT-REP-CS04_CL18-000011 provides a summary of the results of the analysis of sulphate and pH testing that has been undertaken for this asset.

E4.5 Existing Geotechnical Risks

The geotechnical risks associated with the asset are summarised within Section 4 of this document.

Reference should be made to the IDR 208.4 Geotechnical Risk Register (1MC06-CEK-GT-REG-CS04-000003).

The key Geohazards relating to this asset are

- Presence of soft ground
- Shallow groundwater
- Relic shear surfaces
- Shrink / Swell
- Faulting
- Flood risk and adverse weather conditions
- Contaminated Ground

E5 Proposed Strengthened Earthwork

E5.1 Description of Strengthened Earthwork

The strengthened earthworks are to be formed at slope angles of 1H:3V and will be reinforced by geogrids. They will be formed for slope heights of between 1.5m and 2.5m high..

The strengthened earthworks are to be formed at slope angles of 1H:3V and will be reinforced by geogrids with 100kN/m tensile capacity required at base of the highway embankment. They will be formed for slope heights of between 0.5m and 2.0m high, chainage 0+500 to 0+600. In addition to the geogrid, a 300mm topsoil strip should be undertaken for the proposed extents of the geogrid. Topsoil is to be excavated and replaced with Class 6C1 (starter layer material). The geogrid is to be placed at the midpoint of this thickness of Class 6C1.

The fill to the proposed geogrid reinforcement array will comprise Class 7C3 Cohesive Fill material.

These features are shown on the Typical MWCC Earthworks Standard Detail - Typical Reinforced Earth Embankments (MC06-CEK-GT-DSE-C002-000111).

E5.2 Foundation Preparation, including measures to deal with Geotechnical Risks

The reinforced slopes shall be constructed with primary geogrids in lifts of a maximum of 1.0m height, supported by intermediary secondary geogrids of a minimum of 2m in length. All geogrid sheets are to be laid in a bottom-up sequence following appropriate compaction as the surrounding embankment is constructed.

No specific ground treatment is proposed, other than topsoil strip at locations of all embankments and the provision of a basal drainage blanket / starter layer for embankments >3m high.

Soft spots, defined as soils with an undrained shear strength of <45kPa, will be required to be removed by inspection following exposure of the Formation (Ch.0+000 to 0+500).

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Soft spots, defined as soils with an undrained shear strength of <20kPa, will be required to be removed by inspection following exposure of the Formation (Ch.0+500 to 0+619).

E5.3 Materials to be used in Construction

The highway embankments will comprise reworked Kimmeridge Clay Formation Class 2A4 material. The proposed Class 2A4 fill has also been categorised as a Class 7C3 fill for use within the geogrid array, refer to Appendix 6/1 Table 6/1 for acceptability criteria and Appendix 1/5 for testing requirements.

A departure has been submitted and accepted to permit the use of Kimmeridge Clay as Class 7C3, due to the fact that it exceeds the Plasticity Index for Class 7C3 material. Further details are given below on the Departure

The slopes of the embankment will be reinforced with layers of polymeric geogrids with long term design strength of at least 15 kN/m over a 120-year expected service life. The design properties of geogrids are as detailed in Appendix 6/9.

As part of the construction highway embankment, it is also proposed to install geogrid reinforcement to prevent localised slope failures into the drainage ditch from extending beneath the highway earthworks. Between Ch.0+0500 and Ch.0+0600, a 6.0m wide geogrid is required, measured from the toe of the highway embankment. In addition to the geogrid, a 300mm topsoil strip should be undertaken for the proposed extents of the geogrid. Topsoil is to be excavated and replaced with Class 6C1 (starter layer material). The geogrid is to be placed at the midpoint of this thickness of Class 6C1. The tensile capacity of the geogrid is to be (minimum) 100kN/m as per Appendix 6/9 of the IDR2084. Specification(1MC06-CEK-AU-SPE-CS04-000003).

Where over-the-edge drainage is to be adopted at embankment heights nominally greater than 0.75m, to 3.00m high, a topsoil retention system in accordance with Standard Earthworks Drawing 1MC06-CEK-GT-DSE-C002-000107 is required.

E5.4 Drainage Measures

A 300mm thick Class 1 starter/basal drainage layer is to be installed beneath embankments greater than 3m in height. For the purposes of assessing the stability of the embankments an excess pore water pressure parameter, r_u, has been adopted in the embankment, and in the upper Vadose Zone as directed by the HS2 Technical Standard – Earthworks.

Over the edge drainage is to be applied on 3m high highway embankments or lower in conjunction with drainage ditches at the toe of the earthworks. Kerb and gully drainage details are to be applied on sloped embankments above 3m.

A topsoil retention system should be adopted as identified in E5.3 where over-the-edge drainage is proposed.

E5.5 Arrangements for Highway Furniture and Buried Services and Landscaping

A clash check has been undertaken between highway furniture, such as lighting/VRS and also buried services within the reinforced embankments.

The geogrid reinforcement design allows for the upper primary geogrid to be placed no higher than 500mm below the proposed verge level to accommodate shallow foundations and utilities. Where deeper utilities (such as filter drains) cannot be moved sufficiently to avoid a clash with the geogrid array, or deep foundations to VRS are proposed, the geogrid design (and length of geogrid) has been modified to accommodate these potential clashes.

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E5.6 Inspection and Maintenance

Instrumentation, monitoring is not required for this asset, however, in-situ testing is required to monitor settlements of embankments, pore water pressure dissipation, verification of ground conditions and to ensure safety during construction.

Asset specific instrumentation and monitoring can be found in Section 12.1.1 of this GDR. For this asset, the criticality of the earthworks is such that no Instrumentation and Monitoring is proposed

The assets should be inspected on a regular cycle to be agreed with the adopting authority. Any evidence of slope movement should be noted and relayed to the Designer.

Maintenance is normally undertaken from the front of the embankment.

The provision of geogrid reinforcement should not have a significant impact on the cycles of inspection.

E 5.7 Interface with Structures

There will be interaction with the proposed ancillary Culverts only for this asset.

E6 Design Methods

E6.1 Internal Stability

The analysis of the internal stability of the strengthened earthworks was carried out by limit state design in accordance with BS 8006-1:2010 – *Code of practice for strengthened/reinforced soils and other fills*, and Morgenstern-Price's method for slope stability. Design was undertaken using appropriate computer software (SLOPE/W Version 10. by Geo-Slope). The internal stability analysis was undertaken using ultimate limit state design in accordance with BS 8006-1:2010 to achieve a Degree of Utilisation (Dou) of less than 1.

The geogrid sheet length, inclination, spacing, and required strength were determined by selecting the slope failure mechanism that passes through the reinforced soil block, giving the greatest activating force that leads to failure. The contribution from reinforcement was limited by the tensile failure and pull-out resistance in accordance with BS 8006:2010.

Stability analyses were carried out for various typical embankment heights for reinforced soil design covering height ranges of 1.5 to 3.0m, at locations as identified in E3.3

E6.2 External Stability

The stability of the overall embankment slope including failure beneath and behind the strengthened earthwork was assessed in accordance with BS EN 1997-1:2004+A1:2013 *Eurocode 7: Geotechnical Design*.

E7 Design/Assessment Criteria

E7.1 List of Relevant Documents

a. BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills
b. BS EN 1997-1:2004+A1:2013 Eurocode 7: Geotechnical Design inc. National Annex
c. BS 8002:2015 Code of practice for earth retaining structures
d. BS EN 14475:2006 Execution of Special Geotechnical Works – Reinforced Fill
e. MCHW Volume 1 Specification for Highways
f. HS2-HS2-GT-STD-000-000001 P08 Technical Standard – Earthworks

E7.2 Limit State Design Criteria

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A 120-year design life has been specified that is consistent with the requirements of the HS2 Technical Standard - Earthworks.

Limit state partial factors have been applied in accordance with:

a. BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills
b. BS EN 1997-1:2004+A1:2013 Eurocode 7: Geotechnical Design inc. National Annex

E7.3 Serviceability Design Criteria

Slope stability design has been undertaken incorporating an allowance for long term degradation of the embankment slopes and climate change in accordance with HS2 requirements reference Table 3.8/1 of the HS2 Technical Standard – Earthworks.

Internal settlement of the fill will be controlled using suitable compaction and will be built out during construction. Internal drainage layers will be provided to the embankment at critical locations i.e., on the approach to major structures where control of post construction differential settlement is required.

Where required, mitigation to control settlement in the foundation soils has been specified, but no such measures are required for this asset.

In order to prevent excessive deformation in the slope face the design strength of the reinforcement has been limited to a post-construction strain of 5% under permanent loading in accordance with BS 8006:2010+A1:2016.

E7.4 Design Parameters for Soils and Materials

The highways embankments for this asset will be constructed with Class 2A4 material using excavated Kimmeridge Clay Formation. A minimum undrained shear strength of compacted Class 2A4 embankment material and Class 7C reinforced embankment material has been specified as a minimum of 60kN/m^2 for analysis of embankment stability.

Any embankments less than a nominal 750mm above existing ground level will be constructed of granular fill material (Class 1/6, sub-base, and structural pavement layers) for practical and material handling requirements.

The characteristic material parameter values used for the reinforced slope designs are summarised in Table E2 below.

Table E2 – Fill Material Design Properties

Material	Unit Weight (kN/m^3)	Angle of Shearing Resistance (°)	Cohesion, c' (kN/m^2)	Undrained Shear Strength, c_u (kN/m^2)
Class 2A4 / 7C3 Fill	20	23, (21)*	1.0	50
Class 6C1 Fill	20	35	35	0
Alluvium	18	22, (21)*	0.0	20
Portland Group	20	25, (23)	1.0	45
Purbeck Group	20	24 (22)	1.0	80

* A lower angle of shearing resistance to be adopted in the Vadose zone in line with Technical Standard -Earthworks HS2-HS2-Gt-STD-000-000001 is given in brackets().

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The design properties of the geogrids are as detailed in Appendix 6/9. The properties of the geogrids have been selected based on a design life of 120 years.

The category of the earthwork in accordance with Table 2.2/2 of Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001), the highways earthworks are designated as Geotechnical Category 2 and require Cat II checking. In accordance with BS8006-1:2010+A1:2016 they are subject to an implication factor of f_n=1.0.

E7.5 Design Groundwater Conditions

In line with Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001), an r_u of 0.25 was adopted for the Vadose Zone to be applied to the earthwork side-slopes in intermediate or high plasticity clay.

A phreatic groundwater level was adopted at 0.1 m below the foundation level of embankment.

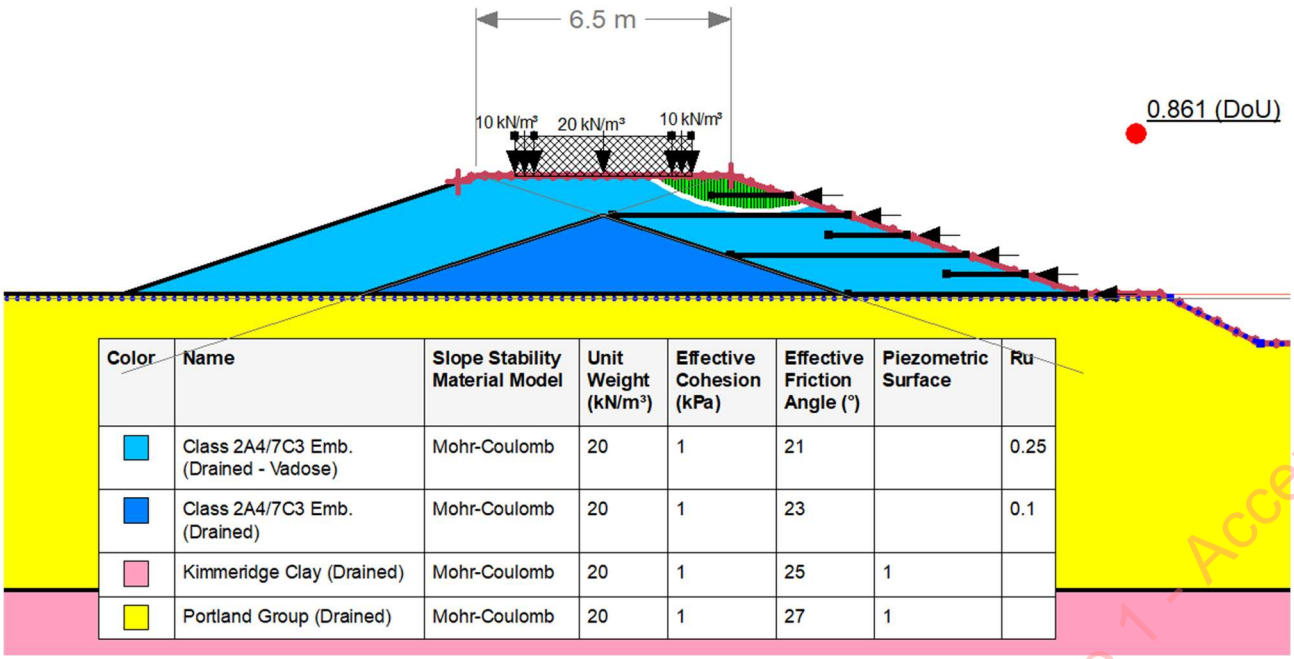
E7.6 Live Loadings

A permanent 20kPa surcharge has been adopted crest to crest for the embankment stability check in the short term (undrained) condition.

For the long term (drained) stability analysis, a permanent surcharge of 10 kPa is applied over the verge between the carriageway and 1.0m from the crest of the embankment side-slope. A permanent surcharge load of 20 kPa is applied across the trafficked width.

E7.7 Idealised Soil Structure Model used in Analysis

A range of slope heights from 1.0m to 3.0m has been analysed to verify the stability for the reinforced soil slopes. The idealised model is summarised in Figure E1 below.



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface	Ru
	Class 2A4/7C3 Emb. (Drained - Vadose)	Mohr-Coulomb	20	1	21		0.25
	Class 2A4/7C3 Emb. (Drained)	Mohr-Coulomb	20	1	23		0.1
	Kimmeridge Clay (Drained)	Mohr-Coulomb	20	1	25	1	
	Portland Group (Drained)	Mohr-Coulomb	20	1	27	1	

Figure E1 - Idealised Reinforced Slope Model for Highways Earthworks

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E7.8 Precautions against Chemical Attack to Materials

Extremely acidic or alkaline environments can cause chemical degradation of geosynthetic materials and a pH of between 4 and 9 is required to protect the geotextiles from Chemical Attack

Analysis of the pH range of the Kimmeridge Clay and Mudstones which is proposed for use as 7C fill has a pH of 8.3 (1 test undertaken), which indicates no exceedance of the upper pH range.

Where specified as a Class 7C material, suitability testing of the Kimmeridge Clay should be undertaken to confirm the upper range of the soil pH does not exceed 9 prior to placement.

E7.9 Proposed Departures from Design Standards

The highway earthworks in the Aylesbury Embankment Group area are to be constructed from reworked Kimmeridge Clay Formation. This material has a Plasticity Index (PI = 35%) higher than the normal limit of PI = 25% for cohesive fill (Class 7C) as backfill to reinforced soil in Series 600 of the Specification for Highways Works. Note that BS 8006-1 allows the use of Class 2 fill for reinforced slopes below 45 degrees.

Departures DJV-DEP-000466, DJV-DEP-000499 and DJV-DEP-000543 as described in Section 14.2 have been accepted by HS2.

E8 Checking

The design of the highway earthworks have been subject to a Category I check in accordance with Table 2.2/2 of Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001).

E9 Drawings and Documents

E9.1 List of Drawings and Documents accompanying submission

E9.1.1 Drawings and documents

Drawings relating to the highway earthwork Design are as follows

Report / Drawing Titles	Document No
Maintenance Access Road to Pond (South Arm off Calley Farm - South - Highway Earthworks General Arrangement	1MC06-CEK-HW-DGA-CS04_CL18-000172
Maintenance Access Road to Pond (South Arm off Calley Farm - South – Highway Cross-Sections Sheet 1	1MC06-CEK-HW-DSE-CS04_CL18-000035
Maintenance Access Road to Pond (South Arm off Calley Farm - South) – Highway Earthwork Cross-Sections Sheet 2	1MC06-CEK-HW-DSE-CS04_CL18-000036
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 1	1MC06-CEK-GT-DSE-CS04_CL18-000090
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 2	1MC06-CEK-GT-DSE-CS04_CL18-000091
Maintenance Access Road to Pond (South Arm off Calley Farm - South) Earthworks Detail Design - Reinforced Slope Sheet 3	1MC06-CEK-GT-DSE-CS04_CL18-000092

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Reference should also be made to the Standard Earthwork Drawing, Reference MC06-CEK-GT-DSE-C002-000111.

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E.2 Geotechnical Calculation Report – Footpath SBH/19 Oat Close Bridleway Realignment

Number of Pages: 108

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1 Introduction

1.1 Scope and Objective of Calculation Pack

The purpose of this document is to present the geotechnical design calculations for the highway assets associated with the Oat Close SBH/19 Realignment, which spans across the Level 1 asset Aylesbury Embankment. This report is to be read in conjunction with Geotechnical Design Report (1MC06-CEK-GT-REP-CS04_CL18-000037), which provides further details relating to the geotechnical conditions and design across the Level 1 asset.

This calculation pack is to be submitted as part of IDR 207.2.

1.2 Summary of Assets

Oat Close SBH/19 Realignment is located at Mainline Ch. 60+920m (SnakeGrid) within Section C2 (Aylesbury Area), centred at approximate National Grid Reference E 480996 N 211435 (OSGB). The asset is part of the Aylesbury Embankment Level 1 Asset Group.

Mainline assets to the north and the south are Aylesbury North Cutting Group and Aylesbury South Cutting Group, respectively.

Figure 1-1 and Table 1-1 identify the location and properties of the assets, respectively.

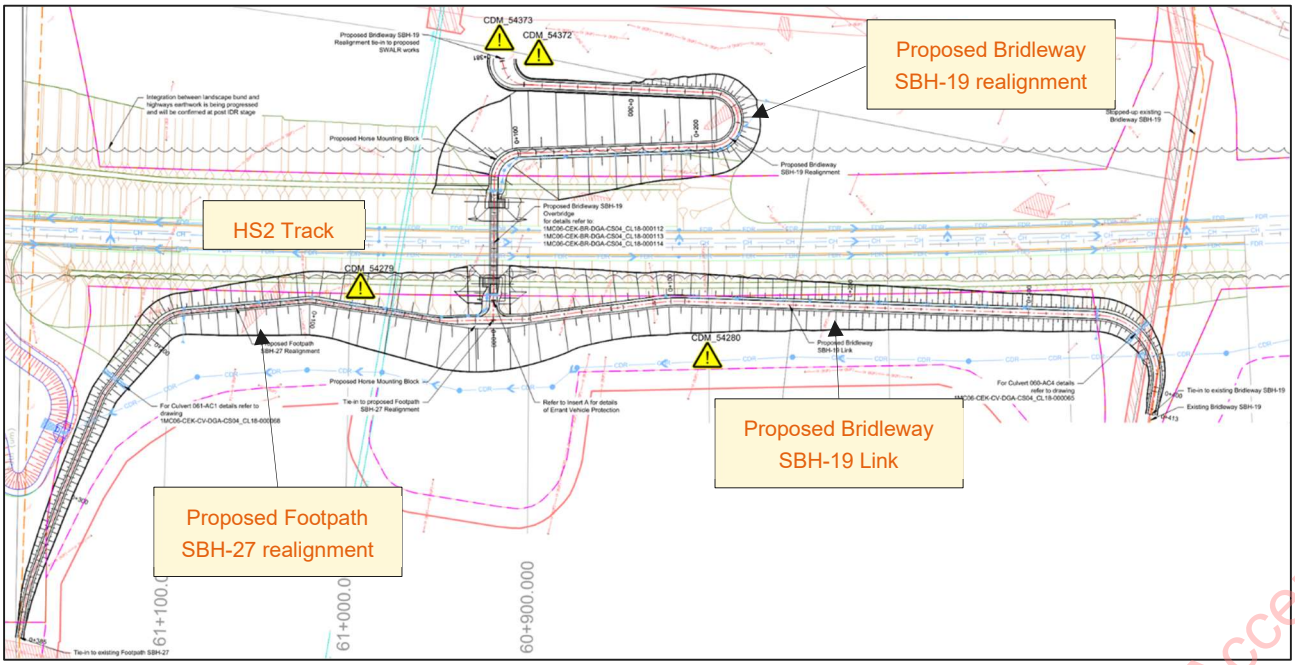


Figure 1-1: Bridleway SBH/19 Realignment Asset Plan and Location

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Table 1-1: Asset Details

Asset Description	Detail		
Name of design element	**Bridleway SBH/19 Realignment**	**Bridleway SBH/19 Link**	**Footpath SBH/27 Realignment**
Contract/section/lot	AA	AA	AA
HS2 Asset ID	HS2-000020RKL		
Type	Public Right of Way (PRoW)	Public Right of Way (PRoW)	Public Right of Way (PRoW)
Highway Start - End chainage (approx.)	0+000 to 0+381	0+000 to 0+414	0+000 to 0+387
Length	381	414	387
Maximum height	10.35 at 0+070 (ML Ch. 60+920)	8.53 at 0+000 (ML Ch. 60+920)	8.53 at 0+000 (ML Ch. 60+920)
Detailed design side slope	Max: 1:3 Min: 1:8.7	Max: 1:2.5 Min: 1:4	Max: 1:3 Min: 1:4
Landscape bunds	Yes, on northern side (design ongoing)	Yes (design ongoing)	Yes (design ongoing)
Geotechnical Category (Design Category)	GC2 (CAT II)	GC2 (CAT II)	GC2 (CAT II)

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1.3 Sources of Information

The following documents should be consulted to provide supporting information for the geotechnical assessment and design of this asset:

Table 1-2: Sources of Information

Report / Drawing Titles	Document No
Aylesbury Embankment - Supplementary GIR	1MC06-CEK-GT-REP-CS04_CL18-000011
Aylesbury Embankment GDR	1MC06-CEK-GT-REP-CS04_CL18-000037
Model: Aylesbury Embankment – 2D Plan Model	1MC06-CEK-GT-DMA-CS04_CL18-000005
Model: Oat Close Bridleway (SBH-19) Realignment – Asset Group 2D XREF DM2	1MC06-CEK-HW-DMA-CS04_CL18-000006
IDR Package 207.2 – Specification	1MC06-CEK-AU-SPE-CS04_CL18-000003
IDR Package 207.2 – Geotechnical Risk Register	1MC06-CEK-GT-REG-CS04_CL18-000006
Assumptions register IDR 207.2	1MC06-CEK-AU-REG-CS04_CL18-000059
IDR Package 207.2 - CDM Risk Register	1MC06-CEK-AU-REG-CS04_CL18-000043
Departures Register IDR 207.2	1MC06-CEK-AU-REG-CS04_CL18-000059
Bridleway SBH-19 Overbridge Earthwork Transition Sketch	1MC05-CEK-BR-DGA-CS04_CL18-000240
Oat Close Bridleway (SBH-19) Realignment – Highway General Arrangement	1MC06-CEK-HW-DGA-CS04_CL18-000208
Oat Close Bridleway (SBH-19) Realignment - Highway Earthworks General Arrangement	1MC06-CEK-HW-DGA-CS04_CL18-000209
Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 1	1MC06-CEK-HW-DSE-CS04_CL18-000051
Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 2	1MC06-CEK-HW-DSE-CS04_CL18-000052
Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 1	1MC06-CEK-HW-DSE-CS04_CL18-000053
Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 2	1MC06-CEK-HW-DSE-CS04_CL18-000054
Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 3	1MC06-CEK-HW-DSE-CS04_CL18-000100
Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 3	1MC06-CEK-HW-DSE-CS04_CL18-000101

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C2.L.22 Catchment – Pond C2.L.22-PND-0606 General Arrangement	1MC06-CEK-DR-DGA-CS04_CL18-000139
C2.L.22 Catchment – Pond C2.L.22-PND-0606 Cross Section Drawing	1MC06-CEK-DR-DGA-CS04_CL18-000076
C2.L.22 Catchment – Pond C2.L.22-PND-0609 General Arrangement	1MC06-CEK-DR-DGA-CS04_CL18-000140
C2.L.22 Catchment – Pond C2.L.22-PND-0609 Cross Section Drawing	1MC06-CEK-DR-DGA-CS04_CL18-000077
C2.T.21 Catchment – Pond C2.T.21-PND-0611 General Arrangement	1MC06-CEK-DR-DGA-CS04_CL18-000046
C2.T.21 Catchment – Pond C2.T.21-PND-0611 Cross Section Drawing	1MC06-CEK-DR-DGA-CS04_CL18-000027
Bridleway SBH-19 Overbridge Construction Sequence Sheet 1	1MC06-CEK-BR-DGA-CS04_CL18-000125

The following other sources of information have also been used within the assessment:

- Ground Investigation (GI) data and monitoring data from the HS2 Main Ground Investigation (GI);
- Data available from Stage 2 GI, where available (Data Drop 39);
- GI data and monitoring data from the Supplementary GI;
- ASC GIS Viewer;

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2 Summary of Scheme Design

At scheme design Footpath SBH-27 and Bridleway SBH-19 were previously situated in two separate locations in Aylesbury Embankment, at Chainages 61+200 and 60+550, respectively. However, for detailed design, it was concluded to combine these two L2 Assets into Oat Close SBH/19 Realignment. Subsequently, the earthwork design of the previous assets has been altered to suit the new combined layout.

Due to this change, no specific geotechnical analysis was undertaken for the new combined Oat Close SBH/19 Realignment layout at scheme design.

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3 Detailed Design Proposals

The detailed design proposals for the asset can be found in the table below.

Table 3-1: Asset Details, Detailed Design

Asset Description	Bridleway SBH/19 Realignment	Bridleway SBH/19 Link	Footpath SBH/27 Realignment
Type	Public Right of Way (PRoW)	Public Right of Way (PRoW)	Public Right of Way (PRoW)
Start - end mainline chainage (approx.)	Ch 60+770 to 60+950	Ch 60+550 to 60+920	Ch 60+920 to 61+170
Highway Start - End chainage (approx.)	0+000 to 0+381	0+000 to 0+414	0+000 to 0+387
Length (approx.)	381	414	387
Carriageway/verge width	3.5m/1.5m		
Pavement Design	Ch 0 to 5 = AT2 (Unbound) Ch 5 to 20 = C1 (Bound) Ch 65 to 80 = C1 (Bound) Ch 80 to 381 = AT2 (Unbound)	Ch 0 to 414 = AT2 (Unbound)	Ch 0 to 387 = AT2 (Unbound)
Max structure height at asset chainage	10.35 at 0+070 (ML Ch. 60+920)	8.53 at 0+000 (ML Ch. 60+920)	8.53 at 0+000 (ML Ch. 60+920)
Proposed Detailed Design side slope	Max: 1:3 Min: 1:8.7	Max: 1:2.5 Min: 1:4	Max: 1:3 Min: 1:4
Landscape bunds	Yes, on northern side (design ongoing)	Yes (design ongoing)	Yes (design ongoing)

3.1 Relevant Boreholes and Other Information

A list of the exploratory holes under consideration when deriving the ground model for this document are included in the tables in Appendix B

A total of 18 Exploratory Holes (from the Stage 2 GI and the supplementary GI) have been considered in the design for these assets, the location of which are shown in the plan included within Appendix B.

At the time of producing this calculation pack, there are no outstanding Stage 2 GI exploratory points within 10m of the asset footprint.

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The Uncertainty in Ground Conditions Classification for the asset group is Class 1 considering the available GI data and Geotechnical Category of the earthworks and has been undertaken in line with the Ground Engineering Design Basis Statement – Management of Uncertainty in Ground Conditions (1MC06-CEK-GT-REP-C002-000127). This classification confirms sufficient GI data is available for Detailed Design.

3.2 Site Constraints

3.2.1 Drainage Features

3.2.1.1 Land Drainage

Land drainage is present at the toe of the embankments and landscape bunds. This will in turn intercept overland flow and intercept post development run-off. There are two land drainage ponds adjacent to the highways earthwork, C2.L.22-PND-0606 at Chainage 60+600 and C2.L.22-PND-0609 at Chainage 60+930. Due to the distance between the pond, C2.L.22-PND-0606, and the highways earthwork (3.70m), the stability of the highway may be impacted and has therefore been assessed (refer to Section 5.5). However, pond C2.L.22-PND-0609 is approximately 14.00m away from the toe of the highway and is not considered an impact.

For more information, refer to the following report and drawings:

- Land Drainage C2.L.26 Catchment – Design and Calculation Report (1MC06-CEK-DR-REP-CS04_CL18-000022)
- C2.L.22-PND-0606 Cross Section (1MC06-CEK-DR-DSE-CS04_CL18-000076)
- C2.L.22-PND-0606 General Arrangement (1MC06-CEK-DR-DGA-CS04_CL18-000139)
- C2.L.22-PND-0609 Cross Section (1MC06-CEK-DR-DSE-CS04_CL18-000077)
- C2.L.22-PND-0609 General Arrangement (1MC06-CEK-DR-DGA-CS04_CL18-000140)

3.2.1.2 Track Drainage

Track drainage, through a combination of carrier drains and filter drain systems, is present to adjacent to SBH-19 Oat Close Bridleway Realignment which runs for 636m, from Chainage 60+984 to 61+620. The track drainage is collected in the attenuation pond, C2.T.21-PND-0611 at Chainage 61+200, near Sedrup Culvert. Due to the distance between the pond, C2.T.21-PND-0611, and the highways earthwork, the stability of the highway may be impacted and has therefore been assessed (refer to Section 5.5).

For more information, refer to the following report:

- Track Drainage C2.T.23 Catchment – Design and Calculation Report (1MC06-CEK-DR-REP-CS04_CL19-000024)

3.2.1.3 Highway Drainage

The run-off that is generated from SBH-19 Oat Close Bridleway Realignment, a combination of unbound and bound surfaces will be collected and attenuated in C2.L.22 land drainage ditch.

The highways drainage will ensure that any surface water run-off from the bridleway/footpath and the associated verges will not lead to any deterioration.

3.2.2 Ancillary Structures

There are two culverts present, 060-AC4 (HS2-00002N7VP) and 061-AC1 (HS2-00002N7VQ), located at approximate Ch0+360 in Bridleway SBH/19 Link and at Ch. 0+230 in Footpath SBH-27 Realignment, respectively. Details of the culverts, 060-AC4 and 061-AC1, are presented in drawings 1MC06-CEK-CV-DGA-CS04_CL18-000262 and 1MC06-CEK-CV-DGA-CS04_CL18-000264, respectively.

Reference should be made to the main text of the Aylesbury Embankment GDR, for details of design and calculations for culverts and ditch slabs.

3.2.3 Landscape Earthworks

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Landscaping earthworks are proposed on the south western side of the mainline along the entire length of Bridleway SBH/19 Link and Footpath SBH/27 Realignment. However, the landscape earthworks on the north eastern side of the mainline only extends northward from Chainage 60+770, at which it is to be integrated with the highway's earthwork for Bridleway SBH/19 Realignment.

3.2.4 Utilities

Clash detection between the geogrid reinforcement and any utilities and drainage that may be present within the embankments will be undertaken post IDR.

3.2.5 Flood Risk

Flooding has been identified as a risk within the Aylesbury Embankment level 1 asset area. As shown below in Figure 3-1: Aylesbury Embankment Flooding

, there will be some level of flood water interaction with the north eastern side of the highway earthworks (Bridleway SBH/19 Realignment), between Chainages 60+820 and 60+960. However, the risk is considered low, so flood protection is not required.



Figure 3-1: Aylesbury Embankment Flooding

3.3 Contaminated Ground Risk Assessment

The following text has been summarised from the Aylesbury Embankment Supplementary GIR (Ref: 1MC06-CEK-GT-REP-CS04_CL18-000011), refer to the GIR for further details.

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The contaminant risk assessment has been restricted to identified Land Quality (LQ) areas within an area immediately surrounding the Aylesbury Embankment only. Additionally, any location outside of the identified LQ areas, the potential for site wide ground contamination within greenfield agricultural areas is considered to be very low, although there remains the potential that residual herbicides or pesticides and isolated areas of unforeseen contamination may be present.

Two LQ constraints are noted within the vicinity of the asset which is discussed in the Aylesbury Embankment Supplementary GIR (Ref: 1MC06-CEK-GT-REP-CS04_CL18-000011) is as follows:

- Hartwell Landfill (LQ 11.04) which is approximately 110m northwest of the LOD of the Embankment (at its nearest point);
- Former Hartwell clay, brick/tile works & landfill (LQ 11.03) located 200m north of the LOD of the Embankment.

The two identified LQ areas, 11.04 and 11.03, are located approximately 952m and 1012m northwest of Oat Close SBH/19, respectively, and therefore no adverse impacts are anticipated.

3.4 Construction Sequence

The construction sequence of the mainline earthworks, Oat Close Bridleway SBH/19 Overbridge and the approach is assumed to be as follows.

It is proposed to construct Aylesbury Embankment and the highways earthwork in accordance with the geotechnical design documents. Following construction of the mainline embankment and the highways earthwork, it is assumed that the permanent landscape bunds will be constructed within 2 years of the highway earthwork.

The Overbridge Construction Sequence is assumed to be as follows:

Stage 1 – Backfill the earthwork blister to the abutment founding level.

Stage 2 – Construct RC bankseat abutments and wingwalls, backfill behind and in front of abutments and backfill around the wing walls and bridge abutments.

Stage 3 – Steelwork transportation and assembly, deck waterproofing and surfacing, install parapet mesh and finally lift the span into place.

Stage 4 – Install the bridge finishes and complete the soft landscaping.

The GDR specified a 6-month post-construction hold period for Oat Close Bridleway SBH/19 Overbridge between the installation of the abutments and wingwalls and the installation of the bridge deck. The construction sequence of the bridge is not anticipated to affect the design of the proposed highway earthworks.

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4 Ground Conditions

4.1 General

Detailed geological descriptions, test results of the encountered materials and recorded groundwater levels, are presented in the Aylesbury Embankment Supplementary GIR.

4.2 Ground Model and Characteristic Parameters

This section describes the proposed earthwork materials and natural stratigraphy along with the characteristic parameters used within the design as presented in the tables below.

The main ground model to be used in analysis is referenced as Ground Model 1 (“GM1”) and is detailed in Table 4-1 and the potential for Head deposits underlying the eastern leg of Bridleway SBH/19 Link refer to GM2, detailed in Figure 4-1. Figure 4-1 covers the highways earthworks for Bridleway SBH/19 Link, due to the presence of Head deposits at shallow depth and the potential for Head deposits underlying the eastern leg of Bridleway SBH/19 Realignment.

To account for the limited ground investigation data to the east of the SBH/19 Overbridge, i.e., underlying much of the Bridleway SBH/19 Realignment earthworks, both GM1 and GM2 have been applied during design of this section of earthworks, as detailed in Figure 4-1.

Figure 4-1Figure 4-1 covers the highways earthworks for Bridleway SBH/19 Link, due to the presence of Head deposits at shallow depth and the potential for Head deposits underlying the eastern leg of Bridleway SBH/19 Realignment.

To account for the limited ground investigation data to the east of the SBH/19 Overbridge, i.e., underlying much of the Bridleway SBH/19 Realignment earthworks, both GM1 and GM2 have been applied during design of this section of earthworks, as detailed in Figure 4-1.

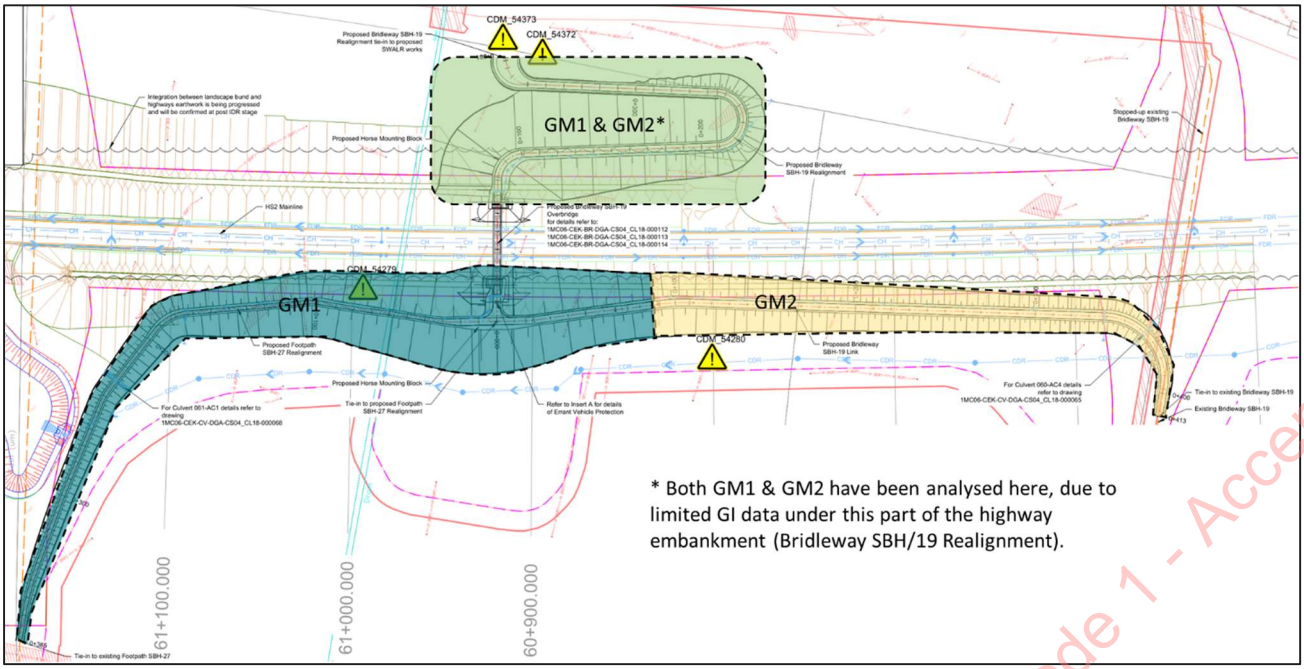


Figure 4-1: Ground Model Plan

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The general stratigraphy for the site consists of Head overlying the Gault Clay Formation, Whitchurch Sand Formation, Purbeck Group, Portland Stone/Portland Sand Formations and the Kimmeridge Clay Formation encountered at depth.

There are localised occurrences of Made Ground / Topsoil and it is assumed that this will be assessed on site and removed during construction where necessary.

Table 4-1: Ground Model 1 and Typical Geological Descriptions

Stratigraphy	Top of Stratum Range (m AOD)	Strata Thickness Range (m)	Design Top of Stratum (m AOD)	Design Strata Thickness (m)	Summary Description
Gault Clay Formation (GLT – C Z)	80.10 – 83.18	1.70 – 16.35	81.34	16.35	Firm, becoming stiff to very stiff, grey and brown slightly sandy CLAY. Occasionally extremely weak grey sandy MUDSTONE, recovered as a firm, occasionally stiff, CLAY.
Whitchurch Sand Formation (WHS – C Z)	64.99	1.47	64.99	1.47	Firm to stiff grey occasionally slightly sandy silty CLAY with occasional shell fragments.

Table 4-2: Ground Model 2 and Typical Geological Descriptions

Stratigraphy	Top of Stratum Range (m AOD)	Strata Thickness Range (m)	Design Top of Stratum (m AOD)	Design Strata Thickness (m)	Summary Description
Head (HEAD – C Z)	81.14 - 83.98	0.80 – 2.30	83.96	2.30	Soft to firm brown slightly sandy slightly gravelly CLAY
Gault Clay Formation (GLT – C Z)	80.13 - 83.19	2.00 - 7.55	81.66	7.13	Same as GM1
Whitchurch Sand Formation (WHS – C Z)	74.53	1.50	74.53	1.50	Firm grey slightly sandy slightly gravelly CLAY
Purbeck Group (PB – LMST)	73.03 – 73.92	1.85 – 2.30	73.03	1.85	Moderately weak to medium strong grey shelly LIMESTONE.
Portland Stone / Portland Sand Formations	65.48 – 71.732	3.90 – 10.10	71.18	9.60	Extremely weak to very weak, occasionally medium strong, grey SILTSTONE.

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Stratigraphy	Top of Stratum Range (m AOD)	Strata Thickness Range (m)	Design Top of Stratum (m AOD)	Design Strata Thickness (m)	Summary Description
(POST – SLST/POSA – LMST)					Moderately weak to medium strong LIMESTONE.

Notes:

1.Occurrences of Made Ground are encountered in localised areas. It is assumed at these locations the formation will be inspected and soft spots will be highlighted. It is also assumed these will be removed during construction and replaced in line with the IDR 207.2 Specification (1MC06-CEK-AU-SPE-CS04_CL18-000003).

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Table 4-3: Summary of Characteristic Parameters for Slope Stability and Settlement Assessment

Strata/ Fill	γ (kN/m³)	PI (%)	ϕ' peak [ϕ' cv] (°)	c' (kN/m²)	Cu (kN/m²)	E' (MPa)	E_u (MPa)	v	v_u
Class 2A4 / 7C3	19 (1)		23 [21] (4 & 5) (**)	1	50 (12)	-	-	0.2	-
Head	18 (2)	40 (3)	22 [22] (4 & 5)	1(*)	40 (6)	9.6 (7)	12 (8)	0.2	0.5
Gault Clay Formation	19(2)	47 (3)	23 [21] (4 & 5)	1(*)	0 – 3m = 50 +3m = 50+10z <150 (9)	0 - 3m = 12 +3m = 12+2.4z (7)	0 - 3m = 15 +3m = 15+3z (8)	0.2	0.5
Whitchurch Sand Formation (***)	20 (2)	20 (11)	-	-	50 (9)	24 (7)	30 (8)	0.2	0.5

Notes:

(*) The cohesion value in the Head and the Gault Clay is assumed as 1kPa to consider the affect from rooting and negative pore pressure.

(**) Design assumption for drained parameters based on TQ4280.

(***) Parameters for the Whitchurch Sand Formation are only applicable where the Whitchurch Sand Formation is encountered at shallow depth due to geological faulting.

Z is the depth from 3.00m bgl.

(1) Characteristic unit weight obtained from Figure 1 in BS 8004:2015+A1 2020.

(2) Characteristic unit weight obtained from Figure 2 in BS 8004:2015+A1 2020.

(3) Derived using characteristic plasticity index using L2 asset specific design line for Head and the Gault Clay Formation (see plasticity plots)

(4) For cohesive material, the φ'cv has been derived the following equation:

$$\varphi' cv = (42° - 12.5 log 10 Ip)$$

Where Ip the soils plasticity index.

(5) For cohesive material, the peak angle of friction is based on the following equation:

$$\varphi' pk = \varphi' cv + \varphi' dil$$

Where $\varphi' dil$ is 2° for the Gault Clay Formation

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Where $\varphi' dil$ is 0° for the Head

(6) Derived using characteristic Cu using L2 asset specific design line for Head (see Cu plot)

(7) E' is based on 0.8Eu.

(8) Eu is based on relationship between Eu and Cu (after Jamiolkowski et al.).

For Head a PI of 40, Eu/Cu=300.

For the Gault Clay Formation a PI of 50, Eu/Cu=300.

For the Whitchurch Sand Formation a PI of 20, Eu/Cu=600

(9) Cu for the Gault Clay Formation and the Whitchurch Sand Formation have been taken from the Mainline Design in Aylesbury Embankment GDR for (1MC06-CEK-GT-REP-CS04_CL18-000037), based on the following rules:

(a) The Cu is adopted from Chainage 60+500 to 61+200, based on the Mainline Design Cu plot B-14 in the GDR.

(10) The Purbeck (PB), Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. Based on the geological descriptions, these Formations are assumed to be rigid stratum with a E'=100MPa as per the overbridge design.

(11) Parameters have been assessed using local ground investigation data or in the absence of data have been taken from Aylesbury Embankment GDR.

(12) Based on Table 6/1 in Series 600 for Class 2A4.

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4.3 Groundwater

There are six groundwater monitoring locations in the vicinity of Oat Close Bridleway SBH/19 and the results are presented within the table below. Monitoring was undertaken in phases between the period of July 2016 and May 2023. The high groundwater levels are thought to be perched groundwater pockets rather than being representative of a continuous groundwater profile. Considering the recorded high groundwater level, a design groundwater level has been assigned at existing ground level.

Table 4-4: Summary of Groundwater Readings

Borehole Reference	Elevation (mAOD)	No. of Readings	Min Reading (mbgl)	Min Reading (mAOD)	Average Reading (mbgl)	Average Reading (mAOD)	Max Reading (mbgl)
ML060-CP002	82.62	13	1.98	78.49	3.45	79.17	4.13
ML060-CR004	81.63	12	1.67	78.92	2.28	79.34	2.71
ML060-CR011	81.64	13	0.60	79.32	1.53	80.11	2.32
ML060-CR424	81.53	64	1.02	79.09	1.93	79.59	2.44
ML060-RC432	80.95	9	1.54	77.97	2.09	78.86	2.98
ML060-RC433	81.58	9	0.48	79.78	1.17	80.41	1.80

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4.4 Geohazards

A summary of the key potential geotechnical geohazards that may affect the Oat Close SBH/19 are provided in Table 4-5.

Table 4-5: Summary of Geohazards

Risks	Description
High / Shallow Groundwater	High or perched groundwater levels may be encountered in excavations during construction. The latest AGS data at the time of writing this document reports groundwater levels ranging between 0.48m (81.10m AOD) and 4.13m bgl (78.49m AOD).
Made Ground / Topsoil Material	Minimal Made Ground / topsoil was encountered in the exploratory holes in the immediate area of the asset and the mainline to a maximum depth of 0.30m bgl. The extents and depths of this are not proven and to be proven on site during excavation. Risk is considered to be low for the location of this asset
Contaminated Ground	The potential risk of encountering unexpected, contaminated soils and contaminated water during construction works has been identified due to the presence of LQ 11.03 and 11.04 within close proximity to SBH/19. However, the risk is considered low for the location of this asset.
Chemically Aggressive Ground	The Gault Clay Formation is expected to have elevated sulphates and sulphides and therefore sulphate resistant concrete may be required foe foundations in contact with it.
Shrink / Swell Potential	Susceptibility of highly plastic fine-grained soils (Gault Clay Formation) to changes in volume arising from changes in water content from seasonal variation from the action of vegetation and climate, leading to unacceptable total or differential settlements and degradation of embankment slopes.
Soft / Compressible Ground	Soft ground within Made Ground, Head deposits and weathered clay in the upper Gault Clay Formation could cause instability of embankment slopes and/or unacceptable total or differential settlements/ insufficient stiffness. Potential reduction in the strength of likely caused by exposure to rainfall during excavation works.
Relict periglacial features	Low strength relic shear surfaces may be present in the superficial Head Deposits and the Gault Clay beneath the embankment. Possible slickenside surfaces were encountered in the Gault Clay in borehole ML060-CR011 at depths of between 14.23m bgl and 16.65m bgl. Reactivation of these low strength shear surfaces may result in local slips which may destabilise the embankment.
Variable weathering profile	Potential variable weathering profiles may be present below formation level. Resulting in variations in strength and compressibility in the foundation soils, leading to a higher risk of excessive settlement and instability.

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Risks	Description
Geological faulting	The presence of faulting may lead to an incorrect ground model adopted for Oat Close Bridleway (SBH/19) Realignment. Faulting may result in highly variable ground conditions, increased risks associated with ground stability, settlement and high groundwater flows in the area which could affect the foundation design for Oat Close Bridleway (SBH/19) Realignment. Differential movements are more likely in a faulted zone.

The above key geotechnical hazards and their locations within Aylesbury Embankment Group and associated assets are detailed in the Supplementary GIR.

The proposed mitigation measures which are required to eliminate or reduce these risks with respect to Highways Earthworks design to an acceptable level are outlined in this Calculation Pack (Section 12.1.1) and the GDR.

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5 Slope Stability Assessment

5.1 General

Slope stability analyses for static performance is outlined in this section.

5.2 Basis of Geotechnical Design

The static global stability of highway's earthworks has been calculated adopting the principles of limit state equilibrium in accordance with BS EN 1997-1 Geotechnical Design (EC7). Specifically, Ultimate Limit State (ULS) Design Approach 1 Combination 1 (DA1 C1) and Combination 2 (DA1 C2) with the requirement to achieve a Degree of Utilisation (DoU) of less than or equal to 1.

Where geogrid reinforced slopes have been adopted for the design of earthwork side slopes, the internal stability of the geogrid arrangement has been assessed in accordance with BS 8006-1:2010+A1:2016.

The following design assumptions have been adopted in the analyses:

General:

1. Slope stability was modelled using the commercial geotechnical software SLOPE/W.
2. Morgenstern-Price method of slope stability analysis was used.
3. Circular slips were assessed adopting the 'Entry and Exit' methods of slip surface determination within the software SLOPE/W.
4. A vadose zone (2.00m thick) has been modelled for embankment slopes comprising cohesive materials at critical state, and subject to surface water infiltration, to represent disturbance/softening and long-term degradation at the slope surface as per the HS2 Technical Standard for Earthworks.
5. An r_u coefficient of 0.10 has been used for the embankment core.
6. An r_u coefficient of 0.20 has been applied to the vadose zone for embankments.
7. To model the stability of the embankments in the intermediate term (i.e., before they are supported by landscape bunds in the long term), peak shear strength parameters and a reduced Vadose Zone depth of 1m have been adopted for the 'partially drained' condition.
8. Slope stability has assessed shallow (confined to the embankment slope) and deeper (global) potential slip surfaces.
9. A depth of 0.3m was used as minimum slip surface depth in order to rule out insignificant shallow slip surfaces.
10. Both the short-term (undrained) and the long-term (drained) conditions were analysed for the static condition.
11. The offline highway embankment slopes are proposed to be constructed at a gradient of 1V:3H.
12. A variety of embankment cross sections (Cross Section AA to Cross Section KK, as seen in Appendix A) were analysed, which represent the critical sections of the earthwork encountered for the SBH-19 Oat Close Bridleway Realignment under assessment.

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5.3 Actions/ Loading

The surcharge loadings applied in the embankment slope stability analyses are summarised below. Partial factors in accordance with Eurocode 7 and BS8006 were applied to static and geogrid reinforcement assessments respectively where appropriate.

Load Case A (static – short-term condition):

- A permanent 20kPa load over the footpath/bridleway and verge (from crest to crest) was adopted to check stability of embankment under construction and in the short-term conditions using undrained parameters.

Load Case B (static – Interim condition):

- A permanent 5kPa load over the footpath/bridleway and verge (from crest to crest) was adopted to check stability of embankment under interim analysis, prior to the construction of the landscape bund and in the long-term conditions using drained parameters.

Load Case C (static – long-term condition):

- A permanent 5kPa load over the footpath/bridleway was adopted to check the stability of embankment under long term conditions with drained parameters adopted.

5.4 Calculation Results

The results of the slope stability calculations for the unsupported embankments (construction and long term conditions) are provided in Table 5-2 and for geogrid reinforced embankments within Table 5-3. The results of the embankments under temporary interim conditions prior to the placement of landscape material is presented within Table 5-4. Finally, an assessment of two drainage ponds has been undertaken, and is summarised in Table 5-5.

The four tables for each design solution cover two ground models (Ground Model 1 and Ground Model 2). A selection of the calculation outputs is presented in Appendix D. A summary of the analysis carried out is presented below in Table 5-1:

Table 5-1: Summary of Slope/W Analysis

Analysis	Applied Ground Model		Comments
	GM1	GM2	
Analysis 1	No	Yes	A generic assessment of embankments with Class 2A4, unsupported and a 1V:3H slope gradient (2.00m High).
Analysis 2	No	Yes	A generic assessment of embankments with Class 2A4, unsupported and a 1V:3H slope gradient (2.50m High).
Analysis 3	Yes	No	A generic assessment, using GM1, of embankments with Class 2A4, unsupported and a 1V:3H slope gradient (2.50m High).
Analysis 4	No	Yes	A generic assessment of embankments with Class 2A4, unsupported and a 1V:3H slope gradient (3.00m High).
Analysis 5	Yes	No	An assessment of Cross Section AA (SBH/19 realignment, Ch. 0+120), An unsupported embankment with Class 2A4 fill.
Analysis 6	No	Yes	An assessment of Cross Section AA (SBH/19 realignment, Ch. 0+120), An unsupported embankment with Class 2A4 fill.
Analysis 7	Yes	No	An assessment of Cross Section CC (SBH/19 realignment, Ch. 0+230), An unsupported embankment with Class 2A4/7C3 fill.
Analysis 8	No	Yes	An assessment of Cross Section CC (SBH/19 realignment, Ch. 0+230), An unsupported embankment with Class 2A4/7C3 fill.

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Analysis 9	Yes	No	An assessment of Cross Section GG (SBH/27 realignment, Ch. 0+100), An unsupported embankment with Class 2A4 fill.
Analysis 10	Yes	No	An assessment of Cross Section HH (SBH/27 realignment, Ch. 0+020), An unsupported embankment with Class 2A4/7C3 fill.
Analysis 11	Yes	No	An assessment of Cross Section II (SBH/19 link, Ch. 0+050), An unsupported embankment with Class 2A4/7C3 fill.
Analysis 12	No	Yes	An assessment of Cross Section JJ (SBH/19 link, Ch. 0+110), An unsupported embankment with Class 2A4 fill.
Analysis 13	Yes	No	A generic unsupported embankment assessment with Class 2A4/7C3, with a slope gradient of 1V:3H slope (9.60m High). Assessment of Chainage 0+090 on Bridleway SBH/19 Realignment
Analysis 14	No	Yes	A generic unsupported embankment assessment with Class 2A4/7C3, with a slope gradient of 1V:3H slope (9.60m High). Assessment of Chainage 0+090 on Bridleway SBH/19 Realignment
Analysis 15	Yes	No	An assessment of Cross Section CC, an embankment supported with geogrids and Class 2A4/7C3 fill.
Analysis 16	No	Yes	An assessment of Cross Section CC, an embankment supported with geogrids and Class 2A4/7C3 fill.
Analysis 17	Yes	No	An assessment of Cross Section HH, an embankment supported with geogrids and Class 2A4/7C3 fill.
Analysis 18	Yes	No	An assessment of Cross Section II, an embankment supported with geogrids and Class 2A4/7C3 fill.
Analysis 19	Yes	No	An assessment of Cross Section II, an embankment supported with geogrids, including a basal geogrid layer and Class 2A4/7C3 fill.
Analysis 20	Yes	No	A generic embankment assessment with Class 2A4/7C3 and supported with geogrid, 1V:3H slope (9.60m High). Assessment of Chainage 0+090 on Bridleway SBH/19 Realignment
Analysis 21	No	Yes	A generic 1V:3H (9.60m high) embankment assessment with Class 2A4/7C3 and supported with geogrid. Assessment of Chainage 0+090 on Bridleway SBH/19 Realignment
Analysis 22	No	Yes	A generic 1V:3H (9.60m high) embankment assessment with Class 2A4/7C3 and supported with geogrid, including a basal geogrid layer. Assessment of Chainage 0+090 on Bridleway SBH/19 Realignment
Analysis 23	Yes	No	A generic embankment with Class 2A4 which is unsupported with a 1V:3.5H slope gradient (9.60m High).
Analysis 24	Yes	No	A generic embankment with Class 2A4 which is unsupported with a 1V:3.6H slope (9.60m High).
Analysis 25	Yes	No	A generic unsupported embankment with Class 2A4 in the temporary (i.e., <2 years) scenario, with a 1V:3H slope gradient (10.30m High).
Analysis 26	No	Yes	A generic unsupported embankment with Class 2A4 in the temporary (i.e., <2 years) scenario, with a 1V:3H slope gradient (10.30m High).
Analysis 27	No	Yes	Analysis of Pond C2.L.22-PND-0606 (Section KK) adjacent to an unsupported highway embankment.
Analysis 28	Yes	No	Analysis of Pond C2.T.21-PND-0611 (Section EE) adjacent to an unsupported highway embankment.

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Table 5-2: Summary of Slope Stability Results – Unreinforced Embankments

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
1	2.00	1V:3H	Class 2A4	0.966	0.326
2	2.50	1V:3H	Class 2A4	1.000	0.355
3	2.50	1V:3H	Class 2A4	1.000	0.253
4	3.00	1V:3H	Class 2A4/7C3	1.029	0.403
5 (Section AA)	8.10	1V:3.75H	Class 2A4	0.949	0.473
6 (Section AA)	8.10	1V:3.75H	Class 2A4	0.949	0.619
7 (Section CC)	3.40	1V:3H	Class 2A4/7C3	1.071	0.294
8 (Section CC)	3.40	1V:3H	Class 2A4/7C3	1.136	0.415
9 (Section GG)	4.70	1V:3.9H	Class 2A4	0.868	0.299
10 (Section HH)	8.40	1V:3.3H	Class 2A4/7C3	1.064	0.493
11 (Section II)	7.00	1V:2.5H	Class 2A4/7C3	1.395	0.571
12 (Section JJ)	4.10	1V:4H	Class 2A4	0.901	0.419
13	9.60	1V:3H	Class 2A4/7C3	1.178	0.649
14	9.60	1V:3H	Class 2A4/7C3	1.179	0.847
23	9.60	1V:3.5H	Class 2A4/7C3	1.034	0.544
24	9.60	1V:3.6H	Class 2A4	0.984	0.533

Notes:

1) The results for the construction (short term) analysis are presented as the DOU for the undrained scenario. Whereas the long term analysis are presented as the DOU for the drained scenario.

2) Analysis 1 to 4 shows the maximum height (3.00m) a 1V:3H slope can sustain whilst been unsupported by either the Landscape Bunds or Geogrid reinforcement.

5.4.1 Summary of Unreinforced Embankments

Based on the results above, the following conclusions can be made:

Bridleway SBH/19 Realignment:

- The left-hand side slope, between Chainages 0+085 and 0+110, require geogrid reinforcement due sides slopes having a proposed gradient of 1V:3H and the earthwork heights between 9.00m and 10.00m.
- Additionally, the right-hand side slope, which is no longer constrained by the Landscape Bund, between Chainages 0+220 and 0+240, require geogrid reinforcement due to the proposed gradient of 1V:3H and the earthwork heights exceeding 3.00m.
- The left-hand side slope between Chainages 0+110 and 0+220 does not require geogrid reinforcement, due to the side slopes having a proposed gradient greater then 1V:3.5H.

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Footpath SBH/27 Realignment:

- The left-hand side slope of Footpath SBH/27 realignment requires geogrid reinforcement between Chainages 0+000 and 0+090, due to the slope gradients being between 1V:3H and 1V:3.5H, and slope heights in excess of 3.00m.
- Beyond Chainage 0+090, on the left-hand side, the slope gradient is greater than 1V:3.5H, so geogrid reinforcement is not required. Similarly, beyond Chainage 0+225, on the right-hand side, the highway earthwork is no longer constrained by the Landscape Bund and is less than 3.00m in height, therefore geogrid reinforcement is not required, despite the slope gradient of 1V:3H.

Bridleway SBH/19 Link:

- The right-hand side slope of Bridleway SBH/19 Link requires geogrid reinforcement between Chainages 0+000 and 0+080, due to the slope gradients being between 1V:2.5H and 1V:3.5H, and slope heights in excess of 3.00m.
- Beyond Chainage 0+080, on the right-hand side, the slope gradient is greater than 1V:3.5H, so geogrid reinforcement is not required. Similarly, beyond Chainage 0+360, on the left-hand side, the highway earthwork is no longer constrained by the Landscape Bund and is less than 3.00m in height, therefore geogrid reinforcement is not required, despite the slope gradient of 1V:3H.

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Table 5-3: Summary of Slope Stability Results - Reinforced Embankments (Class 2A4/7C3)

Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	Location (m above base)	Length (m)	DoU (As Per BS8006)	DoU (As Per DA1 C2)
15 (Section CC)	3.40	1V:3H	3 (Primary Reinforcement) 3 (Secondary Reinforcement)	0	6.0	0.854	0.942
				0.5	2.0		
				1.0	6.0		
				1.5	2.0		
				2.0	6.0		
				2.5	2.0		
16 (Section CC)	3.40	1V:3H	3 (Primary Reinforcement) 3 (Secondary Reinforcement)	0	6.0	0.879	0.990
				0.5	2.0		
				1.0	6.0		
				1.5	2.0		
				2.0	6.0		
				2.5	2.0		
17 (Section HH)	8.40	1V:3.3H	7 (Primary Reinforcement) 7 (Secondary Reinforcement)	0	7.0	0.774	0.918
				0.5	2.0		
				1.0	7.0		
				1.5	2.0		
				2.0	7.0		
				2.5	2.0		
				3.0	7.0		
				3.5	2.0		
				4.0	7.0		
				4.5	2.0		
				5.0	7.0		
				5.5	2.0		
				6.0	7.0		
				6.5	2.0		
18 (Section II)	6.40	1V:2.5H	6 (Primary Reinforcement) 6 (Secondary Reinforcement)	0	8.0	0.955	1.016
				0.5	2.0		
				1.0	8.0		
				1.5	2.0		
				2.0	8.0		
				2.5	2.0		
				3.0	8.0		
				3.5	2.0		
				4.0	8.0		
				4.5	2.0		
				5.0	8.0		
				5.5	2.0		
19 (Section II)	6.40	1V:2.5H	1 Basal Reinforcement 6 (Primary Reinforcement) 6 (Secondary Reinforcement)	-0.5 (Basal Reinforcement)	18	0.949	0.967
				0.0	7.0		
				0.5	2.0		
				1.0	7.0		
				1.5	2.0		
				2.0	7.0		
				2.5	2.0		
				3.0	7.0		
				3.5	2.0		
				4.0	7.0		
				4.5	2.0		
				5.0	7.0		
				5.5	2.0		
20	9.60	1V:3H	9 (Primary Reinforcement) 9 (Secondary Reinforcement)	0	6.0	0.856	0.993
				0.5	2.0		
				1.0	6.0		
				1.5	2.0		
				2.0	6.0		
				2.5	2.0		
				3.0	6.0		
				3.5	2.0		
				4.0	6.0		
				4.5	2.0		
				5.0	6.0		
				5.5	2.0		
				6.0	6.0		
				6.6	2.0		
				7.0	6.0		
				7.5	2.0		
				8.0	6.0		
				8.5	2.0		
21	9.60	1V:3H	9 (Primary Reinforcement) 9 (Secondary Reinforcement)	0	6.0	0.867	1.034
				0.5	2.0		
				1.0	6.0		

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Analysis No.	Embankment Height (m)	Slope Gradient	No. of layers	Location (m above base)	Length (m)	DoU (As Per BS8006)	DoU (As Per DA1 C2)
				1.5	2.0		
				2.0	6.0		
				2.5	2.0		
				3.0	6.0		
				3.5	2.0		
				4.0	6.0		
				4.5	2.0		
				5.0	6.0		
				5.5	2.0		
				6.0	6.0		
				6.5	2.0		
				7.0	6.0		
				7.5	2.0		
				8.0	6.0		
				8.5	2.0		
				0 (Basal Reinforcement)	21		
				0.5	2.0		
				1.0	6.0		
				1.5	2.0		
				2.0	6.0		
				2.5	2.0		
				3.0	6.0		
				3.5	2.0		
22	9.60	1V:3H	1 Basal Reinforcement 8 (Primary Reinforcement) 9 (Secondary Reinforcement)	4.0	6.0	0.851	0.986
				4.5	2.0		
				5.0	6.0		
				5.5	2.0		
				6.0	6.0		
				6.5	2.0		
				7.0	6.0		
				7.5	2.0		
				8.0	6.0		
				8.5	2.0		

Notes:

1) Results are not presented for DA1C1 as they do not govern the slope design.

2) All results are presented as degree of utilisation.

3) A 2m deep vadose zone has been modelled within the highway embankment with an r_u of 0.2 with the remaining embankment fill modelled with an r_u of 0.1 for all embankment heights.

4) The results for Analysis 18 (GM1) show that for slope gradients of between 1V:2.5H and 1V:3H, a basal geogrid reinforcement layer is required in order to maintain the stability of the side slope. Adding a basal geogrid reinforcement layer will decrease the DOU to <1.000 in DA1 C2, as shown by Analysis 19 (GM1).

The results for Analysis 21 (GM2) show that for slope gradients of 1V:3H at 9.60m in height, a basal geogrid reinforcement layer is required in order to maintain the stability of the side slope. Adding a basal geogrid reinforcement layer will decrease the DOU to <1.000 in DA1 C2, as shown by Analysis 22 (GM1).

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Large sections of the highway earthworks for Oat Close SBH/19 are to be constrained by landscape bunds. To model the stability of these unsupported embankments in the intermediate term (i.e., <2 years before they are supported by landscape bunds in the long term), peak shear strength parameters and a reduced Vadose Zone depth of 1.00m have been adopted for the 'partially drained' condition. The results are summarised in Table 5-4.

Although the drained DOU, as derived using EC7, is exactly 1.000 in Table 5-4 for the maximum slope height encountered, conservative parameters have been adopted and the exact DOU value is considered acceptable for short term temporary conditions and the shallow nature of the critical slip surface.

In summary, it is considered that where landscape bunds are to be adopted to support the proposed highway earthworks, the embankments will stand unsupported in the short to medium term (i.e., <2 years) pending construction of the landscape bunds.

Table 5-4: Slope Stability Results – Temporary Condition

Analysis No.	Embankment Height (m)	Slope Gradient	Fill Class	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
25	10.3	1V:3H	Class 2A4	1.000	0.642
26	10.3	1V:3H	Class 2A4	1.000	0.835

5.5 Pond Side Slope Stability

There are three drainage ponds (two land drainage ponds and one for track drainage), associated with IDR 207.2, close to Oat Close SBH/19 Bridleway Realignment. Two of the ponds, C2.L.22-PND-0606 and C2.T.21-PND-0611 are within 4.00m of the toe of the highway embankment. The results to determine whether the presence of the ponds impacts the stability of the highway earthwork are presented in Table 5-5.

Table 5-5: Slope Stability Results – Pond Analysis

Analysis No.	Highway Embankment Height (m)	Pond Slope Gradient	Pond Distance from Embankment Toe (m)	Static DOU (As Per EC7 - DA1C2)	
				Drained	Undrained
27	0.42	1V:3H	3.80	0.989	0.173
28	1.64	1V:5H	1.60	0.942	0.185

In summary, the above results show a DOU of <1.000 for the global stability checks, taking into account the stability between the highway earthwork and the ponds. Therefore, it is considered that the global stability of the ponds and the highway earthwork is suitable and will stand unsupported.

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5.6 Summary of Reinforcement Design

The following sections details the proposed design solution(s) along the alignment for both sides of the embankment.

Table 5-6: Summary of Reinforcement Design

Left Hand Side		Right Hand Side	
Ch.	Solution	Ch.	Solution
Bridleway SBH/19 Realignment			
0+000 to 0+015	Supported by the Landscape Bund	0+000 to 0+015	Supported by the Landscape Bund
0+015 to 0+071	Overbridge	0+015 to 0+071	Overbridge
0+071 to 0+085	Supported by the Landscape Bund		
0+085 to 0+110	Geogrid Reinforcement	0+071 to 0+220	Supported by the Landscape Bund
		0+220 to 0+240	Geogrid Reinforcement
0+110 to 0+220	Unsupported	0+240 to 0+381	Unsupported
Bridleway SBH/19 Link			
0+000 to 0+360	Supported by the Landscape Bund	0+000 to 0+080	Geogrid Reinforcement
0+360 to 413	Unsupported	0+080 to 0+413	Unsupported
Footpath SBH/27 Realignment			
0+000 to 0+090	Geogrid Reinforcement	0+000 to 0+225	Supported by the Landscape Bund
0+090 to 0+385	Unsupported	0+225 to 0+385	Unsupported

Notes:

Side of embankment taken as if looking up chainage.

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5.7 Transition to Structures

Figure 5-1 below, shows the indicative arrangements of the highway earthwork and the structure. In summary the structure including the impact walls, bank seat and wing walls are being placed on a granular wedge of class 6N material.

The Class 6N internal wedge fill for the overbridge design is anticipated to approximately extend 13m (to Ch. 0+002) and 14m (to Ch. 0+085) back from the west and east abutments, respectively. Based on that, the Class 6N material will not interfere with the ability to place geogrid reinforcement within the cohesive Class 2 embankment fill require achieve stability of the proposed side slope gradients.

This interface will be translated into the detailed design drawings and 3D model.

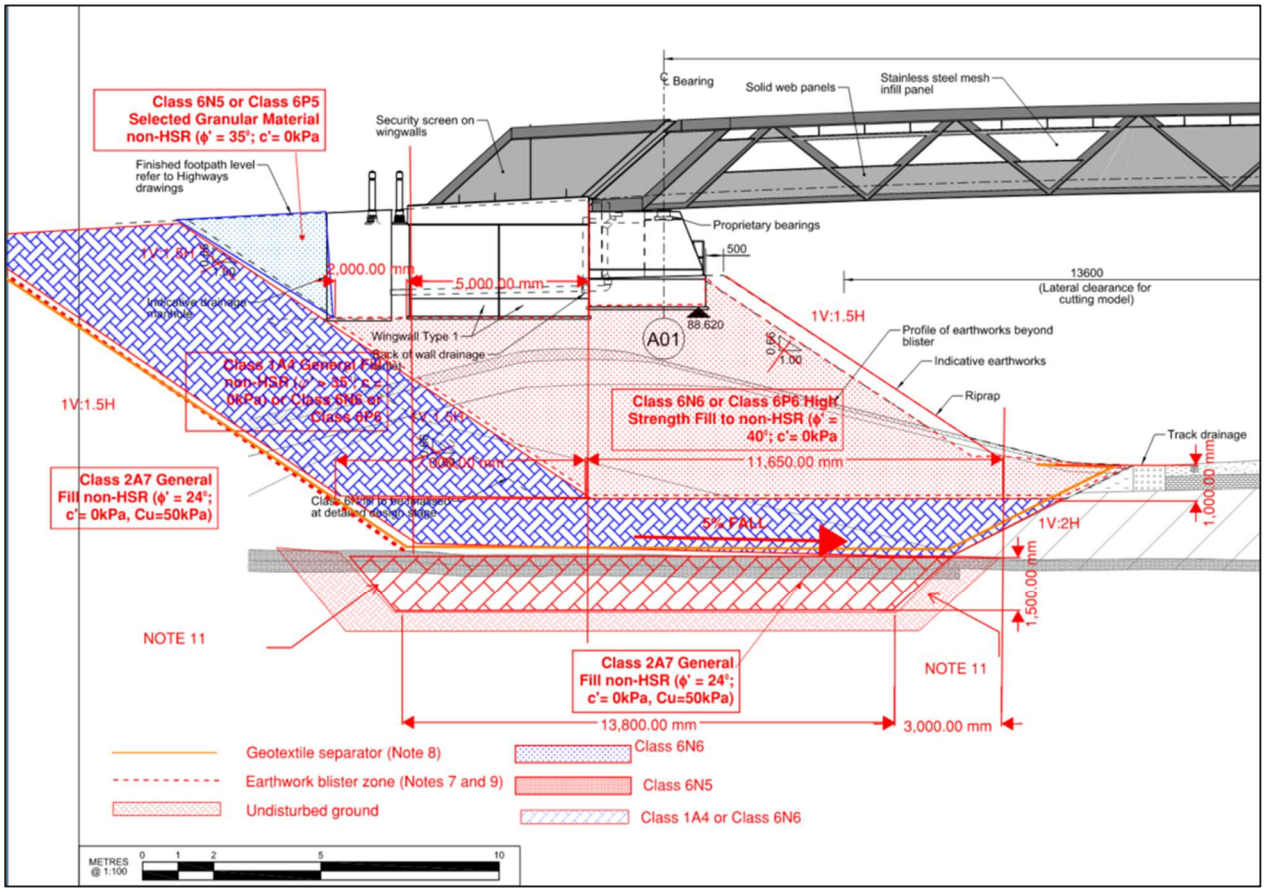


Figure 5-1: Indicative Transition to Structure (NTS)

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6 Settlement Analysis

6.1 Design Considerations and Assumption

6.1.1 Design Standards

The design has been carried out to assess settlement in accordance with EC7 – SLS and in accordance with the HS2 Technical Standard Earthworks (HS2-HS2-GT-STD-000-000001).

6.1.2 Design Methodology

The Boussinesq method of stress distribution has been used. Post consolidation settlement has been determined as the total settlement (drained conditions) minus the immediate settlement (undrained conditions) i.e., no hold period has been allowed for between construction of the embankment and placement of the pavement.

The subgrade assessment considers vertical loads from the embankment to be calculated and applied over its footprint. The effects of the vertical load applied to the soil result in displacements, with the longitudinal settlement calculated as part of the analysis.

6.1.3 Geometry

The highway embankment slopes across Oat Close Bridleway (SBH/19) Realignment are proposed to be constructed at varying gradients of between 1V:2.5H and 1V:4H. The landscape bunds running parallel to the mainline will tie into the highway earthworks have been included in the calculation for long term settlement.

The settlement assessment has modelled the eastern embankment for Bridleway SBH/19 Realignment due to the varying embankment heights and gradients. The varying heights of the embankment have been taken from the elevations along the longitudinal centre of the bridleway.

The eastern embankment for Bridleway SBH/19 Realignment is approximately 10.35m in height at the centre of the bridleway within the wingwalls of the overbridge, with 10m being applied for the settlement analysis.

The embankment and landscaping bund widths have been measured using the highway cross sections provided at every 10m chainage.

The boundary between the embankment and the overbridge is taken as Chainage 0+071, with the model displacement lines ranging from Ch. 0+040 to Ch. 0+231.

A summary of the embankment geometry and loading zones is presented in Figure 6-1.

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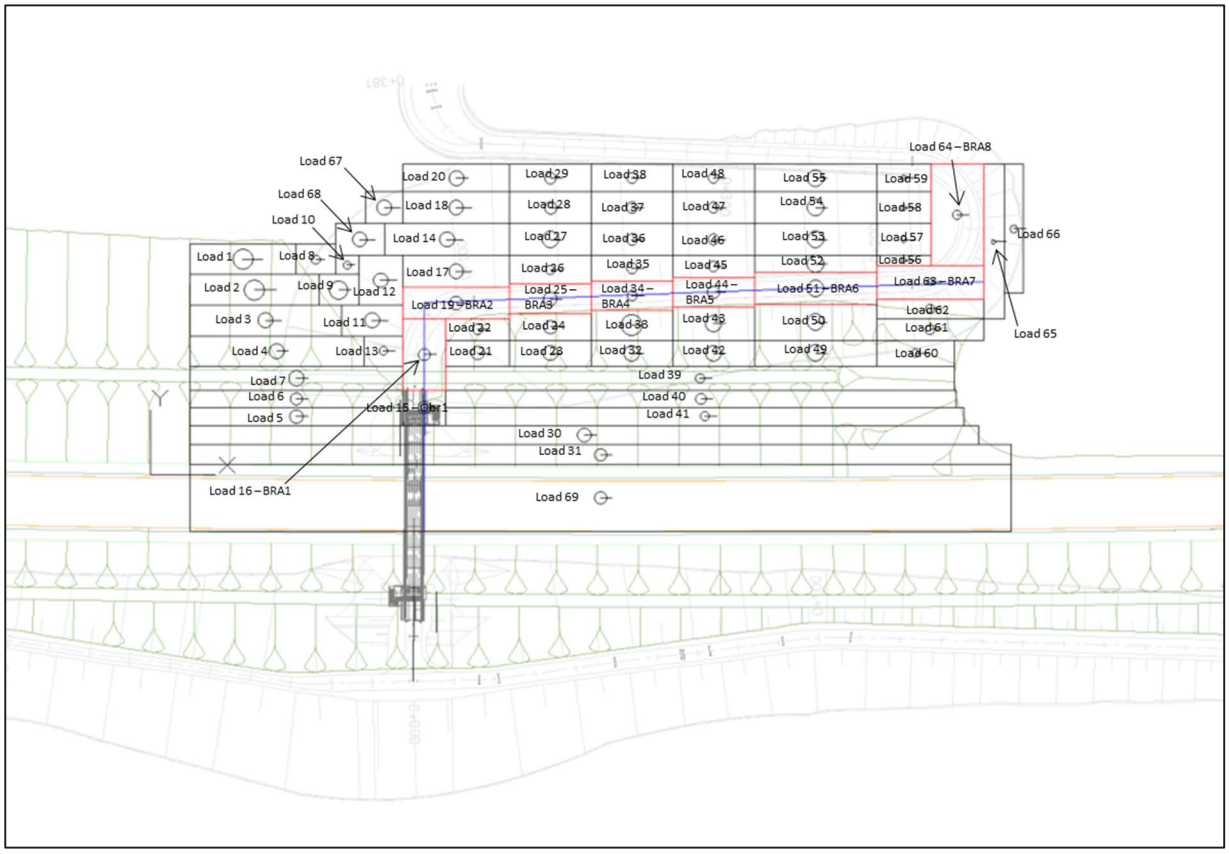


Figure 6-1: Settlement Loading Zones and Geometry

6.1.4 Ground Conditions

The settlement analysis has combined the worst case ground conditions from Ground Model 1 (Table 4-1) and Ground Model 2 (Table 4-2), such that Head has been modelled above thick deposits of cohesive strata from the Gault Clay Formation and the Whitchurch Sand Formation, and subsequently above the bedrock of the Purbeck Group and the Portland Sand / Stone Formations. The parameter values have been adopted from Table 4-3. In keeping with the overbridge design, an area of 1.50m thick Excavate and Replace has been modelled in below Zone 15 – OBr1, and as per the overbridge design the material is cohesive and uses a Young's Modulus of 45 MPa.

For the other strata, the Young's Modulus (E') of Soil has been derived based Notes 7 and 8 below Table 4-3, as set out the Ground Engineering Technical Note – Guidance on the Assessment of Settlement 1MC06-CEK-GT-NOT-C002-000023 C02 Page 26.

The Poissons' Ratio for all cohesive soils has been taken as 0.2 for drained conditions and 0.5 for undrained conditions.

Design groundwater level was assumed at ground level at the base of embankment.

6.1.5 Loading

For calculation of settlement, it has been assumed that no surcharge is applied to the top of the embankment. To account for variability in the fill material a unit weight of 20kN/m^3 has been considered for the weight of the embankment as a conservative approach.

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6.1.6 Modelling Details

Settlement has been modelled and calculated using the geotechnical software Oasys PDisp.

Only settlement within the foundation soils has been considered. Between Ch. 0+071 and Ch. 0+110, granular or geo-fabric drainage layers have been specified at 1.5m centres vertically to allow consolidation settlement to occur during and immediately after construction. Internal settlement of the embankment is expected to occur during construction and therefore will not affect the long-term settlement predictions. Reference should be made to SED G105 on Drawing No 1MC06-CEK-GT-DSE-C002-000104.

For the western approach of Bridleway SBH/19 realignment horizontal drainage layers will be extended between Ch. 0+000 and 0+015. Additionally, drainage layers will extend from Ch. 0+000 to 0+020 for both Bridleway SBH/19 Link and Footpath SBH/27 Realignment.

The horizontal rigid boundary level has been determined as the level at which the increase in effective vertical stress is 20% of the effective overburden stress for assessing the subgrade settlement, in accordance with EC7.

6.2 Summary of Analysis

6.2.1 Location of analysis

Settlement analysis has been carried out for the highway asset near structures and where there is a significant height of earthworks embankment present. Assessments have been carried out for the following portions of the highway earthworks:

- Oat Close Bridleway SBH/19 Realignment, specifically the eastern approach of Bridleway SBH/19 Realignment.

This is considered to be the critical case and, therefore, no additional analysis of Bridleway SBH/19 Link and Footpath SBH/27 Realignment is required due to lower embankment height and its lack of proximity to the overbridge.

6.2.2 Loadings

The loadings for the settlement analysis are presented within Appendix D, however the model loading zones for the bridge abutment and the bridleway alignment are detailed in Table 6-1. The loadings are based on embankment heights ranging from 3.0m to 10.0m.

Table 6-1: Summary of Embankment Model Zones along Centre Line

Longitudinal Zone No.	Height (m)	Width (m)	Length (m)
Load 15 - OBr1	10	10.41	8.57
Load 16 - BRA1	10	10.41	17.5
Load 19 - BRA2	9	26.14	7.78
Load 25 - BRA3	8	19.98	7.3
Load 34 - BRA4	7	19.98	7.3
Load 44 - BRA5	6	19.98	7.3
Load 51 - BRA6	5	29.98	7.7
Load 63 - BRA7	4.5	26.13	8.29
Load 64 - BRA8	3	12.84	24.7

Notes: OBr = Overbridge, BRA = Bridleway Alignment

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6.2.3 Rigid Boundary

The rigid boundary has been determined as per Section 6.1.6, considering the loads in Section 6.2.2. A summary of the calculated rigid boundaries for each zone are provided in Table 6-2:

Table 6-2: Rigid Boundary for each zone

Longitudinal Zone No.	Embankment Height (m)	Displacement Line Name	Rigid Boundary Depth (m bgl)
Load 15 - OBr1	10	Load 15 - OBr1, centre vertical	44.2
Load 16 - BRA1	10	Load 16 - BRA1, centre vertical	46.2
Load 19 - BRA2	9	Load 19 - BRA2, centre vertical	45.1
Load 25 - BRA3	8	Load 25 - BRA3, centre vertical	45.4
Load 34 - BRA4	7	Load 34 - BRA4, centre vertical	44.1
Load 44 - BRA5	6	Load 44 - BRA5, centre vertical	42.0
Load 51 - BRA6	5	Load 51 - BRA6, centre vertical	38.0
Load 63 - BRA7	4.5	Load 63 - BRA7, centre vertical	30.1
Load 64 - BRA8	3	Load 64 - BRA8, centre vertical	21.2

Notes: OBr = Overbridge, BRA = Bridleway Alignment

6.2.4 Results

Maximum settlement values have been taken from the highest point of the embankments and beneath the centre of the bridleway.

Table 6-3: Maximum Settlement Results

Embankment Height (m)	Immediate Settlement (Si) (mm)	Consolidation Settlement (Sc) (mm)	Total Settlement (St) (mm)
10*	69	88	158
10	64	114	179
9	56	106	162
8	48	99	147
7	40	88	128
6	32	77	109
5	25	65	90
4.5	21	52	73
3	14	34	48

Notes:

* = Maximum settlement results beneath the overbridge abutment.

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6.3 Transitions to Structures

Settlement of non-mainline highway earthworks can result in differential movement between structures and approach earthworks leading to an impact on the quality and can be a hazard to pedestrians.

Currently there is no guidance provided by HS2 on the permissible limits on settlement for approach earthworks to non-mainline structures. In the absence of any guidance, the following limits are proposed:

Bridleways and Footpaths:

- Criterion 1: Between 0 and 10m back from a structure – post construction (consolidation) settlement should not exceed 100mm
- Criterion 2: 0 to 20m back from a structure – post construction differential settlement should not exceed 1V:100H (1.0%)
- Criterion 3: >20m Along alignment – post construction differential settlement should not exceed 1V:100H (1.0%) from design gradient.

The settlement results (see Table 6-3 and Figure 6-2 below) show a maximum consolidation settlement of 114mm, which is slightly above the 100mm criteria limit. Considering that the consolidation settlement will commence as soon as the construction work begins and the embankment construction will take place over a period of months, no specific hold period is required before construction of the permanent works, including the pavement. Additionally, it should also be considered that the Overbridge is expected to settle at the same time.

As shown on the chart below (red line) the predicted differential settlement is less than the 1V:100H allowable differential settlement limit (black line) at a distance of 0m to 10m back from the overbridge.

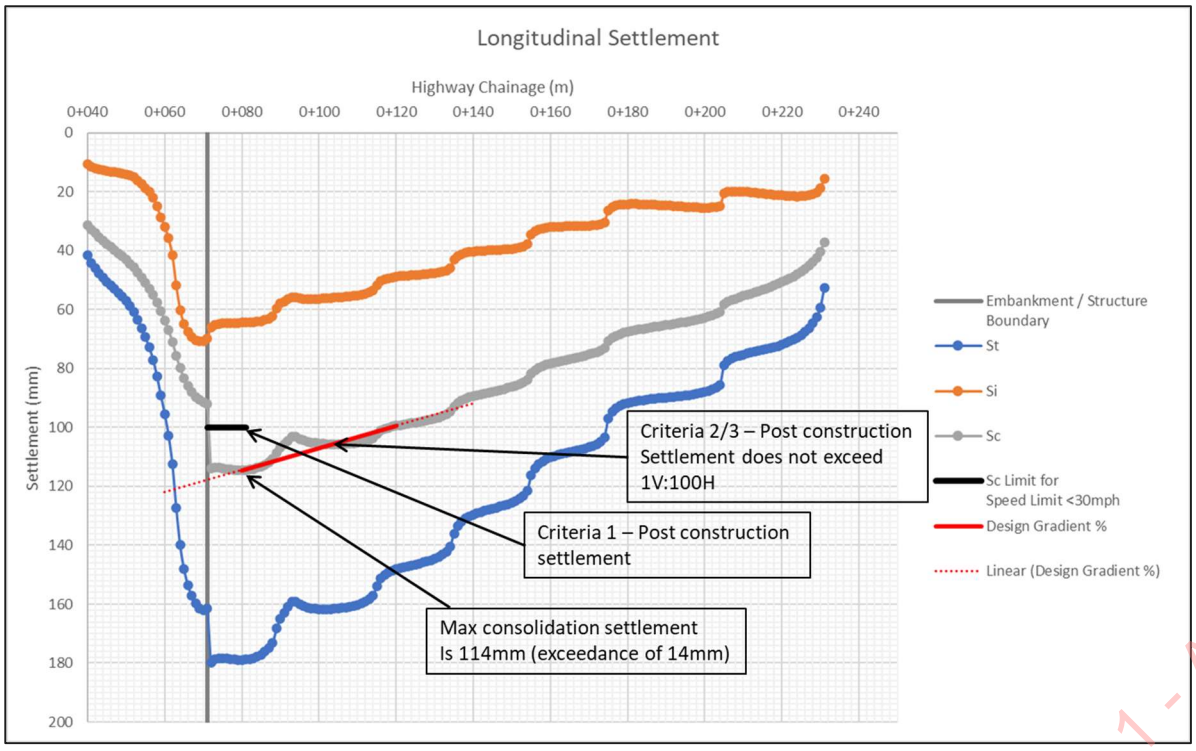


Figure 6-2: Longitudinal Settlement

Instrumentation and monitoring is required for the highway asset and further details are detailed within Section 11 and Appendix F.

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6.4 Internal Embankment Settlement

No significant long term internal embankment settlement is predicted for high embankments as it is assumed that internal drainage layers will be included with embankments over 3m in height as per SED. G105 on Dwg 1MC06-CEK-GT-DSE-C002 000104 C01.

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7 Pavement Design

7.1 Introduction

The pavement design is based on the DMRB, Volume 7: Pavement Design and Construction as well as the HS2 Technical Standard and the HS2 Highway Standard Details.

Footpaths/bridleways are to be designed in accordance with CD239. Design CBR values are derived for embankments and the existing ground considering available in situ and laboratory testing as well as correlations with other derived parameters.

The subgrade underlying the majority of Bridleway SBH/19 Link is anticipated to be Head deposits, whereas Bridleway SBH/19 Realignment and Footpath SBH/27 Realignment is underlain by the Gault Clay Formation and embankments that are proposed to be constructed with Class 2A4/7C3 material (remoulded Gault Clay).

With reference to CD225, for any pavement, the lowest of the short term CBR (for construction traffic loadings) and long-term equilibrium CBR (for long term traffic loadings) should be adopted for design. The short term CBR is based on either plate bearing tests or CBR testing or by corelation with undrained strength testing within the near surface soils. The long-term equilibrium CBR value is based on the Plasticity Index (PI), of the natural subgrade or fill material, based on Table C1 of LR 1132.

7.2 Summary of the CBR

The design CBR for each formation strata are presented in Table 7-1.

Table 7-1: Design CBR Values

Strata	Short Term CBR (%)	Long Term Equilibrium CBR (%)	Design CBR (%)	Comments
Class 6N6 or Class 1A4	>5%	>5%	>5%	-
Class 2A4/7C3	2.0%	2.0%	2.0%	CBR assumes a minimum of 50kPa undrained shear strength and plasticity index of ≤50% as defined in IDR 207.2 Specification
Head	2.0%	2.5%	2.0%	Short term CBR based on available undrained shear strength (using correlation of CBR=cu/23) with design value of 40kPa. Long term CBR based upon design plasticity index of 40% and correlation with table C1 from LR1132. Refer to Appendix C for data plots.
Gault Clay	2.0%	2.0%	2.0%	Short term CBR based on available undrained shear strength (using correlation of CBR=cu/23) with design value of 50kPa. Long term CBR based upon design plasticity index of 50% and correlation with table C1 from LR1132. Refer to Appendix C for data plots.

During construction, verification testing to confirm the design Atterberg Limits, undrained shear strength and CBR testing should be undertaken to verify the design in-situ CBR value. Soft spots are to be identified as cohesive soils with a shear strength/CBR of less than the assumed design value and should be excavated and replaced as set out in the Specification.

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Construction of the bridleway/footpath which is at-grade should be avoided in winter and wet conditions to reduce the possibility of degradation and softening of the near surface soils during construction.

7.3 Geotechnical Input into Pavement Design

Considering the design CBR and the usage of the highway assets, further geotechnical input is often required to ensure a suitable pavement thickness/foundation is considered. The following sections describe further geotechnical input where they're deemed appropriate with a summary presented in Table 7-2.

7.3.1 Bound Pavements

The following assets are to be constructed using a bound pavement design:

- **Bridleway SBH/19 Realignment (Ch 0+005 to 0+020 and Ch 0+065 to 0+085)** – The pavement type is C1 and is to be mainly founded on top of the embankment fill (Class 6N6 or Class 1A4) with a design CBR of >5.0%, thus geotechnical input is not required.

7.3.2 Unbound Pavements

The following assets of Oat Close Bridleway SBH/19 Realignment are to be constructed using an unbound pavement design:

- **Bridleway SBH/19 Realignment (Ch 0+000 to 0+005 and Ch 0+085 to 0+381)** – The pavement type is to be AT2 and shall be mainly founded on Gault Clay or Class 2A4/7C3, both of which have a design CBR of 2.0%.
- **Bridleway SBH/19 Link (Ch 0+000 to 0+414)** - The pavement type is to be AT2 and shall be founded on Head deposits or Class 2A4/7C3, both of which have a design CBR of 2.0%.
- **Footpath SBH/27 Realignment (Ch. 0+000 to 0+387)** - The pavement type is to be AT2 and shall be founded on Gault Clay or Class 2A4/7C3, both of which have a design CBR of 2.0%.

However, as the aforementioned assets are considered to be a minor bridleway/footpath with an allowance of only light traffic, it is considered that no further geotechnical input is required.

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7.3.3 Summary of Geotechnical Input to Pavement Design

Table 7-2: Summary of the Geotechnical Input into the Pavement Design

Permitted Pavement Options								
Description (e.g., mainline)	Chainage (m)		Permitted Pavement Option	Pavement Type	CBR (%)	Total Thickness without GEO input (mm)	Geotechnical Input	Total Thickness with GEO input (mm)
	From	To						
Bridleway SBH/19 Realignment (South)	0	5	AT2	Unbound	2.0	250	None [1]	-
	5	20	C1	Bound	2.0	245	None [2]	-
Bridleway SBH/19 Realignment (North)	65	85	C1	Bound	2.0	245	None [2]	-
	80	381	AT2	Unbound	2.0	250	None [1]	-
Bridleway SBH/19 Link	0	414	AT2	Unbound	2.0	250	None [1]	-
Footpath SBH/27 Realignment	0	387	AT2	Unbound	2.0	250	None [1]	-

Notes:
[1] Based on the use of the asset as a bridleway and footpath, the currently proposed pavement thickness will be sufficient to support the anticipated loadings thus no further GEO input is required.
[2] Based on the overbridge design, the pavement type C1 will be founded on either Class 6N6 or Class 1A4.

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8 Summary of Highway Design

The following table summarises the geotechnical design along the alignment.

Table 8-1: Summary of Design – Bridleway SBH/19 Realignment

Element	Left Hand Side							Right Hand Side						
	0+000 to 0+015	0+015 to 0+071	0+071 to 0+085	0+085 to 0+110	0+110 to 0+230	0+230 to 0+290	0+290 to 0+381	0+000 to 0+015	0+015 to 0+071	0+071 to 0+220	0+220 to 0+240	0+240 to 0+270	0+270 to 0+290	0+290 to 0+381
Embankment Heights (m)	8.63 – 9.72	Overbridge	10.35 – 9.57	9.57 – 8.66	8.66 – 3.39	3.39 - <0.75	<0.75	8.63 – 9.72	Overbridge	10.35 – 3.94	3.94 – 2.91	2.91 – 1.94	1.94 - <0.75	<0.75
Side Slope Angle	1V:3H		1V:3H	1V:3H to 1V:3.5H	>1V:3.5H	>1V:3.5H to 1V:2H [2]	1V:2H to 1V:1H [2]	1V:3H		1V:3H				1V:3H to >1V:3H
Side Slope Design	Soil slope supported by the Landscape Bund		Soil slope supported by the Landscape Bund	Reinforced	Soil Slope			Soil slope supported by the Landscape Bund		Soil slope supported by the Landscape Bund	Reinforced	Soil Slope		
Embankment Fill Class	Class 2A4		Class 2A4	Class 2A4 / 7C3	Class 2A4		Class 1A4 or 2A4	Class 2A4		Class 2A4	Class 2A4 / 7C3	Class 2A4		Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4 / Class 6N6 or 1A4 [1]		Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4		GLT [3]	Class 2A4 / Class 6N6 or 1A4 [1]		Class 2A4 / Class 6N6 or 1A4 [1]	Class 2A4			GLT [3]
Subgrade DSG CBR Value (%)	2.0		2.0					2.0		2.0				
Subgrade Improvement Layer	No		No					No		No				
Pavement thickness (mm)	250		250					250		250				
Earthwork Geotextile Separator Layer	Yes		Yes					Yes		Yes				
Topsoil Strip	No		No		Yes [4]			No		No		Yes		
Starter Layer	Yes		Yes		No [5]			Yes		Yes		No		
Excavate and Replace	Yes [6]		Yes [6]	No				Yes [6]		Yes [6]	No			
Horizontal Drainage Layers	Yes [7]		Yes [7]	No				Yes [7]		Yes [7]	No			
Mitigation SED	G105		G105	G104	G103, G104		G102	G105		G105, G104	G104, G103	G103	G103, G102	G102

Notes:
1 – The Class 6N6 or 1A4 is a requirement from the overbridge design. Refer to Section 7.3 for extent of Class 6N6 or 1A4.
2 – Due to the geometry of the bridleway, there is a small gully on the left-hand side slope. Beyond Ch. 0+260, the right-hand slope of this small gully is 0.30m high with slope gradients of between 1V:2H and 1V:1H (Ch. 360).
3 - GLT = Gault Clay Formation.
4 – The topsoil strip will extend from Chainage 0+240, due to the embankment height being less than 3.00m
5 – Beyond Ch. 0+240, a starter layer is not required, due to the embankment height being less than 3.00m
6 – Excavate and Replace is required below the abutments as per the Overbridge design.
7 – The horizontal drainage layer only extends to Ch. 0+110 on the left-hand side and to Ch. 0+220 on the right-hand side of the embankment to accelerate consolidation. Refer to standard earthwork details G105.

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Table 8-2: Summary of Design –Bridleway SBH/19 Link

Element	Left Hand Side			Right Hand Side					
	0+000 to 0+360	0+360 to 0+390	0+390 to 0+413	0+000 to 0+020	0+020 to 0+080	0+080 to 0+100	0+100 to 0+190	0+190 to 0+390	0+390 to 0+413
Embankment Heights (m)	8.63 – 1.74	1.74 – <0.75	<0.75	8.63 – 8.36	8.36 – 4.97	4.97 – 4.20	4.20 – 2.99	2.99 – <0.75	<0.75
Side Slope Angle	1V:3H			1V:2.9H – 1V:3.1H	1V:2.5H to 1V:3.3H	1V:3.7H to 1V:4H	1V:4H		
Side Slope Design	Soil slope supported by the Landscape Bund	Soil Slope		Reinforced		Soil Slope			
Embankment Fill Class	Class 2A4 [1]		Class 1A4 or 2A4	Class 2A4 / 7C3		Class 2A4			Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4		GLT [2]	Class 2A4		Class 2A4			GLT [2]
Subgrade DSG CBR Value (%)	2.0			2.0					
Subgrade Improvement Layer	No			No					
Pavement thickness (mm)	250			250					
Earthwork Geotextile Separator Layer	Yes			Yes					
Topsoil Strip	No [5]	Yes		No				Yes	
Starter Layer	Yes [6]	No		Yes				No	
Excavate and Replace	No			No					
Horizontal Drainage Layers	Yes [3]	No		Yes [3]	No				
Mitigation SED	G103, G104 and G105	G103	G102	G105	G104			G103	G102

Notes:

1 – The fill material highway embankment between Ch. 0+000 and 0+010 will include the granular 6N Fill used for the Overbridge design.

2 - GLT = Gault Clay Formation.

3 - Horizontal drainage layer required from Ch. 0+000 to 0+020 on the left-hand side and 0+000 to 0+360 on the right-hand side of the embankment to accelerate consolidation (15m back from overbridge abutment). Refer to standard earthwork details G105.

4 – For details of the pavement design (unbound) see Table 7-2.

5 – The topsoil strip will extend from Chainage 0+190, due to the embankment height being less than 3.00m

6 – Beyond Ch. 0+190, a starter layer is not required, due to the embankment height being less than 3.00m

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Table 8-3: Summary of Design –Footpath SBH/27 Realignment

Element	Left Hand Side							Right Hand Side	
	0+000 to 0+020	0+020 to 0+060	0+060 to 0+090	0.090 to 0+100	0+100 to 0+170	0+170 to 0+360	0+360 to 0+385	0+000 to 0+360	0+360 to 0+385
Embankment Heights (m)	8.63 – 8.73	8.73 – 6.49	6.49 – 5.07	5.07 – 4.65	4.65 – 2.85	2.85 - 0.75	<0.75	8.63 – 0.75	<0.75
Side Slope Angle	1V:3.1H	1V:3.3H	1V:3.5H	1V:3.7H	1V:4H			1V:3H	
Side Slope Design	Reinforced			Soil Slope				Soil slope supported by the Landscape Bund [4]	Soil Slope
Embankment Fill Class	Class 2A4/7C3			Class 2A4			Class 1A4 or 2A4	Class 2A4 [5]	Class 1A4 or 2A4
Subgrade DSG Assumed	Class 2A4						GLT [1]	Class 2A4	GLT [1]
Subgrade DSG CBR Value (%)	2.0							2.0	
Subgrade Improvement Layer	No							No	
Pavement thickness (mm)	250							250	
Earthwork Geotextile Separator Layer	Yes							Yes	
Topsoil Strip	No					Yes		No [6]	Yes
Starter Layer	Yes					No		Yes [7]	No
Excavate and Replace	No							No	
Horizontal Drainage Layers	Yes [2]	No						Yes [8]	No
Mitigation SED	G105	G104			G103 G104	G103	G102	G103, G104, G105	G102

Notes:
1 – GLT = Gault Clay Formation.
2 - Horizontal drainage layer required from Ch. 0+000 to 0+020 on the left-hand side of the embankment to accelerate consolidation (15m back from overbridge abutment). Refer to standard earthwork details G105.
3 – For details of the pavement design (unbound) see Table 7-2.
4 – Highway earthwork to be supported by the landscape bund from the edge of the abutment to Ch. 0+225, beyond which the right-hand side is a unsupported soil slope.
5 – The fill material highway embankment between Ch. 0+000 and 0+010 will include the granular 6N Fill used for the Overbridge design.
6 – Beyond Ch. 0+170, a topsoil strip is required due to the embankment height being less than 3.00m.
7 - Beyond Ch. 0+170, a starter layer is not required due to the embankment height being less than 3.00m.
8 – A drainage layer is required between Ch. 0+000 and 0+360 on the right-hand side of the embankment to accelerate consolidation. Refer to standard earthwork details G105.

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9 Reuse of Material

A summary of the material likely to be produced on site:

- A nominal 300mm topsoil strip is anticipated for the entire highway alignment, to be reused as Class 5.
- In areas of minor cutting where Head is encountered, it will be recovered and is assumed to be suitable for Class 4 only, based on anticipated high water contents for near-surface materials.
- Made Ground if encountered shall be stockpiled and tested to determine its suitability for re-use in the Design.

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10 Specification

Please refer to document No.1MC06-CEK-AU-SPE-CS04_CL18-000003 for the IDR Package 207.2 Specification.

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11 Proposed Testing and Verification Plan

11.1 Instrumentation and monitoring

Instrumentation and monitoring (I&M) for this asset is described in the Aylesbury Embankment Designers Monitoring Plan (1MC06-CEK-GT-REP-CS04_CL18-000019). I&M is proposed for the approach embankments as per the typical array for back of highway overbridges (1MC06-CEK-CV-DGA-CS04_CL18-000203). A summary of the inputs to the DMP are provided in Appendix F.

11.2 Inspection methods/controls

Inspection of the formation at time of construction should be undertaken by a qualified Geotechnical Engineer/ Engineering Geologist to identify any soft spots.

Identified soft spots shall be excavated and replaced (E&R), in accordance with the SED G106 (1MC06-CEK-GT-DSE-C002-000106) and the IDR 207.2 Specification, with Class 1A4 or 2A6 material to achieve the minimum shear strength/ CBR.

11.3 Site testing

In-situ tests are required to be undertaken at formation level during construction to confirm the strength of subgrade meets the minimum specified stiffness/ undrained shear strength/ CBR as per the design requirements.

Pre-construction in-situ testing (MEXECone or similar) should be undertaken to confirm the properties of the near surface soils relevant to the design of at-grade pavements, due to the lack of testing data in the upper 2m relevant to pavement design.

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12 Risks

12.1.1 Geotechnical Risk Register

This calculation pack should be read in conjunction with Geotechnical Risk Register (GRR). The GRR is a live GIS database that is under ongoing review and update throughout the life of the project.

A snapshot of the GRR at the time of producing this document is included in Appendix D of the GDR. The following key risks have been identified:

- Soft / compressible ground potentially leading to excessive settlement between the interface of the highway embankment and the overbridge.
- Chemically aggressive ground.
- High shrink / swell potential.
- Variable weathering profile.
- Geological faulting.
- Relict periglacial features.
- High / shallow groundwater.

For a full account of the geotechnical risks refer to the IDR Specific GRR (1MC06-CEK-GT-REG-CS04_CL18-000006)

12.1.2 CDM Risk Register

This calculation pack should be read in conjunction with CDM Risk Register. This register is a live GIS database that is under ongoing review and update throughout the life of the project.

The following key risks have been identified:

- The high plasticity clay used for embankment construction.
- Presence of low or residual strength shear surfaces.
- Presence of soft / loose ground.
- Presence of high / shallow groundwater

For a full account of the CDM risks refer to the IDR Specific CDM Risk Register (1MC06-CEK-AU-REG-CS04_CL18-000043).

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13 Assurance Input

13.1.1 Assumptions

Key assumptions used for the design:

- Recovered Gault Clay Formation will be used for the highway earthworks as Class 2A4/7C3 material.
- A maximum permanent surcharge of 5 kPa has been assumed for long term 'Drained' case stability calculations up to the crest of the embankment for the Oat Close Bridleway SBH/19 Realignment.
- Thin pavement, average construction conditions and high-water table conditions during construction have been assumed for the design of pavements. This means that the exposure and construction of the bridleway / footpath formations should exclude working in adverse weather or wet conditions and preferably be undertaken in the summer months.
- The construction of the highway earthworks and landscaping will be constructed simultaneously or within a short period of time of each other (i.e., <2yrs), such that the vadose zone and fully drained conditions have not developed at the interface of both earthworks.

For assumptions that need further verification refer to the IDR Specific Assumptions Register (1MC06-CEK-AU-REG-CS04_CL18-000059).

13.1.2 Departures

A departure from the standards, the following have been accepted by HS2:

- DJV-DEP-000543 – Departure to clarify the requirements for checking category for highway assets. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000006 which has been accepted by HS2.
- DJV-DEP-000499 – Relaxation of Upper Limits on Liquid Limits and Plasticity Index for 7C selected wet cohesive material for use as fill to reinforced soil as set out in the HS2 SCEW. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000003 which has been accepted by HS2.
- DJV-DEP-000466 – Relaxation of Grading Limits for 7C selected wet cohesive material for use as fill to reinforced soil as set out in the HS2 SCEW. Departure submitted via IAF 1MC06-CEK-GT-FRM-C002-000004 which has been accepted by HS2.

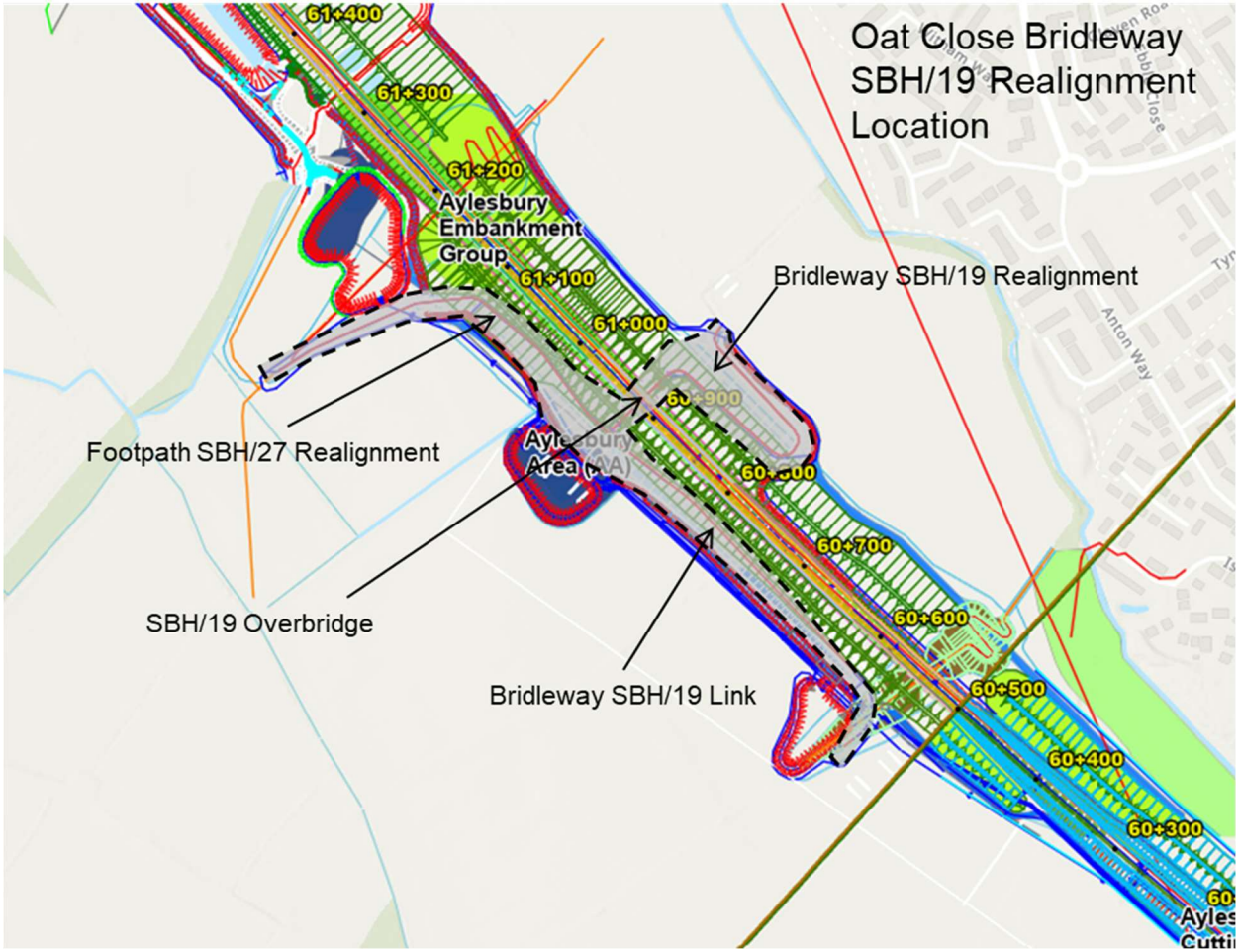
For a full account of the departures refer to the C23 Route wide Master Departures Register (1MC12-EKF-AU-REG-C000-000019)

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APPENDIX A : Maps and Drawings

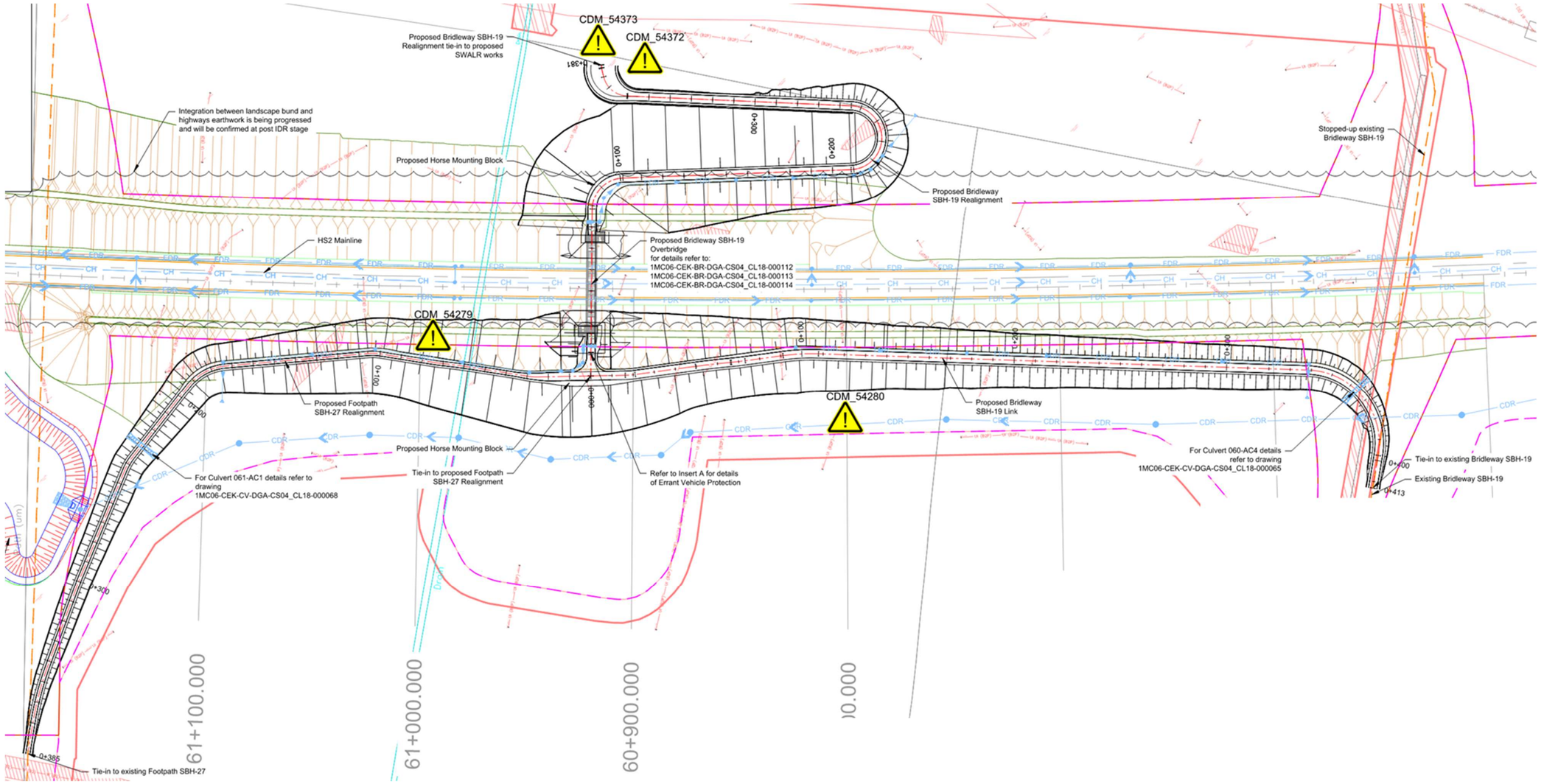
No of pages: 11

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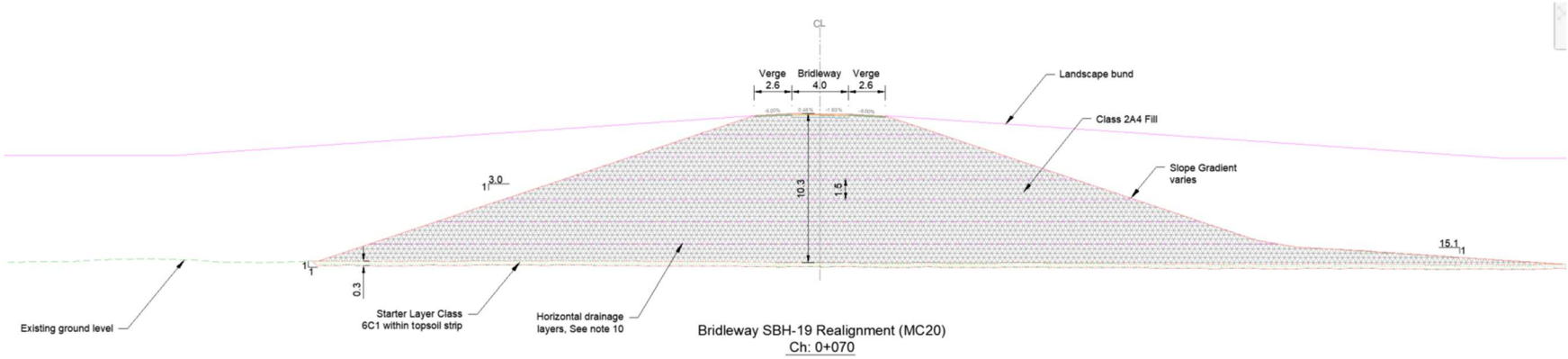
General Site Location Plan – New asset location shown in red

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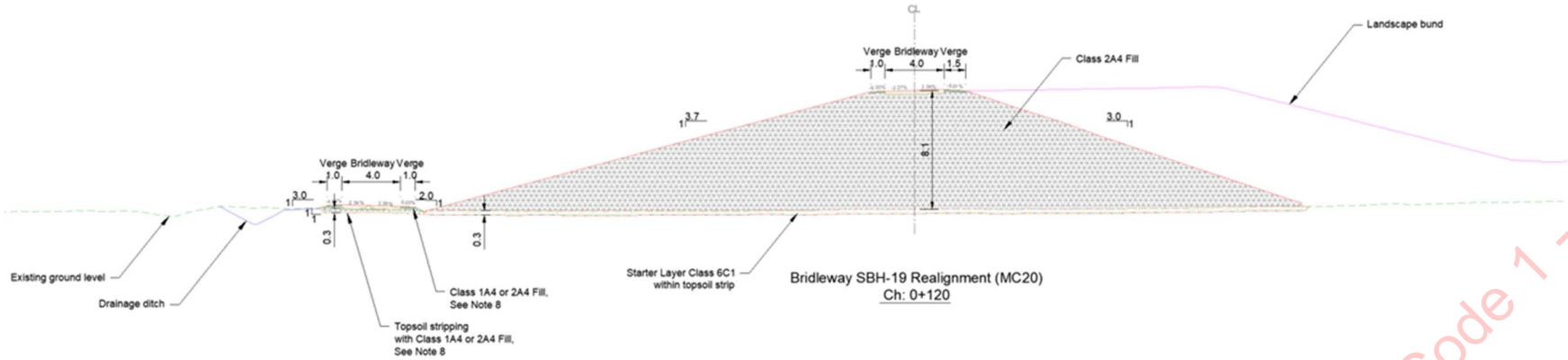


Highway Earthworks General Arrangement Plan – Oat Close Bridleway SBH/19 Realignment

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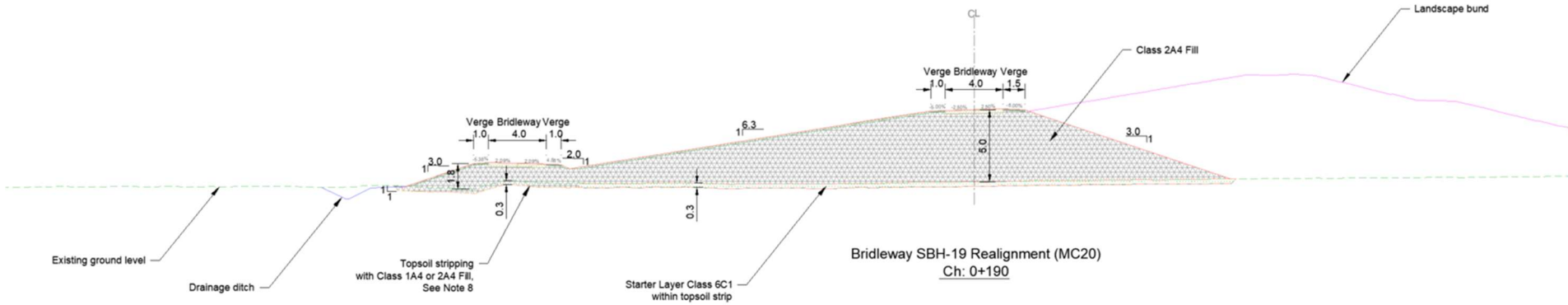


Highway Earthworks Cross Section DD – Bridleway SBH/19 Realignment at Ch. 0+070

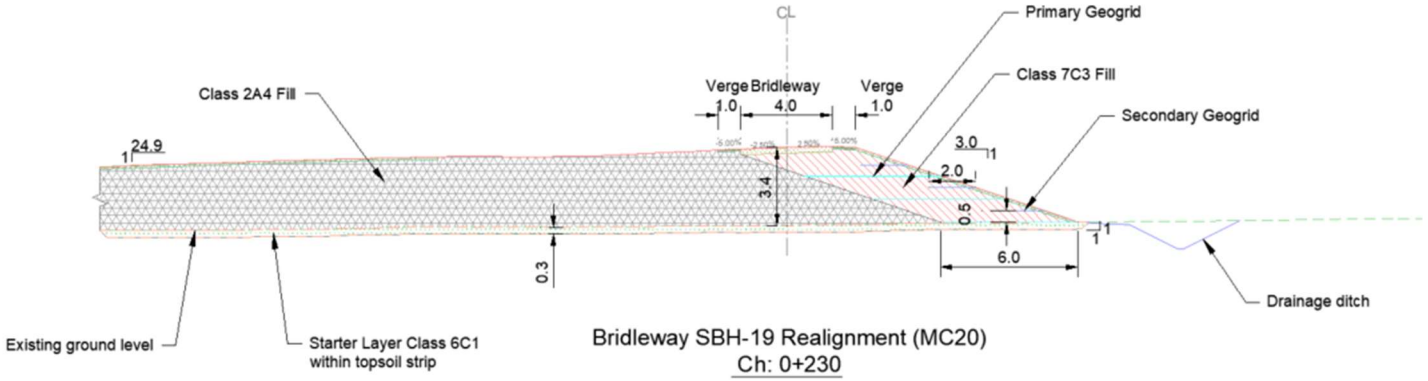


Highway Earthworks Cross Section AA – Bridleway SBH/19 Realignment at Ch. 0+120

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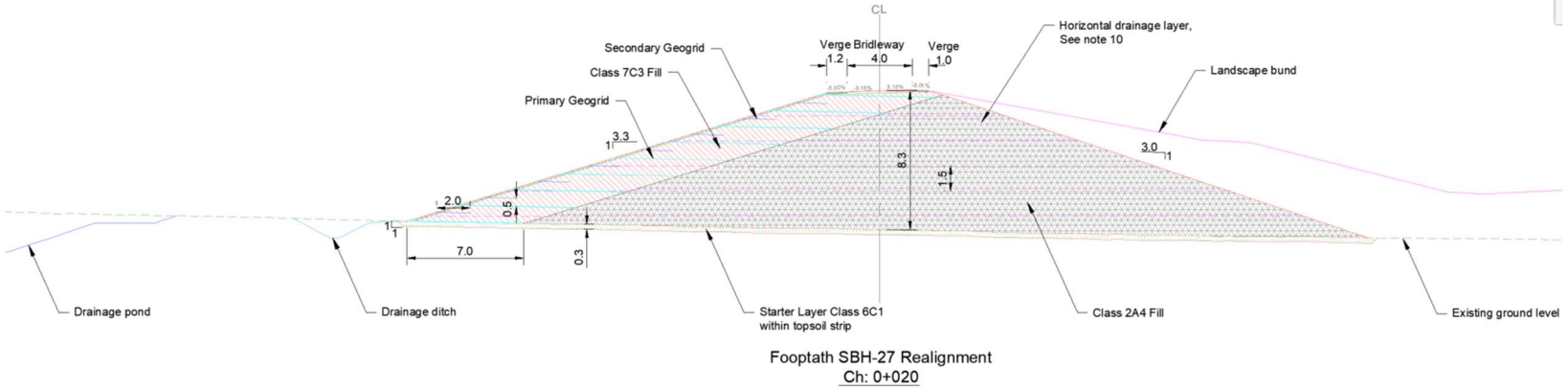


Highway Earthworks Cross Section BB – Bridleway SBH/19 Realignment at Ch. 0+190

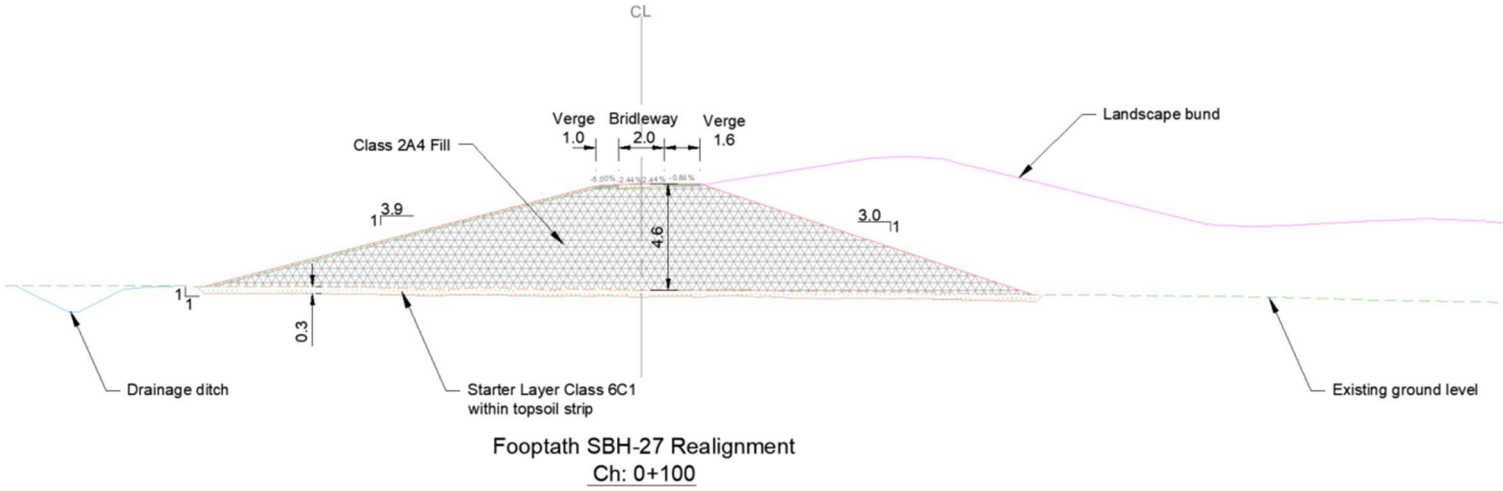


Highway Earthworks Cross Section CC – Bridleway SBH/19 Realignment at Ch. 0+230

HS2 Ltd - Code 1 - Accepted

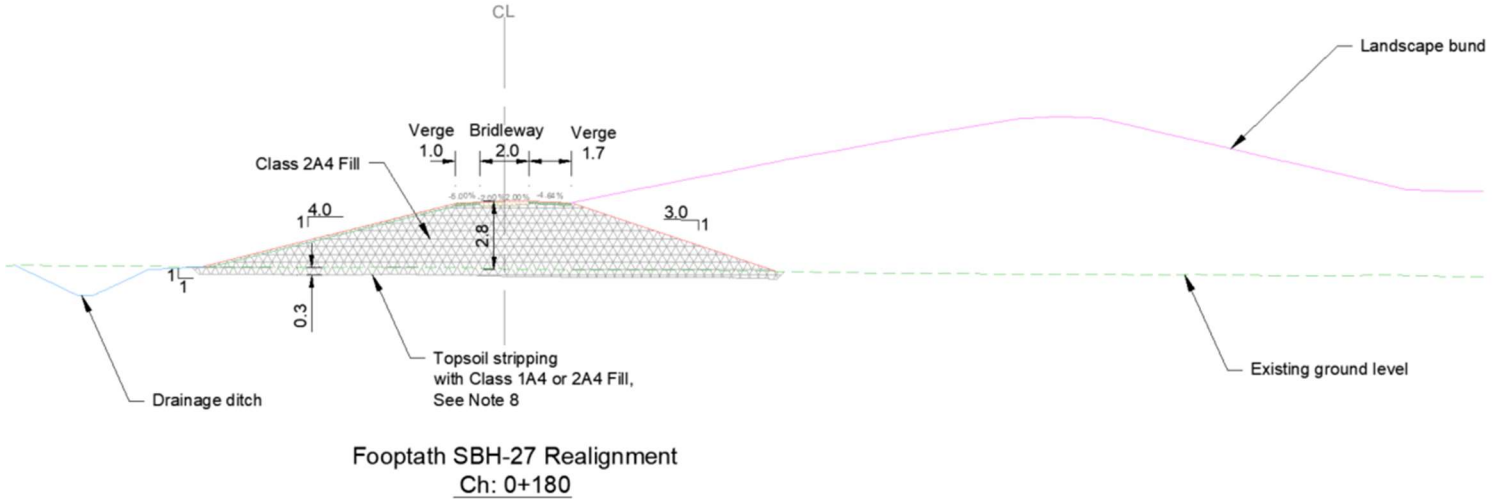


Highway Earthworks Cross Section HH – Footpath SBH/27 Realignment at Ch. 0+020

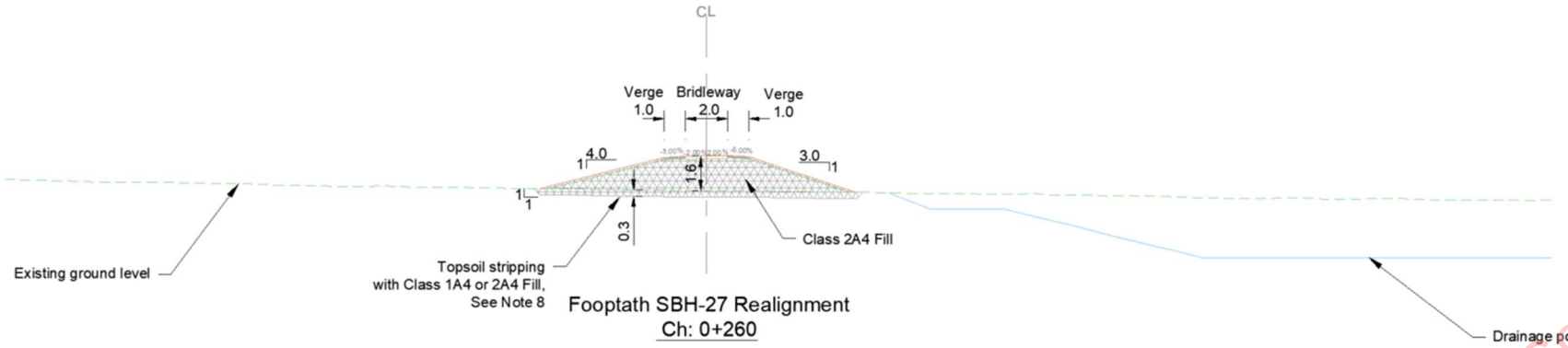


Highway Earthworks Cross Section GG – Footpath SBH/27 Realignment at Ch. 0+100

HS2 Ltd - Code 1 - Accepted

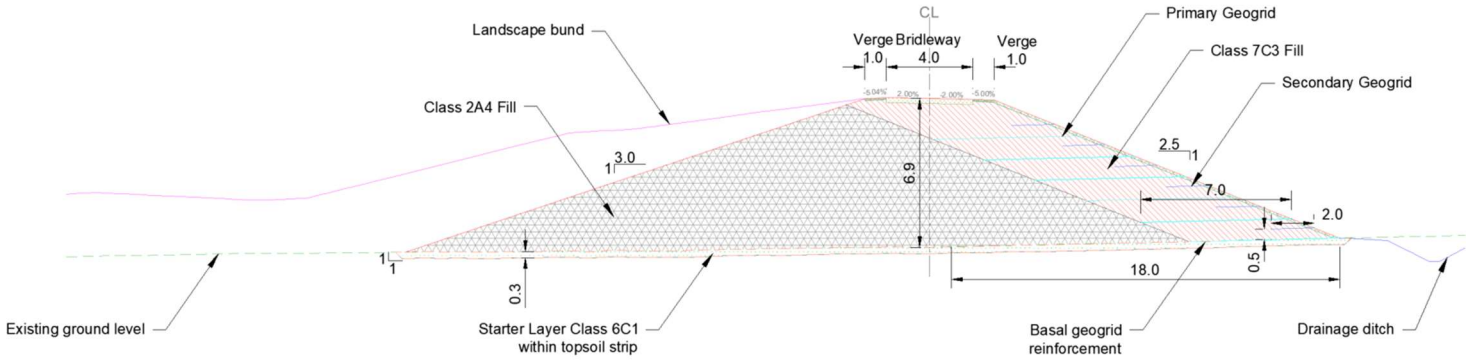


Highway Earthworks Cross Section FF – Footpath SBH/27 Realignment at Ch. 0+180



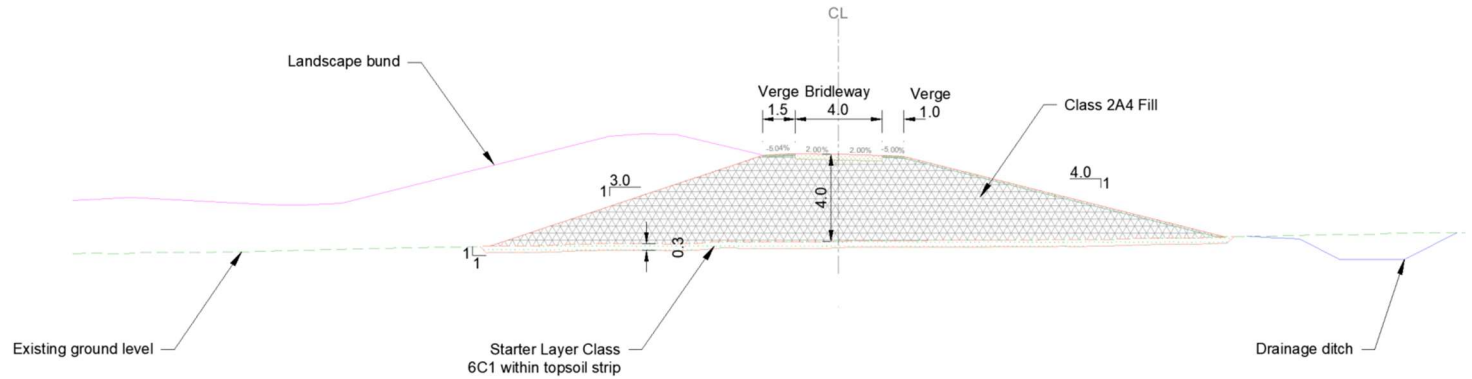
Highway Earthworks Cross Section EE –Footpath SBH/27 Realignment at Ch. 0+260

HS2 Ltd - Code 1 - Accepted



Bridleway SBH-19 Link (MC10)
Ch: 0+050

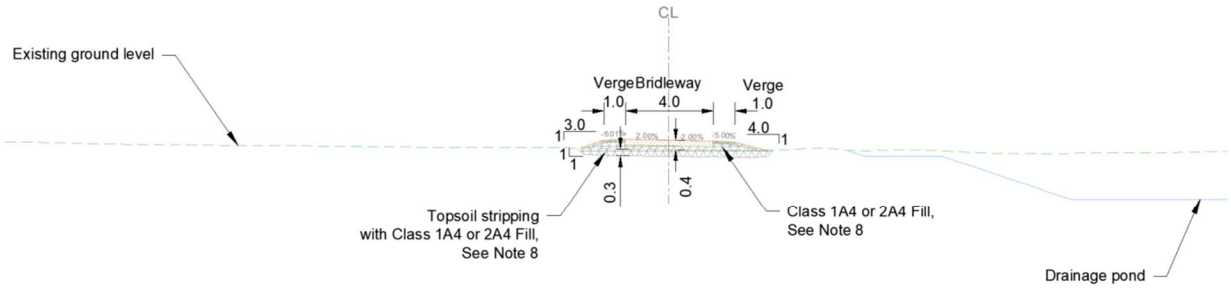
Highway Earthworks Cross Section II – Bridleway SBH/19 Link at Ch. 0+050



Bridleway SBH-19 Link (MC10)
Ch: 0+110

Highway Earthworks Cross Section JJ – Bridleway SBH/19 Link at Ch. 0+110

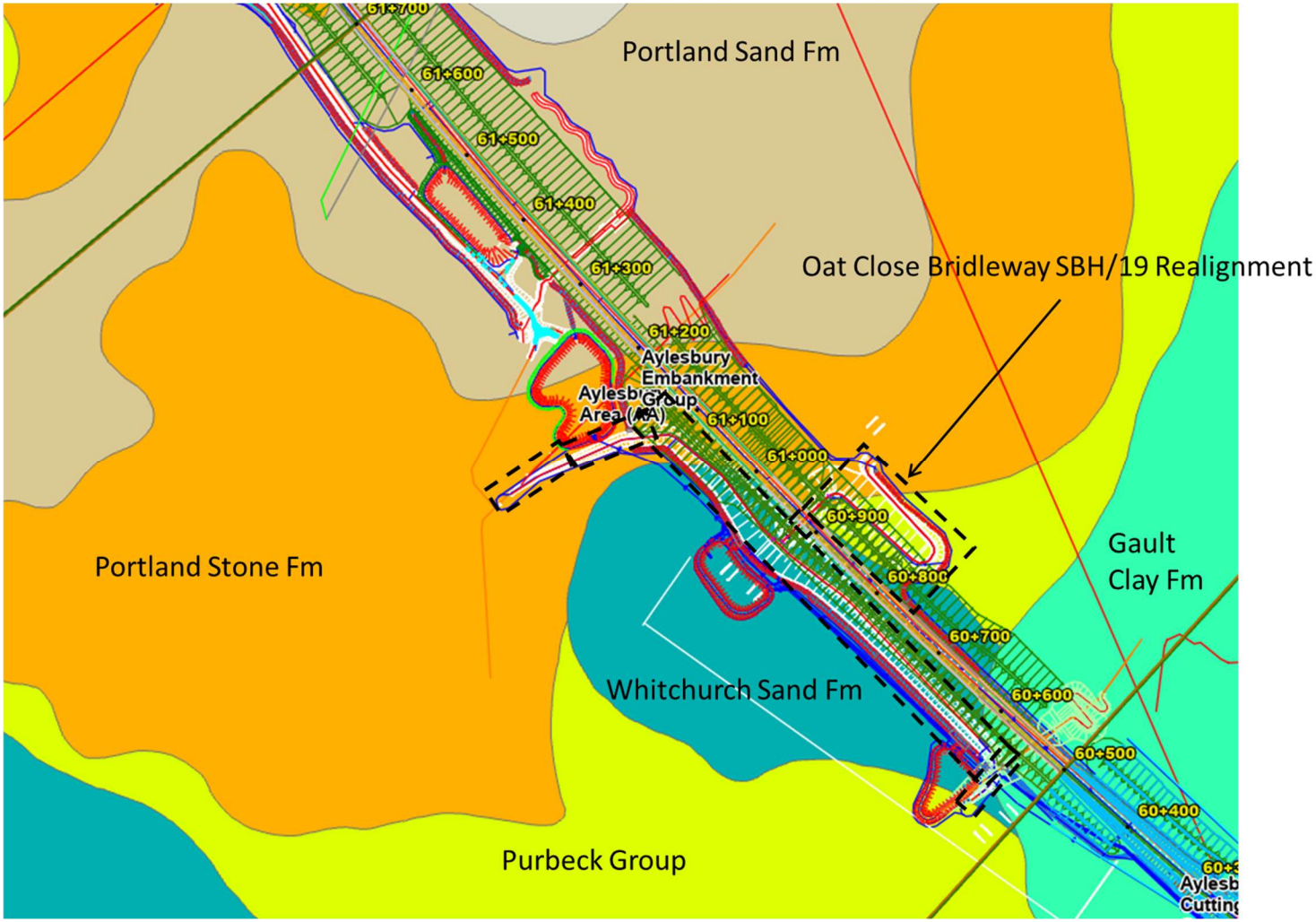
HS2 Ltd - Code 1 - Accepted



Bridleway SBH-19 Link (MC10)
Ch: 0+400

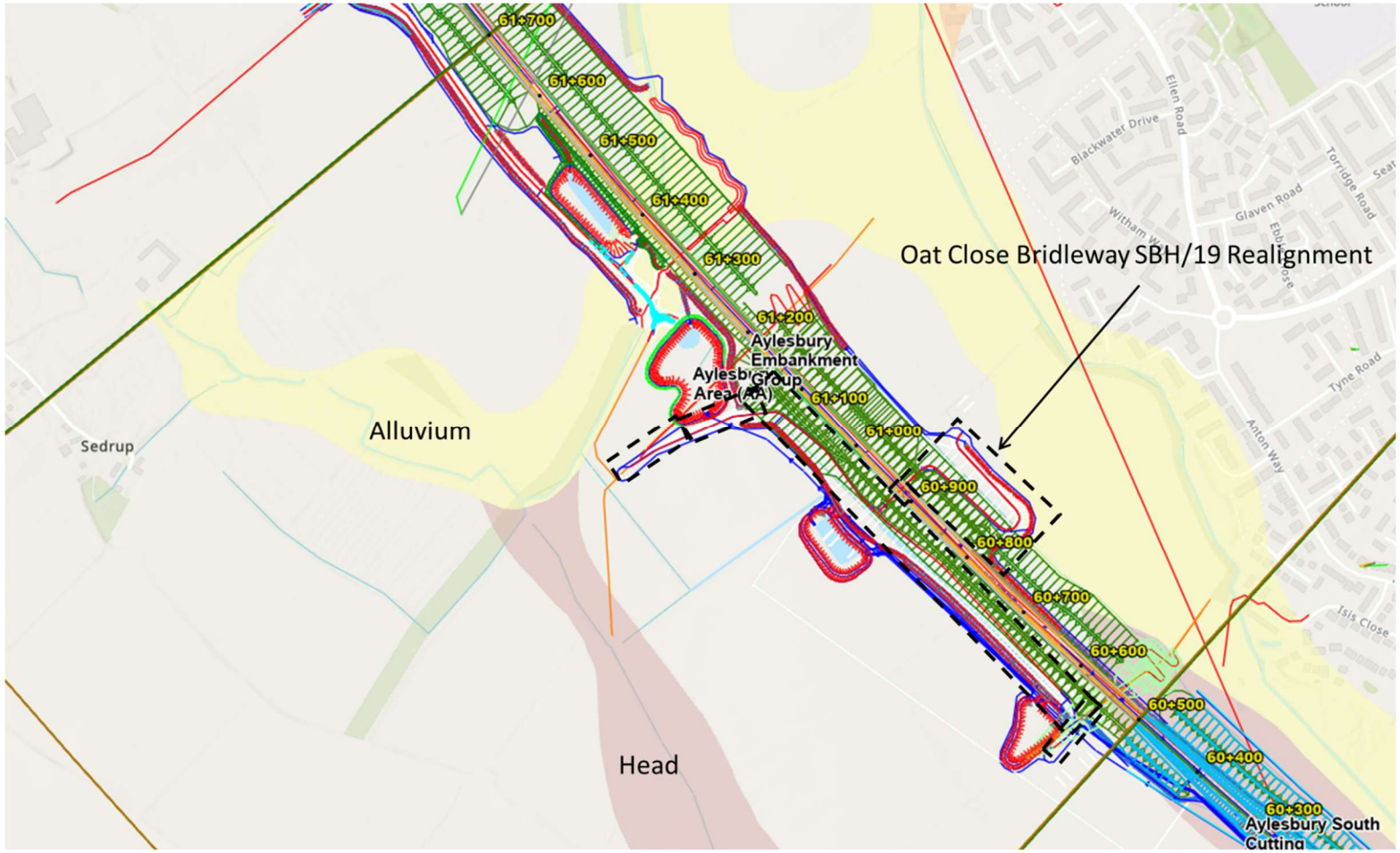
Highway Earthworks Cross Section KK – Bridleway SBH/19 Link at Ch. 0+400

HS2 Ltd - Code 1 - Accepted



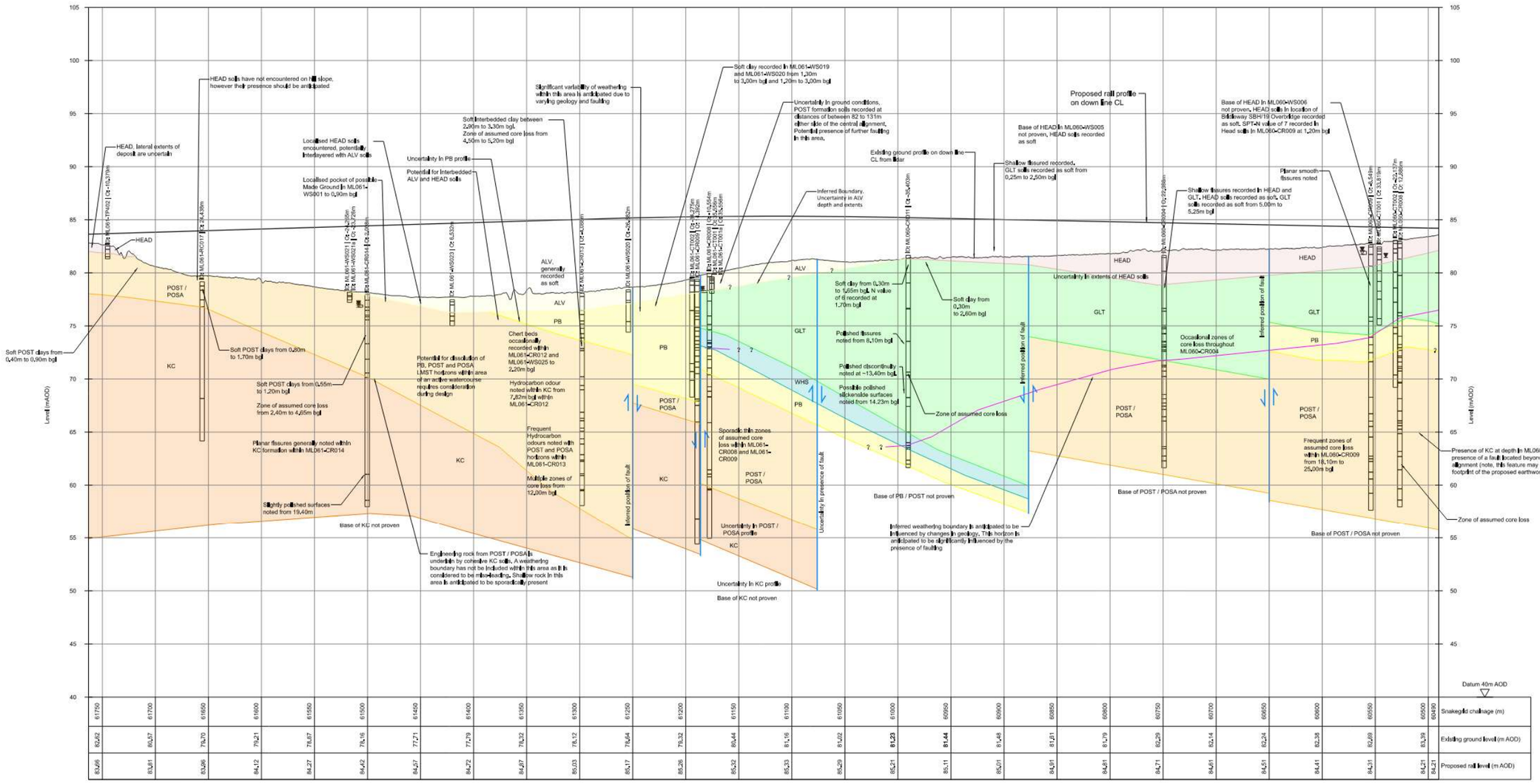
Published Bedrock Geology Map

HS2 Ltd - Code 1 - Accepted



Published Superficial geology Map

HS2 Ltd - Code 1 - Accepted



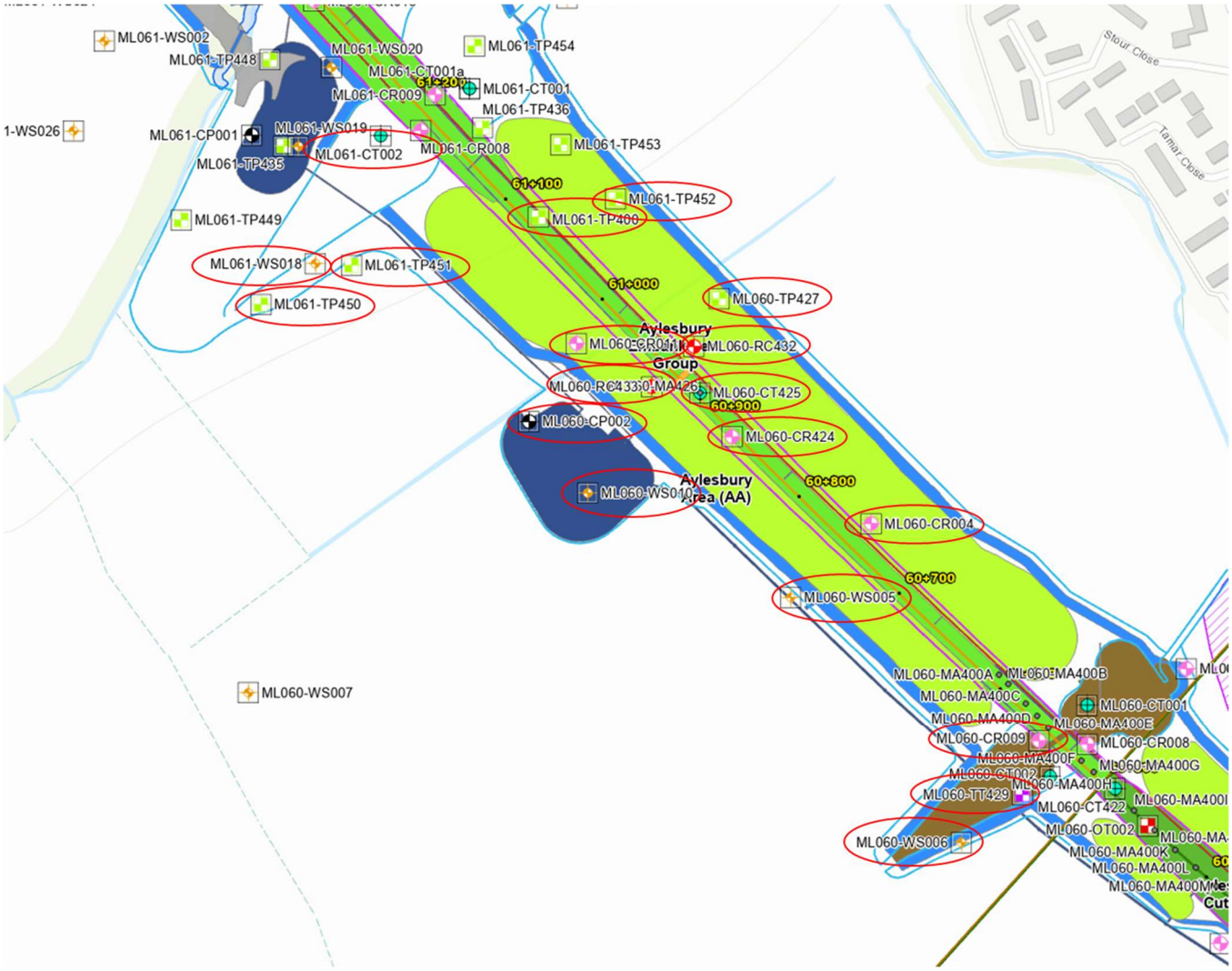
Aylesbury Embankment Long Section extracted from GIR

HS2 Ltd - Code 1 - Accepted

APPENDIX B : Ground Investigation Information

Number of Pages: 2

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GI Location Plan

HS2 Ltd - Code 1 - Accepted

Completed Exploratory Holes

Exploratory Hole ID	Type	Depth (mBGL)	Ground Level AGS (mOD)	Approx. Distance from Asset (m)	Easting	Northing
ML060-CP002	CP	10.00	82.62	50.00	480886.6	211401
ML060-CR004	CP+RC	20.00	81.63	30.00	481133.6	211331.7
ML060-CR009	CP+RC	25.00	82.62	195.00	481256.6	211177.5
ML060-CR011	CP+RC	20.00	81.64	0.00	480919.7	211457.4
ML060-CR424	CP+RC	25.05	81.53	20.00 (E) and 15.00 (W)	481032.7	211392.5
ML060-CT425	CPT	7.24	81.44	20.00 (E) and 20.00 (W)	481009.1	211423.5
ML060-RC432	RC	25.00	80.95	<10.00	481004.1	211456.7
ML060-RC433	RC	25.45	81.58	<10.00	480974.4	211427.4
ML060-TP427	TP	2.00	80.4	0.00	481021.7	211491.3
ML060-TT429	TP	3.00	83.32	220.00	481245.2	211139
ML060-WS005	WLS	4.00	83.24	<10.00	481076.6	211277.8
ML060-WS006	WLS	3.00	83.98	220.00	481201.8	211104.2
ML060-WS010	WLS	4.00	83.44	45.00	480929.7	211350.4
ML061-TP400	TP	4.00	81.09	<10.00	480891.1	211547.7
ML061-TP450	TP	3.00	79.26	120.00	480692.8	211481.9
ML061-TP451	TP	1.50	80.32	45.00	480757.5	211511.1
ML061-TP452	TP	1.50	80.85	95.00	480946.3	211561.6
ML061-WS018	WS	4.00	79.78	75.00	480731.3	211511.8

HS2 Ltd - Code 1 - Accepted

APPENDIX C : Selected Data Plots

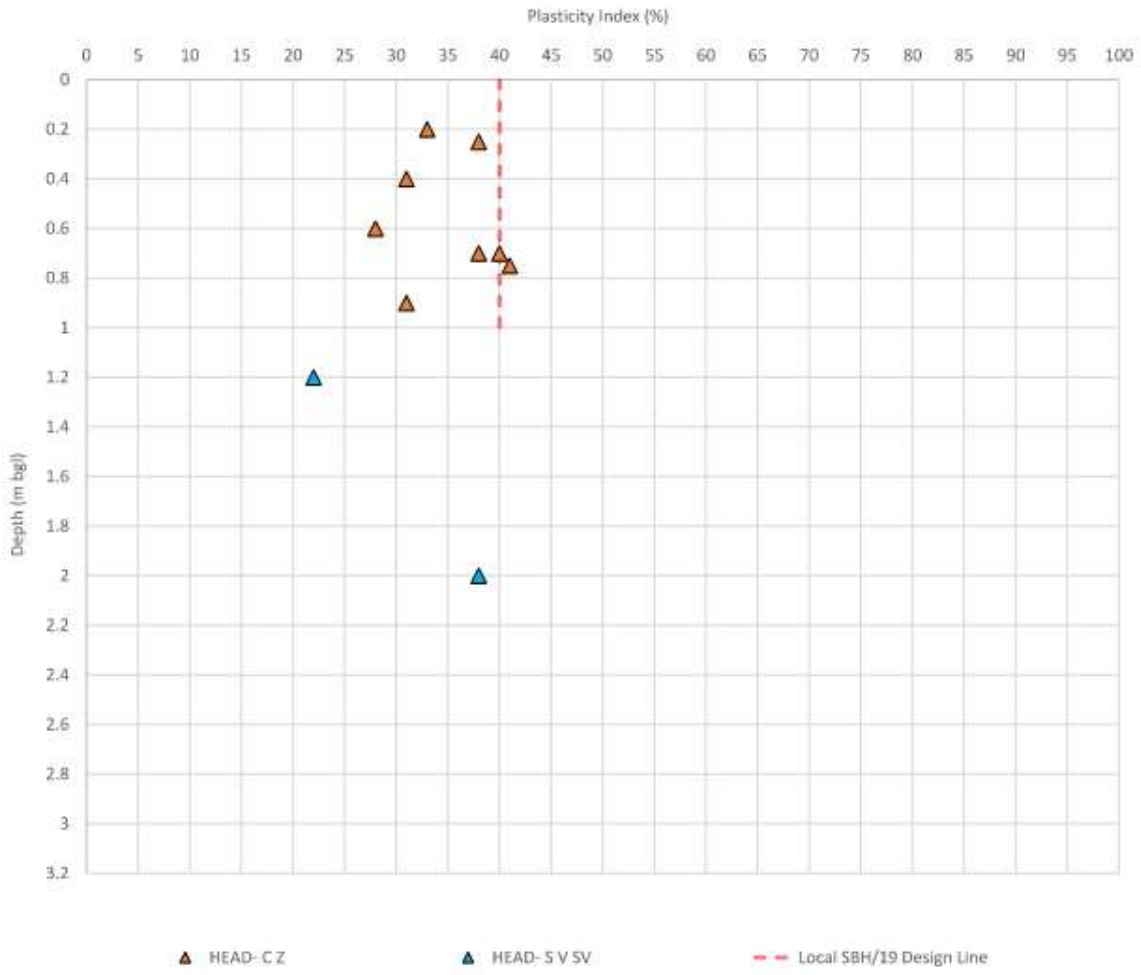
Number of Pages: 4

HS2 Ltd - Code 1 - Accepted

HS2

Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 -MWCC – North Portal of Chiltern Tunnels to Brackley	Location:	Oat Close Bridleway SBH/19 Realignment
Client:	HS2	Geology :	HEAD	Figure:	A1



Number of test	14	Notes Number of test value denotes number of Atterberg Limits tests completed only. Background C2 scheme wide parameters shown are based on OpenGround database geology codes dated 10/09/2021.	Plot Date:	27/11/2023
			Database reference	C2-C3-CONT-ASC

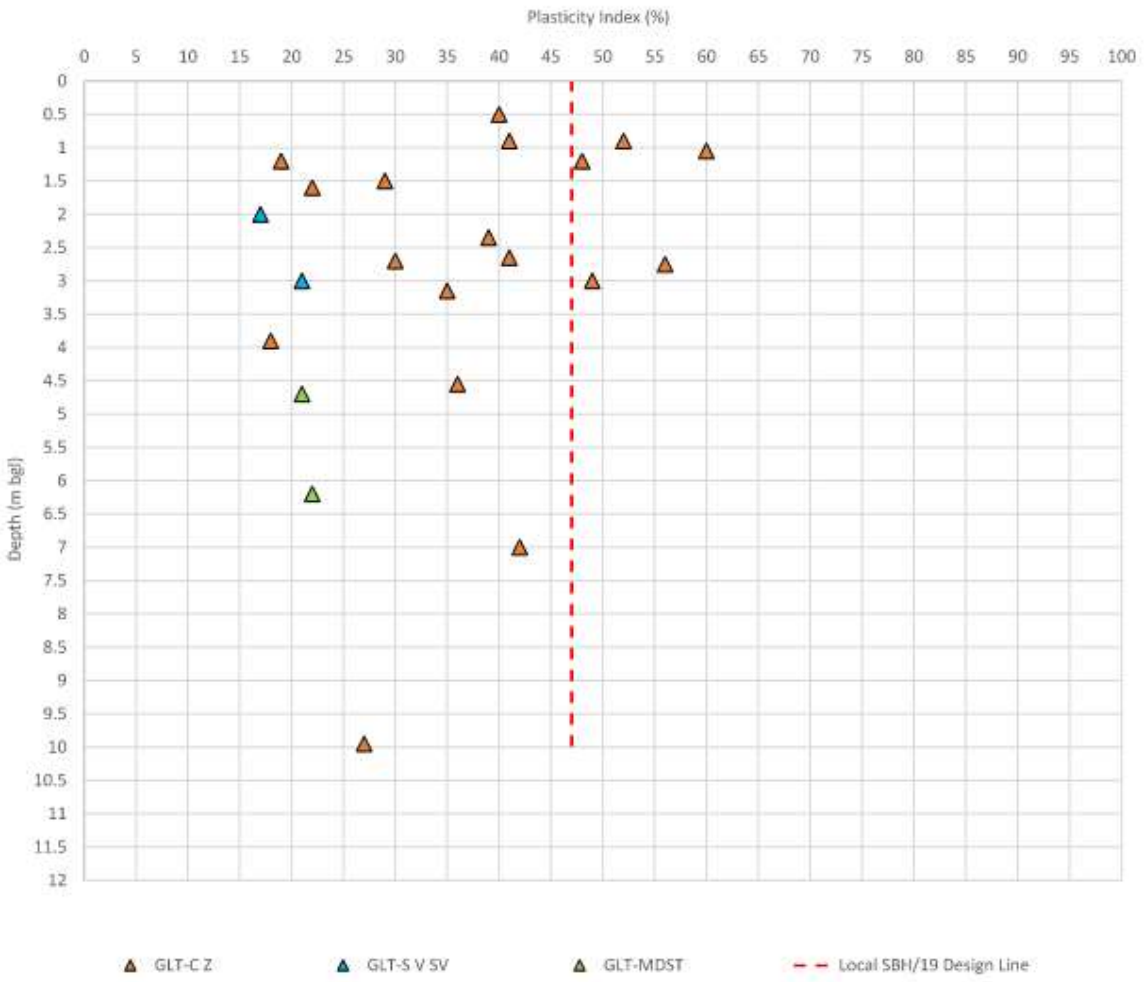
Local Data Plot for Head - Plasticity Index vs Depth (m bgl)

HS2 Ltd - Code 1 - Accepted

HS2

Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 -MWCC – North Portal of Chiltern Tunnels to Brackley	Location:	Oat Close Bridleway SBH/19 Realignment
Client:	HS2	Geology :	GAULT CLAY FORMATION	Figure:	A2



Number of test	22

Notes
Number of test value denotes number of Atterberg Limits tests completed only.

Background C2 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

Plot Date:	27/11/2023
Database reference	C2-C3-CONT-ASC

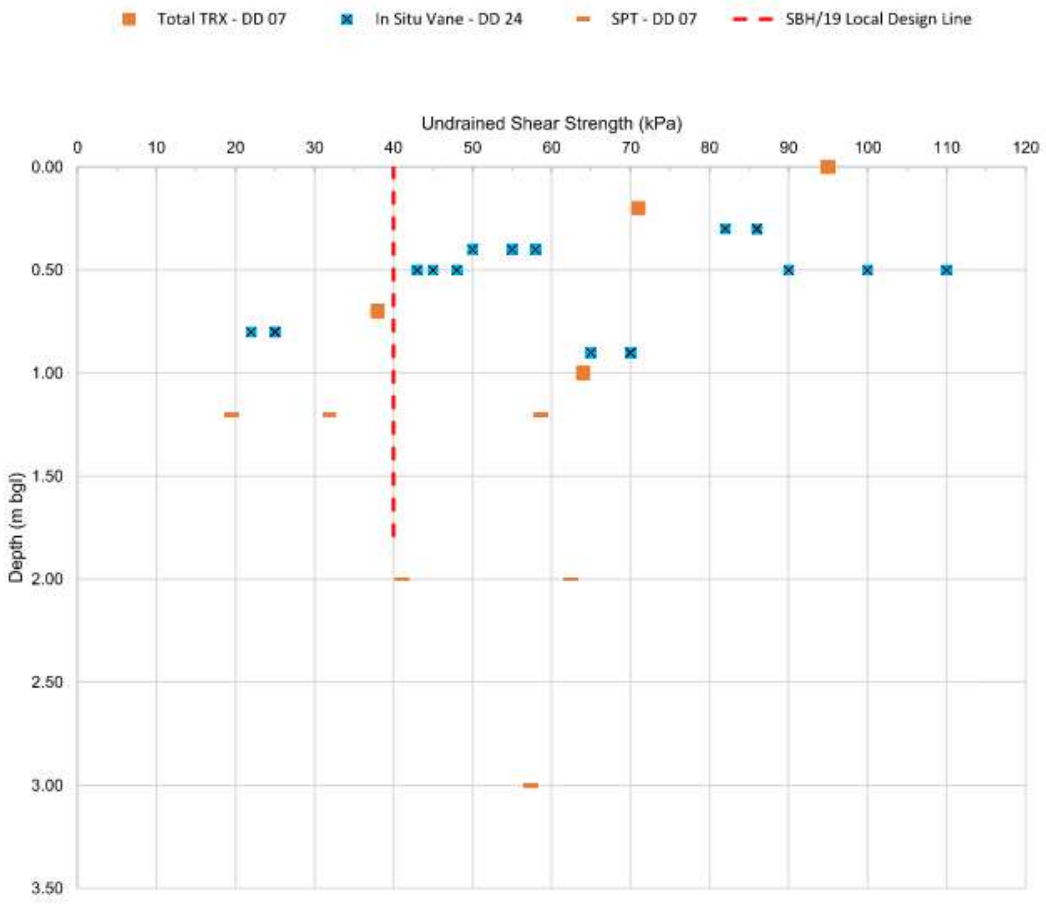
Local Data Plot for the Gault Clay Formation - Plasticity Index vs Depth (m bgl)

HS2 Ltd - Code 1 - Accepted

HS2

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 -MWCC – North Portal of Chiltern Tunnels to Brackley	Location:	Oat Close Bridleway SBH/19 Realignment
Client:	HS2	Geology :	HEAD-C Z	Figure:	B1



	Total
Number of Tests	73
Minimum	20
Maximum	110

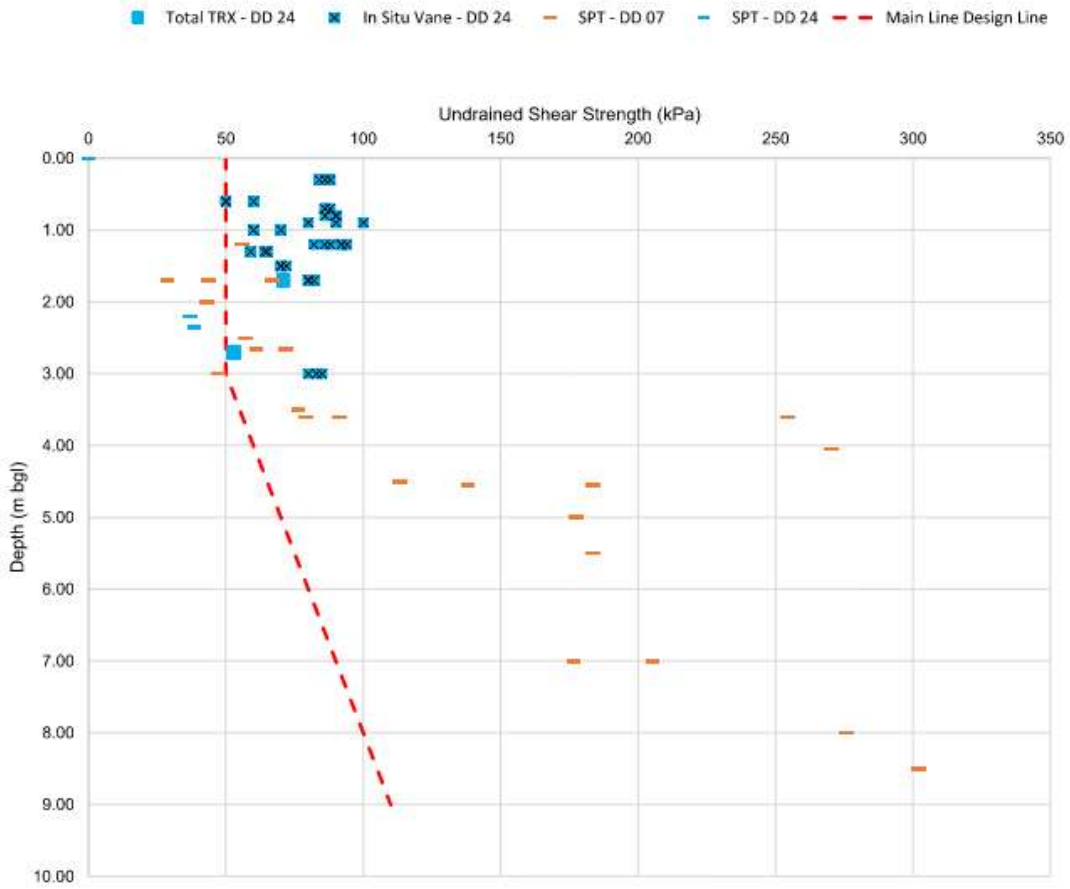
Notes
Background C2 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

Plot Date:	27/11/2023
Database reference	C2-C3-CONT-ASC

Local Data Plot for Head - Undrained Shear Strength vs Depth (m bgl)

HS2 Ltd - Code 1 - Accepted

HS2	Undrained Shear Strength vs. Depth (m bgl)				
Project ID:	10039367	Project Title:	1MC06 - Stage One C2 -MWCC – North Portal of Chiltern Tunnels to Brackley	Location:	Oat Close Bridleway SBH/19 Realignment
Client:	HS2	Geology :	GLT-C Z	Figure:	B2



	Total
Number of Tests	192
Minimum	0
Maximum	302

Notes
Background C2 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

Plot Date:	27/11/2023
Database reference	C2-C3-CONT-ASC

Local Data Plot for the Gault Clay Formation - Undrained Shear Strength vs Depth (m bgl)

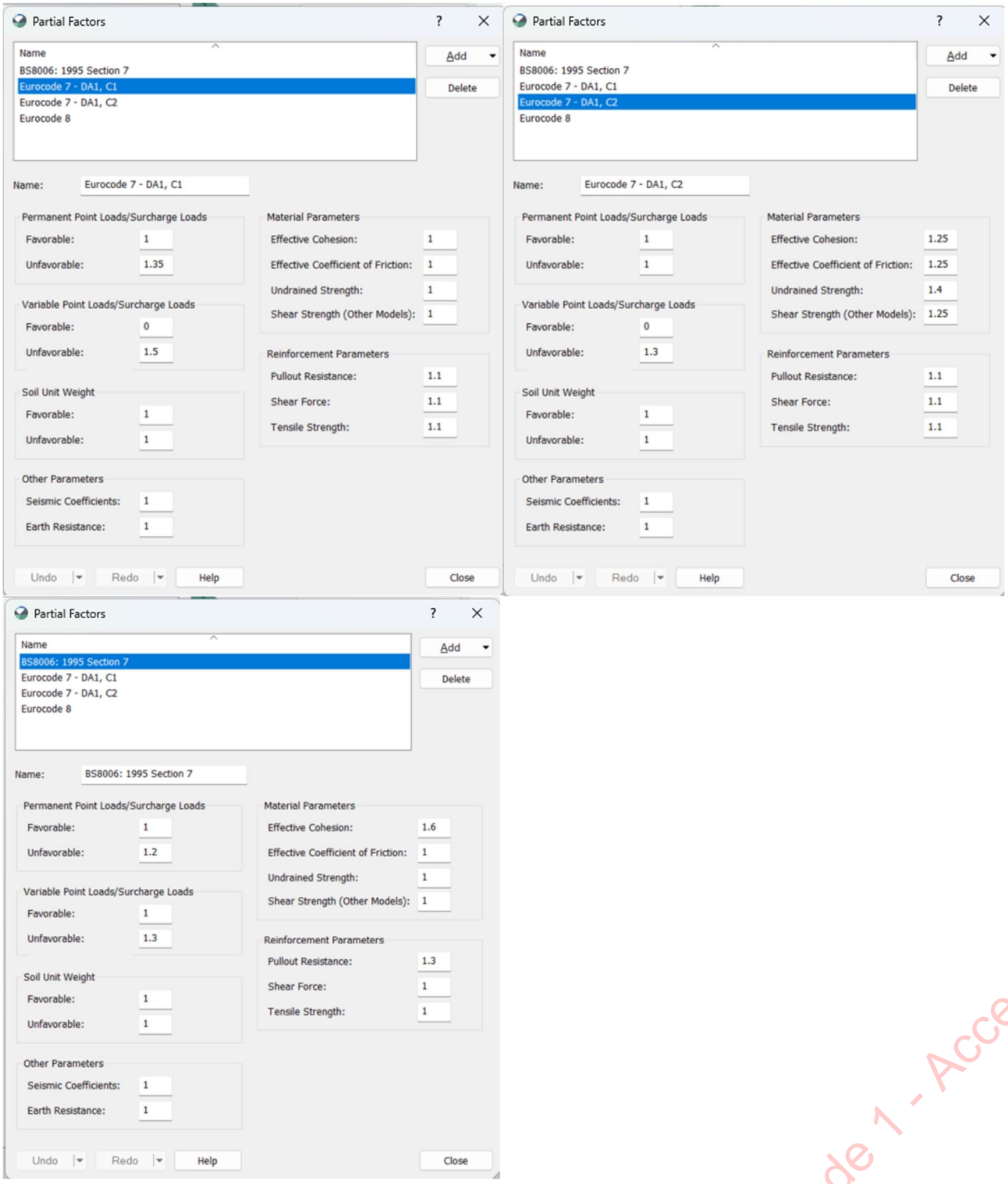
HS2 Ltd - Code 1 - Accepted

APPENDIX D : Design Summary Outputs for Slope Stability and Settlement

Number of Pages:25

HS2 Ltd - Code 1 - Accepted

Slope Stability Partial Factors



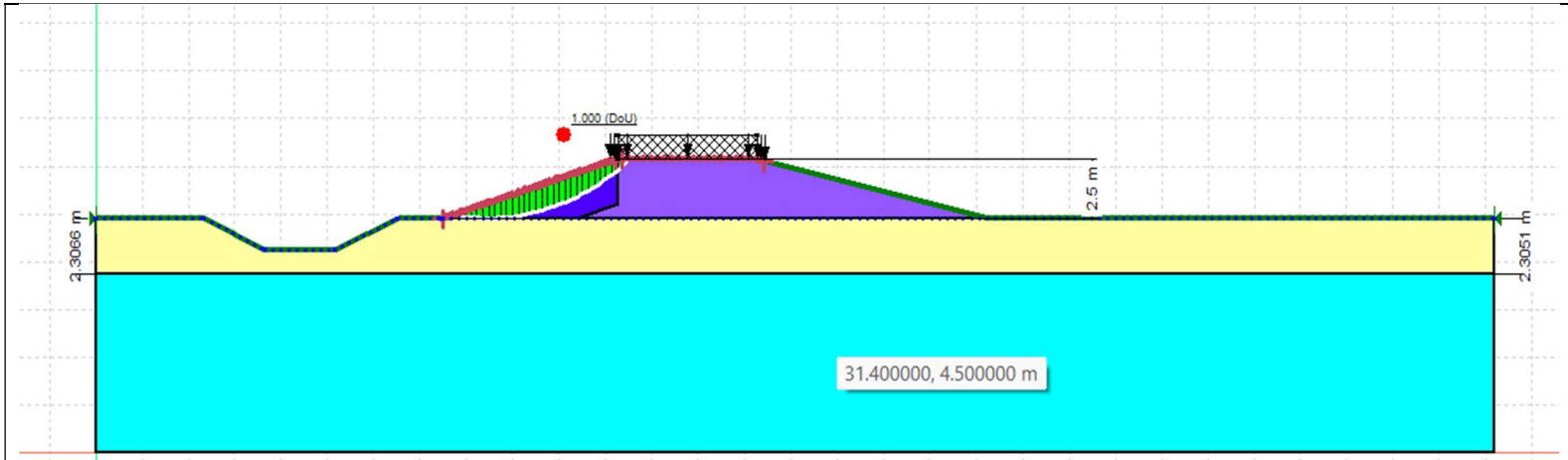
HS2 Ltd - Code 1 - Accepted

Note:

The selected Slope/W results for the reinforcement analysis show results for DA1 C2 high strength analysis. The analysis for the BS8006 condition tests the stability of the geogrid reinforcement, whereas the DA1 C2 condition forces a global failure of the embankment beyond the extent of the geogrid, thus assuming no failure is occurring in the high strength geogrid.

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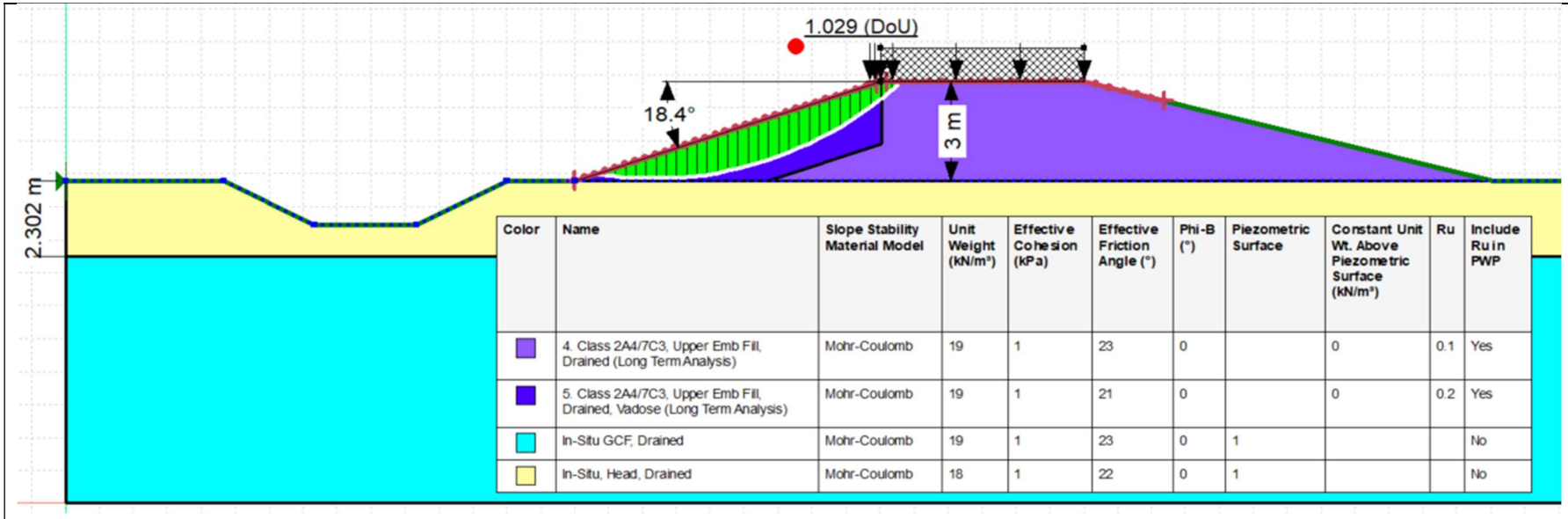
Selected Results



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No.2.: Embankment Height 2.0m Drained conditions EC7 - DA1C2 Unsupported DoU=1.000

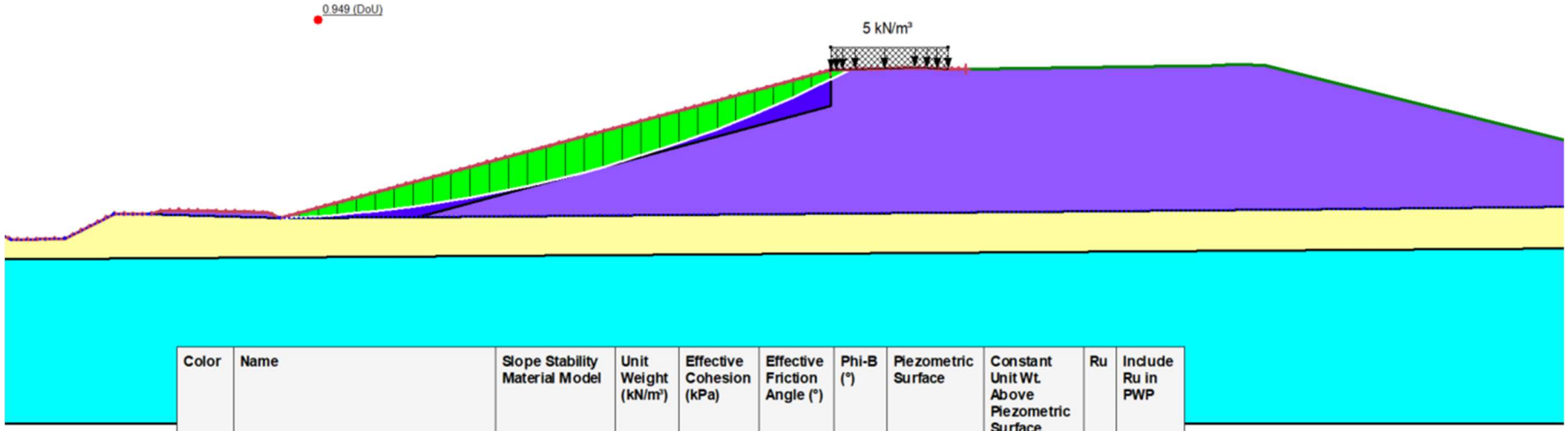
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No.4.: Embankment Height 3.0m Drained conditions EC7 - DA1C2 Unsupported DoU=1.029

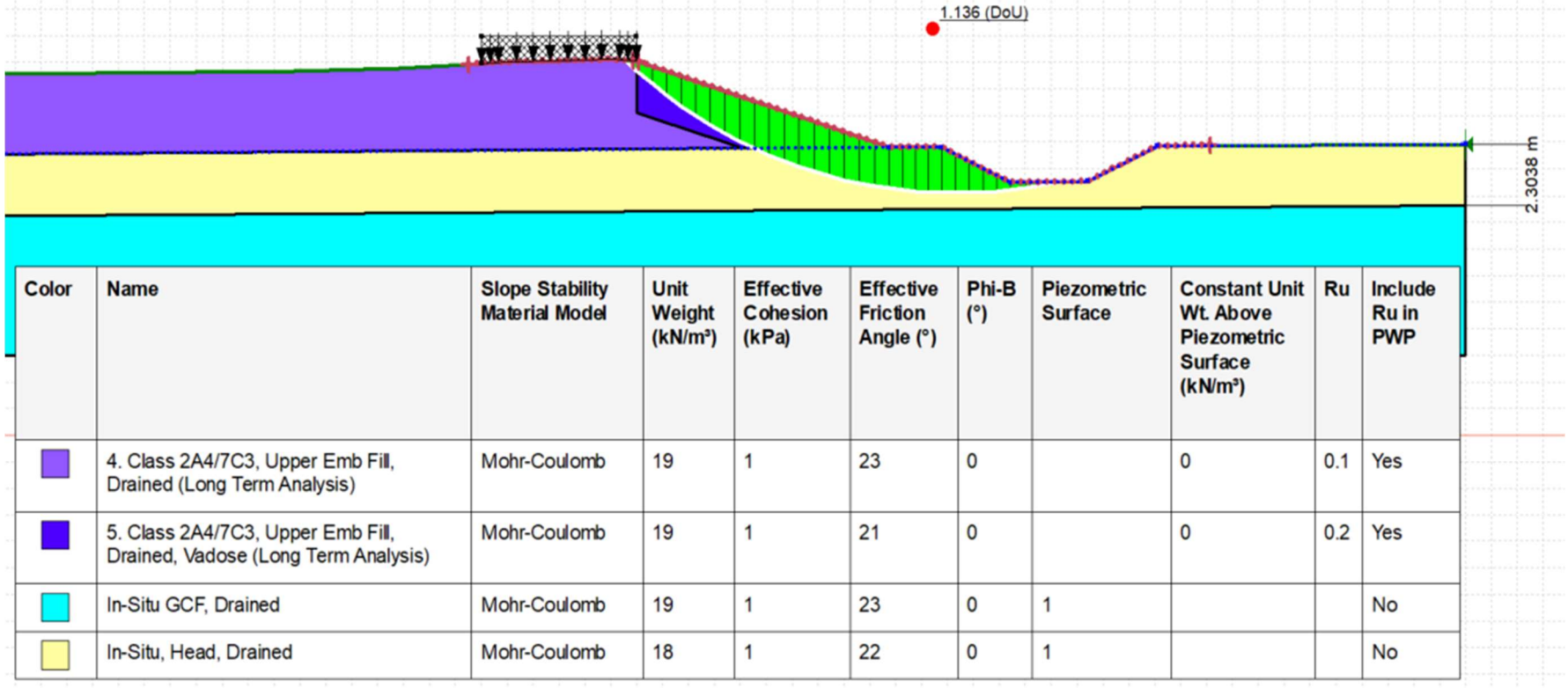
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No.6.: Embankment Height 8.1m Drained conditions EC7- DA1C2 Unsupported DoU=0.949

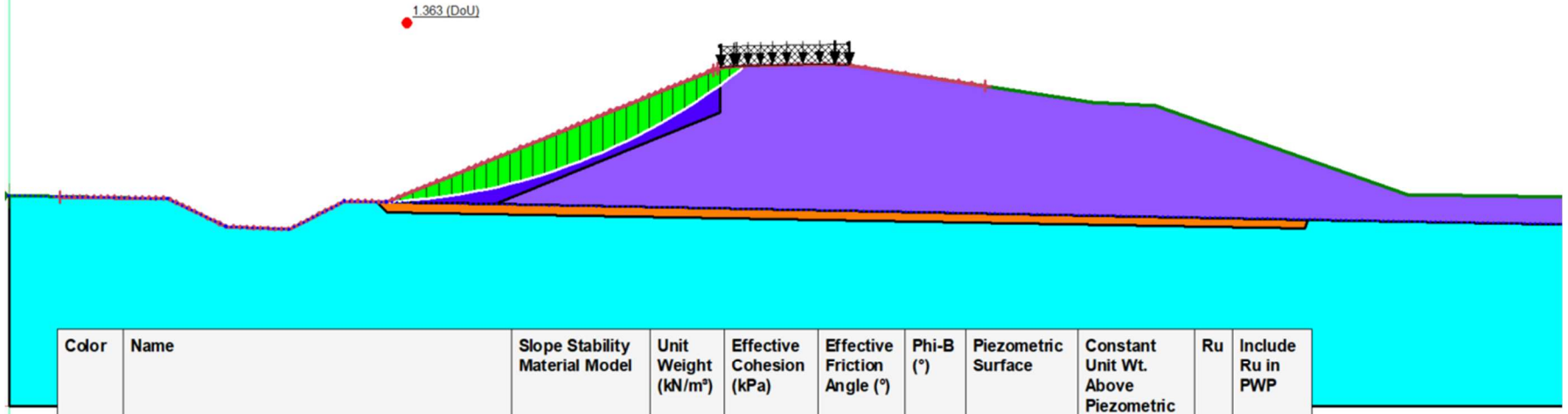
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No 8.: Embankment Height 3.24 Drained conditions EC7 - DA1C2 Unsupported DoU=1.136

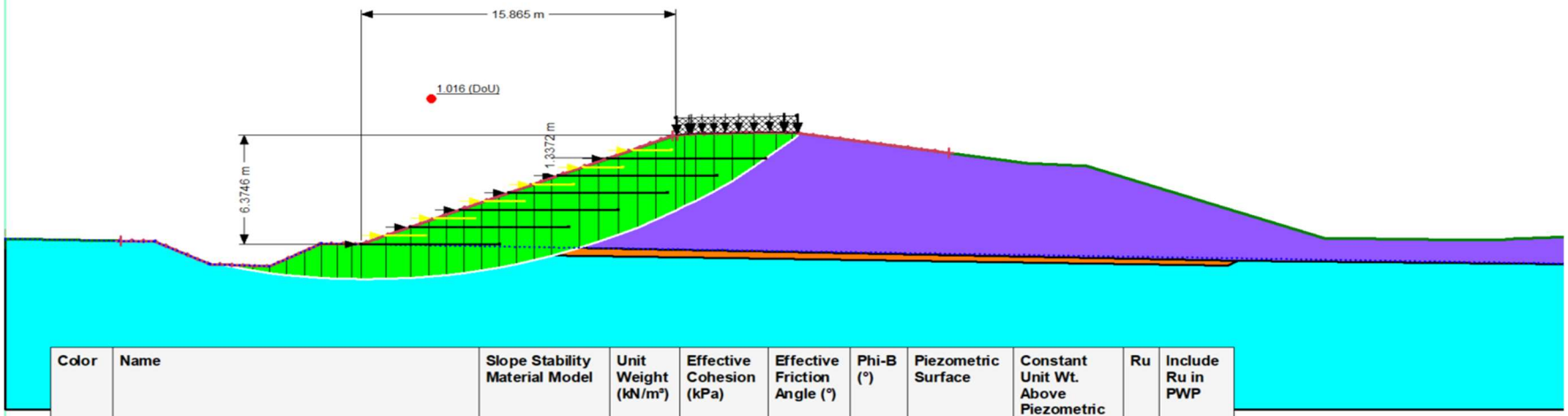
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	Class 2A4/7C3, Upper Emb Fill, Drained below WT(Long Term Analysis)	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No

Analysis No.11.: Embankment Height 6.40m Drained conditions EC7 - DA1C2 Unsupported DoU=1.363

HS2 Ltd - Code 1 - Accepted

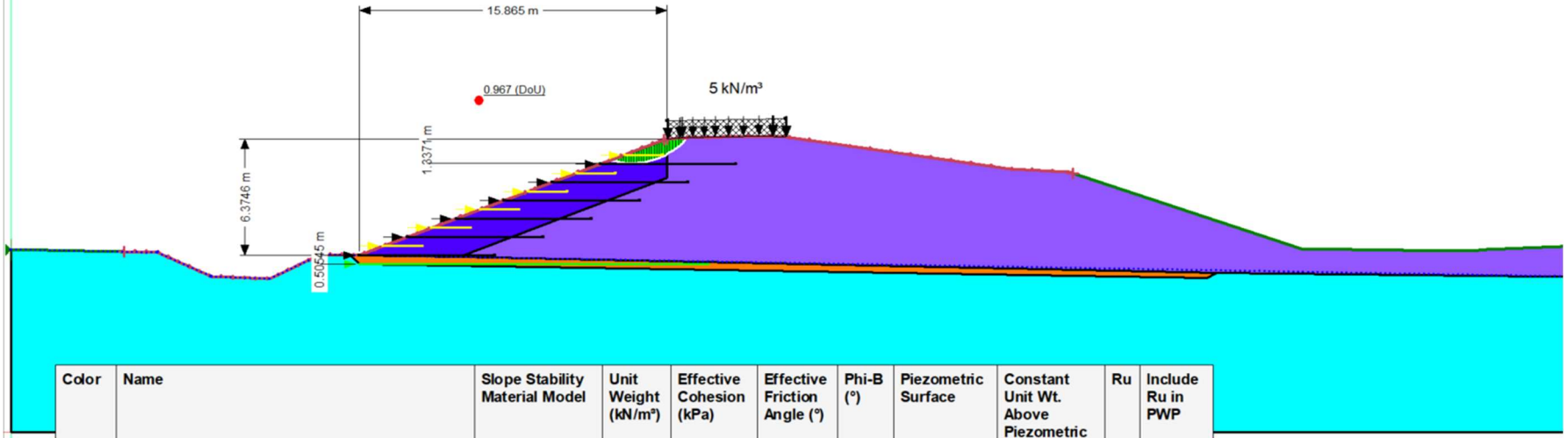


Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	Class 2A4/7C3, Upper Emb Fill, Drained below WT(Long Term Analysis)	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No

Color	Name	Type	Interface Adhesion (kPa)	Interface Shear Angle (°)	Surface Area Factor	Pullout Resistance (kPa)	Tensile Capacity (kN)	Face Anchorage	Factored Pullout Resistance	Factored Tensile Capacity
	High Strength	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	Secondary	Geosynthetic	0	12	2		15	Yes		15 kN/m

Analysis No.18.: Embankment Height 6.40 m Undrained conditions EC7-DA1C2 Supported DoU=1.016

HS2 Ltd - Code 1 - Accepted

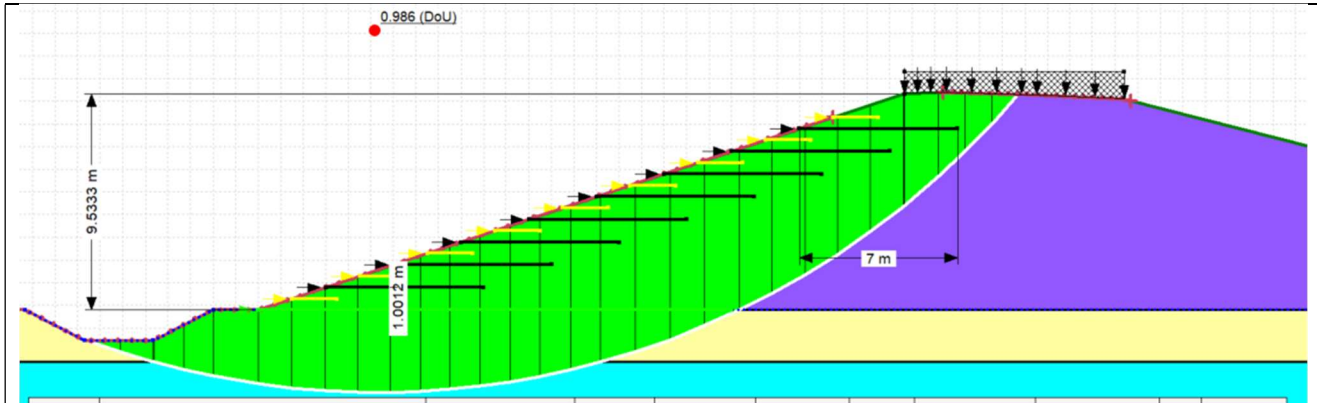


Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	Class 2A4/7C3, Upper Emb Fill, Drained below WT(Long Term Analysis)	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No

Color	Name	Type	Interface Adhesion (kPa)	Interface Shear Angle (°)	Surface Area Factor	Pullout Resistance (kPa)	Tensile Capacity (kN)	Face Anchorage	Factored Pullout Resistance	Factored Tensile Capacity
	Basal	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	High Strength	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	Secondary	Geosynthetic	0	12	2		15	Yes		15 kN/m

Analysis No.19.: Embankment Height 6.40 m Undrained conditions EC7-DA1C2 Supported with Basal layer DoU=0.967

HS2 Ltd - Code 1 - Accepted

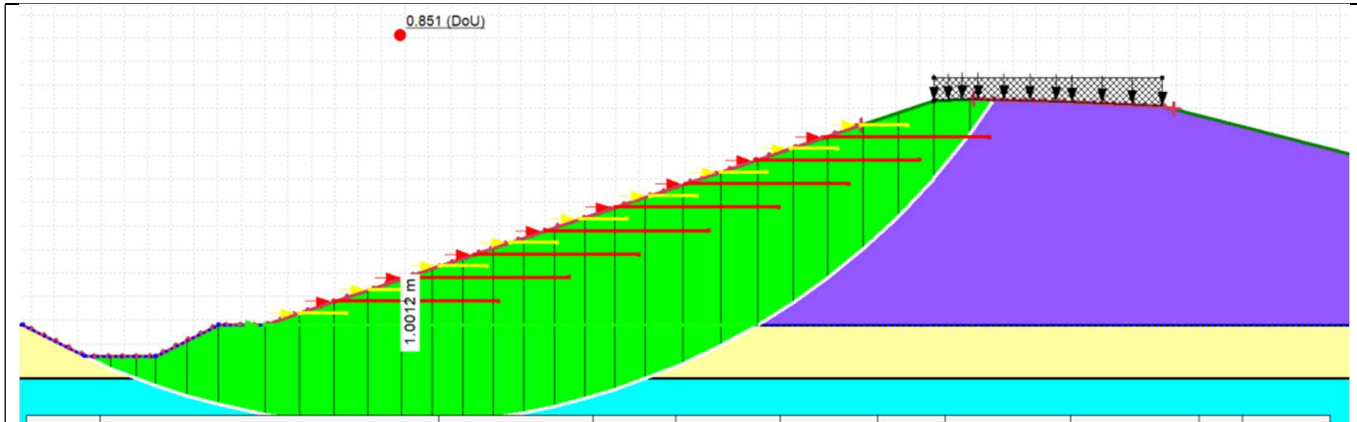


Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Color	Name	Type	Interface Adhesion (kPa)	Interface Shear Angle (°)	Surface Area Factor	Pullout Resistance (kPa)	Tensile Capacity (kN)	Face Anchorage	Factored Pullout Resistance	Factored Tensile Capacity
	Basal	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	High Strength	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	Secondary	Geosynthetic	0	12	2		15	Yes		15 kN/m

Analysis No.22.: Embankment Height 9.60m Drained conditions EC7 - DA1C2 Soil Reinforced with Basal layer DoU=0.986

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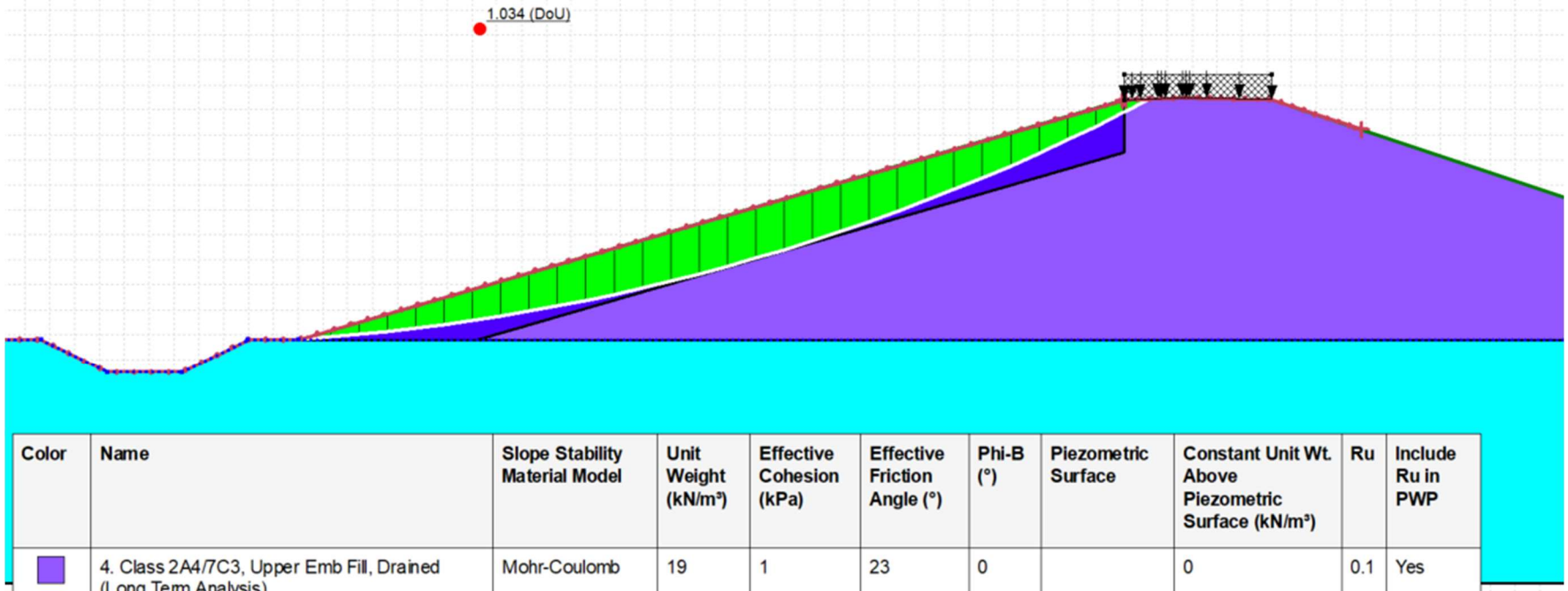


Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m²)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Color	Name	Type	Interface Adhesion (kPa)	Interface Shear Angle (°)	Surface Area Factor	Pullout Resistance (kPa)	Tensile Capacity (kN)	Face Anchorage	Factored Pullout Resistance	Factored Tensile Capacity
	Basal	Geosynthetic				100	100	Yes	100 kPa	100 kN/m
	BS8006	Geosynthetic	0	12	2		15	Yes		15 kN/m
	Secondary	Geosynthetic	0	12	2		15	Yes		15 kN/m

Analysis No.22.: Embankment Height 9.60m Drained conditions BS8006 Soil Reinforced with Basal layer DoU=0.851

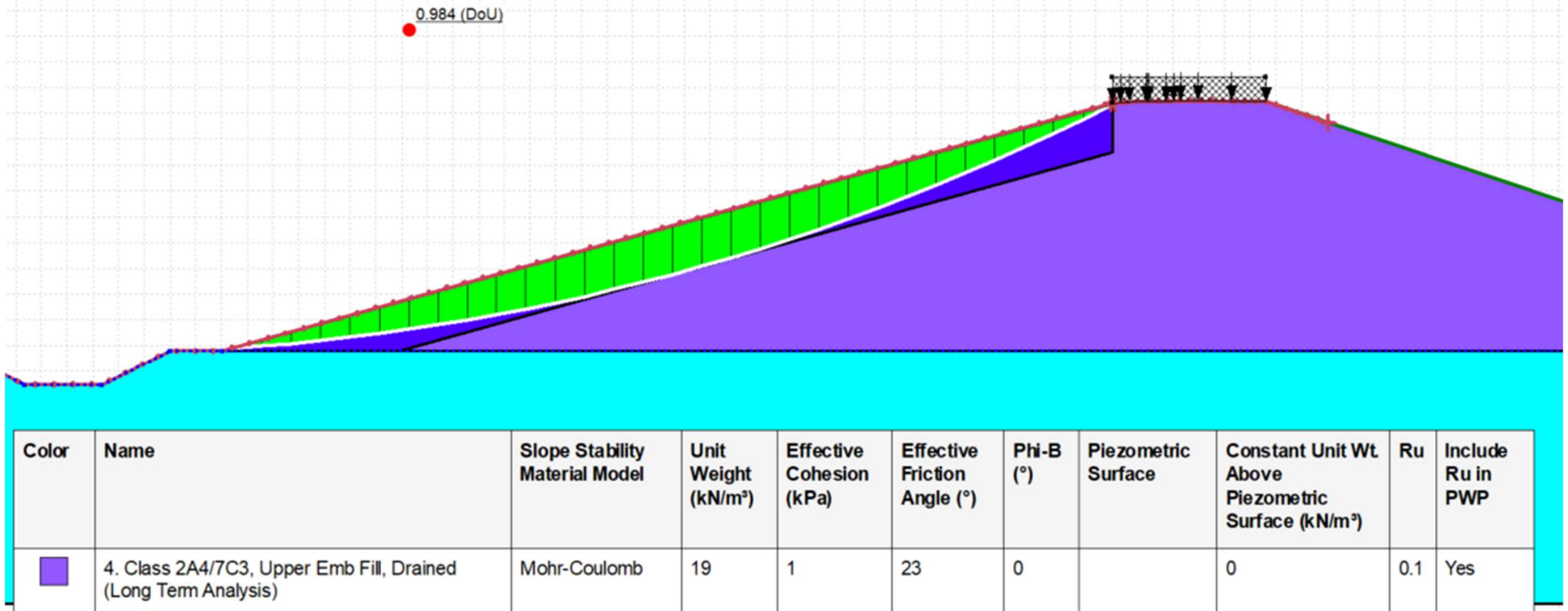
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No

Analysis No.23.: Embankment Height 9.6 m Drained conditions EC7-DA1C2 Unsupported DoU=1.034

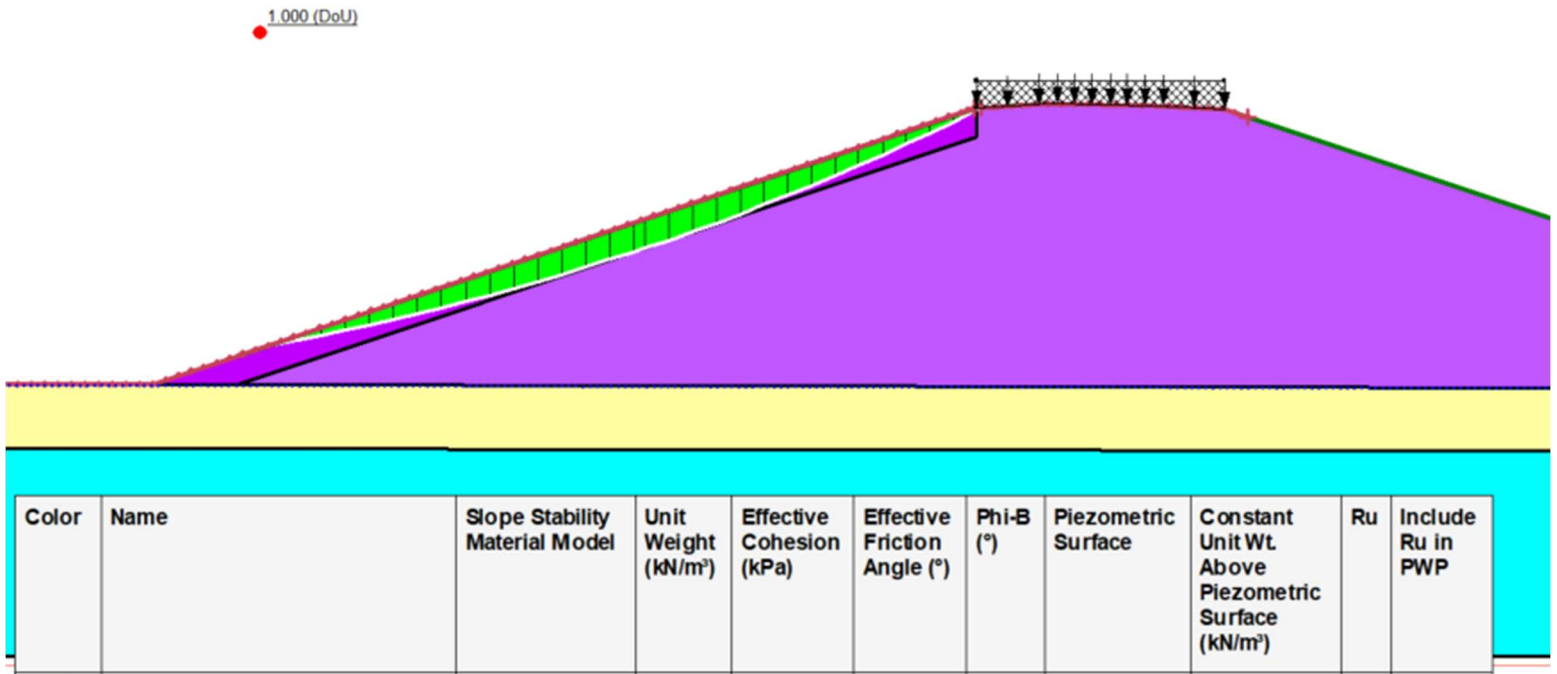
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fill, Drained (Long Term Analysis)	Mohr-Coulomb	19	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	19	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No

Analysis No.24.: Embankment Height 9.6m Drained conditions EC7 - DA1C2 Unsupported DoU=0.984

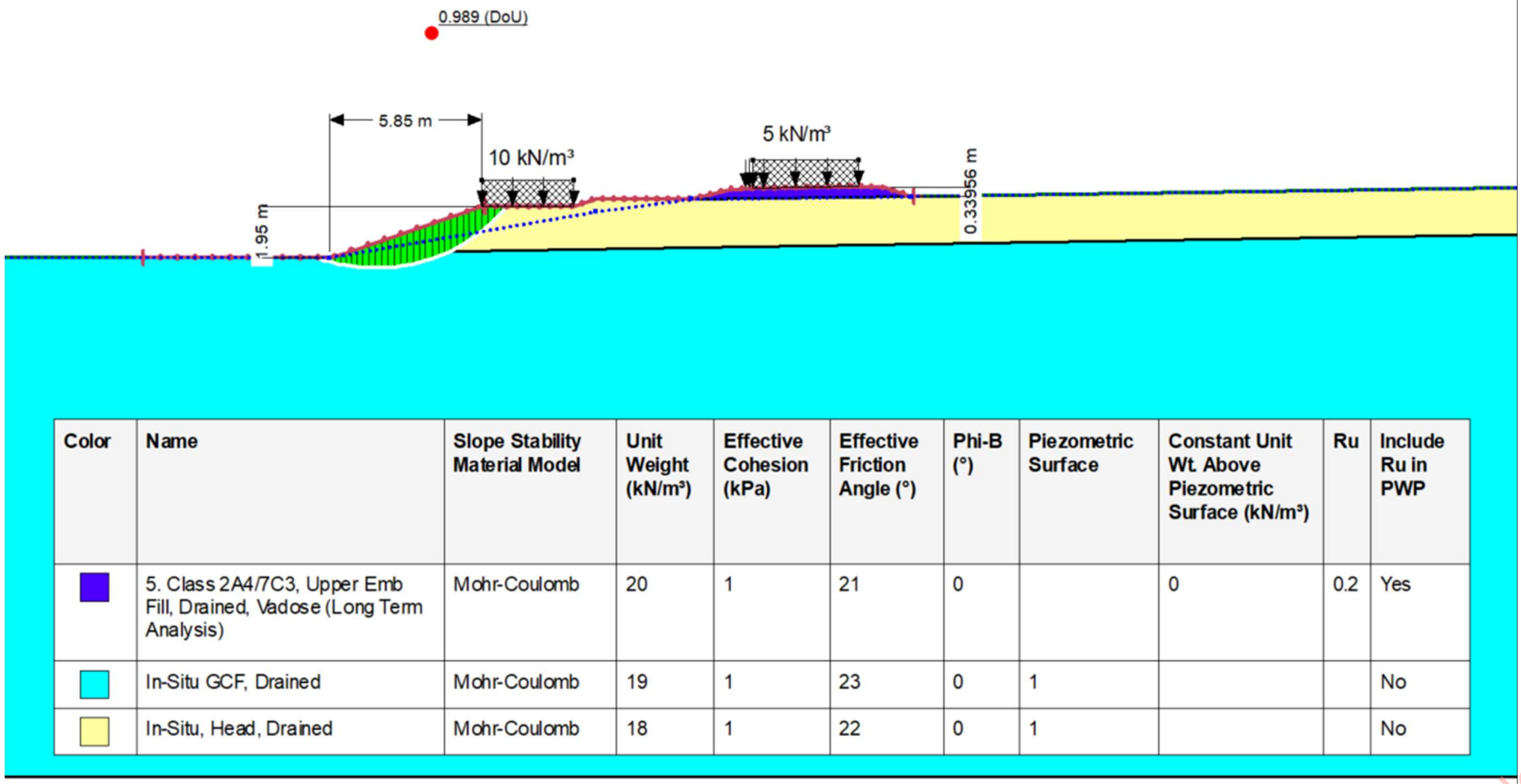
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	2. Class 2A4, Upper Emb Fill, Drained (Interim Analysis)	Mohr-Coulomb	19	3	23	0		0	0.1	Yes
	3. Class 2A4, Upper Emb Fill, Drained, Vadose (Interim Analysis)	Mohr-Coulomb	19	1	23	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No.26.: Embankment Height 10.1 m Drained conditions EC7-DA1C2 Unsupported DoU=1.000

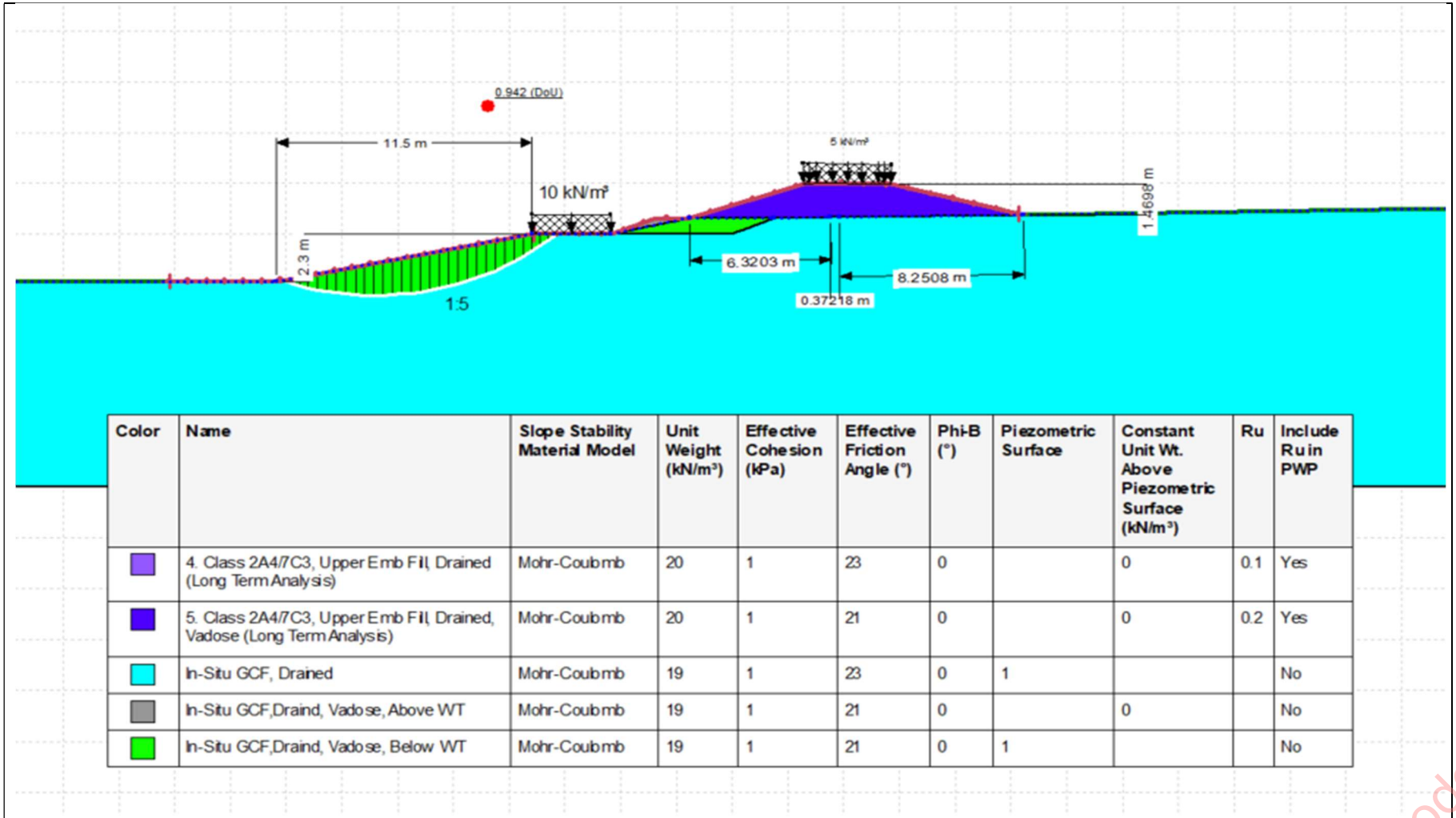
HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	5. Class 2A4/7C3, Upper Emb Fill, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	20	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ, Head, Drained	Mohr-Coulomb	18	1	22	0	1			No

Analysis No.27.: Embankment Height 0.4 m Drained conditions EC7-DA1C2 Unsupported DoU=0.989

HS2 Ltd - Code 1 - Accepted



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Surface	Constant Unit Wt. Above Piezometric Surface (kN/m³)	Ru	Include Ru in PWP
	4. Class 2A4/7C3, Upper Emb Fil, Drained (Long Term Analysis)	Mohr-Coulomb	20	1	23	0		0	0.1	Yes
	5. Class 2A4/7C3, Upper Emb Fil, Drained, Vadose (Long Term Analysis)	Mohr-Coulomb	20	1	21	0		0	0.2	Yes
	In-Situ GCF, Drained	Mohr-Coulomb	19	1	23	0	1			No
	In-Situ GCF,Draind, Vadose, Above WT	Mohr-Coulomb	19	1	21	0		0		No
	In-Situ GCF,Draind, Vadose, Below WT	Mohr-Coulomb	19	1	21	0	1			No

Analysis No.28.: Embankment Height 1.5 m Drained conditions EC7-DA1C2 Unsupported DoU=0.942

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Settlement Analysis - Embankment Loads

Name								Load Value		
	Orientation of Plane	Centre of load (global)			Angle of x from global X	Width x or radius	Length (y)	Normal	Tangential	
		X	Y	Z				Local (z)	Local (x)	Local (y)
		(m)	(m)	(m)						
Load 15 - OBr1	Horizontal plane	57.426	13.8621	10	0	10.41	8.57	230.00	0	0
Load 16 - BRA1	Horizontal plane	57.4257	26.8958	10	0	10.41	17.5	200.00	0	0
Load 19 - BRA2	Horizontal plane	65.2899	39.5371	9	0	26.14	7.78	180.00	0	0
Load 25 - BRA3	Horizontal plane	88.3509	40.568	8	0	19.98	7.3	160.00	0	0
Load 34 - BRA4	Horizontal plane	108.335	41.3662	7	0	19.98	7.3	140.00	0	0
Load 44 - BRA5	Horizontal plane	128.319	42.1644	6	0	19.98	7.3	120.00	0	0
Load 51 - BRA6	Horizontal plane	153.2991	43.1597	5	0	29.98	7.7	100.00	0	0
Load 63 - BRA7	Horizontal plane	181.3519	44.6533	4.5	0	26.13	8.29	90.00	0	0
Load 64 - BRA8	Horizontal plane	187.995	61.1463	3	0	12.84	24.7	60.00	0	0
Load 1	Horizontal plane	13.033	50.2716	1	0	26.07	7.5	20.00	0	0
Load 2	Horizontal plane	15.8469	42.7714	3	0	31.69	7.5	60.00	0	0
Load 3	Horizontal plane	18.5687	35.2711	5.5	0	37.14	7.5	110.00	0	0
Load 4	Horizontal plane	21.2986	27.7709	7	0	42.6	7.5	140.00	0	0
Load 5	Horizontal plane	26.1105	11.7199	6	0	52.22	4.28	120.00	0	0
Load 6	Horizontal plane	26.1105	16.0042	7	0	52.22	4.28	140.00	0	0
Load 7	Horizontal plane	26.1105	21.0836	8	0	52.22	5.87	160.00	0	0
Load 8	Horizontal plane	30.8786	50.2716	1	0	9.62	7.5	20.00	0	0
Load 9	Horizontal plane	36.5057	42.7714	4	0	9.62	7.5	80.00	0	0
Load 10	Horizontal plane	38.504	48.8813	3	0	5.63	4.72	60.00	0	0

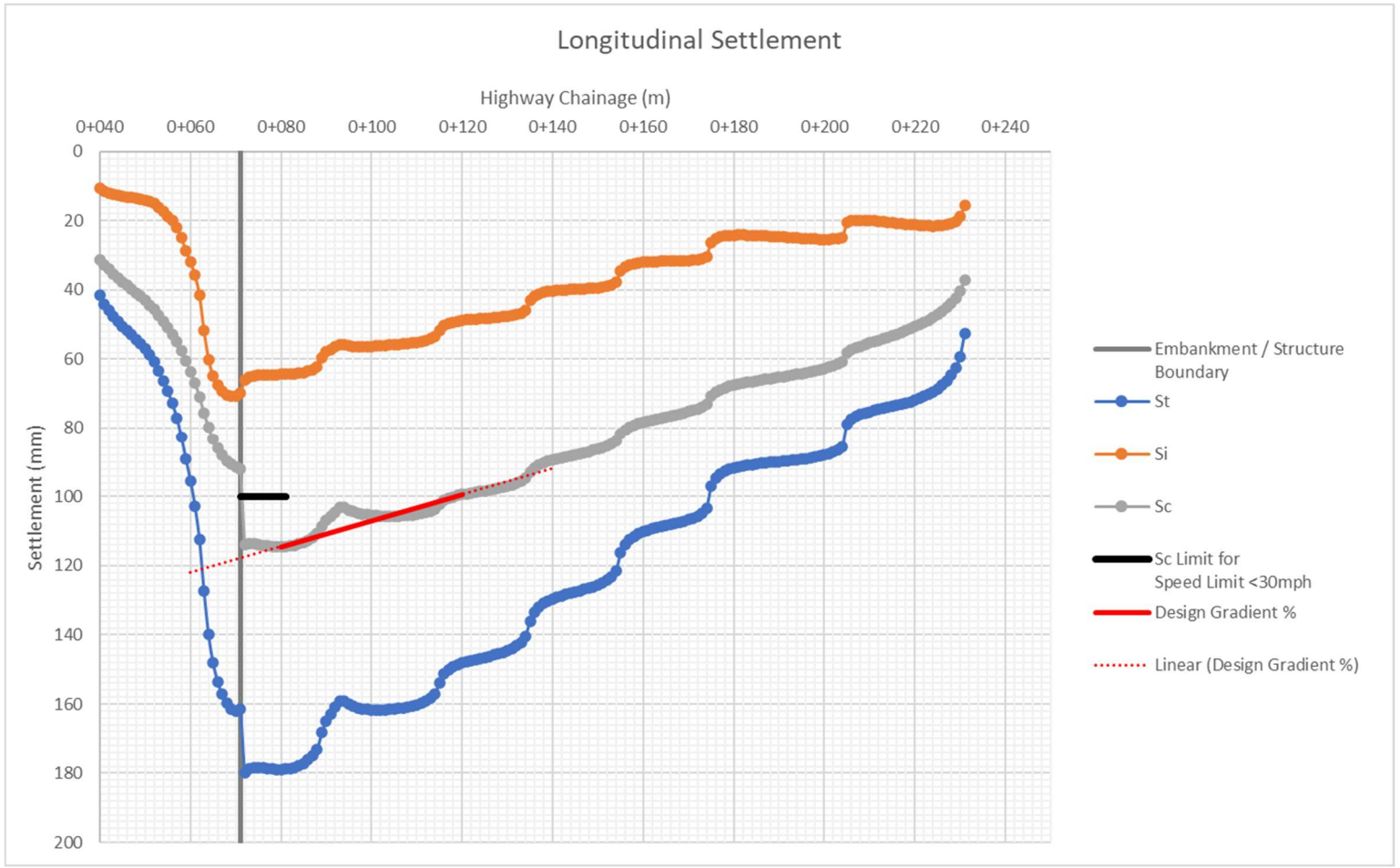
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Load 11	Horizontal plane	44.6792	35.2711	7	0	15.08	7.5	140.00	0	0
Load 12	Horizontal plane	46.7693	45.1312	5.5	0	10.9	12.22	110.00	0	0
Load 13	Horizontal plane	47.4091	27.7709	9	0	9.62	7.5	180.00	0	0
Load 14	Horizontal plane	63.0117	55.1402	5	0	30.69	7.8	100.00	0	0
Load 17	Horizontal plane	65.2899	47.335	7.5	0	26.14	7.81	150.00	0	0
Load 18	Horizontal plane	65.2899	62.931	3	0	26.14	7.78	60.00	0	0
Load 20	Horizontal plane	65.2899	70.1584	1	0	26.14	6.67	20.00	0	0
Load 21	Horizontal plane	70.4946	27.2392	8	0	15.73	6.44	160.00	0	0
Load 22	Horizontal plane	70.495	33.0514	9	0	15.73	5.19	180.00	0	0
Load 23	Horizontal plane	88.3509	27.2392	8	0	19.98	6.44	160.00	0	0
Load 24	Horizontal plane	88.3509	33.687	8	0	19.98	6.46	160.00	0	0
Load 26	Horizontal plane	88.3509	47.7305	7	0	19.98	7.02	140.00	0	0
Load 27	Horizontal plane	88.3509	55.1402	5	0	19.98	7.8	100.00	0	0
Load 28	Horizontal plane	88.3509	62.931	3	0	19.98	7.78	60.00	0	0
Load 29	Horizontal plane	88.3509	70.1584	1	0	19.98	6.67	20.00	0	0
Load 30	Horizontal plane	96.6187	7.1833	5	0	193.24	4.79	100.00	0	0
Load 31	Horizontal plane	100.5863	2.3944	3.5	0	201.17	4.79	70.00	0	0
Load 32	Horizontal plane	108.335	27.2392	8	0	19.98	6.44	160.00	0	0
Load 33	Horizontal plane	108.335	34.0861	7	0	19.98	7.26	140.00	0	0
Load 35	Horizontal plane	108.335	48.1295	6	0	19.98	6.22	120.00	0	0
Load 36	Horizontal plane	108.335	55.1402	4	0	19.98	7.8	80.00	0	0
Load 37	Horizontal plane	108.335	62.931	2.5	0	19.98	7.78	50.00	0	0
Load 38	Horizontal plane	108.335	70.1584	0.5	0	19.98	6.67	10.00	0	0
Load 39	Horizontal plane	125.0037	21.0836	8	0	124.75	5.87	160.00	0	0
Load 40	Horizontal plane	125.2443	16.0042	7	0	125.23	4.28	140.00	0	0
Load 41	Horizontal plane	126.1367	11.7199	6	0	127.01	4.28	120.00	0	0
Load 42	Horizontal plane	128.319	27.2392	7	0	19.98	6.44	140.00	0	0
Load 43	Horizontal plane	128.319	34.4852	6	0	19.98	8.06	120.00	0	0

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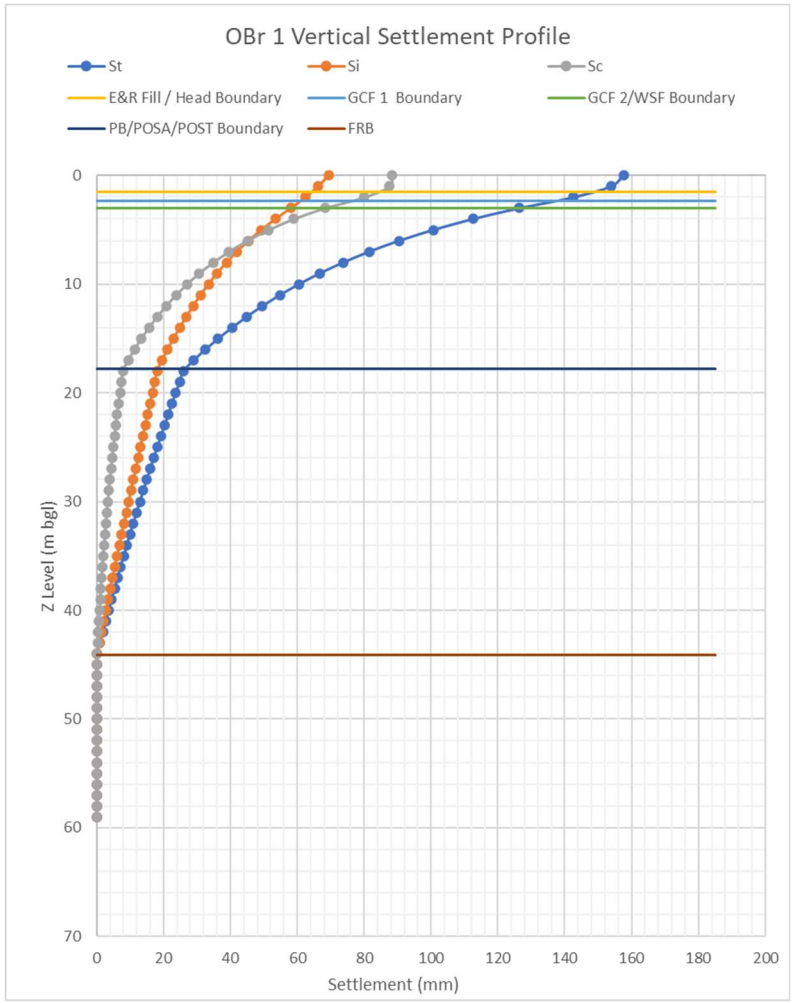
Load 45	Horizontal plane	128.319	48.5286	5	0	19.98	5.43	100.00	0	0
Load 46	Horizontal plane	128.319	55.1402	4	0	19.98	7.8	80.00	0	0
Load 47	Horizontal plane	128.319	62.931	2	0	19.98	7.78	40.00	0	0
Load 48	Horizontal plane	128.319	70.1584	0.5	0	19.98	6.67	10.00	0	0
Load 49	Horizontal plane	153.2991	27.2392	6	0	29.98	6.44	120.00	0	0
Load 50	Horizontal plane	153.2991	34.8843	5.5	0	29.98	8.85	110.00	0	0
Load 52	Horizontal plane	153.2991	49.1249	4.5	0	29.98	4.23	90.00	0	0
Load 53	Horizontal plane	153.299	55.1402	3.5	0	29.98	7.8	70.00	0	0
Load 54	Horizontal plane	153.2991	62.931	2	0	29.98	7.78	40.00	0	0
Load 55	Horizontal plane	153.2991	70.1584	1	0	29.98	6.67	20.00	0	0
Load 56	Horizontal plane	174.9302	50.0198	4	0	13.29	2.44	80.00	0	0
Load 57	Horizontal plane	174.9302	55.1402	3.5	0	13.29	7.8	70.00	0	0
Load 58	Horizontal plane	174.9302	62.931	2.5	0	13.29	7.78	50.00	0	0
Load 59	Horizontal plane	174.9302	70.1584	2	0	13.29	6.67	40.00	0	0
Load 60	Horizontal plane	177.8321	27.2392	5	0	19.09	6.44	100.00	0	0
Load 61	Horizontal plane	181.3519	33.0514	5	0	26.13	5.19	100.00	0	0
Load 62	Horizontal plane	181.352	38.0767	4	0	26.13	4.86	80.00	0	0
Load 65	Horizontal plane	196.9782	54.5696	2.5	0	5.12	37.85	50.00	0	0
Load 66	Horizontal plane	201.8704	57.6774	1	0	4.66	31.63	20.00	0	0
Load 67	Horizontal plane	47.6645	62.931	1	0	9.11	7.78	20.00	0	0
Load 68	Horizontal plane	41.6775	55.1402	2	0	11.97	7.8	40.00	0	0
Load 69	Horizontal plane	100.5863	-8.1709	3	0	201.17	16.34	60.00	0	0

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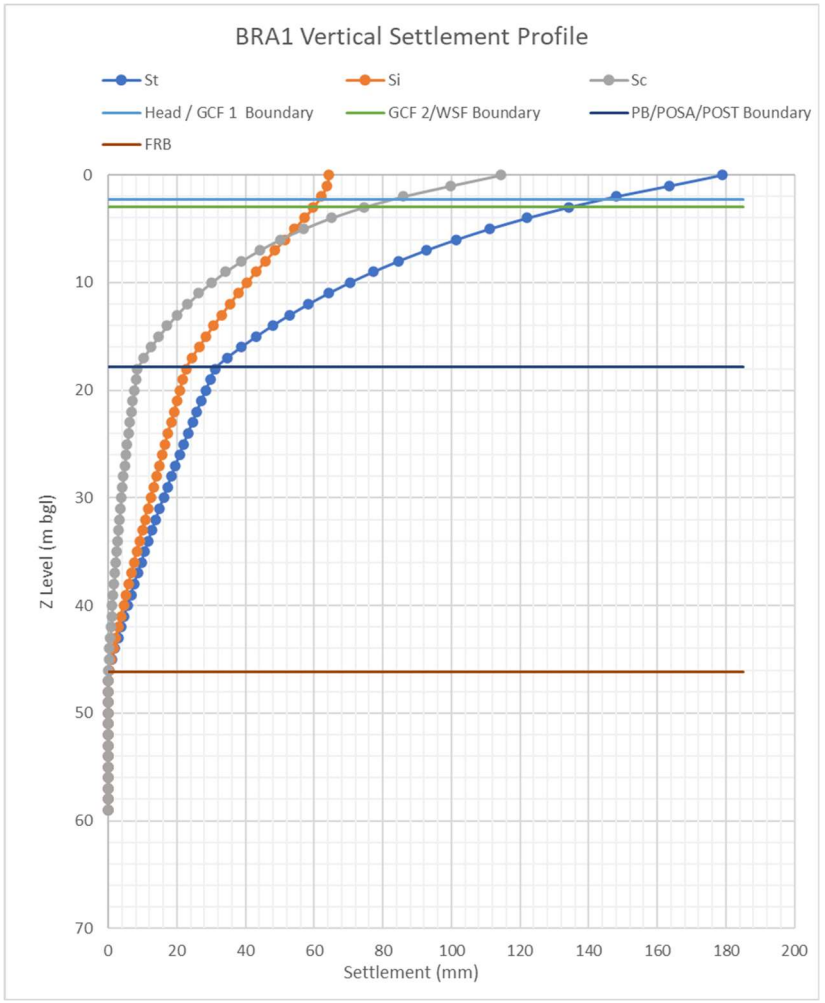


Longitudinal Settlement Profile

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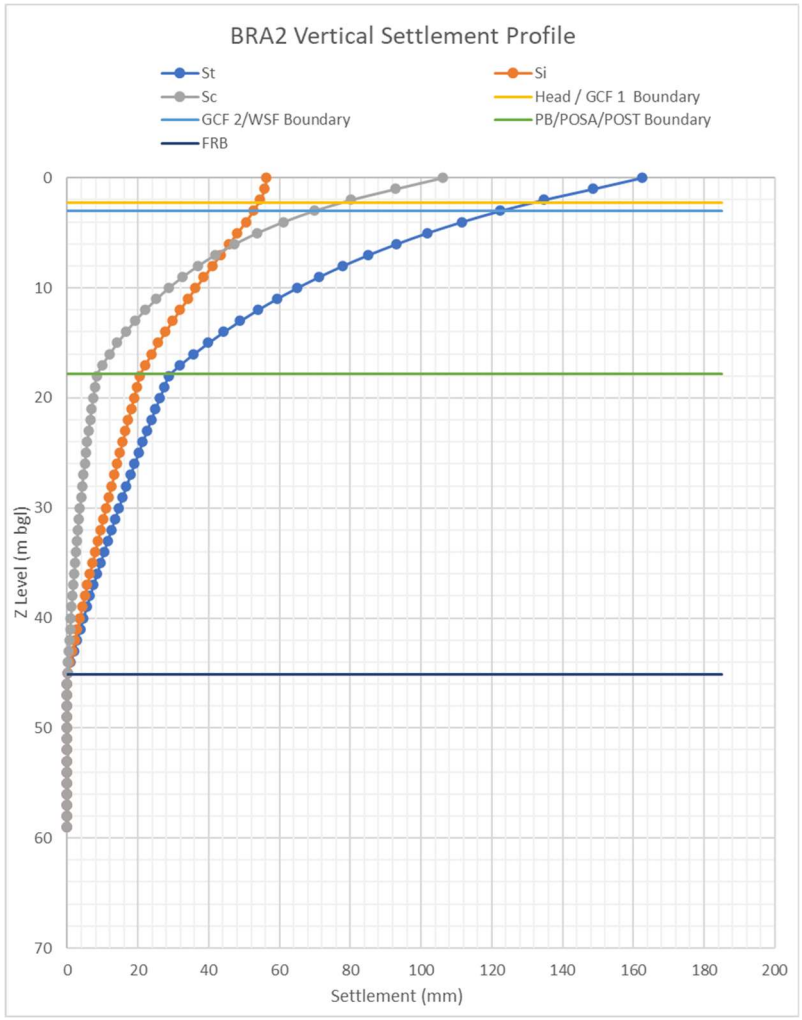


Load 15 – OBr1, 10m Vertical Settlement Profile

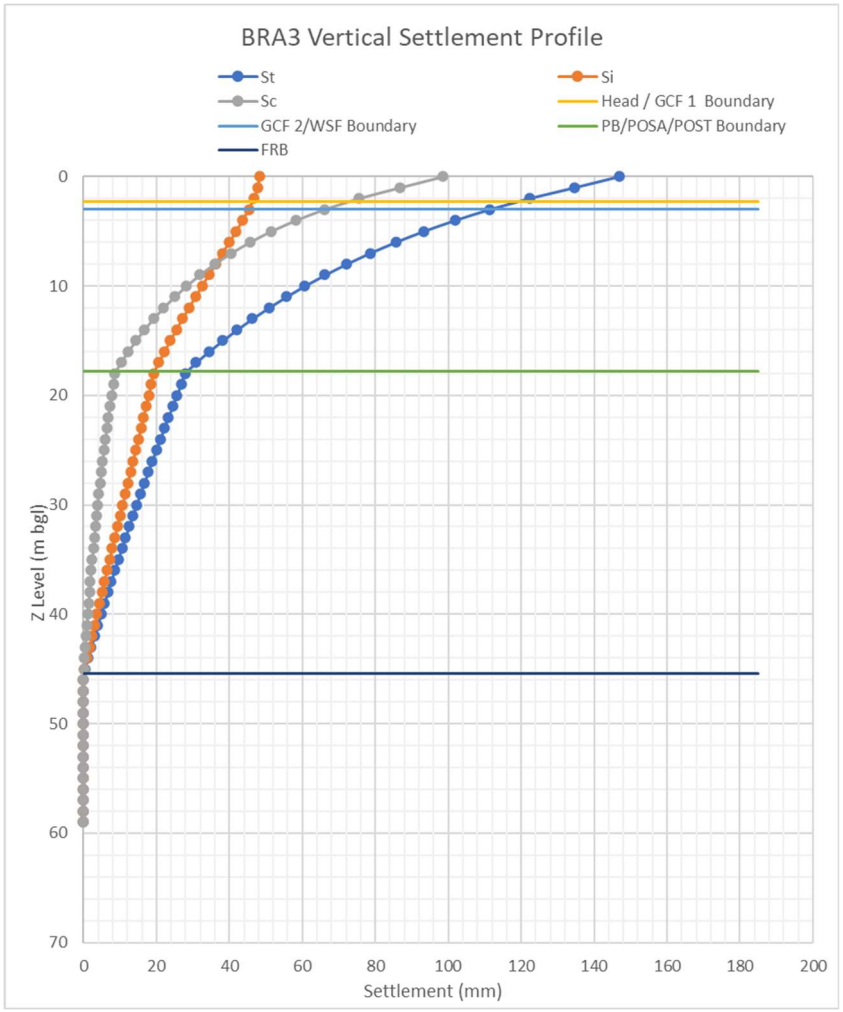


Load 15 – BRA1, 10m Vertical Settlement Profile

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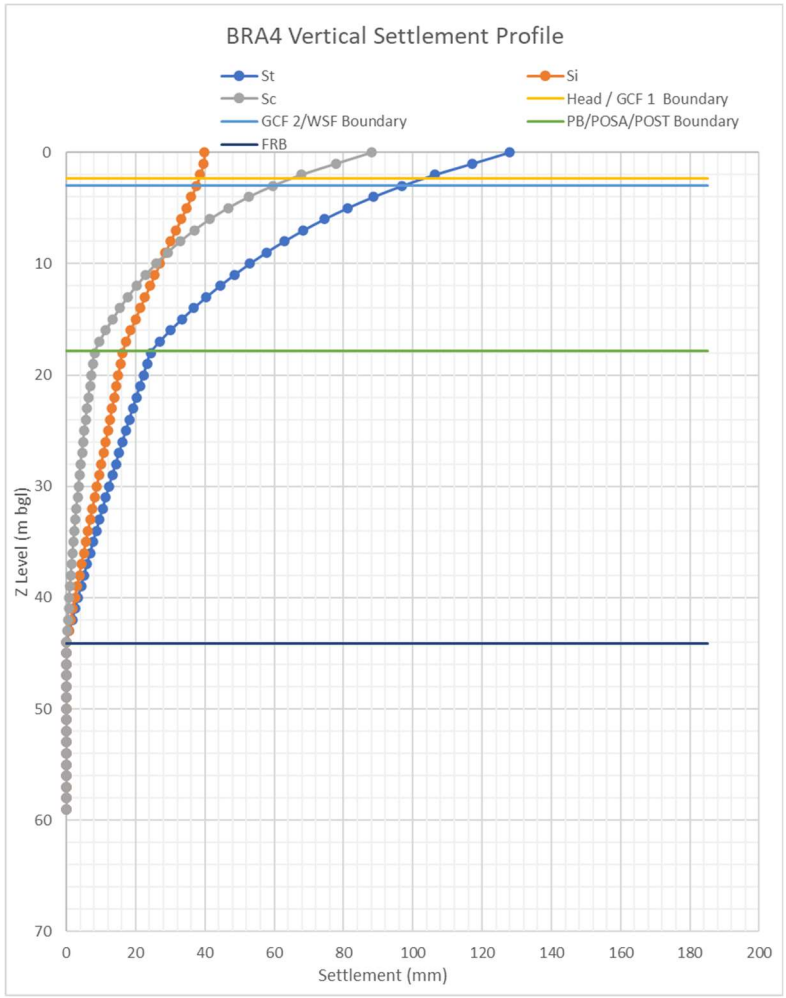


Load 19 – BRA2, 9m Vertical Settlement Profile

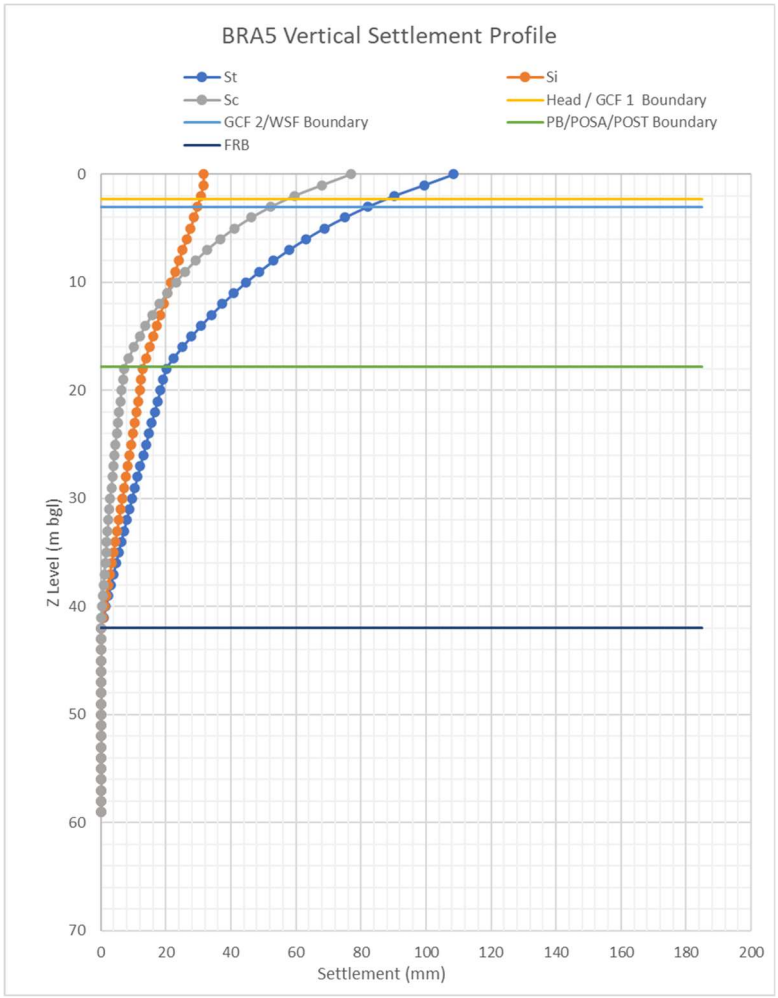


Load 25 – BRA3, 8m Vertical Settlement Profile

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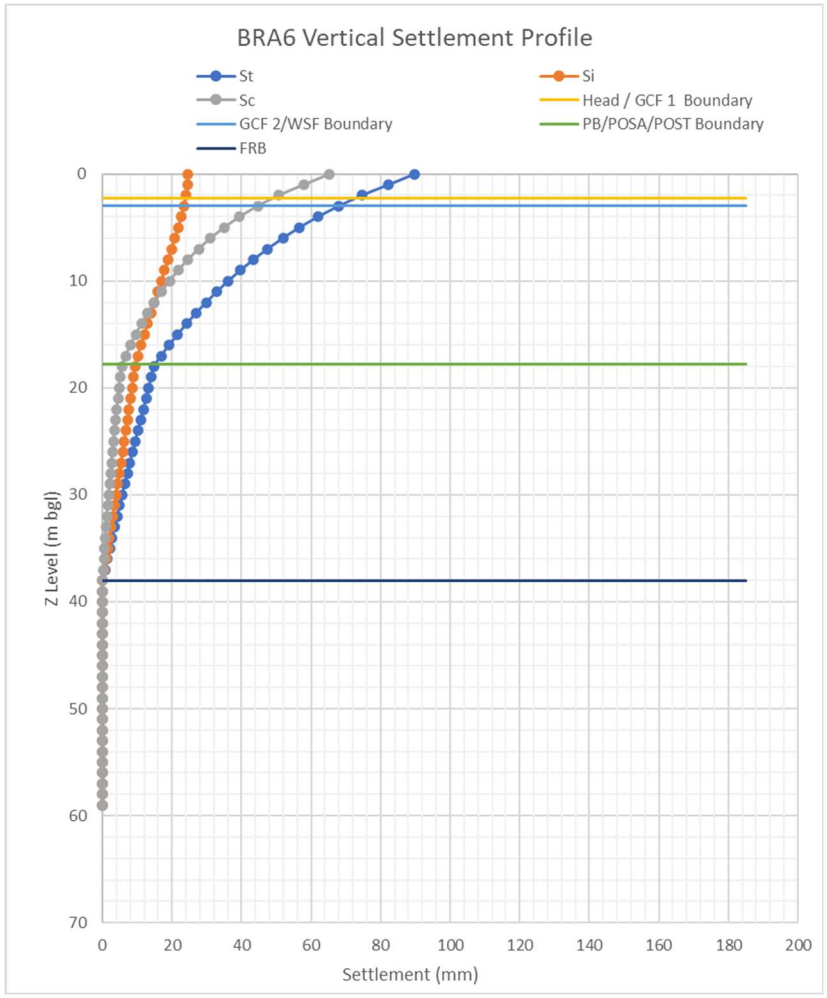


Load 34 – BRA4, 7m Vertical Settlement Profile

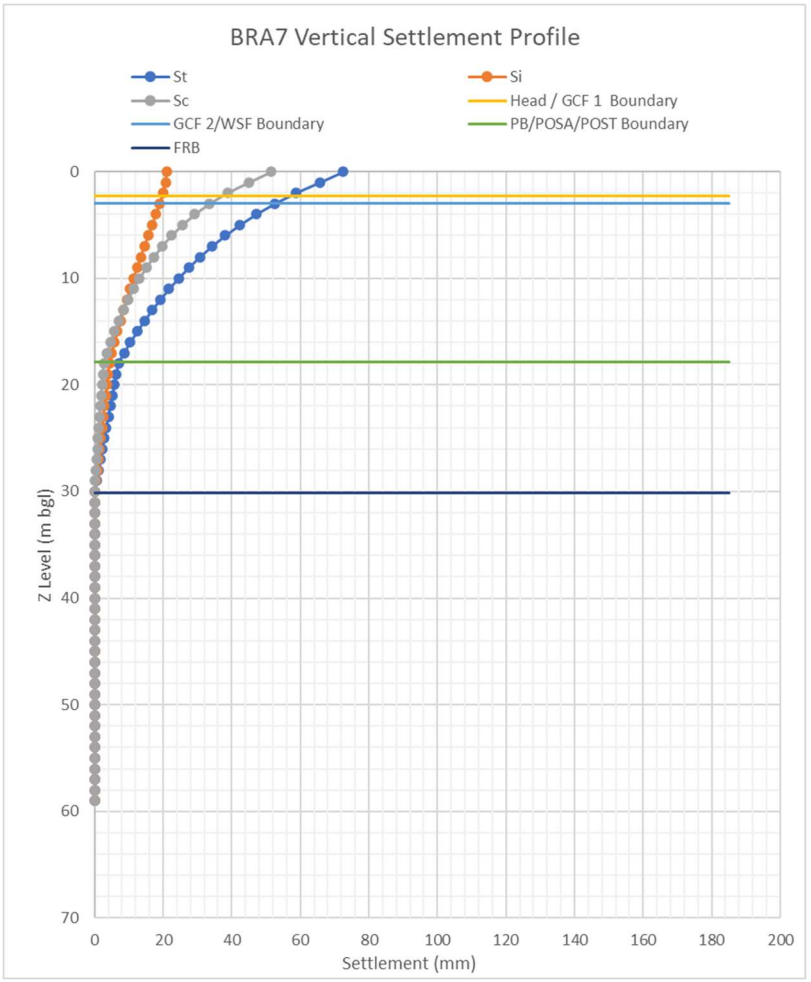


Load 44 – BRA5, 6m Vertical Settlement Profile

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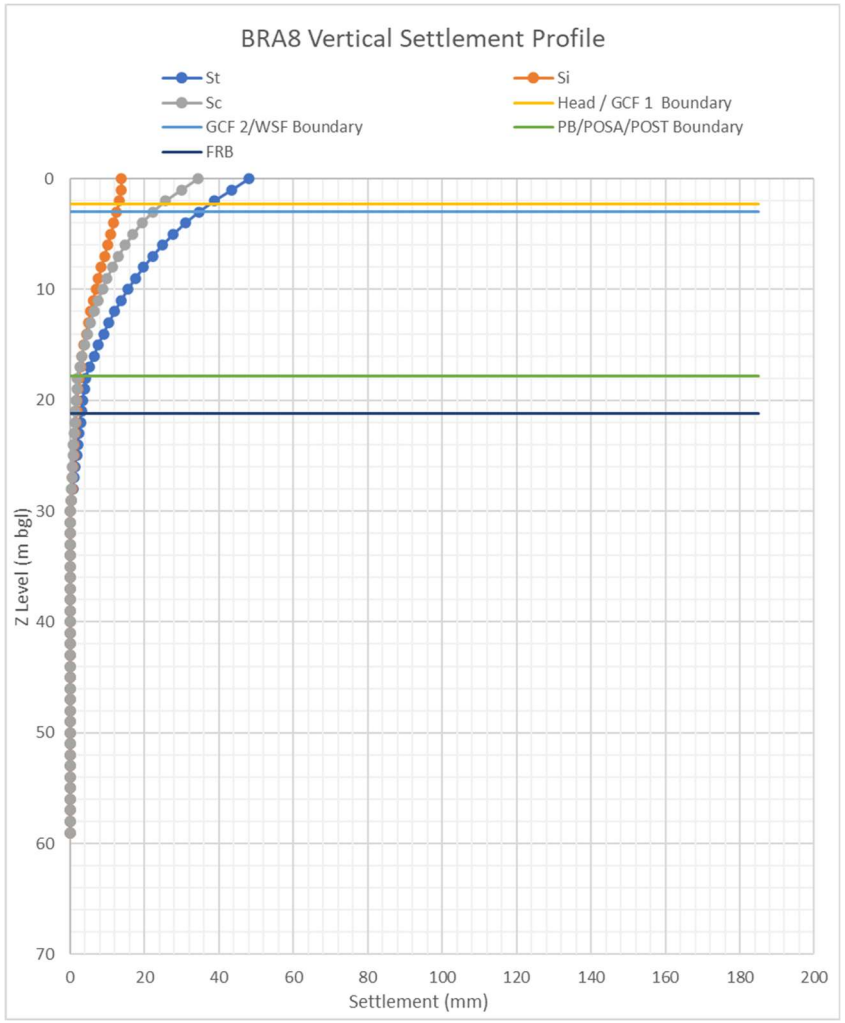


Load 51 – BRA6, 5m Vertical Settlement Profile



Load 63 – BRA7, 4.5m Vertical Settlement Profile

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Load 64 – BRA8, 3m Vertical Settlement Profile

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Appendix E : Strengthened Earthworks Appraisal Form

Number of Pages: 10

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E1 Introduction

This section presents the Strengthened Earthworks Appraisal Form (SEAF) for the Highways Embankments associated with Oat Close Bridleway (SBH/19) Realignment located in Section C2 Central.

The proposed scheme includes embankments that are to be constructed using Strengthened Earthworks to support embankments at heights and gradients that do not comply with the stability requirements of EC7 and the HS2 Technical Standard - Earthworks

E2 Scheme Details

E2.1 Name of Scheme

HS2 Aylesbury Area – Oat Close Bridleway (SBH/19) Realignment – HS2 Asset ID HS2-000020RKL

E2.2 Type of Highway

Public Right of Way/Bridleway/Footpath

E2.3 Permitted Traffic Speed

N/A

E2.4 Nature of Scheme Element

New highway construction to provide bridleway/footpath access to new SBH/19 Overbridge as part of the HS2 Scheme.

E3 Strengthened Earthwork Type, Purpose, and Location

E3.1 Generic Type of Strengthened Earthwork

Strengthened Soil Slopes

E3.2 Purpose of Strengthened Earthwork

The embankment slopes are to be constructed at an inclination greater than the natural safe slope angle due to restricted land take.

E3.3 Intended Locations for Use

Highway embankments which are not able to stand up unsupported for sections of Oat Close Bridleway (SBH/19) earthworks. Specific Locations for the use of reinforced soils slopes are detailed in Table E1 below.

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Table E1 – Locations For Use of Reinforced Soil

Asset	Asset Chainage	Embankment Height (m)
Bridleway SBH/19 Realignment	LHS: 0+085 to 0+110 RHS: 0+220 to 0+240	LHS: 9.57 to 8.66 RHS: 3.94 to 2.91
Bridleway SBH/19 Link	RHS: 0+000 to 0+080	8.63 to 4.97
Footpath SBH/27 Realignment	LHS: 0+000 to 0+090	8.63 to 5.07

E4 Outline of Existing Ground Conditions and Groundwater Conditions

E4.1 Ground Investigation Data

The ground investigation data available for Oat Close Bridleway (SBH/19) Realignment is set out in within Section 4 and Appendix B in the corresponding Supplementary Ground Investigation Report (GIR) 1MC06-CEK-GT-REP-CS04_CL18-000011.

A total of 18 no Exploratory Holes have been considered in the design of this asset, the locations of which are shown the plan included within Appendix B.

At the time of producing this calculation pack, there are no outstanding Stage 2 GI exploratory points within 10m of the asset footprint.

The Uncertainty in Ground Conditions Classification for the asset group is Class 1 considering the available GI data and Geotechnical Category of the earthworks.

E4.2 Existing Ground Conditions

The ground conditions are presented in in Section 4 of this document. In summary, Ground Model 1 is as follows:

Stratigraphy	Depth range (mbgl)
Gault Clay Formation (GLT – C Z)	0.00 – 16.35
Whitchurch Sand Formation (WHS – C Z)	16.35+ (Base not Proven)

Ground Model 2 is summarised below.

Stratigraphy	Depth range (mbgl)
Head (HEAD – C Z)	0.00 – 2.30
Gault Clay Formation (GLT – C Z)	2.30 – 9.43
Whitchurch Sand Formation (WHS – C Z)	9.43 – 10.93
Purbeck Group (PB – LMST)	10.93 – 12.78
Portland Stone / Portland Sand Formations (POST – SLST/POSA – LMST)	12.78+ (Base not Proven)

E4.3 Existing Groundwater Conditions

The groundwater conditions are summarised in Section 4.3 of this document. A conservative Design Groundwater Level of 0.00m has been adopted for this asset.

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E4.4 Soil and Groundwater Chemistry

At the time of writing the Ground Investigation Report (GIR) (1MC06-CEK-GT-REP-CS04_CL18-000011) no geochemical testing had been carried out on the in-situ strata, besides limited samples for the Portland Stone Formation, which is encountered at depth below Oat Close Bridleway SBH/19 Realignment.

However, the results of the contaminated land assessment are provided in Section 4.5 of the GIR.

E4.5 Existing Geotechnical Problems and Risks

The geotechnical risks associated with the asset are summarised in Section 4.4 of this document.

- Reference should be made to the IDR Package 207.2 – Geotechnical Risk Register (1MC06-CEK-GT-REG-CS04_CL18-000006)

The key Geohazards relating to this asset are:

- High / Shallow Groundwater
- Made Ground / Topsoil Material
- Contaminated Ground
- Chemically Aggressive Ground
- Shrink / Swell Potential
- Soft / Compressible Ground
- Relict periglacial features
- Variable weathering profile
- Geological Faulting

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E5 Proposed Strengthened Earthwork

E5.1 Description of Strengthened Earthwork

The strengthened earthworks are to be formed at slope angles between 1V:2.5H and 1V:3.5H and will be reinforced by geogrids. They will be formed for slope heights of between 3.00m and 9.60m high.

The fill to the proposed geogrid reinforcement array will comprise Class 7C3 Cohesive Fill material.

These features are shown on the G031 Typical Highway Earthworks Section Embankment Reinforced Earth (1MC06-CEK-GT-DSE-C002-000111).

E5.2 Foundation Preparation, including measures to deal with Geotechnical Problems

The reinforced slopes shall be constructed with primary geogrids in lifts of a maximum of 1.0m height, supported by intermediary secondary geogrids of a minimum of 2m in length. All geogrid sheets are to be laid in a bottom-up sequence following appropriate compaction as the surrounding embankment is constructed.

No specific ground treatment is proposed, other than topsoil strip for all embankments and the provision of a basal drainage blanket for embankments >3m high.

Soft spots, defined as soils with an undrained shear strength of <50kPa, will be required to be removed by inspection following exposure of the Formation.

E5.3 Materials to be used in Construction

The highway embankments will comprise of reworked Gault Clay Formation Class 2A4 material. The proposed Class 2A4 fill has also been categorised as a Class 7C3 fill for use within the geogrid array, refer to Appendix 6/1 Table 6/1 for acceptability criteria and Appendix 1/5 of the SCEW (1MC06-CEK-AU-SPE-CS04_CL18-000003) for testing requirements.

Departures have been submitted to permit the use of Gault Clay Formation as Class 7C, due to the fact that it exceeds the Plasticity Index for Class 7C material as set out in the SCEW and the proposed Class 7C material has a finer grading curve than the Class 7C as set out in the SCEW. Further details are given below on the Departures, which have now been accepted.

The slopes of the embankment will be reinforced with layers of polymeric geogrids with long term design strength of at least 15 kN/m over a 120-year expected service life. The design properties of geogrids are as detailed in Appendix 6/9 of the SCEW (1MC06-CEK-AU-SPE-CS04-000003).

E5.4 Drainage Measures

A 300mm thick Class 1 starter/basal drainage layer is to be installed beneath embankments greater than 3m. For the purposes of assessing the stability of the embankments an excess pore water pressure parameter, r_u, has been adopted in the embankment, and in the upper Vadose Zone as directed by the HS2 Technical Standard - Earthworks

Over the edge drainage is to be proposed for all high highway embankments in conjunction with drainage ditches at the toe of the earthworks.

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E5.5 Arrangements for Highway Furniture and Buried Services and Landscaping

A clash check has been undertaken between highway furniture, such as lighting/VRS and also buried services within the reinforced embankments.

The geogrid reinforcement design allows for the upper primary geogrid to be placed no higher than 1000mm below the proposed verge level to accommodate shallow foundations and utilities. Where deeper utilities (such as filter drains) cannot be moved sufficiently to avoid a clash with the geogrid array, or deep foundations to VRS are proposed, the geogrid design (and length of geogrid) has been modified to accommodate these potential clashes.

E5.6 Inspection and Maintenance

Instrumentation, monitoring and in-situ testing is required in general to monitor settlements of embankments, pore water pressure dissipation, verification of ground conditions and to ensure safety during construction.

Asset specific instrumentation and monitoring can be found in Section 11 of this calculation pack. For this asset, monitoring has been specified as set out in Appendix F to this calculation pack.

The assets should be inspected on a regular cycle to be agreed with the adopting authority. Any evidence of slope movement should be noted and relayed to the Designer.

Maintenance is normally undertaken from the front of the embankment.

The provision of geogrid reinforcement should not have any impact on the cycles of inspection.

E 5.7 Interface with Structures

The abutments to all bridge structures are to be backfilled with Class 6N/6P or 1A or fill as per their specific design requirements and no interaction with geogrid reinforced slopes are anticipated at this interface other than provision for VRS foundations.

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E6 Design Methods

E6.1 Internal Stability

The analysis of the internal stability of the strengthened earthworks was conducted by limit state design in accordance with BS 8006-1:2010 – *Code of practice for strengthened/reinforced soils and other fills*, and Morgenstern-Price's method for slope stability. Design was undertaken using appropriate computer software (SLOPE/W Version 11.4.0.18. by GeoStudio). The internal stability analysis was undertaken using ultimate limit state design in accordance with BS 8006-1:2010 to achieve a DoU less than or equal 1.

The geogrid sheet length, inclination, spacing, and required strength were determined by selecting the slope failure mechanism that passes through the reinforced soil block, giving the greatest activating force that leads to failure. The contribution from reinforcement was limited by the tensile failure and pull-out resistance in accordance with BS 8006:2010.

Stability analyses were conducted for various embankment heights for reinforced soil design covering height ranges of 3.40 to 9.40m, at Chainage locations as identified in Section E3.3.

E6.2 External Stability

The stability of the overall embankment slope including failure beneath and behind the strengthened earthwork was assessed in accordance with BS EN 1997-1:2004+A1:2013 *Eurocode 7: Geotechnical Design*.

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E7 Design/Assessment Criteria

E7.1 List of Relevant Documents

a. BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills.
b. BS EN 1997-1:2004+A1:2013 Eurocode 7: Geotechnical Design inc. National Annex.
c. BS 8002:2015 Code of practice for earth retaining structures.
d. BS EN 14475:2006 Execution of Special Geotechnical Works – Reinforced Fill.
e. MCHW Volume 1 Specification for Highways.
f. HS2-HS2-GT-STD-000-000001 P08 Technical Standard – Earthworks.

E7.2 Limit State Design Criteria

A 120-year design life has been specified that is consistent with the requirements of the HS2 Technical Standard - Earthworks.

Limit state partial factors have been applied in accordance with:

a. BS 8006-1:2010 Code of practice for strengthened/reinforced soils and other fills.
b. BS EN 1997-1:2004+A1:2013 Eurocode 7: Geotechnical Design inc. National Annex.

E7.3 Serviceability Design Criteria

Slope stability design has been undertaken incorporating an allowance for long term degradation of the embankment slopes and climate change in accordance with HS2 requirements reference Table 3.8/1 of the HS2 Technical Standard – Earthworks

Internal settlement of the fill will be controlled using suitable compaction and will be built out during construction. Internal drainage layers will be provided to the embankment at critical locations i.e., on the approach to major structures where control of post construction differential settlement is required.

Where required, mitigation to control settlement in the foundation soils has been specified, but no such measures are required for this asset.

In order to prevent excessive deformation in the slope face the design strength of the reinforcement has been limited to a post-construction strain of 5% under permanent loading in accordance with BS 8006:2010+A1:2016.

E7.4 Design Parameters for Soils and Materials

The highways embankments for this asset will be constructed with Class 2A4 material using excavated Gault Clay Formation. A minimum undrained shear strength of compacted Class 2A4 embankment material and Class 7C3 reinforced embankment material has been specified as 50kN/m^2 for analysis of embankment stability.

Any embankments less than a nominal 750mm above existing ground level will be constructed of either cohesive Class 2A4 fill or granular Class 1A4 fill material (Class 1/6, sub-base, and structural pavement layers) for practical and material handling requirements.

The characteristic material parameters used for the reinforced slope designs are summarised in Table E2 below.

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Table E2 – Fill Material Design Properties

Material	Unit Weight (kN/m^3)	Angle of Shearing Resistance (°)	Cohesion, c' (kN/m^2)	Undrained Shear Strength, c_u (kN/m^2)
Class 1A4	19	35	0	-
Class 2A4/7C3	19	23 [21] *	1	50

* A lower angle of shearing resistance to be adopted in the Vadose zone in line with Technical Standard -Earthworks HS2-HS2-Gt-STD-000-000001 is given in brackets [].

The design properties of the geogrids are as detailed in Appendix 6/9. The properties of the geogrids have been selected based on a design life of 120 years.

The category of the earthwork in accordance with Table 2.2/2 of Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001), the highways earthworks are designated as Geotechnical Category 2 and require Cat II checking. In accordance with BS8006-1:2010+A1:2016 they are subject to an implication factor of f_n=1.0.

E7.5 Design Groundwater Conditions

In line with Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001), an r_u of 0.20 was adopted for the Vadose Zone to be applied to the earthwork side-slopes in intermediate or high plasticity clay as no rapid run off is expected due to the pavement being a permeable surface.

An r_u of 0.1 was adopted for the embankment core (cohesive) fill.

A phreatic groundwater level was adopted at 0.0 m below the foundation level of embankment.

E7.6 Live Loadings

A short term permanent 20kPa surcharge has been adopted crest to crest for the embankment stability in the short term “construction” (undrained) condition.

For the interim (<2 years) and long term (drained) conditions, a permanent surcharge of 5 kPa is applied crest to crest of the embankment.

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E7.7 Idealised Soil Structure Model used in Analysis

A range of slope heights from 2.00m to 10.50m has been analysed to verify the stability for the reinforced soil slopes. An example reinforced slope model is summarised in Figure E1 below.

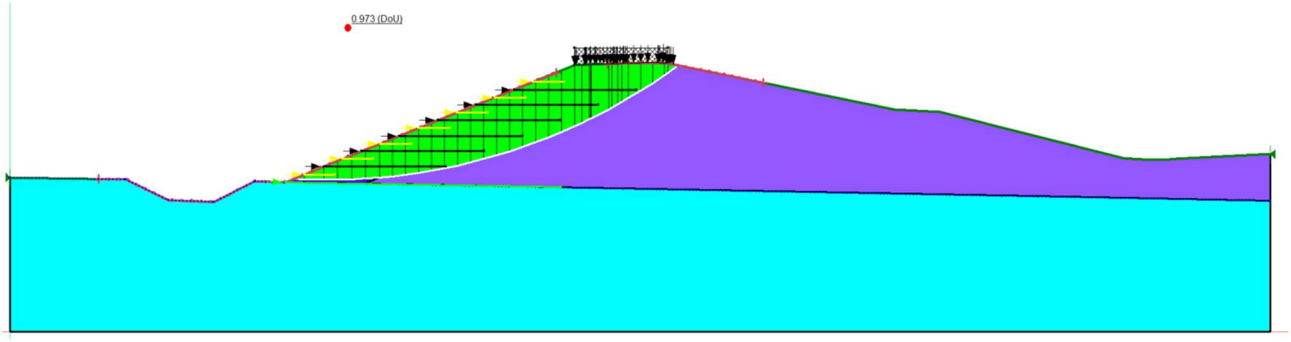


Figure E1: Example Reinforced Slope Model for Highways Earthworks

E7.8 Precautions against Chemical Attack to Materials

Extremely acidic or alkaline environments can cause chemical degradation of geosynthetic materials and a pH of between 4 and 9 is required to protect the geotextiles from Chemical Attack

Analysis of the pH range of the Gault Clays and Mudstones which is proposed for use as 7C fill is within the pH range of 8.11 to 8.42, which indicates no exceedance of the pH range.

Where specified as a Class 7C material, suitability testing of the Gault Clay Formation should be undertaken to confirm the lower and upper range of the soil pH does not exceed 4 and 9 respectively prior to placement.

E7.9 Proposed Departures from Design Standards

The highway earthworks in the Aylesbury Embankment Group area are to be constructed from reworked Gault Clay Formation. This material has a Plasticity Index (PI = 60%) higher than the normal limit of PI = 25% for cohesive fill (Class 7C) as backfill to reinforced soil in the SCEW. Note that BS 8006-1 allows the use of Class 2 fill for reinforced slopes below 45 degrees.

Departures DJV-DEP-000466, DJV-DEP-000499 and DJV-DEP-000543 as described in Section 13.2 have been accepted by HS2.

E8 Checking

The design of the highway earthworks will be subject to a Category II check in accordance with Table 2.2/2 of Technical Standard - Earthworks (HS2-HS2-GT-STD-000-000001).

E9 Drawings and Documents

E9.1 List of Drawings and Documents accompanying submission

E9.1.1 Drawings and documents

Drawings relating to the highway earthwork Design are as follows

- Oat Close Bridleway (SBH-19) Realignment – Highway General Arrangement - Highway Earthworks General Arrangement (1MC06-CEK-HW-DGA-CS04_CL18-000209)
- Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 1 (1MC06-CEK-HW-DSE-CS04_CL18-000053)
- Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 2 (1MC06-CEK-HW-DSE-CS04_CL18-000054)

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- Oat Close Bridleway (SBH-19) Realignment – Highway Earthwork Cross-Sections Sheet 3 (1MC06-CEK-HW-DSE-CS04_CL18-000101)
- Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 1 (1MC06-CEK-HW-DSE-CS04_CL18-000051)
- Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 2 (1MC06-CEK-HW-DSE-CS04_CL18-000052)
- Oat Close Bridleway (SBH-19) Realignment – Highway Cross-Sections Sheet 3 (1MC06-CEK-HW-DSE-CS04_CL18-000100)

Reference should also be made to the Standard Earthwork Drawing, Reference MC06-CEK-GT-DSE-C002-000111.

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Appendix F : I&M Summary

Number of Pages: 2

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Table F1 – Instrumentation and monitoring Schedule

Array Number	Location	Approximate Chainage (SnakeGrid)	Instrument Type	Total Instrument Length (m)	Quantity of Borehole Instrument	Time of Installation	Comments
1MC06_AEB_LM_60920_A to 1MC06_AEB_LM_60920_C	Oat Close Bridleway (SBH/19) Realignment	Eastern side ML Ch. 60+920 Bridleway SBH/19 Realignment Ch. 0+071 to 0+086	Settlement Monuments (LM)	N/A	3 no. at 0m, 5m and 15m distance from back of abutment	After Construction	Exact locations to be verified depending on construction activities.
1MC06_AEB_LM_60920_D to 1MC06_AEB_LM_60920_F		Western side ML Ch. 60+920 Bridleway SBH/19 Realignment Ch. 0+010 to 0+015 Ch. 0+005 for Footpath SBH/27 Realignment			3 no. at 0m, 5m and 17m distance from back of abutment.		

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Table F2 – Trigger Levels for Settlement

Instruments	Chainage	Trigger Levels			
		Green	Amber	Red	Black
Surface monitoring point (LM)	Mainline Ch. 60+920 both sides	75% of amber (75mm)	100 mm (predicted settlement from end of Monitoring Period 3 to end of Monitoring Period 4)	150% of amber (150mm)	200% of amber (200mm)

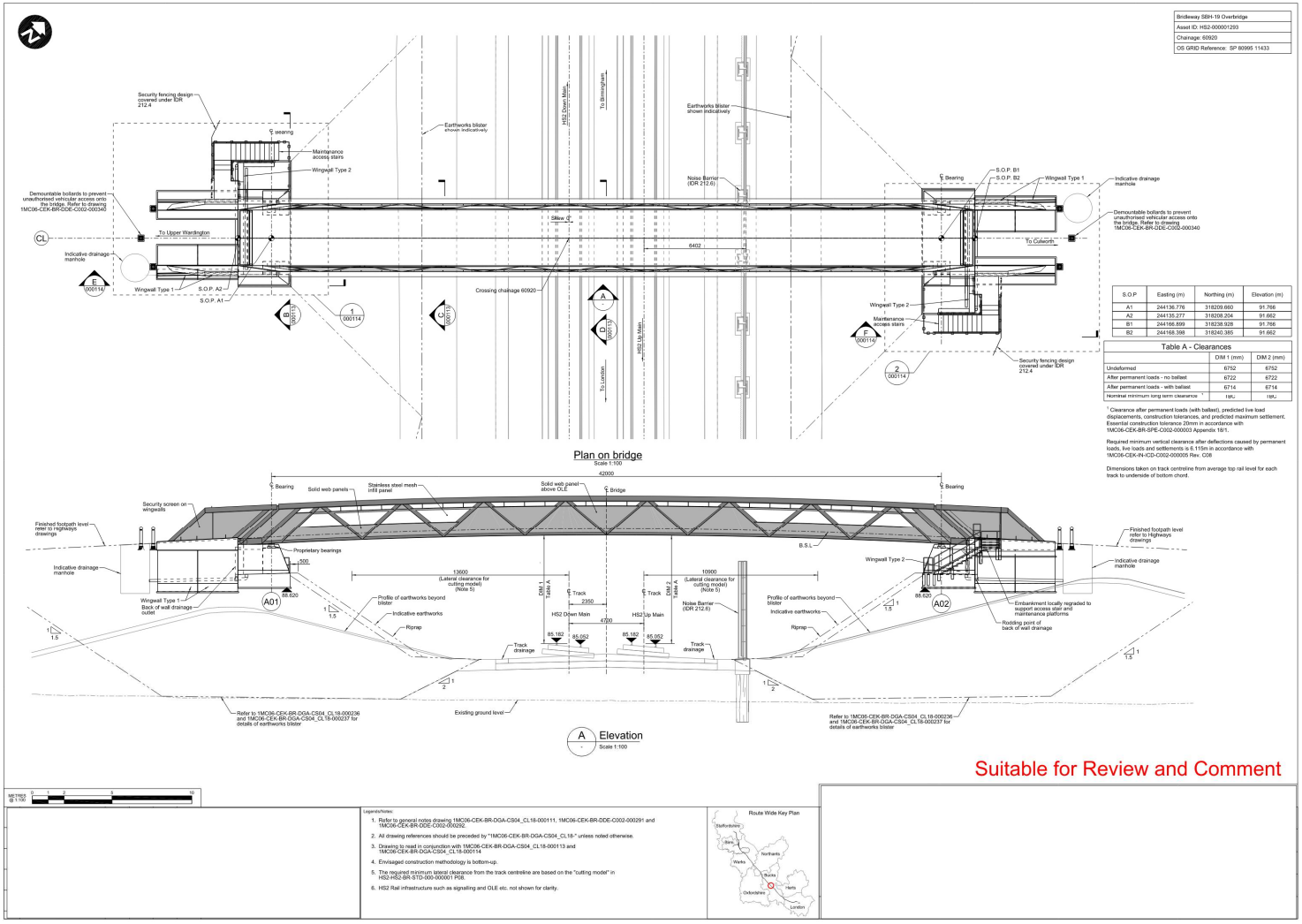
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Appendix F

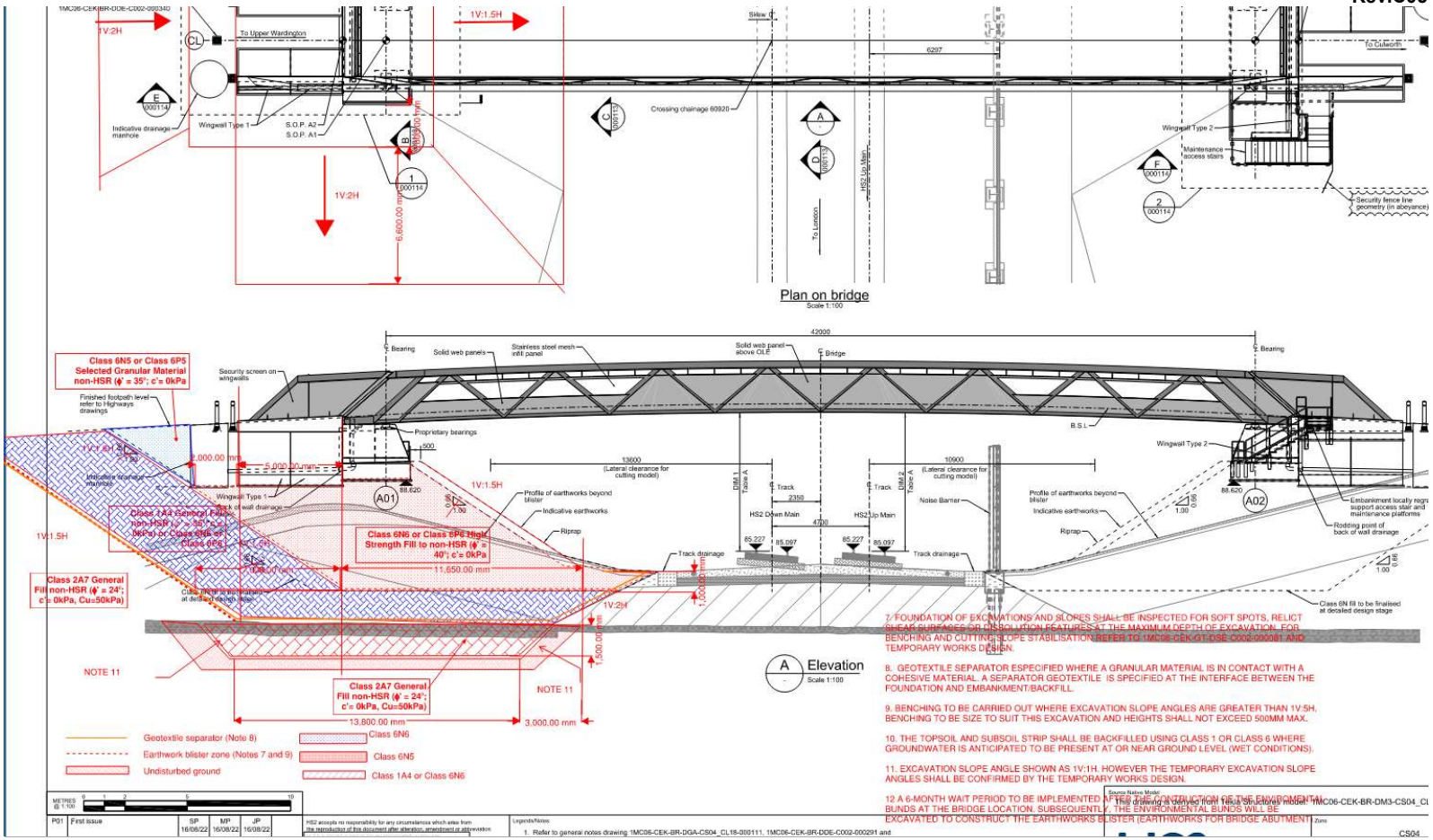
Summary of Design – Bridleway SBH/19 Overbridge HS2-000001293

Number of Pages: 25

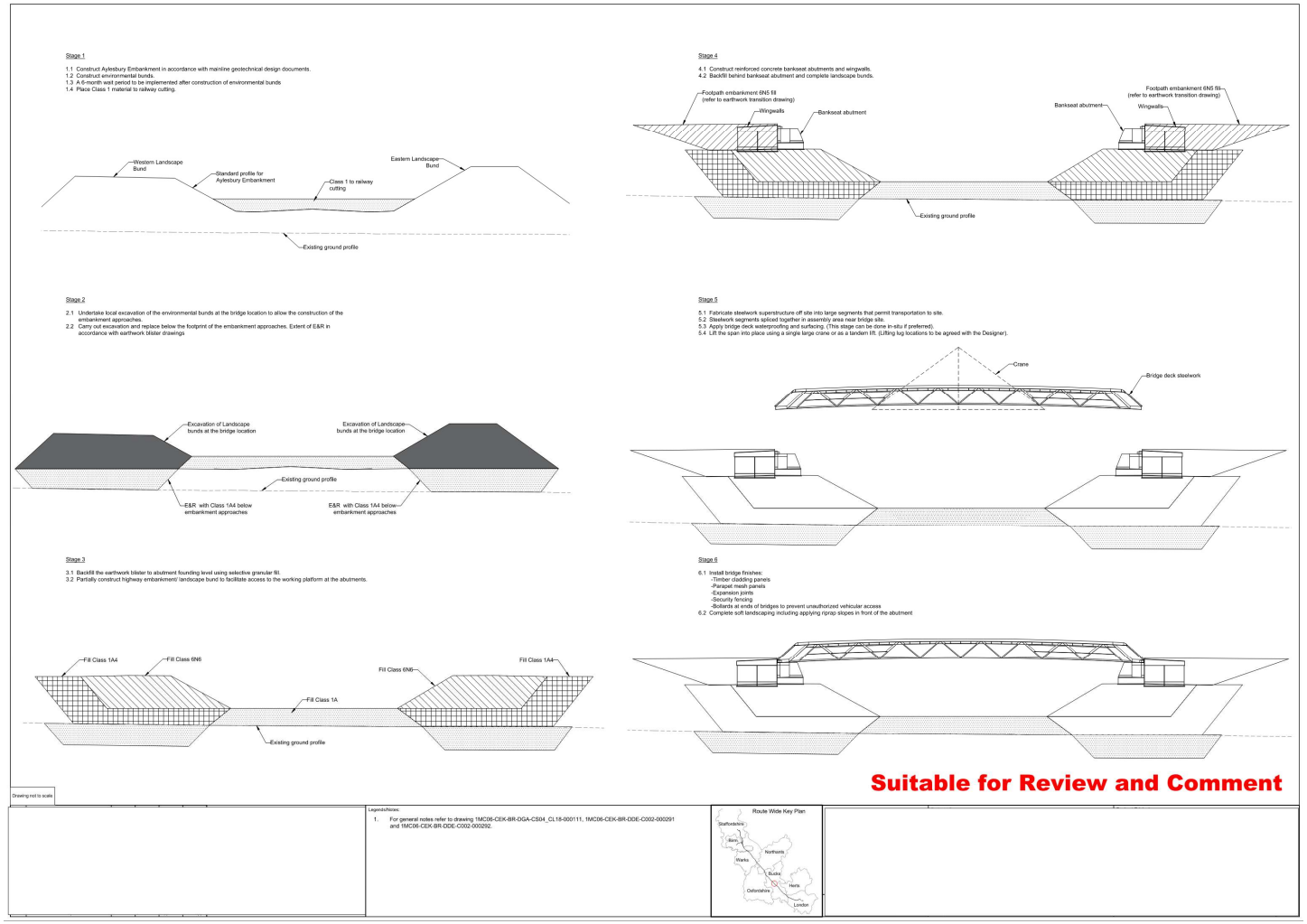
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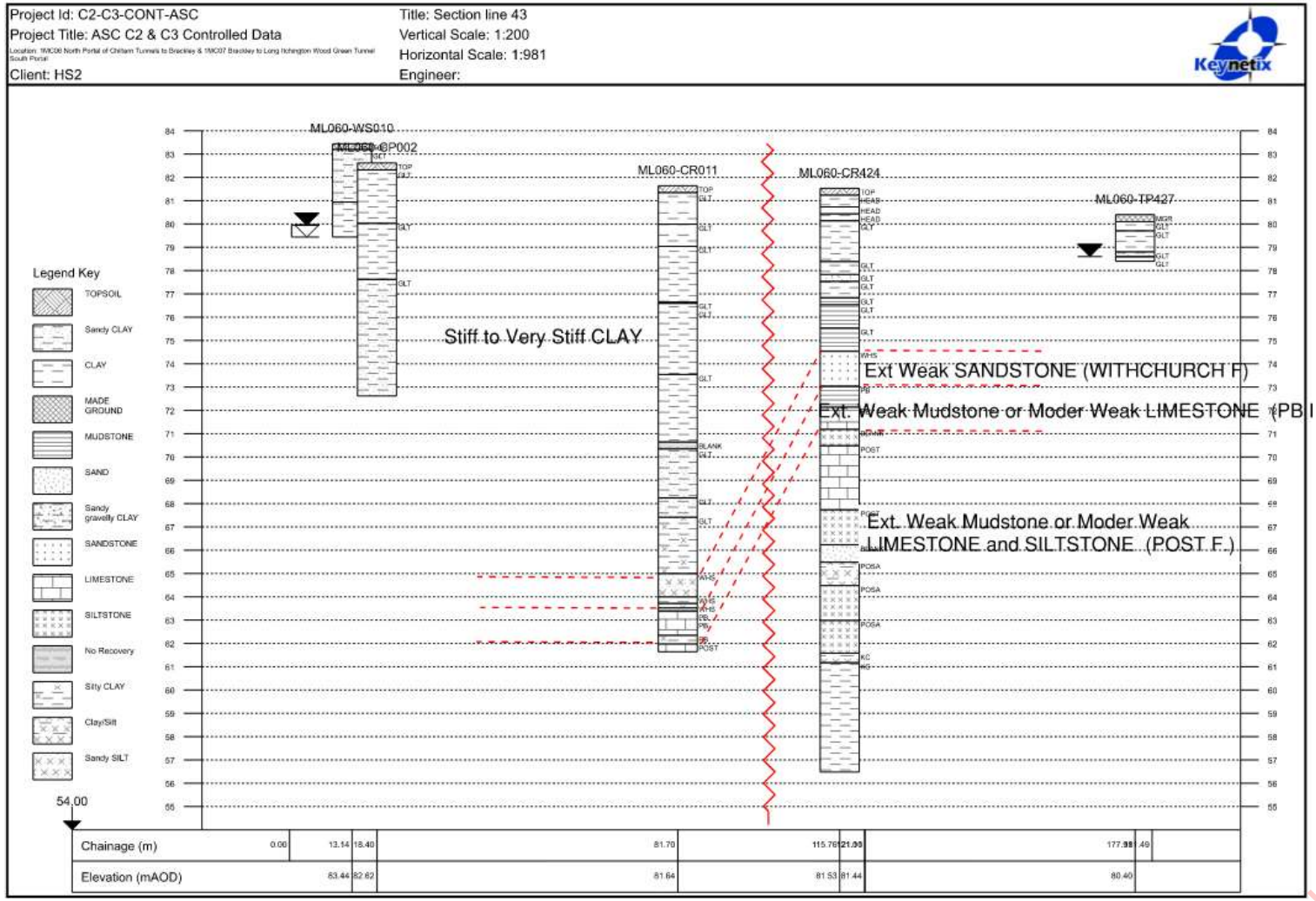
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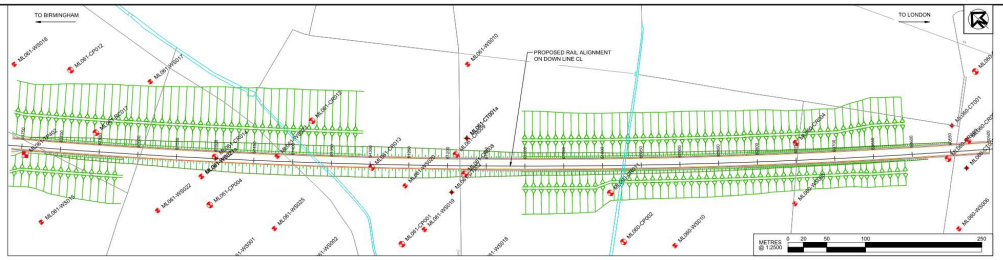
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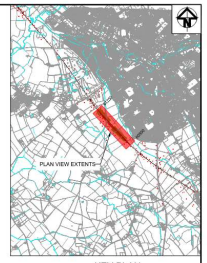
HS2 Ltd - Code 1 - Accepted



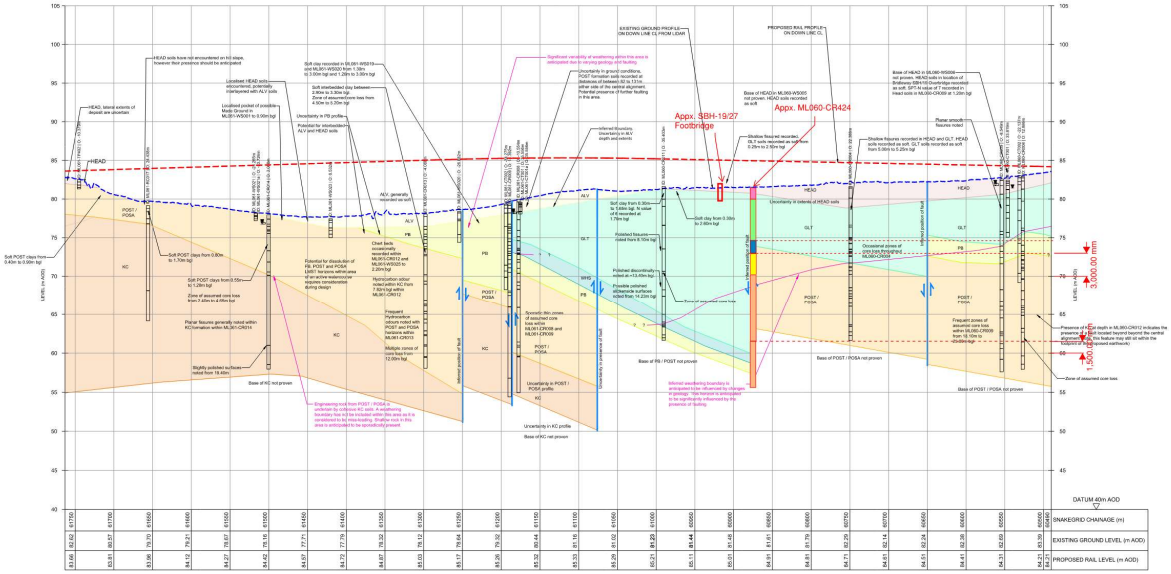
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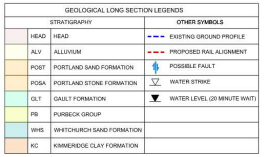
PLAN
SCALE 1:2500 at A1

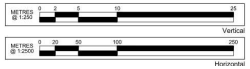


KEY PLAN
NOT TO SCALE



GEOLOGICAL LONG SECTION
HORZ 1:2500 & VERT 1:250 at A1





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Figures

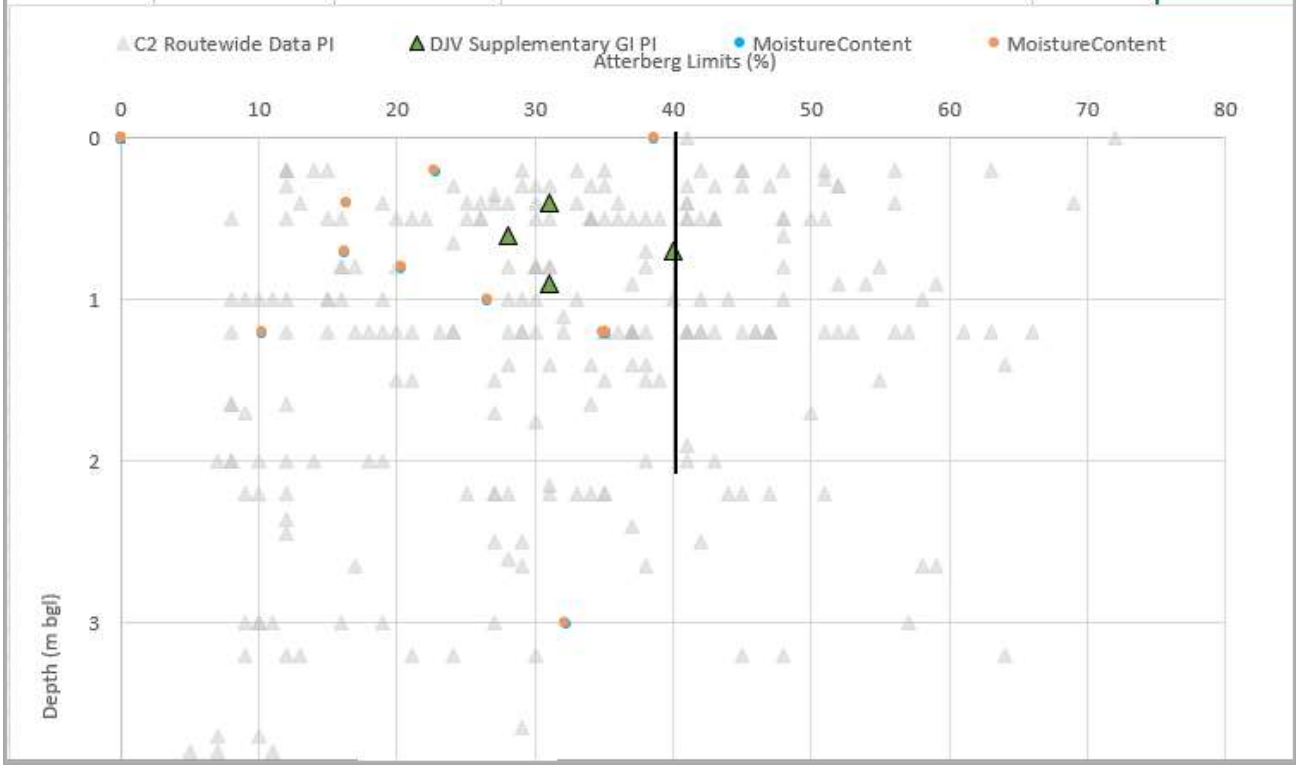
Number of Pages: 04

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HS2

Liquid limit, Plastic limit and Plasticity Index with depth (mbgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnels to Brackley	Location:	Aylesbury Embankment
Client:	HS2	Geology :	HEAD-C Z	Figure:	1

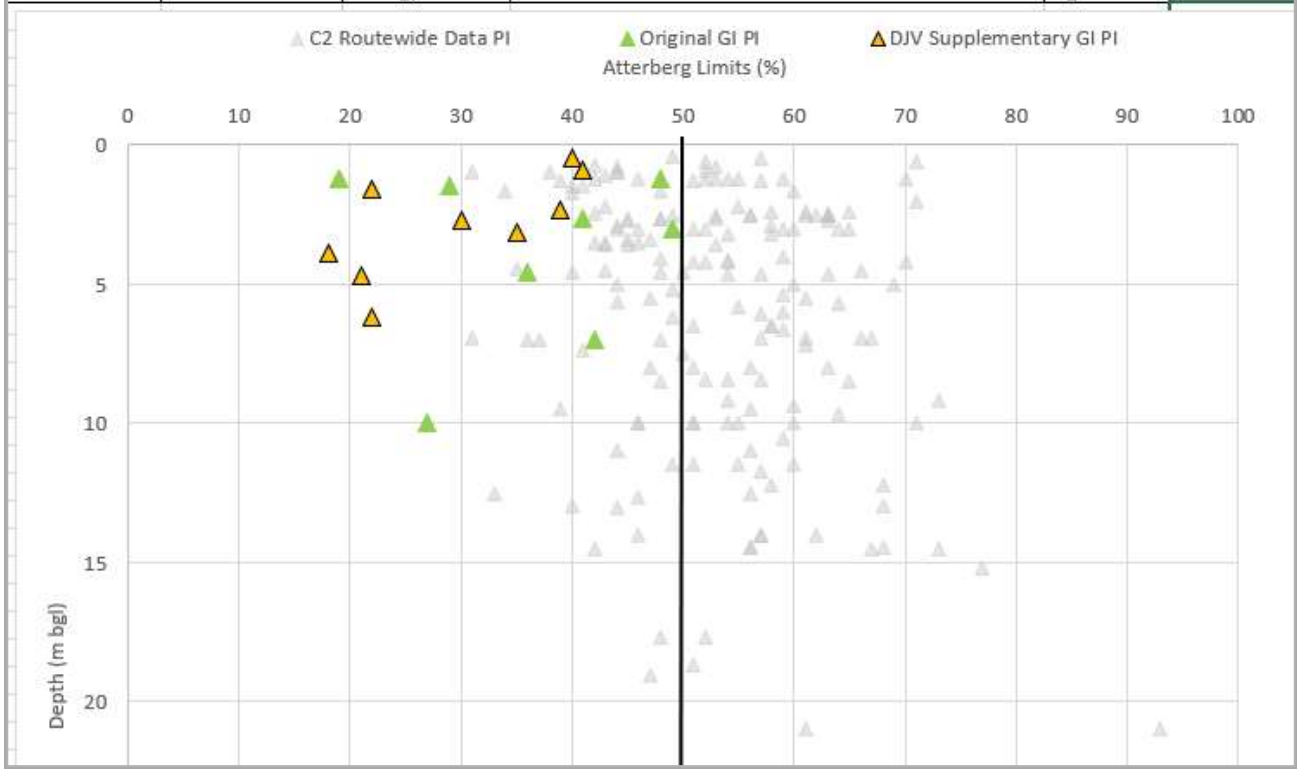


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HS2

Liquid limit, Plastic limit and Plasticity Index with depth (mbgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnels to Brackley	Location:	Aylesbury Embankment
Client:	HS2	Geology :	GLT-C 2	Figure:	2

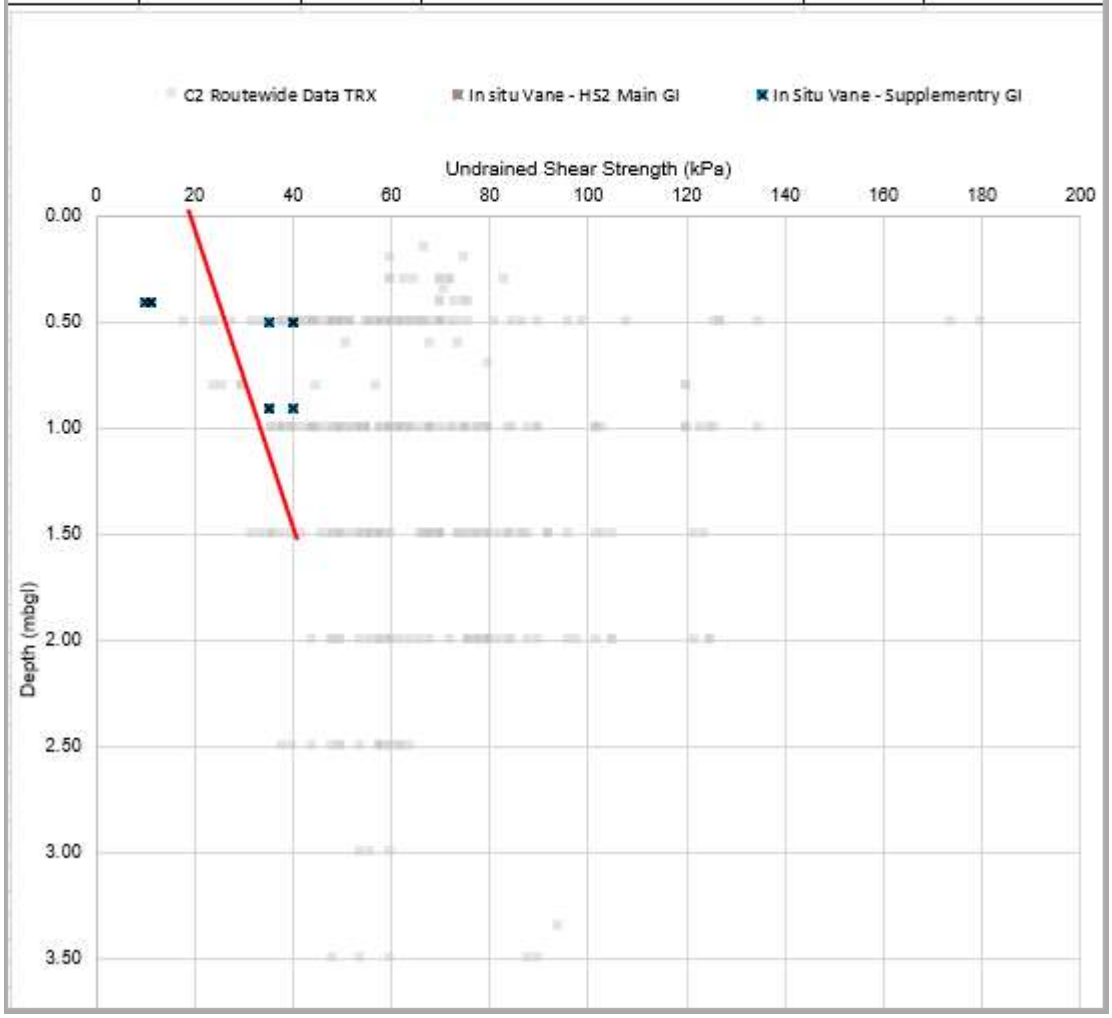


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HS2

Undrained Shear Strength vs. Depth (mbgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnels to Brackley	Location:	Aylesbury Embankment
Client:	HS2	Geology:	HEAD C Z	Figure:	3

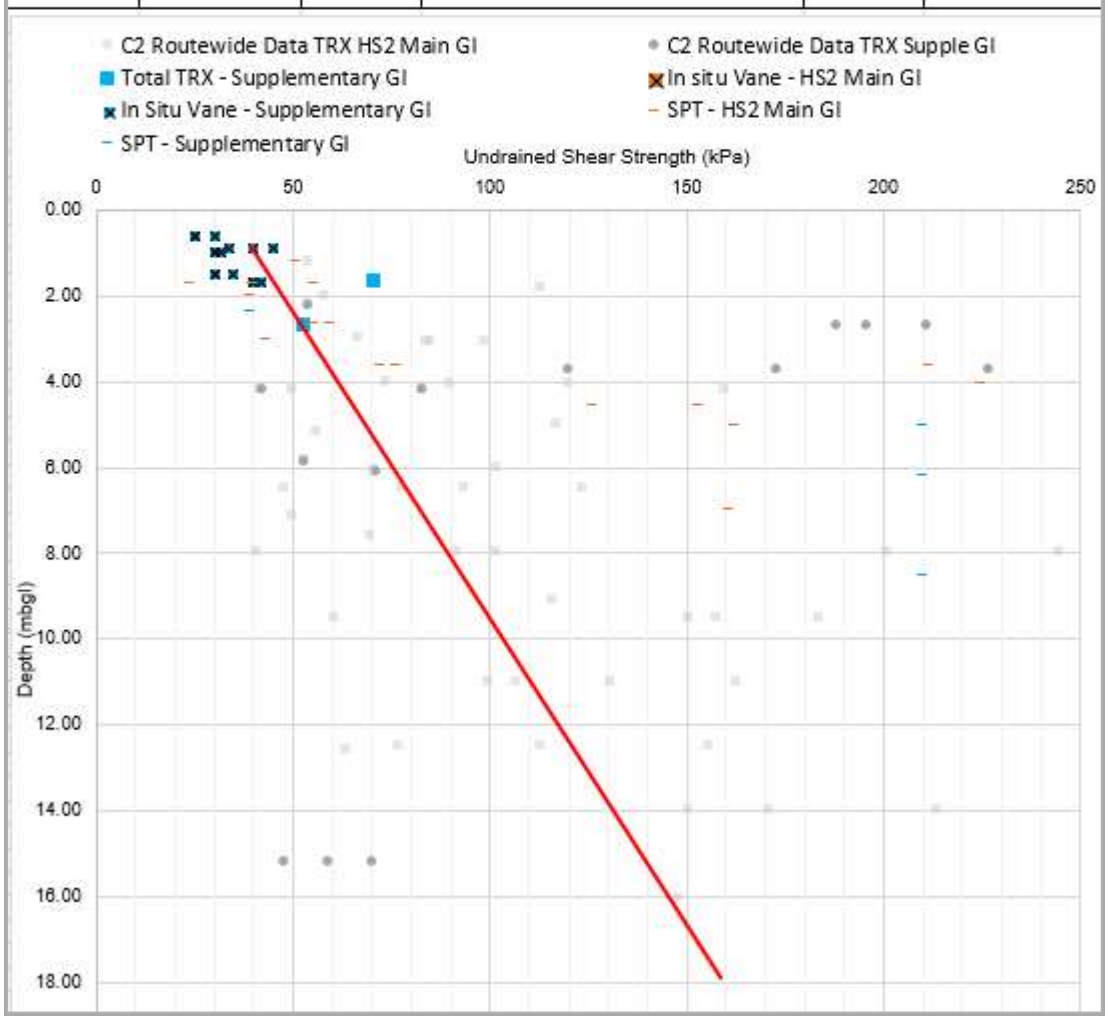


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HS2

Undrained Shear Strength vs. Depth (mbgl)

Project ID:	10039367	Project Title:	1MC06 – Stage One C2 – MWCC – North Portal of Chiltern Tunnels to Brackley	Location:	Aylesbury Embankment
Client:	HS2	Geology:	GAULT C Z	Figure:	4



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Slope/W Runs

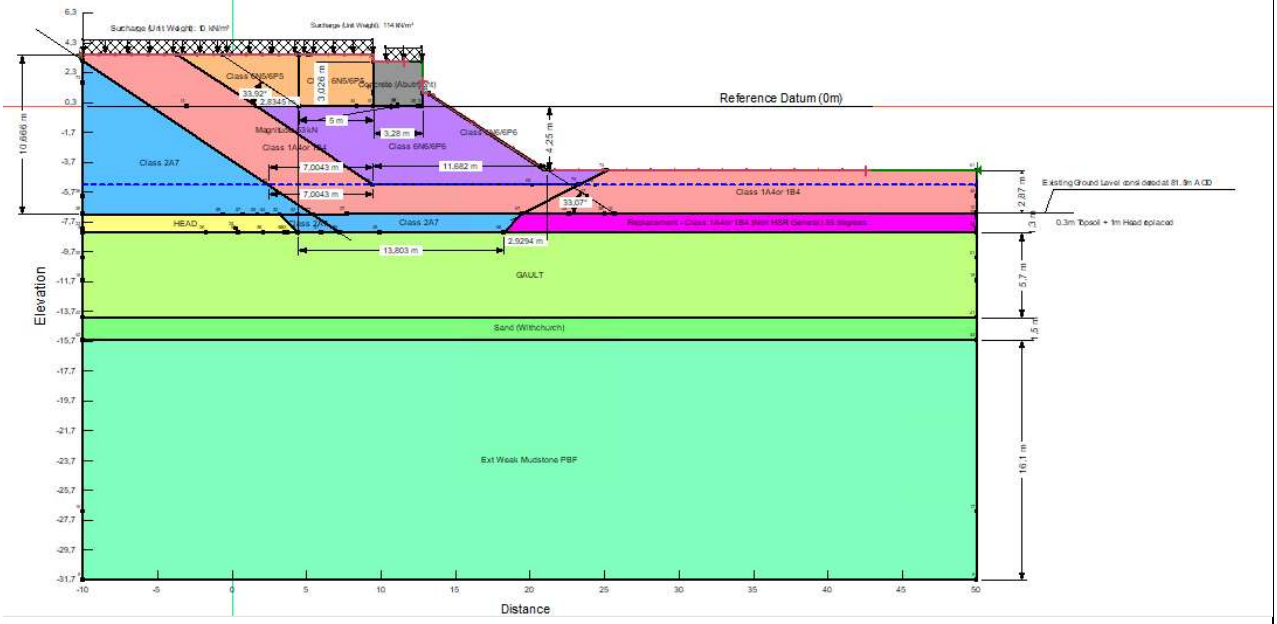
Number of Pages: 10

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Geometry East Abutment

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Class 1A4or 1B4	Mohr-Coulomb	20	0	35	0	1
	Class 2A7	Mohr-Coulomb	20	0	24	0	1
	Class 6N5/6P5	Mohr-Coulomb	20	0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20	0	40	0	1
	Concrete (Abutment)	High Strength	17				
	Ext Weak Mudstone PBF	Mohr-Coulomb	19	15	23	0	1
	GAULT	Mohr-Coulomb	19,5	0,01	21	0	1
	HEAD	Mohr-Coulomb	19,5	0	22	0	1
	Replacement - Class 1A4or 1B4 (Non HSR General) 35 degrees	Mohr-Coulomb	20	0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19	15	23	0	1

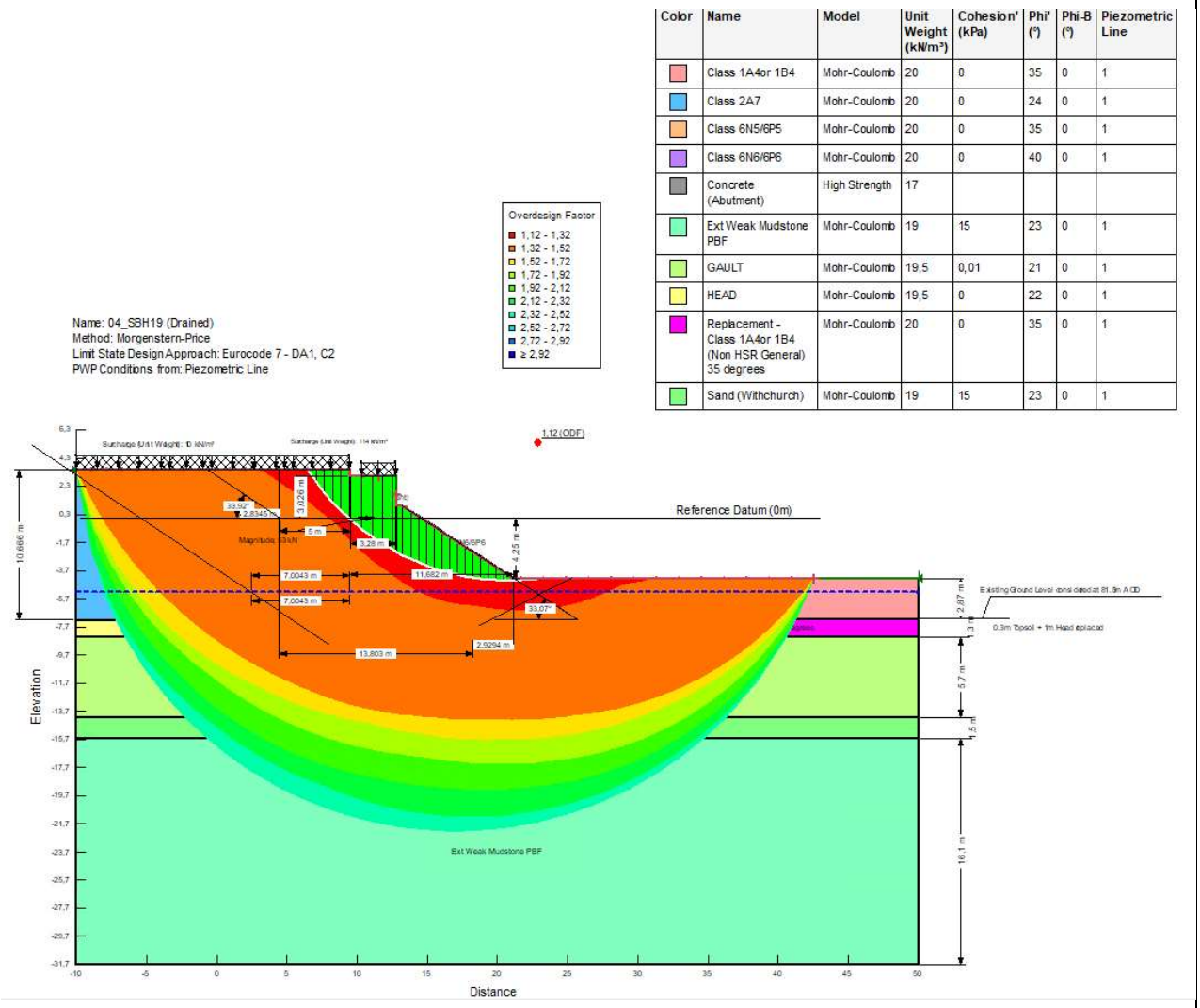
Name: 04_SBH19 (Drained)
Method: Morgenstern-Price
Limit State Design Approach: Eurocode 7 - DA1, C2
PWP Conditions from: Piezometric Line



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FIGURE 1: C/1: DA1C2 – Long Term (Drained) – FoS 1.12

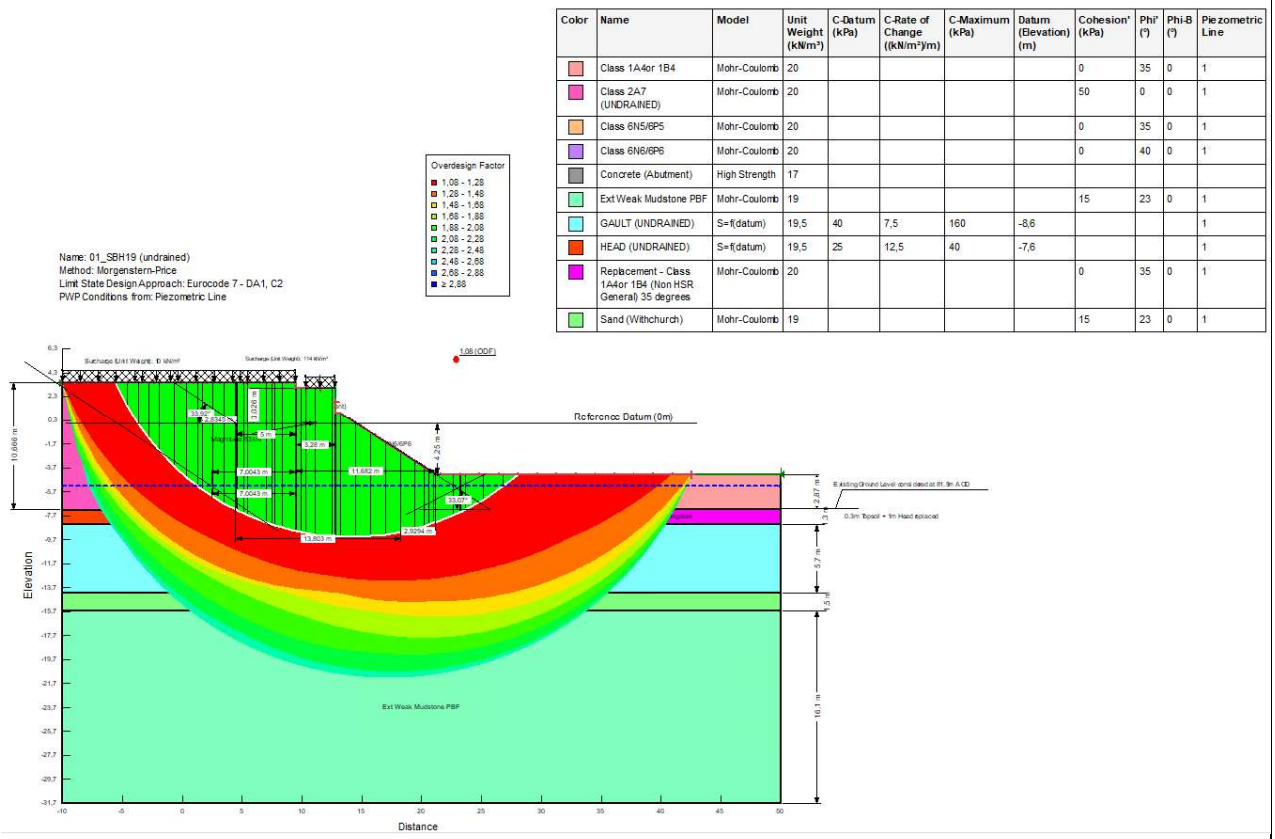
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Class 1A4or 1B4	Mohr-Coulomb	20	0	35	0	1
	Class 2A7	Mohr-Coulomb	20	0	24	0	1
	Class 6N5/6P5	Mohr-Coulomb	20	0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20	0	40	0	1
	Concrete (Abutment)	High Strength	17				
	Ext Weak Mudstone PBF	Mohr-Coulomb	19	15	23	0	1
	GAULT	Mohr-Coulomb	19,5	0,01	21	0	1
	HEAD	Mohr-Coulomb	19,5	0	22	0	1
	Replacement - Class 1A4or 1B4 (Non HSR General) 35 degrees	Mohr-Coulomb	20	0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19	15	23	0	1



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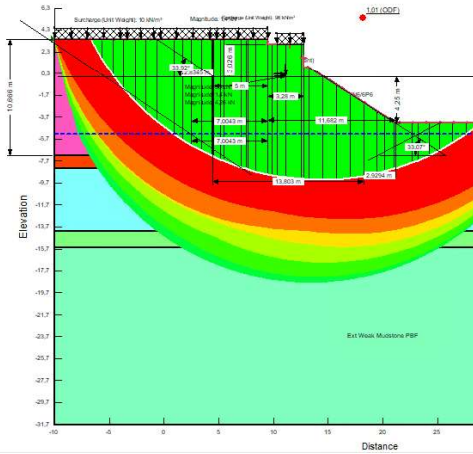
FIGURE 2: C/2: DA1C2 – Short Term (Undrained) – FoS 1.08

Color	Name	Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Class 1A4or 1B4	Mohr-Coulomb	20					0	35	0	1
	Class 2A7 (UNDRAINED)	Mohr-Coulomb	20					50	0	0	1
	Class 6N5/6P5	Mohr-Coulomb	20					0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20					0	40	0	1
	Concrete (Abutment)	High Strength	17								
	Ext Weak Mudstone PBF	Mohr-Coulomb	19					15	23	0	1
	GAULT (UNDRAINED)	S=f(datum)	19,5	40	7,5	160	-8,6				1
	HEAD (UNDRAINED)	S=f(datum)	19,5	25	12,5	40	-7,6				1
	Replacement - Class 1A4or 1B4 (Non HSR General) 35 degrees	Mohr-Coulomb	20					0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19					15	23	0	1



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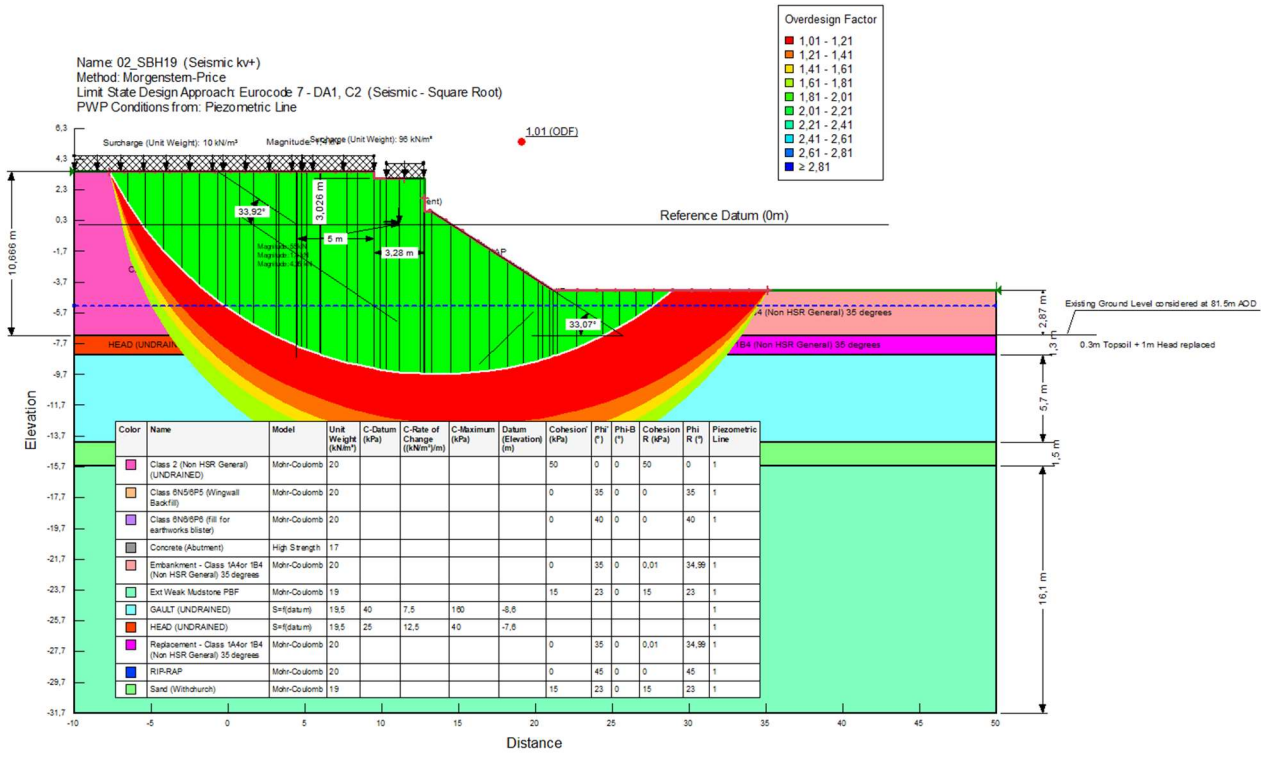
FIGURE 3: C/3: DA1C2 – Short Term Seismic Kv- (Undrained) – FoS 1.01



Color	Name	Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Class 1A4or 1B4	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Class 2A7 (UNDRAINED)	Mohr-Coulomb	20					50	0	0	50	0	1
	Class 6N5/6P5	Mohr-Coulomb	20					0	35	0	0	35	1
	Class 6N6/6P6	Mohr-Coulomb	20					0	40	0	0	40	1
	Concrete (Abutment)	High Strength	17										
	Ext Weak Mudstone PBF	Mohr-Coulomb	19					15	23	0	15	23	1
	GAULT (UNDRAINED)	S= f(datum)	19,5	40	7,5	160	-8,6						1
	HEAD (UNDRAINED)	S= f(datum)	19,5	25	12,5	40	-7,6						1
	Replacement - Class 1A4or 1B4 (Non HSR General) 35 degrees	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Sand (Withchurch)	Mohr-Coulomb	19					15	23	0	15	23	1

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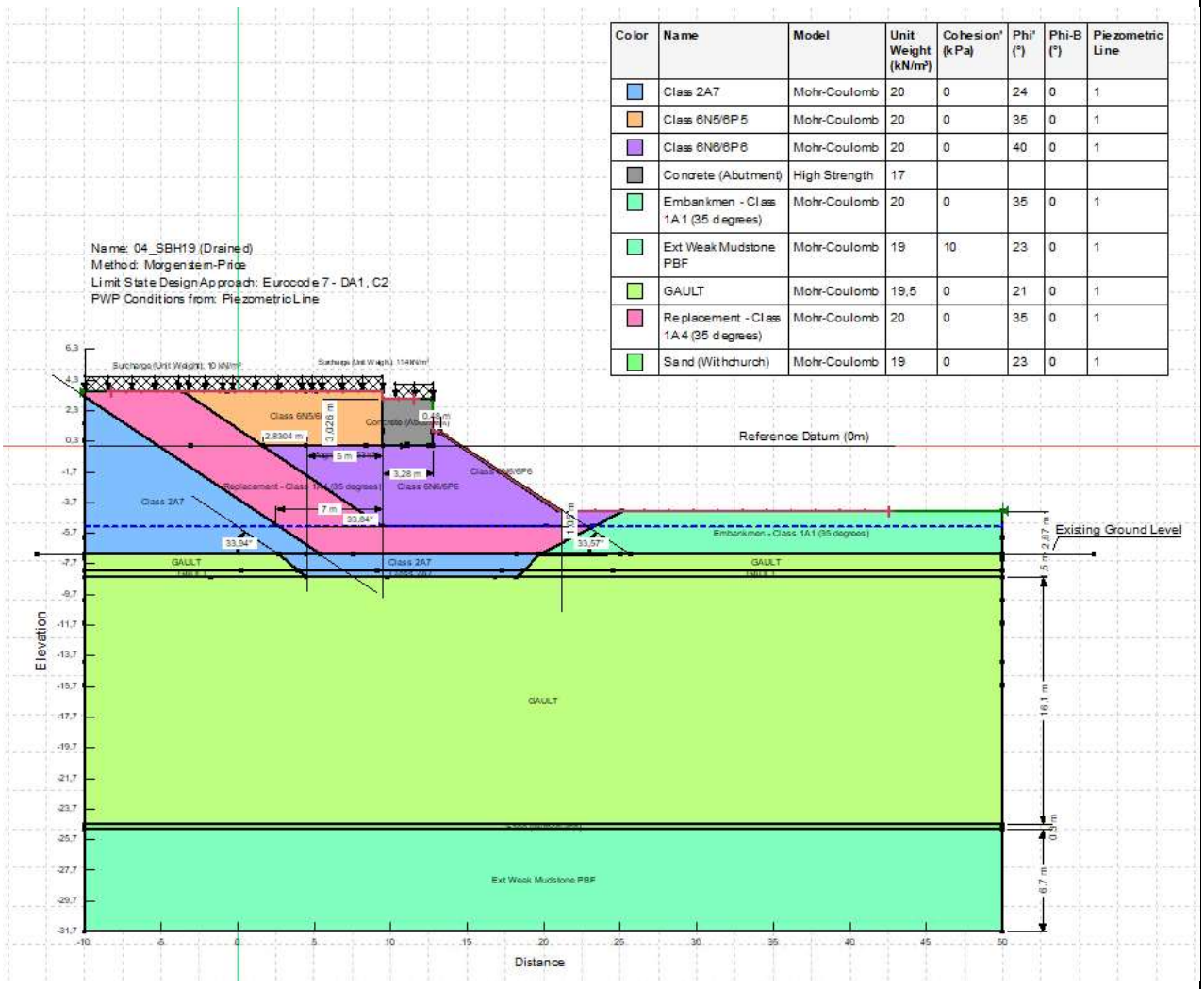
FIGURE 4: C/4: Circle DA1C2 – Short Term Seismic Kv+ (Undrained) – FoS 1.01



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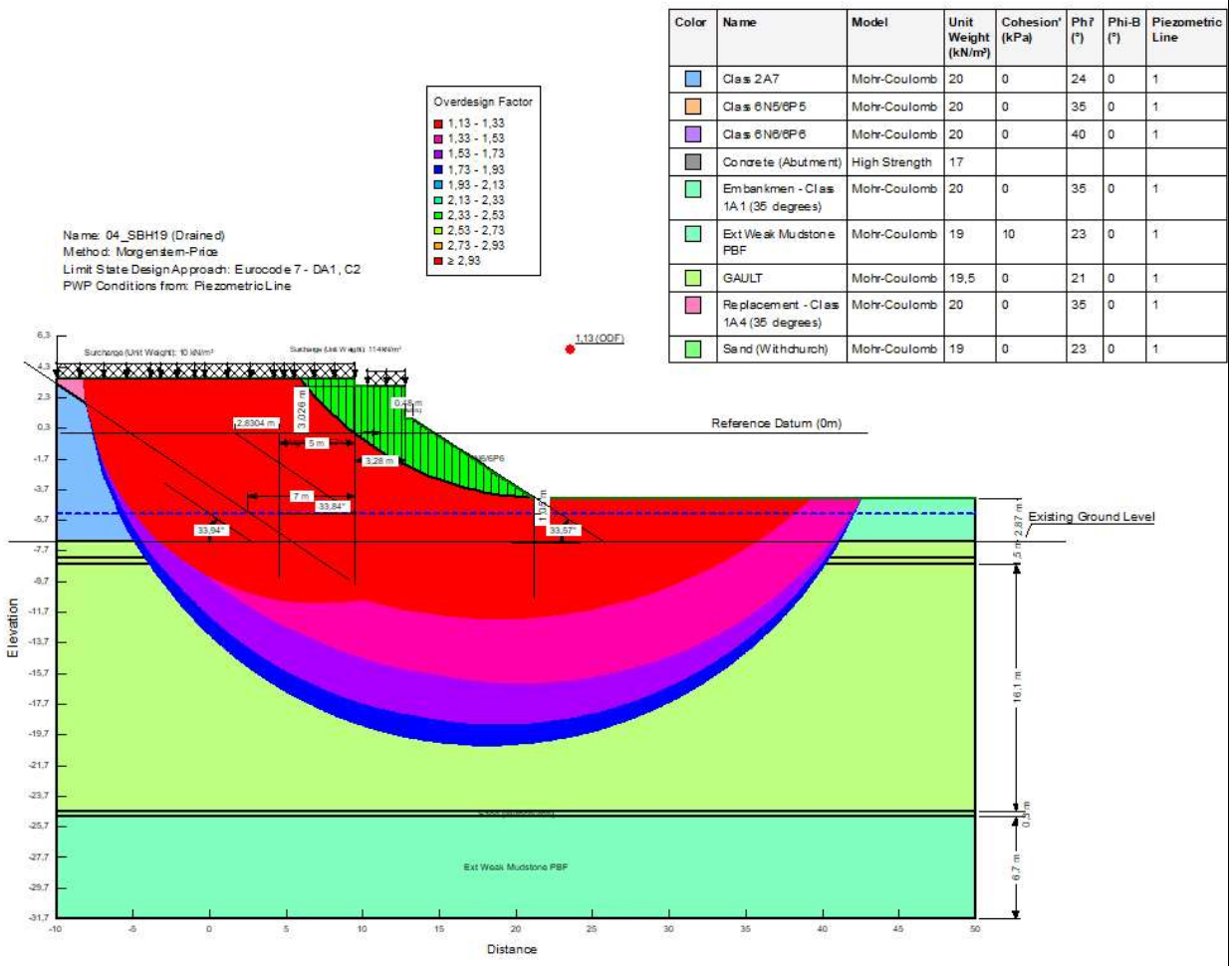
Geometry West Abutment

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Class 2A7	Mohr-Coulomb	20	0	24	0	1
	Class 6N5/6P5	Mohr-Coulomb	20	0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20	0	40	0	1
	Concrete (Abutment)	High Strength	17				
	Embankmen - Class 1A1 (35 degrees)	Mohr-Coulomb	20	0	35	0	1
	Ext Weak Mudstone PBF	Mohr-Coulomb	19	10	23	0	1
	GAULT	Mohr-Coulomb	19,5	0	21	0	1
	Replacement - Class 1A4 (35 degrees)	Mohr-Coulomb	20	0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19	0	23	0	1



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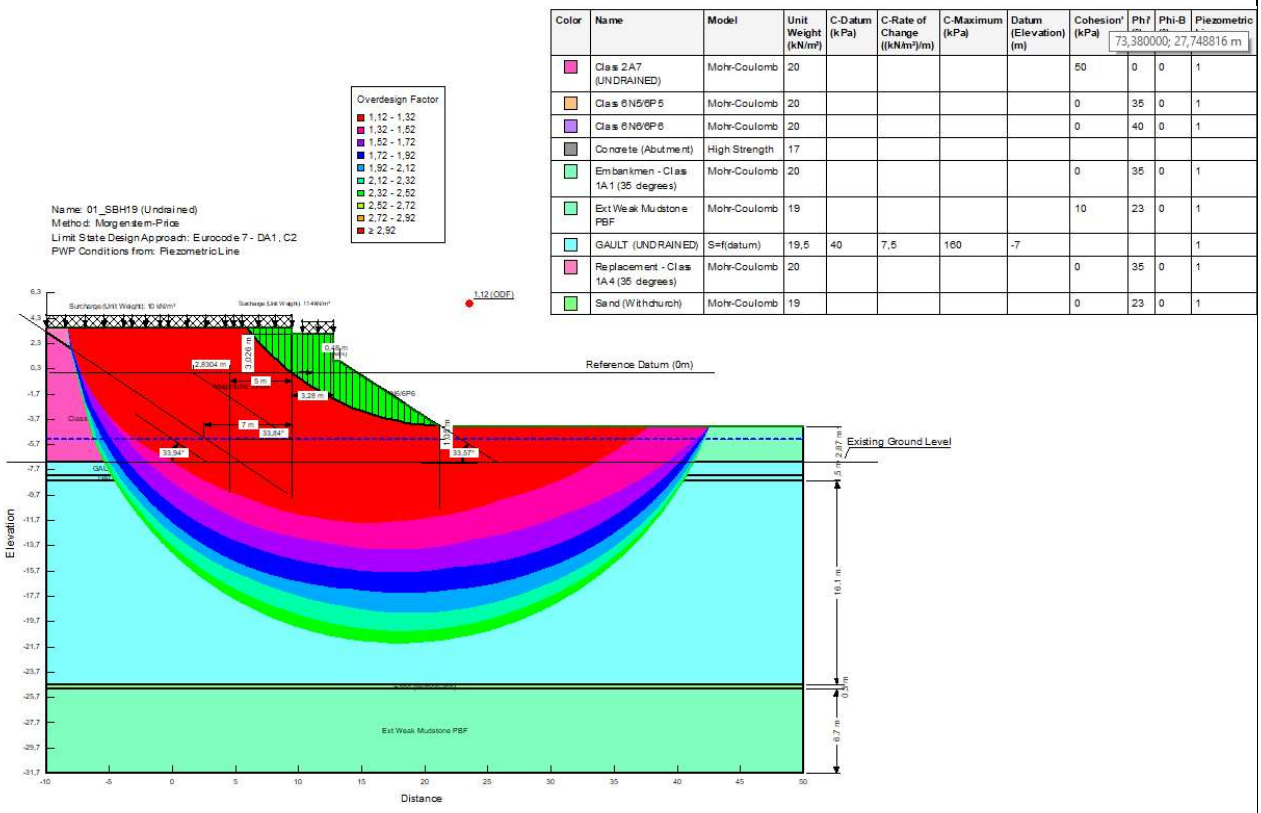
FIGURE 5: C/1: DA1C2 – Long Term (Drained) – FoS 1.13



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Class 2A7	Mohr-Coulomb	20	0	24	0	1
	Class 6N5/6P5	Mohr-Coulomb	20	0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20	0	40	0	1
	Concrete (Abutment)	High Strength	17				
	Embankmen - Class 1A1 (35 degrees)	Mohr-Coulomb	20	0	35	0	1
	Ext Weak Mudstone PBF	Mohr-Coulomb	19	10	23	0	1
	GAULT	Mohr-Coulomb	19,5	0	21	0	1
	Replacement - Class 1A4 (35 degrees)	Mohr-Coulomb	20	0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19	0	23	0	1

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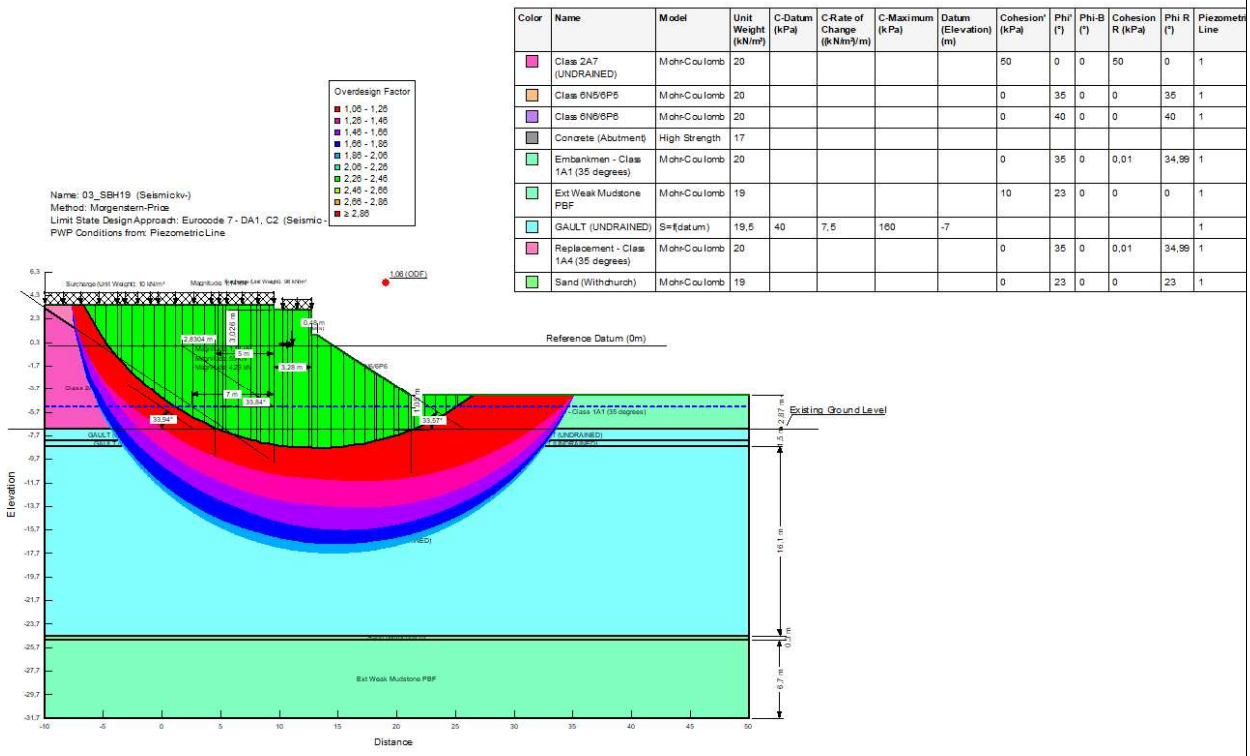
FIGURE 6: C/2: DA1C2 – Short Term (Undrained) – FoS 1.12



Color	Name	Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Cohesion' (kPa)	Phi'	Phi-B	Piezometric
	Class 2A7 (UNDRAINED)	Mohr-Coulomb	20					50	0	0	1
	Class 6N5/6P5	Mohr-Coulomb	20					0	35	0	1
	Class 6N6/6P6	Mohr-Coulomb	20					0	40	0	1
	Concrete (Abutment)	High Strength	17								
	Embankmen - Class 1A1 (35 degrees)	Mohr-Coulomb	20					0	35	0	1
	Ext Weak Mudstone PBF	Mohr-Coulomb	19					10	23	0	1
	GAULT (UNDRAINED)	S=f(datum)	19,5	40	7,5	160	-7				1
	Replacement - Class 1A4 (35 degrees)	Mohr-Coulomb	20					0	35	0	1
	Sand (Withchurch)	Mohr-Coulomb	19					0	23	0	1

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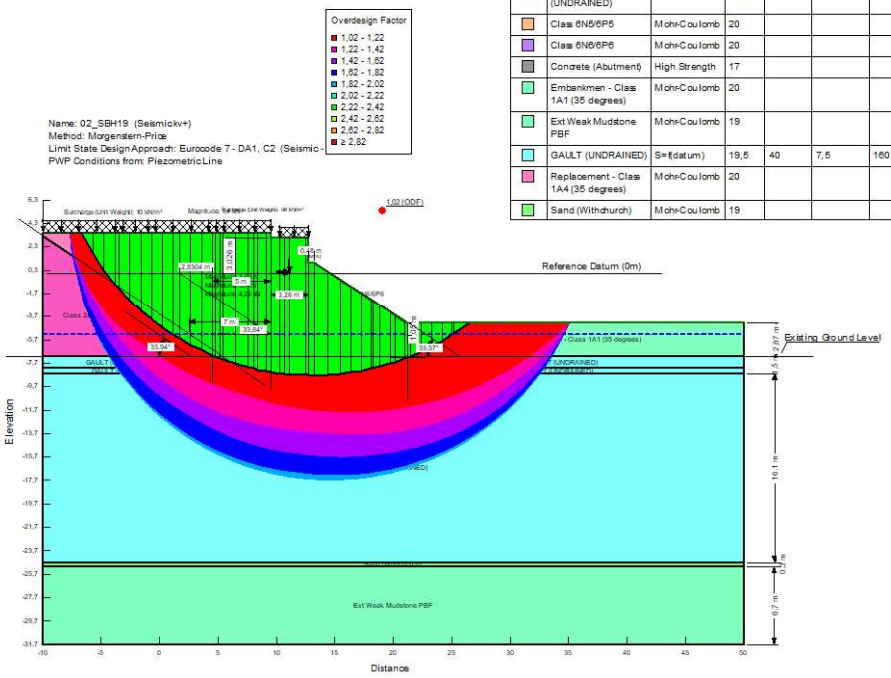
FIGURE 7: C/3: DA1C2 – Short Term Seismic Kv- (Undrained) – FoS 1.06



Color	Name	Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Class 2A7 (UNDRAINED)	Mohr-Coulomb	20					50	0	0	50	0	1
	Class 6N5/6P5	Mohr-Coulomb	20					0	35	0	0	35	1
	Class 6N6/6P6	Mohr-Coulomb	20					0	40	0	0	40	1
	Concrete (Abutment)	High Strength	17										
	Embankmen - Class 1A1 (35 degrees)	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Ext Weak Mudstone PBF	Mohr-Coulomb	19					10	23	0	0	0	1
	GAULT (UNDRAINED)	S=f(datum)	19,5	40	7,5	160	-7						1
	Replacement - Class 1A4 (35 degrees)	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Sand (Withchurch)	Mohr-Coulomb	19					0	23	0	0	23	1

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FIGURE 8: C/4: Circle DA1C2 – Short Term Seismic Kv+ (Undrained) – FoS 1.02



Color	Name	Model	Unit Weight (kN/m³)	C-Datum (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Datum (Elevation) (m)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Class 2A7 (UNDRAINED)	Mohr-Coulomb	20					50	0	0	50	0	1
	Class 6N5/6P5	Mohr-Coulomb	20					0	35	0	0	35	1
	Class 6N6/6P6	Mohr-Coulomb	20					0	40	0	0	40	1
	Concrete (Abutment)	High Strength	17										
	Embankmen - Class 1A1 (35 degrees)	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Ext Weak Mudstone PBF	Mohr-Coulomb	19					10	23	0	0	0	1
	GAULT (UNDRAINED)	S=f(datum)	19,5	40	7,5	160	-7						1
	Replacement - Class 1A4 (35 degrees)	Mohr-Coulomb	20					0	35	0	0,01	34,99	1
	Sand (Withchurch)	Mohr-Coulomb	19					0	23	0	0	23	1

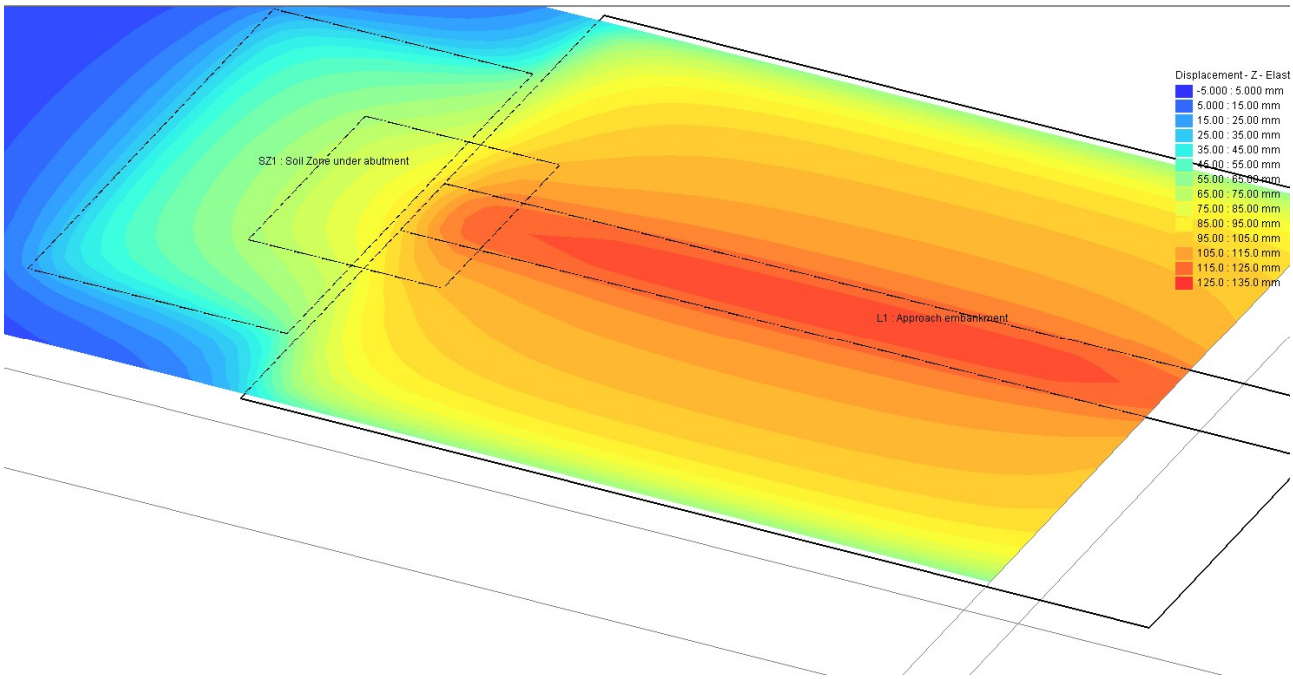
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PDips Runs

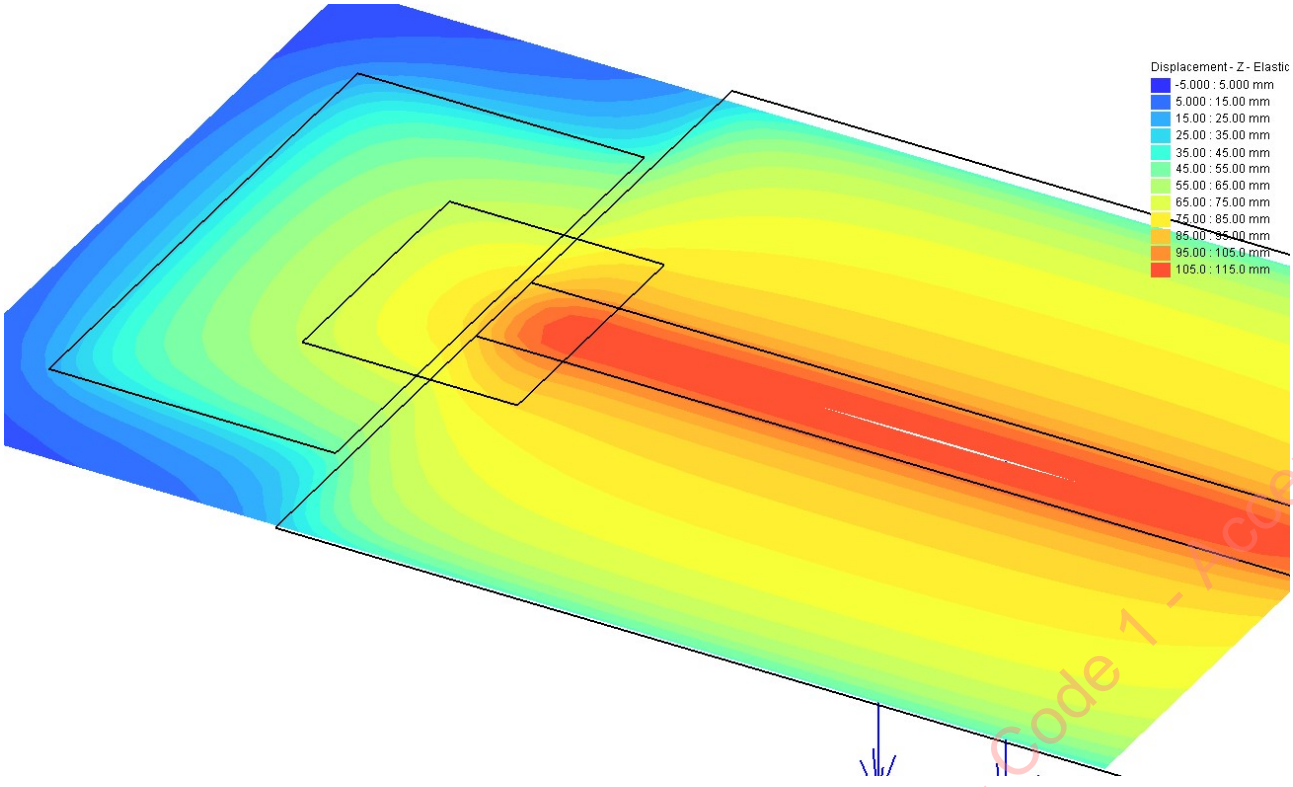
Number of Pages: 2

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CASE 1 East Abutment - Total Settlement

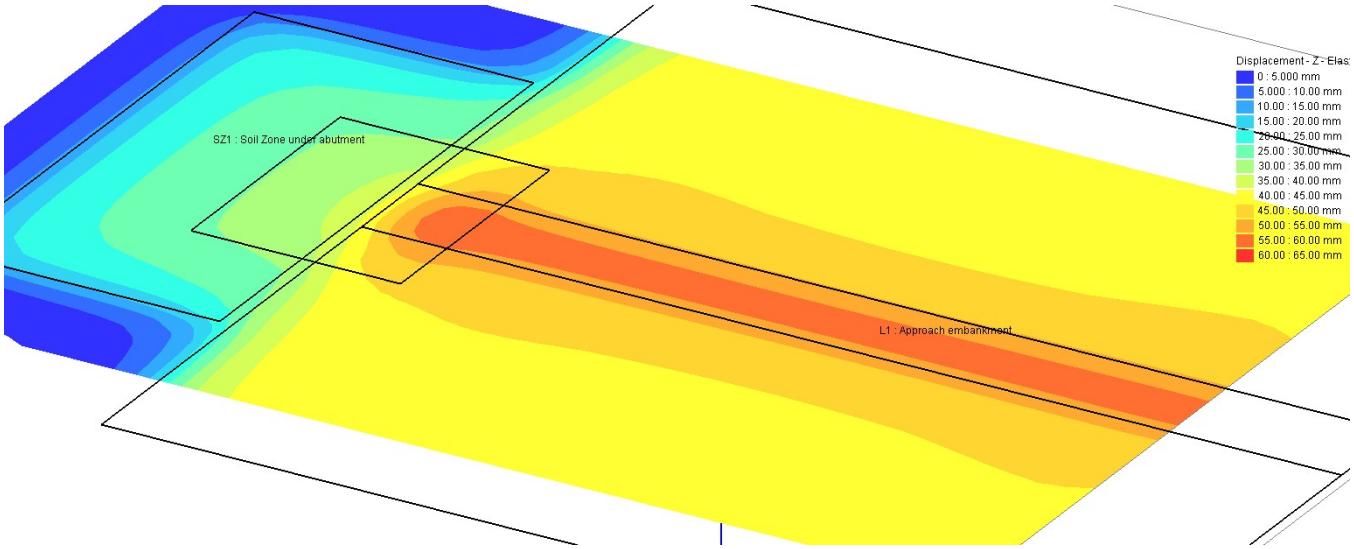


CASE 1 West Abutment - Total Settlement

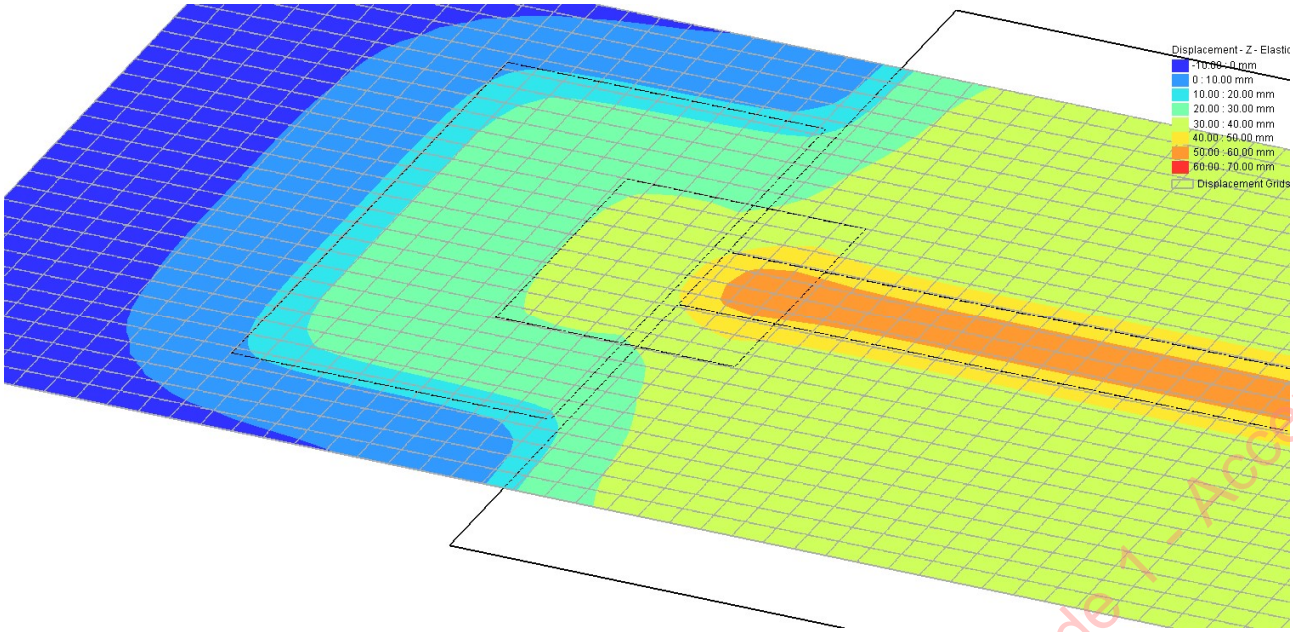


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CASE 2 East Abutment - Total Settlement



CASE 2 West Abutment - Total Settlement



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Appendix G

Summary of Design – Sedrup North Watercourse Diversion

Number of Pages: 14

HS2 Ltd - Code 1 - Accepted

Ch. 12.634 U/S

Solution: Class 1A E&R 0.7m wide and minimum 1.5m deep required for left bank
Class 1A E&R 1m wide and 1.5m deep required for right bank

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 1A Fill	Mohr-Coulomb	20	0	35	1
	Class 2A Highway fill	Mohr-Coulomb	19	0	25	1
	Portland Sand/Stone Formation	Mohr-Coulomb	20	2	26	1
	Weathered Portland Sand/Stone Formation	Mohr-Coulomb	20	1	23	1

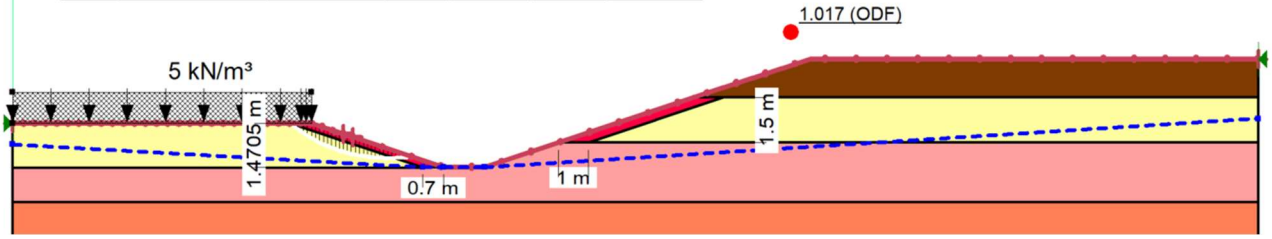


Figure G-1 Slope/W Output for Ch. 12.634 Left Bank

Ch. 12.634 U/S

Solution: Class 1A E&R 0.7m wide and minimum 1.5m deep required for left bank
Class 1A E&R 1m wide and 1.5m deep required for right bank

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 1A Fill	Mohr-Coulomb	20	0	35	1
	Class 2A Highway fill	Mohr-Coulomb	19	0	25	1
	Portland Sand/Stone Formation	Mohr-Coulomb	20	2	26	1
	Weathered Portland Sand/Stone Formation	Mohr-Coulomb	20	1	23	1

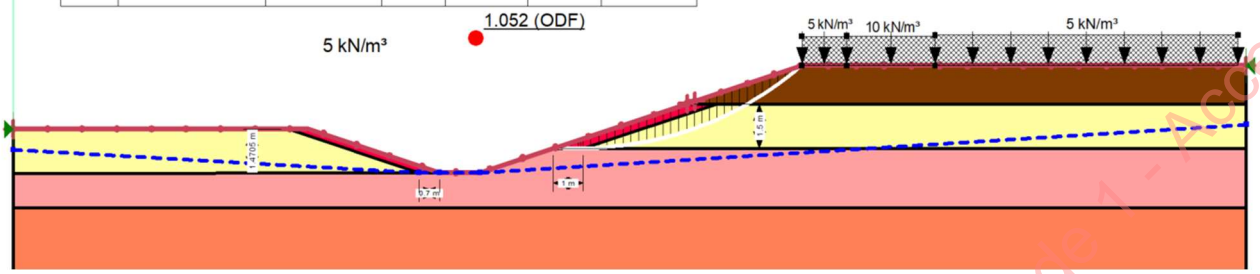


Figure G-2 Slope/W Output for Ch. 12.634 Right Bank

HS2 Ltd - Code 1 - Accepted

Ch. 60 U/S

Solution: No mitigation required for Left Bank
2.0m wide and minimum 1.1m deep E&R with Class 1A Granular Fill for Right Bank

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	18	0	21	1
	Class 1A Fill	20	0	35	1
	Purbeck Group	20	2.5	22	1

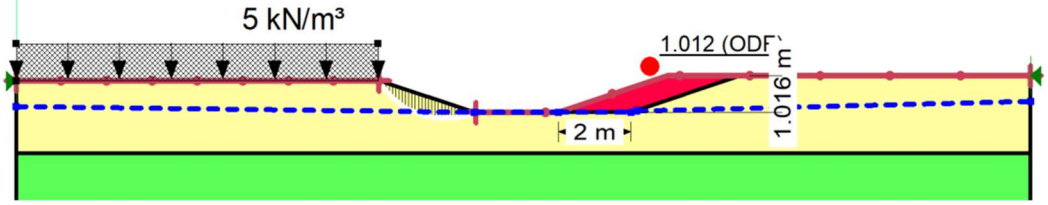


Figure G-3 Slope/W Output for Ch. 60 Left Bank

Ch. 60 U/S

Solution: No mitigation required for Left Bank
2.0m wide and minimum 1.1m deep E&R with Class 1A Granular Fill for Right Bank

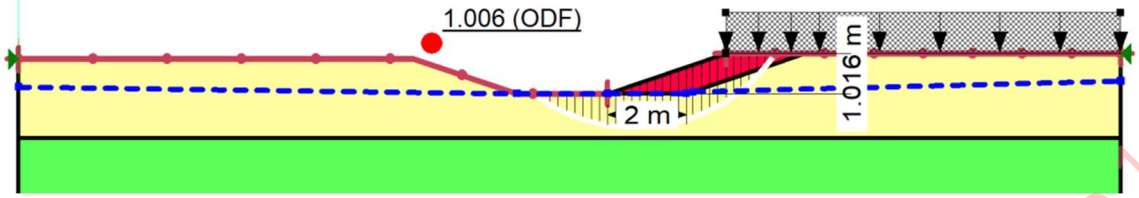


Figure G-4 Slope/W Output for Ch. 60 Right Bank

HS2 Ltd - Code 1 - Accepted

Ch. 80 U/S

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 1A Fill	Mohr-Coulomb	20	0	35	1
	Purbeck Group	Mohr-Coulomb	20	2.5	22	1

Solution: Left bank mitigation E&R with Class 1A 0.7m wide and upto 1.5m deep
Right bank mitigation E&R with Class 1A 2m wide and 1.4m deep

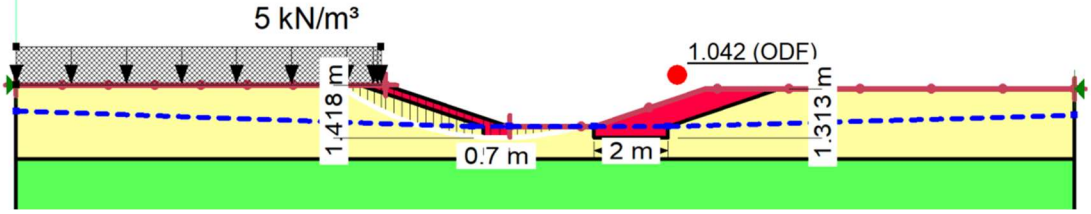


Figure G-5 Slope/W Output for Ch. 80 Left Bank

Ch. 80 U/S

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 1A Fill	Mohr-Coulomb	20	0	35	1
	Purbeck Group	Mohr-Coulomb	20	2.5	22	1

Solution: Left bank mitigation E&R with Class 1A 0.7m wide and upto 1.5m deep
Right bank mitigation E&R with Class 1A 2m wide and 1.4m deep

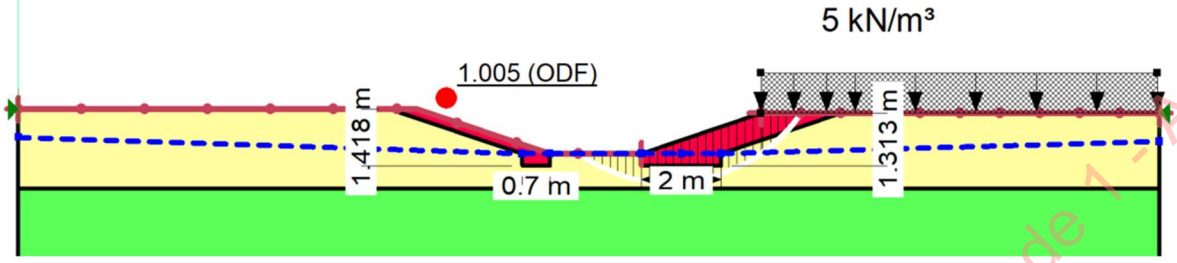


Figure G-6 Slope/W Output for Ch. 80 Right Bank

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061-AC3 Culvert Headwall

Solution: For both banks: 3m long geogrids (3.76m including length within the Flex MSE bags) at 0.42m vertical spacing (3 bags spacing) with 3.5 wide Class 6I/6J backfill.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 6 Fill	Mohr-Coulomb	20	0	35	1
	Flex MSE Support	Mohr-Coulomb	17	2.6	24	1
	Portland Sand/Stone Formation	Mohr-Coulomb	20	2	26	1
	Weathered Portland Sand/Stone Formation	Mohr-Coulomb	20	1	23	1

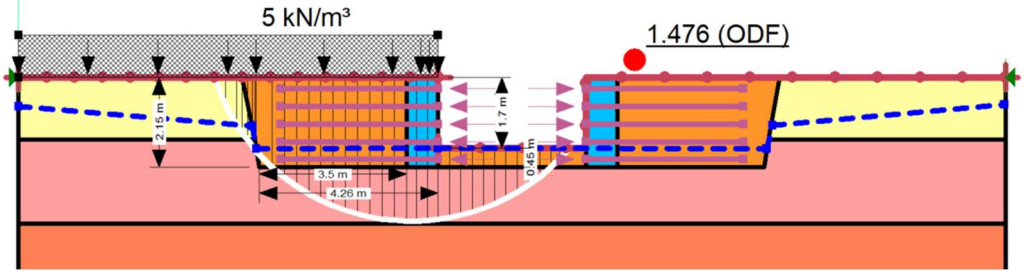


Figure G-7 Slope/W Output for 061-AC3 Ancillary Culvert Headwall

HS2 Ltd - Code 1 - Accepted

Sedrup North Headwall

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
	Alluvium	Mohr-Coulomb	18	0	21	1
	Class 1A Fill	Mohr-Coulomb	20	0	35	1
	Class 6 Fil	Mohr-Coulomb	20	0	35	1
	Flex MSE Support	Mohr-Coulomb	17	2.6	24	1
	Portland Sand/Stone Formation	Mohr-Coulomb	20	2	26	1
	Weathered Portland Sand/Stone Formation	Mohr-Coulomb	20	1	23	1

Solution: For both banks: 3.1m long geogrids (3.86m including length within the Flex MSE bags) at 0.28m vertical spacing (2 bags spacing) with 3.6 wide Class 6I/6J backfill.

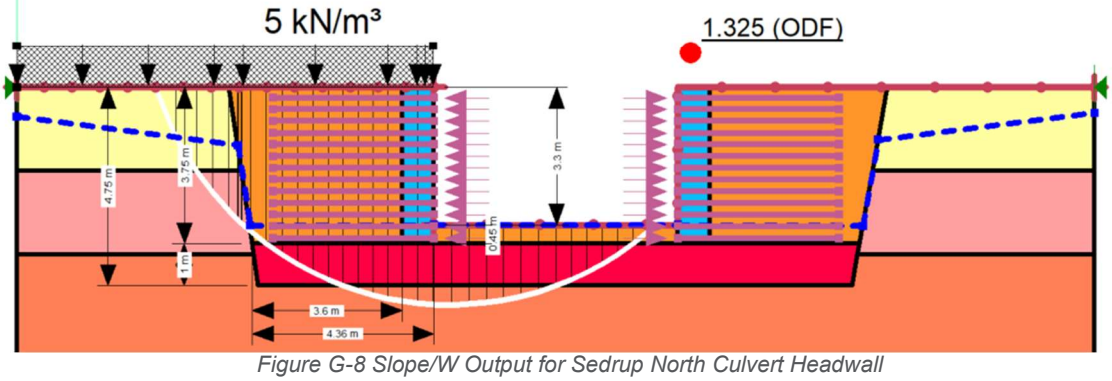


Figure G-8 Slope/W Output for Sedrup North Culvert Headwall

HS2 Ltd - Code 1 - Accepted

Bearing Capacity Check for Sedrup North Culvert Headwall

CHARACTERISTIC SOIL PARAMETERS	Symbol	Units	INPUT
Bulk density (overburden soil)	γ_b	kN/m^3	18.0
Saturated density (overburden soil)	γ_{sat}	kN/m^3	18.5
Bulk density (foundation soil)	γ_b	kN/m^3	19.5
Saturated density (foundation soil)	γ_{sat}	kN/m^3	20.0
Density of water	γ_w	kN/m^3	10.00
Undrained shear strength	$c_{u.k}$	kN/m^2	100.0
Effective cohesion	c'_k	kN/m^2	2.0
Angle of shearing resistance	$ø'_k$	degree	26.0

FOUNDATION DETAILS	Symbol	Units	INPUT
Unit weight of Flex MSE	γ_{conc}	kN/m^3	17.0
Width	B	m	4.300
Length	L	m	5.000
Base thickness	D_B	m	0.50
Depth (ground to founding level)	D	m	1.45

ADDITIONAL DESIGN DETAILS	Symbol	Units	INPUT
Depth of groundwater (from ground level)	D_w	m	0.00
Slope gradient	β	degree	0.00
Foundation inclination	α	degree	0.00

DESIGN ACTIONS	Input loads			Total load		
	DA1 C1	DA1 C2	BS	DA1 C1	DA1 C2	BS
Vertical		1827			1827	
Horizontal		424			424	
Moment		542			542	

Bearing Capacity Check DESIGN SUMMARY

Method	DA1C1	DA1C2	BS8004
Base		Compression	
R_d (Drained)		OK	
R_d (Undrained)		OK	
Utilisation Factor, Δ (%)		73%	

RESULTS

R_d/A' (Drained)= kN/m^2

R_d/A' (Undrained)= kN/m^2

DESIGN ASSUMPTIONS

- Ground condition below foundation extends to depth > B
- Does not calculate bearing resistance of;
  - stronger soil overlying weaker soil
  - weaker soil overlying stronger soil
  - two way eccentrically loaded foundation
- BS8004 method, FOS = 1.00 on P_{max}
- Allow Depth Factor ? NO
- Design groundwater level taken at ground level

HS2 Ltd - Code 1 - Accepted

BS 8006 Internal Stability Checks for Sedrup North Culvert Headwall

2.2 Sliding Along the Base

Horizontal Disturbing Force (Rh)

The horizontal disturbing force is calculated as the sum of the active pressure behind the block (Pa) and other horizontal loadings.

		Characteristic	
Ka (Soil 2)=		0.455	
γ(dry)=		18	
γsat=		18.5	
H (above GWT)=		1.065	
H (Below GWT)=		2.235	
Pa(dry)=		4.64426081	kN/m
Pa(sat)=	19.49281	34.376	kN/m

$P_a = \frac{1}{2} k_a \gamma H^2$ Barnes (2000)

Where:

$k_a = \frac{OC[1-sin(\phi')]}{OC[1+sin(\phi')]} = \frac{1-sin(\phi')}{1+sin(\phi')} = tan^2\left[45 - \frac{\phi'}{2}\right]$ Barnes (2000)

Factored horizontal loadings:

	A	B	C	
Earth Hori-Force from soil 2	87.76989	87.769885	58.51326	kN/m
Traffic load behind the reinfoced soil	11.2603	11.260303	0	kN/m

Horizontal Resisting Force (HRF)

HRF provided by a soil/soil interface for the base of the retained soil slab.

$f_s R_h \le R_v \frac{\tan\phi'_p}{f_{ms}} + \frac{c'}{f_{ms}} L$

	Drained, long term					
	HRF for Fill (soil 1)			HRF for Foundation		
	A	B	C	A	B	C
tanϕ'p/fms=	0.700	0.700	0.700	0.700	0.700	0.700
c'/fms=	0	0	0	0	0	0
L=	3.86	3.86	3.86	3.86	3.86	3.86
Unfactored beneficial Rv=	272.0142	272.0142	272.0142	272.0142	272.0142	272.0142
HRF=	190.466	190.466	190.466	190.466	190.466	190.466
fs	1.2	1.2	NA	1.2	1.2	NA
fsRh=	118.836	118.836	check disp	118.836	118.836	check disp
Utilisation of HRF =	62.39%	62.39%		62.39%	62.39%	

$f_s R_h \le \frac{c_u}{f_{ms}} L$

	Undrained, short term			
	HRF for Foundation			
	A	B	C	
cu/fms=	0	0	0	
L=	3.86	3.86	3.86	
HRF=	0	0	0	
fs	1.2	1.2	NA	
fsRh=	118.836	118.836	check disp	
	#DIV/0!	#DIV/0!		

3.2 Local Stability Check - Rupture

		Layer 1			Layer 2		
		ULS, A	Unfactored	SLS, C	ULS, A	Unfactored	SLS, C
Tensile Force, Tj	kN/m	1.200476	0.8003173	0.43883	13.74186	9.1612379	8.559613
Long term (design life) Reinforcement base strength in accordance with BS8006 A.3 and PD ISO/TR 20432, TD (Huesker 35T geogrid)	kN/m	17	17	10	17	17	10
fn		1.1	1.1	1.1	1.1	1.1	1.1
TD/fn		15.45455	15.454545	9.09091	15.45455	15.454545	9.090909
Utilisation of TD	%	7.77%	5.18%	4.83%	88.92%	59.28%	94.16%

HS2 Ltd - Code 1 - Accepted

3.3 *Local Stability Check, Adherence*

$$P_j \geq \frac{T_j}{\frac{\mu L_{ej}(f_{fs}\gamma_1 h_j + f_f w_s)}{f_p f_n} + \frac{a'_{bc} c' L_{ej}}{f_{ms} f_p f_n}}$$

where

- P_j is the total horizontal width of the top and bottom faces of the reinforcing element at the jth layer, per metre "run";
- T_j is the maximum value from **6.6.3.2.1**;
- f_{fs} is the partial load factor applied to soil self weight taken from the same load combination as T_j, see Table 11;
- f_f is the partial load factor applied to surcharge dead loads taken from the same load combination as T_j, see Table 11;
- μ is the coefficient of friction between the fill and reinforcing elements;
- L_{ej} is the length of reinforcement in the resistant zone outside failure wedge, at the jth layer of reinforcements, see Figure 31;
- w_s is the surcharge due to dead loads only;
- f_p is the partial factor for reinforcement pull-out resistance, see Table 11;
- f_n is the partial factor applied to economic ramifications of failure, see Table 9;
- a'_{bc} is the adhesion coefficient between the soil and the reinforcement;
- c' is the cohesion of the soil measured under effective stress conditions;
- f_{ms} is the partial material factor applied to c', see Table 11.

	Layer 1			Layer 2		
	A	B	C	A	B	C
Tj	1.200476	0.8003173	0.43883	13.74186	9.1612379	8.559613
γ1hj	5.6	5.6	5.6	65.63	65.63	65.63
μ (subject to testing)	0.8	0.8	0.8	0.8	0.8	0.8
Lej	1.528	1.528	1.528	2.985	2.985	2.985
ws	0	0	0	0	0	0
fp	1.3	1.3	1	1.3	1.3	1
fn	1.1	1.1	1.1	1.1	1.1	1.1
a'bc (subject to testing)	1	1	1	1	1	1
c'	0	0	0	0	0	0
Req. horizontal width of top and bottom=	0.167197	0.1671971	0.07052	0.083577	0.0835765	0.060068
Pj=	0.6	0.6	0.6	0.6	0.6	0.6
Utilisation of Pj % =	**27.87%**	**27.87%**	**11.75%**	**13.93%**	**13.93%**	**10.01%**

HS2 Ltd - Code 1 - Accepted

3.4 Wedge Stability Check

	A	B	C
ϕ	35	35	35
β	27.5	27.5	27.5
H	3.30	3.30	3.30
W1 = 0.5 * H * tan(β) * H * γ * ffs	90.79	60.53	60.53
SL =	0	0	0
FL =	0	0	0
R =(W1+SL) / cos(90°-β-ϕ) =	102.36	68.24	68.24
T = (W1+SL) * tan(90°-β-ϕ) =	**47.26**	**31.51**	**31.51**
No. of Reinforcment Layers, n =	11	11	11
ΣTD = n*TD	187	187	110
ΣTD/fn =	170.000	170.000	100.000
Utilisation of T =	**27.80%**	**18.54%**	**31.51%**

	A	B	C
Pj	0.6	0.6	0.6
fp	1.3	1.3	1
fn	1.1	1.1	1.1
μ	0.8	0.8	0.8
ffs	1.5	1	1
ff	1.5	1	1
ws	0	0	0
a'bc	1	1	1
c'	0	0	0
μ ff ws	0	0	0
a'bc c' / fms	0	0	0

				A	B	C	
				γ * hj	5.6	5.6	5.6 kPa
Layer	1	hj =	0.28	μ ffs γ h1	6.720	4.480	4.480 kPa
		Le.avg =	1.527888	(Pj Lej) / (fp fn)	0.641	0.641	0.833
				=	**4.31**	**2.87**	**3.73 kN**
				γ * hj	11.2	11.2	11.2 kPa
Layer	2	hj =	0.56	μ ffs γ h2	13.440	8.960	8.960 kPa
		Le.avg =	1.673646	(Pj Lej) / (fp fn)	0.702	0.702	0.913
				=	**9.44**	**6.29**	**8.18 kN**
				γ * hj	16.8	16.8	16.8 kPa
Layer	3	hj =	0.84	μ ffs γ h3	20.160	13.440	13.440 kPa
		Le.avg =	1.819405	(Pj Lej) / (fp fn)	0.763	0.763	0.992
				=	**15.39**	**10.26**	**13.34 kN**
				γ * hj	22.51	22.51	22.51 kPa
Layer	4	hj =	1.12	μ ffs γ h4	27.012	18.008	18.008 kPa
		Le.avg =	1.965164	(Pj Lej) / (fp fn)	0.825	0.825	1.072
				=	**22.27**	**14.85**	**19.30 kN**
				γ * hj	28.67	28.67	28.67 kPa
Layer	5	hj =	1.4	μ ffs γ h5	34.404	22.936	22.936 kPa
		Le.avg =	2.110923	(Pj Lej) / (fp fn)	0.886	0.886	1.151
				=	**30.47**	**20.31**	**26.41 kN**
				γ * hj	34.83	34.83	34.83 kPa
Layer	6	hj =	1.68	μ ffs γ h6	41.796	27.864	27.864 kPa
		Le.avg =	2.256681	(Pj Lej) / (fp fn)	0.947	0.947	1.231
				=	**39.57**	**26.38**	**34.30 kN**
				γ * hj	40.99	40.99	40.99 kPa
Layer	7	hj =	1.96	μ ffs γ h7	49.188	32.792	32.792 kPa
		Le.avg =	2.40244	(Pj Lej) / (fp fn)	1.008	1.008	1.310
				=	**49.58**	**33.05**	**42.97 kN**
				γ * hj	47.15	47.15	47.15 kPa
Layer	8	hj =	2.24	μ ffs γ h8	56.580	37.720	37.720 kPa
		Le.avg =	2.548199	(Pj Lej) / (fp fn)	1.069	1.069	1.390
				=	**60.49**	**40.33**	**52.43 kN**
					53.31	53.31	53.31 kPa
Layer	9	hj =	2.52	μ ffs γ h9	63.972	42.648	42.648 kPa
		Le.avg =	2.693958	(Pj Lej) / (fp fn)	1.130	1.130	1.469
				=	**72.31**	**48.21**	**62.67 kN**
					59.47	59.47	59.47 kPa
Layer	10	hj =	2.8	μ ffs γ h10	71.364	47.576	47.576 kPa
		Le.avg =	2.839716	(Pj Lej) / (fp fn)	1.191	1.191	1.549
				=	**85.03**	**56.69**	**73.69 kN**
					65.63	65.63	65.63 kPa
Layer	11	hj =	3.08	μ ffs γ h11	78.756	52.504	52.504 kPa
		Le.avg =	2.985475	(Pj Lej) / (fp fn)	1.253	1.253	1.628
				=	**98.65**	**65.77**	**85.50 kN**
							kPa
Layer	Not Used	hj =		μ ffs γ h12			kPa
		Le.avg =		(Pj Lej) / (fp fn)			
				=			**kN**
				Σpullout =	**487.52**	**325.02**	**422.52 kN**
				T =	47.26	31.51	31.51 kN
				Utilisation of T =	**9.69%**	**9.69%**	**7.46%**

HS2 Ltd - Code A - Accepted

4.0 Geogrid/Bags pull out check

Checking the geogrid connection to the wall bags for stability under the applied tension.

	Layer 1			Layer 2				
	A	B	C	A	B	C		
Number of bags above the layer	2	2	2	22	22	22		
Bags Weight at layer	3.6176	3.6176	3.6176	39.7936	39.7936	39.7936	kN/m	
Connection Strength, Huesker	6.13	6.13	6.13	28.73	28.73	28.73	kN/m	Fortrac 35
Connection strength, Synteen	6.51	6.51	6.51	24.15	24.15	24.15	kN/m	SF55
Connection strength, Strata	6.23	6.23	6.23	25.46	25.46	25.46	kN/m	SG150
Connection strength, Miragrid	5.27	5.27	5.27	28.76	28.76	28.76	kN/m	3XT
Pull out safety factor along reinforcement, Fp	1.3	1.3	1	1.3	1.3	1		
Tension applied at wall*Fp (with Safety Factor)	1.561	1.040	0.439	17.864	11.910	8.560	kN/m	
Utilisation of minimum connection strength =	**29.62%**	**19.75%**	**8.33%**	**73.97%**	**49.31%**	**35.44%**		
Utilisation of MAXimum connection strength =	**23.98%**	**15.99%**	**6.74%**	**62.11%**	**41.41%**	**29.76%**	**If min doesn't work has to specify**	

Bearing Capacity Check for 061-AC3 Culvert Headwall

CHARACTERISTIC SOIL PARAMETERS	Symbol	Units	INPUT
Bulk density (overburden soil)	γ_b	kN/m^3	18.0
Saturated density (overburden soil)	γ_{sat}	kN/m^3	18.5
Bulk density (foundation soil)	γ_b	kN/m^3	20.0
Saturated density (foundation soil)	γ_{sat}	kN/m^3	20.5
Density of water	γ_w	kN/m^3	10.00
Undrained shear strength	$c_{u.k}$	kN/m^2	50.0
Effective cohesion	c'_k	kN/m^2	1.0
Angle of shearing resistance	$ø'_k$	degree	23.0

FOUNDATION DETAILS	Symbol	Units	INPUT
Unit weight of Flex MSE	γ_{conc}	kN/m^3	17.0
Width	B	m	4.260
Length	L	m	3.000
Base thickness	D_B	m	0.50
Depth (ground to founding level)	D	m	0.45

ADDITIONAL DESIGN DETAILS	Symbol	Units	INPUT
Depth of groundwater (from ground level)	D_w	m	0.00
Slope gradient	β	degree	0.00
Foundation inclination	α	degree	0.00

DESIGN ACTIONS	Input loads			Total load		
	DA1 C1	DA1 C2	BS	DA1 C1	DA1 C2	BS
Vertical		542			542	
Horizontal		84			84	
Moment		66			66	

Bearing Capacity Check DESIGN SUMMARY

Method	DA1C1	DA1C2	BS8004
Base		Compression	
R_d (Drained)		OK	
R_d (Undrained)		OK	
Utilisation Factor, Δ (%)		70%	

RESULTS

R_d/A' (Drained)=

R_d/A' (Undrained)=

DESIGN ASSUMPTIONS

- Ground condition below foundation extends to depth > B
- Does not calculate bearing resistance of;
  - stronger soil overlying weaker soil
  - weaker soil overlying stronger soil
  - two way eccentrically loaded foundation
- BS8004 method, FOS = 1.00 on P_{max}
- Allow Depth Factor ? NO
- Design groundwater level taken at ground level

HS2 Ltd - Code 1 - Accepted

BS 8006 Internal Stability Checks for 061-AC3 Culvert Headwall

2.2 Sliding Along the Base

Horizontal Disturbing Force (Rh)

The horizontal disturbing force is calculated as the sum of the active pressure behind the block (Pa) and other horizontal loadings.

$P_a = \frac{1}{2} k_a \gamma H^2$ Barnes (2000) Where:

$k_a = \frac{OC[1-sin(\phi')]}{OC[1+sin(\phi')]} = \frac{1-sin(\phi')}{1+sin(\phi')} = tan^2\left[45 - \frac{\phi'}{2}\right]$ Barnes (2000)

		Characteristic		
Ka (Soil 2)=		0.472		
γ(dry)=		18		
γsat=		18.5		
H (above GWT)=		0.7		
H (Below GWT)=		1		
Pa(dry)=		2.08308606	kN/m	
Pa(sat)=	5.951674	6.957	kN/m	

Factored horizontal loadings:

	A	B	C	
Earth Hori-Force from soil 2	22.48822	22.488215	14.99214	kN/m
Traffic load behind the reinfoced soil	6.022528	6.0225277	0	kN/m

Horizontal Resisting Force (HRF)

HRF provided by a soil/soil interface for the base of the retained soil slab.

$f_s R_h \le R_v \frac{\tan\phi'_p}{f_{ms}} + \frac{c'}{f_{ms}} L$

	Drained, long term					
	HRF for Fill (soil 1)			HRF for Foundation		
	A	B	C	A	B	C
tanϕ'p/fms=	0.700	0.700	0.700	0.424	0.424	0.424
c'/fms=	0	0	0	1	1	1
L=	3.76	3.76	3.76	3.76	3.76	3.76
Unfactored beneficial Rv=	135.36	135.36	135.36	135.36	135.36	135.36
HRF=	94.780	94.780	94.780	59.807	59.807	61.217
fs	1.2	1.2	NA	1.2	1.2	NA
fsRh=	34.213	34.213	check disp	34.213	34.213	check disp
Utilisation of HRF =	**36.10%**	**36.10%**		**57.21%**	**57.21%**	

$f_s R_h \le \frac{c_u}{f_{ms}} L$

	Undrained, short term		
	HRF for Foundation		
	A	B	C
cu/fms=	50	50	50
L=	3.76	3.76	3.76
HRF=	188	188	188
fs	1.2	1.2	NA
fsRh=	34.213	34.213	check disp
	18.20%	18.20%	

3.2 Local Stability Check - Rupture

		Layer 1			Layer 2		
		ULS, A	Unfactored	SLS, C	ULS, A	Unfactored	SLS, C
Tensile Force, Tj	kN/m	2.331018	1.5540118	1.00413	8.785627	5.8570849	5.186204
Long term (design life) Reinforcement base strength in accordance with BS8006 A.3 and PD ISO/TR 20432, TD (Huesker 35T geogrid)	kN/m	17	17	10	17	17	10
fn		1.1	1.1	1.1	1.1	1.1	1.1
TD/fn		15.45455	15.454545	9.09091	15.45455	15.454545	9.090909
Utilisation of TD	%	**15.08%**	**10.06%**	**11.05%**	**56.85%**	**37.90%**	**57.05%**

HS2 Ltd - Code 1 - Accepted

$$P_j \geq \frac{T_j}{\frac{\mu L_{ej}(f_{fs}\gamma_1 h_j + f_f w_s)}{f_p f_n} + \frac{a'_{bc} c' L_{ej}}{f_{ms} f_p f_n}}$$

where

- P_j is the total horizontal width of the top and bottom faces of the reinforcing element at the jth layer, per metre "run";
- T_j is the maximum value from **6.6.3.2.1**;
- f_{fs} is the partial load factor applied to soil self weight taken from the same load combination as T_j, see Table 11;
- f_f is the partial load factor applied to surcharge dead loads taken from the same load combination as T_j, see Table 11;
- μ is the coefficient of friction between the fill and reinforcing elements;
- L_{ej} is the length of reinforcement in the resistant zone outside failure wedge, at the jth layer of reinforcements, see Figure 31;
- w_s is the surcharge due to dead loads only;
- f_p is the partial factor for reinforcement pull-out resistance, see Table 11;
- f_n is the partial factor applied to economic ramifications of failure, see Table 9;
- a'_{bc} is the adhesion coefficient between the soil and the reinforcement;
- c' is the cohesion of the soil measured under effective stress conditions;
- f_{ms} is the partial material factor applied to c', see Table 11.

	Layer 1			Layer 2		
	A	B	C	A	B	C
Tj	2.331018	1.5540118	1.00413	8.785627	5.8570849	5.186204
γ1hj	8.4	8.4	8.4	35.56	35.56	35.56
μ (subject to testing)	0.8	0.8	0.8	0.8	0.8	0.8
Lej	2.334	2.334	2.334	2.990	2.990	2.990
ws	0	0	0	0	0	0
fp	1.3	1.3	1	1.3	1.3	1
fn	1.1	1.1	1.1	1.1	1.1	1.1
a'bc (subject to testing)	1	1	1	1	1	1
c'	0	0	0	0	0	0
Req. horizontal width of top and bottom=	0.141704	0.1417036	0.07043	0.098481	0.0984814	0.067078
Pj=	0.6	0.6	0.6	0.6	0.6	0.6
Utilisation of Pj % =	**23.62%**	**23.62%**	**11.74%**	**16.41%**	**16.41%**	**11.18%**

HS2 Ltd - Code 1 - Accepted

3.4 Wedge Stability Check

	A	B	C
ϕ	35	35	35
β	27.5	27.5	27.5
H	1.70	1.70	1.70
W1 = 0.5 * H * tan(β) * H * γ * ffs	23.89	15.93	15.93
SL =	0	0	0
FL =	0	0	0
R =(W1+SL) / cos(90°-β-ϕ) =	26.94	17.96	17.96
T = (W1+SL) * tan(90°-β-ϕ) =	**12.44**	**8.29**	**8.29**
No. of Reinforcment Layers, n =	4	4	4
ΣTD = n*TD	68	68	40
ΣTD/fn =	61.818	61.818	36.364
Utilisation of T =	**20.12%**	**13.41%**	**22.80%**

	A	B	C
Pj	0.6	0.6	0.6
fp	1.3	1.3	1
fn	1.1	1.1	1.1
μ	0.8	0.8	0.8
ffs	1.5	1	1
ff	1.5	1	1
ws	0	0	0
a'bc	1	1	1
c'	0	0	0
μ ff ws	0	0	0
a'bc c' / fms	0	0	0

			A	B	C	
		γ * hj	8.4	8.4	8.4	kPa
Layer 1	hj = 0.42	μ ffs γ h1	10.080	6.720	6.720	kPa
	Le.avg = 2.333674	(Pj Lej) / (fp fn)	0.979	0.979	1.273	
		=	**9.87**	**6.58**	**8.55**	**kN**
		γ * hj	17.08	17.08	17.08	kPa
Layer 2	hj = 0.84	μ ffs γ h2	20.496	13.664	13.664	kPa
	Le.avg = 2.552312	(Pj Lej) / (fp fn)	1.071	1.071	1.392	
		=	**21.95**	**14.63**	**19.02**	**kN**
		γ * hj	26.32	26.32	26.32	kPa
Layer 3	hj = 1.26	μ ffs γ h3	31.584	21.056	21.056	kPa
	Le.avg = 2.77095	(Pj Lej) / (fp fn)	1.163	1.163	1.511	
		=	**36.72**	**24.48**	**31.82**	**kN**
		γ * hj	35.56	35.56	35.56	kPa
Layer 4	hj = 1.68	μ ffs γ h4	42.672	28.448	28.448	kPa
	Le.avg = 2.989589	(Pj Lej) / (fp fn)	1.254	1.254	1.631	
		=	**53.53**	**35.68**	**46.39**	**kN**
		γ * hj				kPa
Layer Not Used	hj =	μ ffs γ h5				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
		γ * hj				kPa
Layer Not Used	hj =	μ ffs γ h6				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
		γ * hj				kPa
Layer Not Used	hj =	μ ffs γ h7				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
		γ * hj				kPa
Layer Not Used	hj =	μ ffs γ h8				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
						kPa
Layer Not Used	hj =	μ ffs γ h9				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
						kPa
Layer Not Used	hj =	μ ffs γ h10				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
						kPa
Layer Not Used	hj =	μ ffs γ h11				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
						kPa
Layer Not Used	hj =	μ ffs γ h12				kPa
	Le.avg =	(Pj Lej) / (fp fn)				
		=				**kN**
		Σpullout =	**122.07**	**81.38**	**105.79**	**kN**
		T =	12.44	8.29	8.29	kN
		Utilisation of T =	**10.19%**	**10.19%**	**7.84%**	

HS2 Ltd - Code 1 - Accepted

4.0 Geogrid/Bags pull out check

Checking the geogrid connection to the wall bags for stability under the applied tension.

	Layer 1			Layer 2				
	A	B	C	A	B	C		
Number of bags above the layer	3	3	3	12	12	12		
Bags Weight at layer	5.4264	5.4264	5.4264	21.7056	21.7056	21.7056	kN/m	
Connection Strength, Huesker	7.26	7.26	7.26	17.43	17.43	17.43	kN/m	Fortrac 35
Connection strength, Synteen	7.01	7.01	7.01	13.59	13.59	13.59	kN/m	SF35
Connection strength, Strata	7.19	7.19	7.19	15.85	15.85	15.85	kN/m	SG150
Connection strength, Miragrid	6.44	6.44	6.44	17.01	17.01	17.01	kN/m	3XT
Pull out safety factor along reinforcement, Fp	1.3	1.3	1	1.3	1.3	1		
Tension applied at wall*Fp (with Safety Factor)	3.030	2.020	1.004	11.421	7.614	5.186	kN/m	
Utilisation of minimum connection strength =	**47.04%**	**31.36%**	**15.59%**	**84.07%**	**56.05%**	**38.17%**		
Utilisation of MAXimum connection strength =	**41.75%**	**27.83%**	**13.83%**	**65.52%**	**43.68%**	**29.75%**	**If min doesn't work has to specify**	

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Appendix H

Summary of Design – Sedrup North Culvert

Appendix H.1: Box Culvert Calculation Pack

Number of Pages: 30

Appendix H.2: Maps & Drawings

Number of Pages: 01

Appendix H.3: Geological Section

Number of Pages: 04

Appendix H.4: Data Plots

Number of Pages: 11

Appendix H.5: Stability Checks

Number of Pages: 66

Appendix H.6: Headwall Calculation Pack

Number of Pages: 49

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SUBJECT

Sedrup North Culvert

SHEET NO.

Appendix H-1 : Caclualtion pack/Asset Summary Sheet

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A High-Speed Design Partnership
ARCADIS COWI

ASSET SUMMARY SHEET

SUBJECT

Sedrup North Culvert

SHEET NO. **1** of **30**

Rev	TOTAL SHEETS	AUTHOR	DATE	CHECKED BY	DATE	APPROVED BY	DATE	COMMENTS
0		AIM	18/08/21	LB	13/09/21			Ground model and parameters
1		AIM	28/10/21	LB	28/10/21			Settlement, rigid boundary, springs checks completed. TR comments addressed. Includes note on CPT data and EW1283.
2		NCOB	04/11/21	LB	04/11/21			Update to stability section. Assumption register, geohazard register and notes for the STR drawings.
3		NCOB	10/11/21	LB	10/11/21	BEJ	18/11/21	Note included in section 1.6 clarifying route to L3. Addressing TR comments
4		AK	29/03/23	KY	03/04/23	SK	08/04/23	Included findings of Stage2 GI, the potential fault line is considered in calculations

SUPERSEDES DOC No.

ASSET SUMMARY SHEET

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SECTION 1: ASSET INFORMATION

1.1: Structure Type and Site Description

- Sedrup North Culvert is a box culvert of 2.5m x 3.0m internal dimensions and 94.09m in length located within Aylesbury Embankment Group - C2 Section 2 at chainage 061+330.

1.2: Proposed Earthworks Design

- Refer to Aylesbury Embankment Geotechnical Design Report (1MC06-CEK-GT-REP-CS04_CL18-000037) for details about the proposed earthworks design and ground treatment and for any earthworks and foundation risks and mitigations proposed.
- There is an existing Aylesbury Embankment GDR 1MC06-CEK-GT-REP-CS04_CL18-000002, Rev. C02, where the initial design was presented.
- The earthworks design requires a combined mitigation measure: 2mbgl of excavate and replacement (E&R) with 5 months of holding period in the vicinity of the culvert as per Table 2-22 of Aylesbury Embankment - GDR (1MC06-CEK-GT-REP-CS04_CL18-000037).

1.3: Local GI Plan

- Refer to Figure H1.1 for the local GI plan from ArcGIS online viewer

1.4: Relevant Boreholes and other Information

- The following boreholes were considered to determine the stratigraphy and properties of the ground at this asset.

Borehole ID	Type	Top of Hole (m AOD)	Eastings	Northings	Final Depth (m)	Distance from Asset (m)	GI Status
ML061-CR012	CP+RC	77.68	243882	318594	20.00	55	Final
ML061-CR013	CP+RC	78.06	243890	318497	20.00	47	Final
ML061-WS020	WLS	78.40	243902	318449	4.00	84	Final
ML061-WS023	WLS	77.46	243818	318596	2.45	78	Final
ML061-WS025	WLS	77.87	243747	318535	5.00	20	Final

*CP- Cable purcussion, RC- Rotary Core, WLS- Dynamic Windowless sampling

The ML061-CT400 (CPT) was proposed at the location of the Sedrup North culvert. However considering the geology, and according to the draft log data received, the CPT was not able to penetrate any past 1.27m, thus no information regarding the deeper strata and presence of the potential fault line was gained.

ML061-CT400	CPT	77.8	243869.4	318535.8	1.3	11	Final

Additional GI was requested (EWN 1283) and it was made available in Data Drop 30.

ML061-CR462	RC	77.9	243902.3	318550.4	20.2	At Asset	Final

In addition, the following deeper exploratory holes approximately 200m on the upline and downline from the Culvert along the mainline of the asset are considered to confirm the variation in strata and presence of the potential fault line at the Culvert (Refer Figure H2.3 & Figure H2.4)

ML061-CR014	CP+RC	78.0	243762.3	318654.0	20.0	170.0	Final
ML061-CR442	CP+RC	77.6	243777.0	318608.0	20.3	125.0	Final
ML061-CR008	CP+RC	79.9	243965.0	318403.0	25.0	155.0	Final
ML061-CR009	CP+RC	79.4	243976.0	318428.0	25.0	140.0	Final

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1.5: Geological Cross Section

- Geological cross - section is provided in Figure 2.

1.6: Route to L3

- The available GI is sufficient to support the development of ground model and derivation of geotechnical parameters.

The geological section at asset chainage indicates the presence of the potential fault line within the footprint of the Culvert. Therefore Pre-construction GI, i.e. a trench of width 11.3m, length 105m and depth 2m below culvert founding level is required to be carried out at the culvert location to identify the location of the potential fault line before construction of the culvert. A qualified and competent Geotechnical Engineer shall inspect the excavation to confirm the stratigraphy & location of the potential fault line. The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level. If the potential fault line is confirmed the location, then the direction of the fault line, the properties of the soils within the fault line and the site photos etc. shall be included in the report.

- Along with preconstruction GI, the existing GI is sufficient to undertake the detailed design and to proceed with L3 validation

1.7: Backround

- The variation in stratigraphy encountered between ML061-CR012 and ML061-CR013, the presence of the potential fault line within the area of the culvert was identified at IDR and presented in IDR meeting dt. 10th Dec 2021, in addition, the risk of the presence of the potential fault line was recorded in GRR in risk ID GEO_AA_179 and GEO_AA_180.
'- IDR design was based on borehole ML061-CR012 stratigraphy and pessimistic ground parameters.
- To mitigate the risk of uncertainty in the ground, the additional borehole ML061-CR462 was proposed in EW-1283 & TQ-1402.
- The additional borehole ML061-CR462 was made available in Data Drop 30.
- Borehole ML061-CR462 confirmed the stratigraphy observed in borehole ML061-CR012, which includes a superficial deposit of topsoil, a head deposit, and an Alluvium formation overlying the Portland formation, followed by the Kimmeridge clay formation.
- The variation in strata in asset-specific exploratory hole ML061-CR462 and shallow CPT ML061-CT400 provided confirmation of variation in strata along the culvert length.
-As a result, when boreholes in the 200m vicinity on each side of the culvert and along the mainline are studied (Refer Figure H2.3 and Figure H2.4) and following details are observed,
1) Boreholes ML061-CR462, ML061-CR012 and ML061-CR014 demonstrated similar stratigraphy which has superficial deposit followed by Portland clayey formation up to ~7mbgl which is overlying the Kimmeridge Clay formation,
2) Boreholes ML061-CR013, ML061-CR442 and ML061-CR008 demonstrated similar stratigraphy which has superficial deposit followed by Portland/Purbeck Limestone formation up to ~18mbgl overlying the Kimmeridge clay formation. The CPT ML061-CT400 terminated at a shallow depth as it was unable to penetrate the limestone formation.
- This provides the confirmation of the potential fault line across the Sedrup North Culvert.
- Consequently, two ground models are developed for the current design,
    - Ground Model 1 (GM1) - Based on exploratory holes ML061-CR012 and ML061-CR462 stratification,
    - Ground model 2 (GM2) - Based on exploratory hole ML061-CR013. Refer to Section 2.

Although the possibility of presence of the potential fault line below the culvert location has been confirmed, its precise location remains uncertain. Therefore, for the current design, the most pessimistic location i.e., adjacent to borehole ML061-CR462 is being considered.

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SECTION 2: GROUND MODEL

2.1: Stratigraphy

- - At proposed Culvert, the variation in stratigraphy has been observed (Refer Figure 2.2)
- Borehole ML061-CR462 and ML061-CR012 has presented stratigraphy as superficial deposit of topsoil, a head deposit, and an Alluvium formation overlying the Portland formation up to ~6mbgl, followed by the Kimmeridge clay formation.

- While exploratory hole ML061-CR013 has stratigraphy as superficial deposit ovelying Portland Limestone formation upto ~18mbgl, followed by Kimmeridge clay

- Consequently, two ground models are developed for design presented here, the Ground model 1 (GM1) which is based on borehole ML061-CR012 and ML061-CR462 borehole stratification and ground model 2 (GM2), which is based on borehole ML061-CR013.

A summary of the local ground conditions is presented in the table below for GM1 & GM2.

Ground Model -1 (GM1)

Geological Unit	Top of Stratum Range (m AOD)	Design Top Level (m AOD)	Range of Thickness (m)	Design Thickness (m)
Topsoil (TOP-CZ)	77.87-77.38	77.70	0-0.3	0.30
Alluvium (ALV-CZ)	77.57-75.48	77.40	0.3-2.2	1.00
Head Deposits (HEAD-S V SV)	77.18-75.68	76.40	0.5-2.0	1.00
Portland Stone Formation (POST-CZ)	75.48-69.86	75.40	2.2-7.82	3.70
Kimmeridge Clay Formation (KC-CZ)	71.35-57.67	71.70	6.52-20.2	21.7*

Notes:
- The ground model 1 (GM1) is based on ML061-CR012 and additional borehole ML061-CR462. The GM1 is based on cohesive soil strata.
- The maximum depth of investigation data available is for 20m. *Kimmeridge clay depth is assumed to be extended 50mAOD.

Ground Model -2 (GM2)

Geological Unit	Top of Stratum Range (m AOD)	Design Top Level (m AOD)	Range of Thickness (m)	Design Thickness (m)
Topsoil (TOP-CZ)	78.06-77.76	77.70	0-0.3	0.30
Alluvium (ALV-CZ)	77.76-76.06	77.40	0.3-2	1.40
Purbec Limestone (PB-CZ)	76.06-72.86	76.00	2-5.2	3.10
Portland Stone Formation (POST-LMST)	72.86-65.36	72.90	5.2-12.7	7.60
Portland Sand Formation (POSA-CZ)	65.36-59.46	65.30	12.7-18.6	5.90
Kimmeridge Clay Formation (KC-CZ)	59.46-50.0	59.50	18.6-20*	9.4*

Notes:
- The ground model 2 (GM2) is based on ML061-CR013.
- The maximum depth of investigation data available is for 20m. *Kimmeridge clay depth is assumed to be extended 50mAOD.

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2.2: Static Geotechnical Parameters

- The table below summarises the characteristic design parameters used during analysis.
- These parameters are unfactored.

Geological Unit	γ		ϕ'		c'	c_u	E'	Eu	v_d	v_{ud}	UCS	PI	SPT N range	Permeability (m/s)	
	γ_b	γ_s	ϕ'_p	ϕ'_{cv}											
	kN/m^3	kN/m^3	°	°	kPa	kPa	MPa	MPa	-	-	MPa	%		kx	ky
Alluvium (ALV-CZ)	19	19	22	21	-	30	5.4	9	0.15	0.50	-	48	5-22	1.0E-10	1.0E-10
Head Deposits (HEAD-S V SV)	19.5	19.5	32	30	-	-	16	-	0.25	-	-	18	17	6.9E-06	6.9E-06
Portland Stone Formation (POST-CZ)	21	21	22	21	-	60+44z (150)	25	18+12z (45)	0.15	0.50	-	30	-	1.0E-08	1.0E-08
Kimmeridge Clay (KC-CZ)	19.5	19.5	26	25	10	110+15.4z (250)	35.2+4.9z (80)	44+6.1z (100)	0.15	0.50	-	26	17-37	1.0E-08	2.0E-09
Purbeck Formation (PB-LMST)	21	21	25	25	-	-	250	-	0.15	-	1.5	-	-	6.9E-06	6.9E-06
Portland Stone Formation (POST-LMST)	21	21	25	25	-	-	480	-	0.15	-	22.6	-	-	6.9E-06	6.9E-06

* z is depth below the top of the stratum;

-The characteristic unit weight is taken based lab test data and BS8002

- The permeability values are based on test data

Notes:

Alluvium

-A Drained Angle of Shearing Resistance is calculated using the characteristic PI in accordance to the methodology outlined in BS 8002:2015.

- An undrained shear strength is derived based on field and lab test data. The characterstic line can be viewed in Appendix H.4: Data Plots.
- Undrained stiffness is calculated using the correlation with cu - Eu=cu*300, whereas, the drained stiffness (E') is calculated using the correlation E' = 0.6*Eu.

Head Deposits

- E' for HEAD deposits is estimated from E = ~ 1 *N (MPa). Refer to Appendix H.4: Data Plots
- Friction angle was derived based on relation of N-values with Friction angle (Peck et al. 1974).
- The Head deposit formation is granular material, thus k value is considered as 6.9 E-9m/s

Portland Stone Formation (POST-CZ)

- A Drained Angle of Shearing Resistance is calculated using the characteristic PI in accordance to the methodology outlined in BS 8002:2015.
- An undrained shear strength is derived based on field and lab test data. The characterstic line can be viewed in in Appendix G.4: Data Plots.
- Undrained stiffness was calculated using the correlation with cu - Eu=cu*300 , whereas, the drained stiffness (E') is calculated using the correlation E' = 0.8*Eu.

Kimmeridge Clay Formation

- A Drained Angle of Shearing Resistance is calculated using the characteristic PI in accordance to the methodology outlined in BS 8002:2015.
- An undrained shear strength is derived based on field and lab test data. The characterstic line can be viewed in Appendix H.4: Data Plots.
- Undrained stiffness was calculated using the correlation with cu - Eu=cu*400, whereas, the drained stiffness (E') is calculated using the correlation E' = 0.8*Eu.

Purbeck Formation

- An undrained shear strength is derived based on SPT values. The characterstic line can be viewed in Appendix H.4: Data Plots. It provides UCS value at 1.5MPa when Cu and UCS is correlated.
- Undrained stiffness was calculated using the correlation with cu - Eu=cu*400, the effective stiffness, E' is then estimated as E'=0.8Eu and adopted as 250MPa
- The Purbeck limestone formation is highly permeable, thus k value is considered as 6.9 E-9m/s

Portland Stone Formation (POST-LMST)

- The UCS value is 22.6MPa from the lab test result as per GIR of Aylesbury Embankment. However, as per Geological Summary Note – Portland Group, the UCS value for the Portland group is ranging from 16MPa to 19.5MPa based on the Atkins Desk Study, and the UCS of 16MPa is adopted. The correlation, Em=j.Mr. UCS is used to estimate the Em value. RQD is ranging from 0 to 25%, Mr = 150, j = 0.2. The value adopted for Em is 480MPa.
- The Portland Stone limestone formation is highly permeable, thus k value is considered as 6.9 E-9m/s

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2.3: HSS parameters of Kimmeridge Clay formation

- To assess the impact of variation in strata along the length of Culvert, i.e. to assess the impact of the potential fault line on Culvert in terms of total settlement and horizontal separation, the stiffness parameters for KC formation have been derived based on correlations with strength parameters for HSS material model.

Geological Unit	Level (mAOD)		E50	Eoed	Eur	G0	γ0.7
	From	To	MPa	MPa	MPa	MPa	%
Kimmeridge Clay formation	71.7	62.7	48	39	97	97	0.01
	62.7	52.7	110	88	220	220	0.01

Where, ,

E50 is the secant stiffness in standard drained triaxial test (stress-level and strength independent)

Eoed is the tangent stiffness for primary oedometer loading (stress-level and strength independent)

Eur is the unloading / reloading stiffness (stress-level and strength independent)

G0 is the shear modulus at very small strains

γ0.7 is shear strain at which Gs = 0.722G0 ,

m=0.7 is the power for stress-level dependency of stiffness (Adopted from Technical Note on Soil parameters for HSS Model of KC formation

- The parameters mentioned above has been derived based on correlations with site specific shear strength parameters, the following correlations are used to derive the

- The tangent stiffness for primary ooedometer compression is derived from correlation Eoed = 1.1E' as per Ground Engineering Technical Note: Guidance on the Assessment of Settlement, 1MC06-CEK-GT-NOT-C002-000023

- The secant stiffness is derived in correlation with the tangent stiffness modulus, E50= 1.25Eoed

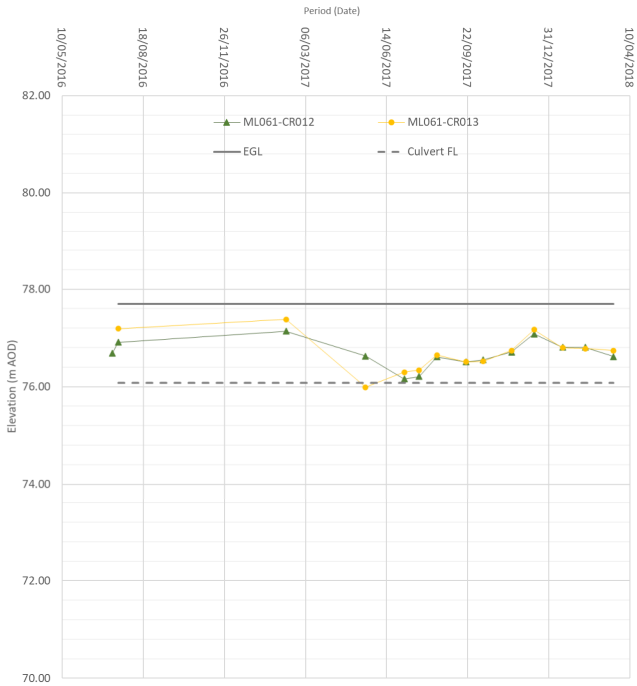
- The unloading / reloading stiffness modulus has been deived as Eur = 2E50

- The shear modulus at very small strain has been derived from MASW tests in Kimmeridge Clay formation

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2.4: Groundwater

- The ground water monitoring was carried out in ML061-CR012 and ML061-CR013 between July 2016 and March 2018. The groundwater level (GWL) varied between 0.54m bgl and 2.07m bgl. The design GWL was considered at Existing ground level.



2.4: Concrete Classification

- No geotechnical chemical testing was carried out in the boreholes located in the footprint of the proposed structure. The design Sulphate and ACEC class is referred as presented in Table 2-23 of the GDR for the Aylesbury Embankment

Table 2-23 Summary of Soil Sulphate Chemistry

Geological Formation		Design Sulphate Class (DS)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) (%)	pH
Head Deposits		No Testing Available.			6.9
Alluvium	No Testing Available.	No Testing Available.	0.15	5.4	
Purbeck Group		No Testing Available			
Portland Sand/Stone Formation		No Testing Available			
Kimmeridge Clay Formation		DS-2	AC-2	9.90	5.7

Source: Table 2-23 in Aylesbury Embankment - GDR (1MC06-CEK-GT-REP-CS04_CL18-000037_Rev C02)

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SECTION 3: GEOHAZARDS AND OTHER RISKS

The geotechnical risk included in the risk register (1MC06-CEK-GT-REG-C002-000076) have been presented below for the geotechnical design of the culvert.

Geotechnical Risk ID	Hazard Title	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Residual Risk Rating
GEO_3048	High / Shallow Groundwater	10	Control	1) Contractor to have a de-watering plan for the site and ensure that all site operatioatives are aware of such plan. Additional site controls including drainage and watching brief may be required during construction. 2) Communicate as CDM risk to contractor.	3
GEO_3049	Soft / compressible ground	4	Control	1) The existing Made Ground and potentialy shallow soft deposits (alluvum/Head) will be removed and replaced with suitable founding granular material. It is anticipated that 2.0m from the base of the culvert will be excavated and replaced with suitable founfing material. 2) In-situ testing of the proposed formation of the culvert will be required to provide assurance of the soil properties at formation level.	2
GEO_AA_0179	Faulting	16	Reduce	1) Additional GI (ML061-CR462) is available and has been considered in the current design. This borehole has also been used to identify the potential fault. As a consequence of this, two different ground models have been derived under the culvert footprint. These two ground models represent the different ground conditions under the side of the fault line, which will have an impact on the predicted movements of the culvert concrete sections. 2) Current design has been carried using out using both ground models and the fault line under the culver and maximum horizontal and vertical movements were used to design the culvert concrete sections. 3) To mitigate the risk of excessive horizontal separation of the concrete sections of the culvert, an internal lining will be installed inside the culvert after both the culvert and embankment over the culvert are constructed. This has been discussed and agreed with EKFB and documented in EW-3114 dated 14th Feb 2023 and RRM on 28th Feb and 15th March 2023. 4) During the excavation to install the culvert, the potential fault line will need to be inspected and its location recorded and mapped as per the SCEW spect for the culvert. Backfilling of the excavation and construction of the culvert cannot be continue until the mapping of the fault is shared with ASC.	4
GEO_AA_0180	Faulting	6	Reduce	1) Additional GI (ML061-CR462) is available and has been considered in the current design. This borehole has also been used to identify the potential fault. As a consequence of this, two different ground models have been derived under the culvert footprint. These two ground models represent the different ground conditions under the side of the fault line, which will have an impact on the predicted movements of the culvert concrete sections. 2) Current design has been carried out using both ground models and the fault line under the culvet and maximum horizontal and vertical movements were used to design the culvert concrete sections. 3) the current estimated maximum total settlement considering the presence of the potential fault line is within the permissible limits for the concrete segments of the culvert. 4) Qualified and competent Geotechnical Engineer shall inspect the excavation/trench to confirm the stratigraphy & location of the potential fault line. 5) The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level.	3
GEO_AA_0181	Inadequate / Uncertain GI Data	12	Reduce	1) Initial HS2 Gi available and used in detailed design. 2) Additional GI (ML061-CR462) was available at the time of the detailed design and was also used to derive the ground model and the geotechnical parameters. 3) Both the initial HS2 GI and additional GI provide sufficient information to mitigate uncertainty in the ground model and geotechnical parameters adopted for the design.	3

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Geotechnical Risk ID	Hazard Title	Risk Rating	Primary Risk Mitigation Strategy	Mitigation Description / Notes	Residual Risk Rating
GEO_AA_0183	Loose / Unstable Ground	15	Control	1) Contractor to ensure suitable temporary works if required during construction. 2) Excavation and Replacement (E&R) of Topsoil, head deposit and alluvial formation has been proposed with 2m of E&R below the culvert founding level. 3) Communicate as CDM risk to the contractor.	4
GEO_AA_0525	Strong or stiff soil / rock	12	Reduce	1) Qualified and competent Geotechnical Engineer shall inspect the excavation/trench to confirm the stratigraphy & location of the potential fault line. 2) The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level. 3) The location of the fault is required to be mapped according to the SCEW spec and included in a site inspection report. Site photos shall be included in the report.	4

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SECTION 4: STRUCTURE

4.1: Introduction

- The proposed culvert is a reinforced concrete structure. The culvert will be constructed with precast reinforced concrete units stitched together to form a continuous structure. The width of the culvert is 3.3m. The design was carried out considering the height of 3.8m.

4.2: Culvert Design Details

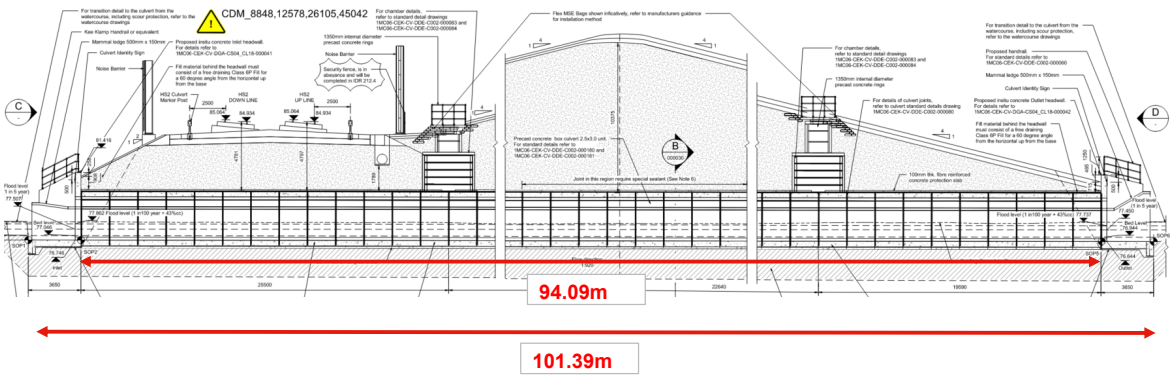
- The dimensions of the culvert are based on the geo-interface sheet provided by the structures team and the following drawings: 1MC06-CEK-CV-DGA-CS04_CL18-000040 and 1MC06-CEK-CV-DSE-CS04_CL18-000031.

Culvert Design Details (settlement calculations)

Total Length	94.09 m			
Width	3.3 m			Analysis conservatively assumed that the loads are at the same level
				Modelled elevation was calculated as average inlet - outlet level
Level of Foundation Base	Inlet	76.746	m AOD	(including culvert wall thickness (0.4m) and blinding (150mm)).
	Outlet	76.644	m AOD	Use 76.145 mAOD

The analysis assumed that 2.0m of the ground below the culvert will be replaced with Class 1A2 fill.

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Source: Drawing No. 1MC06-CEK-CV-DSE-CS04_CL18-000031

4.3: Loadings on Culvert (Embankment and HS2)

1. Fill loads (embankment) are calculated for all heights of embankment and multiplied with unit weight of 20kN/m3 to estimate the embankment load.

2. Load application levels listed are taken from the levels as presented in the drawing 1MC06-CEK-CV-DSE-CS04_CL18-000031. Please note that we have conservatively assumed that the culvert is not inclined, but at the same level (average culvert founding level).

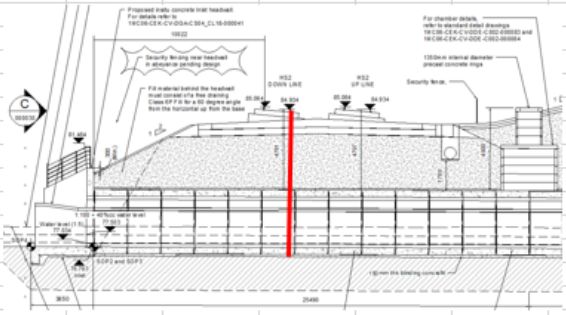
4. The following rail loads: Load due to Rails & fixtures, Load due to Track Slab, Load due to HBL (Hydraulic Bond Layer), Load due to Protection Layer, Load due to prepared subgrade, crossfall & protection. This loads are applied on mainline embankment

5. The height of the embankment was measured from the drawing no. 1MC06-CEK-CV-DSE-CS04_CL18-000031

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Extract from the Geo-Interface sheet

High Speed 2 - 1MC06 - Stage One C2 - MWCC –
North Portal of Chiltern Tunnels to Brackley

Ref					Output
	Load under the rail				
					
	Charateristic Permanent Loads				
				Load Number	
	Load due to Rails & fixtures =	0.22	kN/m^2	(1)	
	Load due to Track Slab =	2.28	kN/m^2	(2)	
	Load due to HBL =	2.94	kN/m^2	(3)	
	Load due to Proctection Layer =	2.79	kN/m^2	(4)	
	Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2	(5)	
	Charateristic Variable Loads				
	Load due to LM71 =	47.45	kN/m^2	(8)	

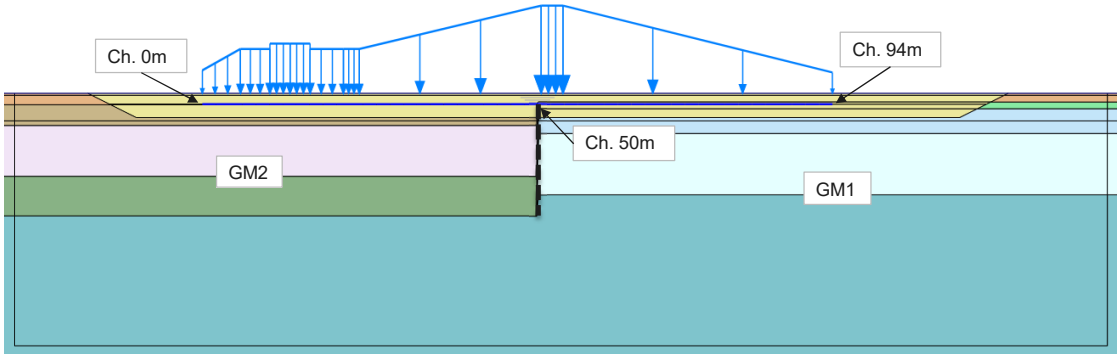
Culvert self weight

Concrete density =	25	kN/m^3
Volume of conc =	10.235	m^3
Load =	255.875	kN
Area applied =	5.8	m^2
Pressure =	44.1164	kN/m^2

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SECTION 5: SETTLEMENT OF STRUCTURE

- The modelling of settlement/ground movements associated with the construction of the proposed HS2 embankment was undertaken using Finite element modelling i.e. Plaxis 2D V_21. The program estimated displacements of a soil mass due to the loads applied, showing the predicted soil settlement pattern beneath the loaded area .
- Fill loads (embankment) are provided as line load with intensity estimated as height of embankment multiplied by unit weight of fill, i.e. 20kN/m3



5.1: Applied loads - Plaxis2D

1. In the Plaxis2D model, the embankment loads and railway loads are applied at the existing ground level conservatively. The following loads are applied at EGL.

Permanent load: Embankment self weight, Rail permanent loads

Temporary loads: LM71

5.2: Details of Plaxis modelling

1. The foundation of culvert is modelled as dummy plate with low values of stiffness and self weight to obtain settlement of ground at every 2m intervals.

2. The Kimmeridge clay is the governing layer for the settlements in GM1 & GM2, therefore the material model is selected as Hardening soil with small strain (HSS) while other layers are modelled with amterial model as Mohr-Coulomb.

3. The location of the potential fault line is considered next to ML061-CR462, i.e. below the Landscape embankment at Chainage 50m. This is the most pessimistic case to obtain horizontal separation due to variation in strata. However, the location of the potential fault line is porposed to be confirmed by trenching in drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000024

4. In Plaxis model above the Chainage 0m is at inlet while chainage 94m is at outlet.

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5.0 SETTLEMENT OF STRUCTURE

- The settlement analysis is carried out to determine the anticipated culvert settlement beneath the proposed embankment.

- As per discussion between STR and GEO team (21/10/2021) the damage assessment and total settlements are critical for structural checks.

5.3: Settlement results

Total settlement for 120yrs = 137 mm
Permissible total settlement = 175 mm
Therefore, the total settlement of culvert is within permissible limit.

Transitions shall be designed to satisfy longitudinal (along the culvert axis) differential settlement during operation of the railway (settlement from which is termed residual settlement), to be less than 1:1000 under the HS2 mainline, in accordance with HS2-HS2-GT-STD-000-000001 - Technical standard – Earthworks, section 4.2.

5.4: Damage Assessment

- A Damage Assessment was undertaken to verify that the structural performance of the culvert under the applied loads, i.e. that the maximum horizontal separation limits are satisfied.
- Horizontal separation expected between two adjacent culvert sections was assessed using an in-house spreadsheet. This considers the relative rotation of the culvert segment to each other based on the settlement profile derived in Section 5.3 over the segment lengths of 2m.
- The permissible limit for maximum localised horizontal separation is 10mm
- Analysis resulted with the maximum horizontal separation as 18.2mm

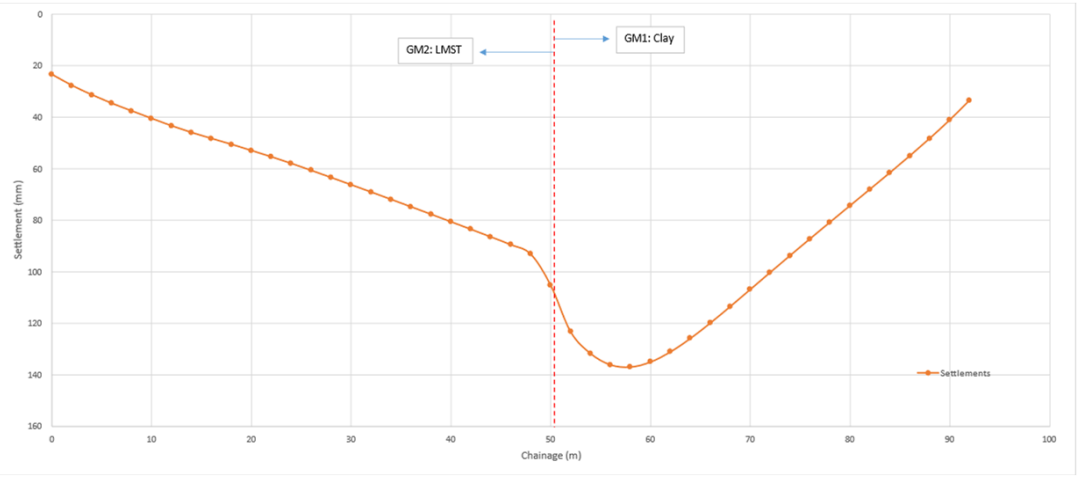
	Max Horizontal separation movement (mm)
Calculated values	18.2
Adopted Limit	10.0

- The maximum horizontal separation of 18.2mm is exceeding the permissible limit of 10mm.
- The horizontal separation is >10mm is observed for the 12m of the length of culvert at the potential fault line (i.e 3 culvert segments on each side)
- Therefore, EW-3114 dated-14th Feb 2023 was issued to EKFB to discuss the mitigation measure for excessive horizontal separation of segments. Following to EW-3114, two Risk Reduction Meetings (Dt.28/02/2023 and Dt.15/03/2023) were conducted with EKFB to present and discuss various Geotechnical and Structural solutions to mitigate the risk of excessive horizontal separation of segments. Out of multiple options, EKFB selected the solution of lining at the potential fault line location after construction of Embankment.
- To identify the location of the potential fault line, the trenching has been proposed in drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000024. It is proposed that a qualified and competent Geotechnical Engineer shall inspect the excavation of trench to confirm the stratigraphy & location of the potential fault line. The size of trench shall be 11.3m x 105m and 2m deep below culvert founding level. Refer drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000069 for liner details, the 9 joints (i.e. 8 culvert segments) are proposed to be provided with special treatment of liner.

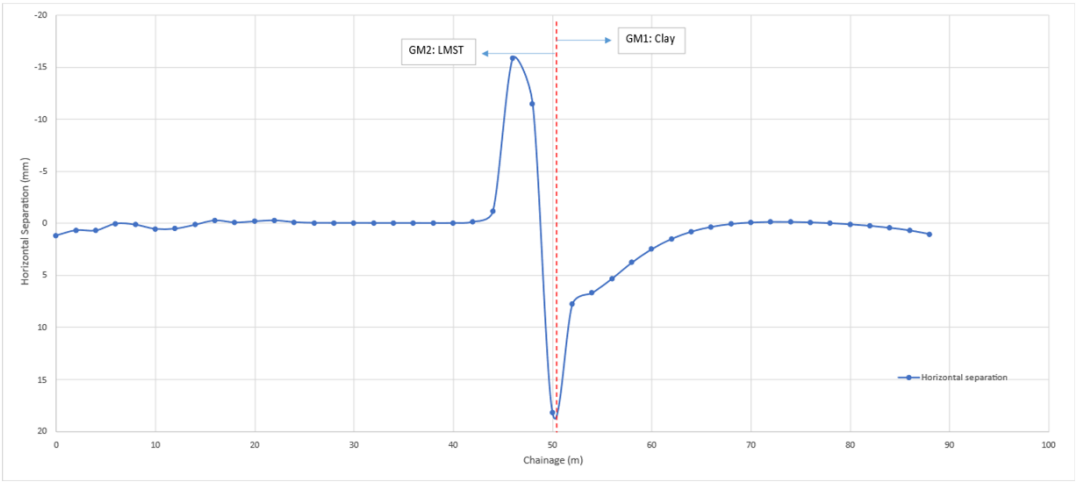
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5.5: Settlement Profiles

- The culvert settlements are estimated for 120yrs of design life. Culvert settles both during construction and throughout the design life of the structure. The total settlement along the culvert alignment is provided below.
- The x- axis shows the chainage. At inlet chainange is assumed as 0m while at outlet of culvert it is 94m.
- The y-axis shows the total seetlement at 120yrs



- The horizontal separation is carried out for each joint of 2 segment, refer following graph presenting horizontal separation for all segments



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5.6: Spring Stiffness

The calculations of subgrade reaction/spring stiffness of the ground below the culvert are required by the structures team to assess the effects of load applied on the culvert.
The embankment self weight and rail loads are applied over the length of culvert and the displacements are estimated from Plaxis2D at every 2m interval below founding level. The maximum and minimum values of spring stiffness has been recommended for structural assessment.

Spring stiffness (kN/m2/m)	
Minimum	Maximum
1875	5317

Conclusion

- Considering the uncertainty around the parameters used to estimate the spring stiffnesses and considering uncertainty in the location of the potential fault line. It is recommended to structures to check their design with springs -20%/+20% of the values provided above.
- The vertical spring stiffnesses should only be used to assess structural actions (moments and shears) in a culvert.

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SECTION 6: Foundation Stability Summary - Detailed Design

The culvert has been designed using the Culvert Design Sheet Rev. 3.0 and using the loading provided by the structures team on 31/03/2023. The design has been carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

The calculations are provided in the following attachments:

- Sedrup North Culvert Design Sheet 2.03.01 - Stage 1 - No cover
- Sedrup North Culvert Design Sheet 2.03.01 - Stage 2A
- Sedrup North Culvert Design Sheet 2.03.01 - Stage 2B
- Sedrup North Culvert Design Sheet 2.03.01 - Stage 2C
- Sedrup North Culvert Design Sheet 2.03.01 - Stage 2D
- Sedrup North Culvert Design Sheet 2.03.01 - Stage 3 - HS2 in Service

The GM1 has the relatively less stiff strata than GM2, consequently the GM1 is critical for stability assessment of the culvert. Therefore, the stability of the culvert is carried out considering GM1 and results are presented in following tables.

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6.1: Culvert cross section

- A cross section of the culvert, including pertinent dimensions, is provided below.

Culvert width =	3.3	m	
Culvert height =	3.8	m	(not including blinding layer or protection slab)
Culvert length* =	94.09	m	(Dwg. No. 1MC06-CEK-CV-DSE-CS04_CL18-000031)

*The distance between the first and the last culvert section

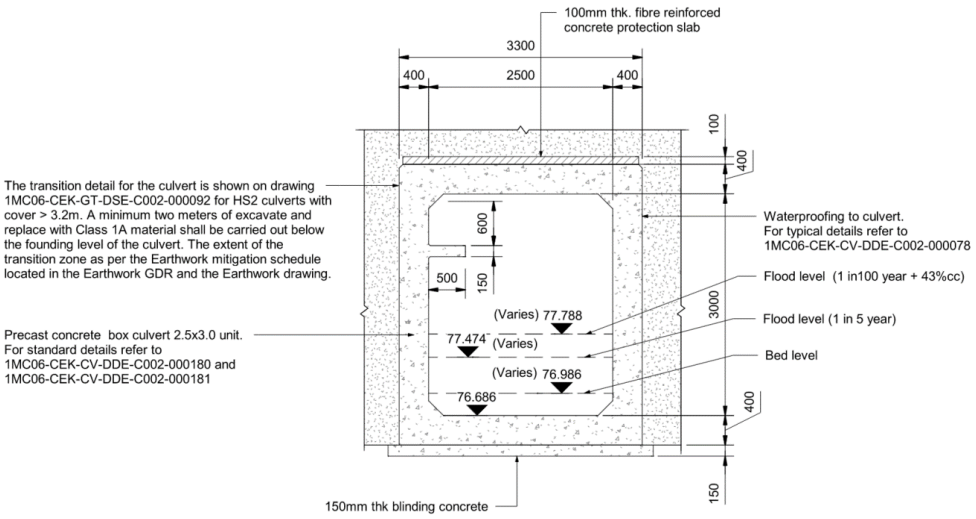


Figure 6.1 - Extract from General Arrangement drawing - Section (1MC06-CEK-CV-DSE-CS04_CL18-000030)

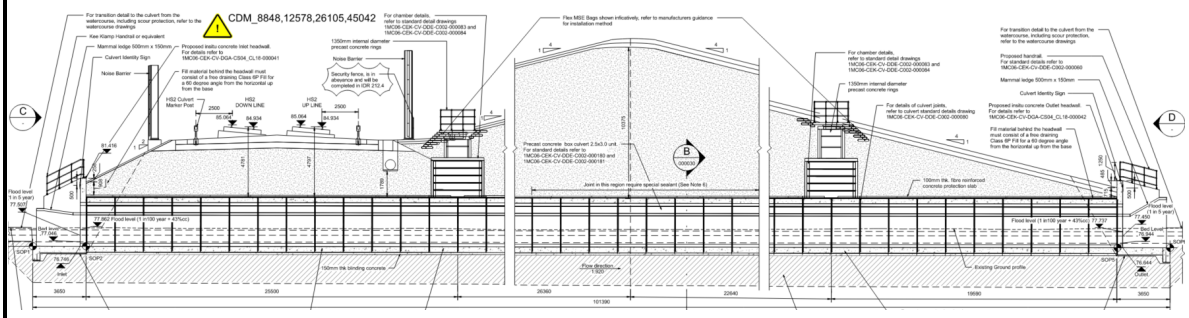


Figure 6.2 - Extract from General Arrangement drawing - Elevation (1MC06-CEK-CV-DSE-CS04_CL18-000031)

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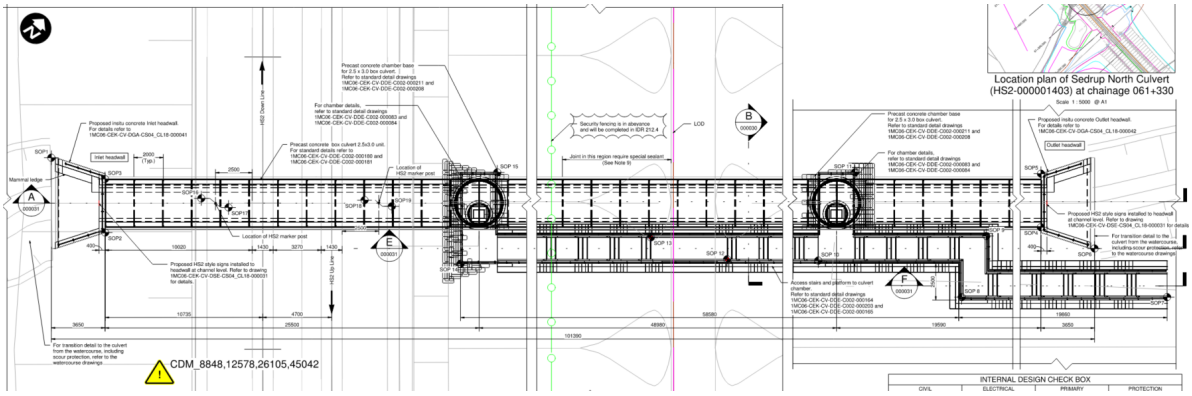


Figure 6.2 - Extract from General Arrangement drawing - HS2 Main Line (1MC06-CEK-CV-DGA-CS04_CL18-000040)

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6.2: Design Standards

- Design was carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available for the culvert.

6.3: Load Cases and Construction Stages

- **PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design**

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic. The minimum cover is 0.8m.

Stage 3 The structure is fully backfilled and in service.

- Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1.9m above the base of culvert.

- Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1.9m above the base of excavation.

- Stage 3 - The structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design.

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6.4: Stage 1 - Summary of Results - Stability Checks

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	89.17	105.23	85%	PASS	kNm/m
DA1,C2	84.10	134.32	63%	PASS	kNm/m
SLS	55.54	108.21	51%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	178.29	221.10	81%	PASS	kNm/m
DA1,C2	166.65	274.92	61%	PASS	kNm/m
SLS	110.55	219.91	50%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	129.69	171.60	76%	PASS
DA1,C2	123.73	244.24	51%	PASS
SLS	81.24	202.48	40%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

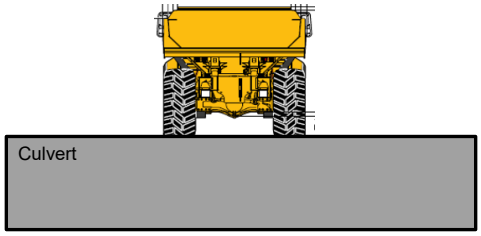
	Granular Subgrade	Natural Subgrade	
DA1,C1	2%	10%	PASS
DA1,C2	3%	12%	PASS

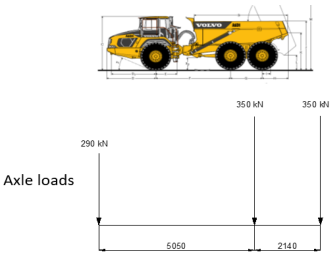
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6.5: Stage 2 - Construction Stage

Check for minimum overlying fill depth before construction traffic (A60H vehicles).

- It is proposed to use A60H vehicles during construction and 0.8m cover is proposed. The vehicles will track directly on top of 0.8m fill placed over the culvert and the embankment either side. The weight of A60H vehicles will surcharge the culvert and the ground at the side of the culverts. Also, any breaking and acceleration of vehicles on the culvert will also result in horizontal forces.

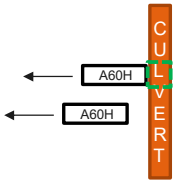




- Four load cases were considered for the analysis on a **single 2m length segment of culvert**:

a) Arrangement of vehicles to give the **maximum axles load** applied directly behind the culvert to give the maximum horizontal force at the side of the culvert.

Schematic Plan of Truck Layout

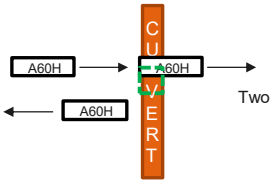


Worst case position of culvert segment analysed

One axle directly behind culvert.

b) Arrangement of vehicles to give the **maximum breaking and acceleration horizontal** with vehicles going in opposite direction (Note: Layout (d) is found more critical).

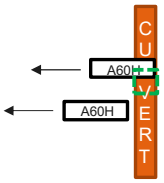
Schematic Plan of Truck Layout



Two Axles on the culvert.

c) Arrangement of vehicles to give the **maximum bearing pressure**.

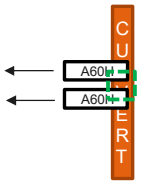
Schematic Plan of Truck Layout



Two Axles on the culvert.

d) Arrangement of vehicles to give the **maximum braking and acceleration horizontal** together with the corresponding maximum horizontal force on the side of the culvert due surcharging from vehicles at the side of the culvert .

Schematic Plan of Truck Layout



Two wheels from 2 x Volvos on culvert and two directly behind the culvert.

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6.5.1: Stage 2a - Summary of Results - Stability Checks

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	216.92	344.6	63%	PASS	kNm/m
DA1,C2	214.94	310.7	69%	PASS	kNm/m
SLS	138.88	171.4	81%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	313.42	637.6	49%	PASS	kNm/m
DA1,C2	302.51	568.5	53%	PASS	kNm/m
SLS	197.85	295.9	67%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	455.03	594.6	77%	PASS
DA1,C2	465.10	584.7	80%	PASS
SLS	294.67	358.9	82%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	5%	18%	PASS
DA1,C2	15%	28%	PASS

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6.5.2: Stage 2b - Summary of Results - Stability Checks

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	174.3	428.6	41%	PASS	kNm/m
DA1,C2	168.6	383.9	44%	PASS	kNm/m
SLS	110.5	259.8	43%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	270.8	701.4	39%	PASS	kNm/m
DA1,C2	256.2	621.3	41%	PASS	kNm/m
SLS	169.4	363.0	47%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	334.4	869.0	38%	PASS
DA1,C2	326.8	873.5	37%	PASS
SLS	214.3	647.7	33%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	6%	20%	PASS

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6.5.3: Stage 2c - Summary of Results - Stability Checks

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	138.0	428.6	32%	PASS	kNm/m
DA1,C2	131.8	383.9	34%	PASS	kNm/m
SLS	86.3	259.8	33%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	234.5	701.4	33%	PASS	kNm/m
DA1,C2	219.3	621.3	35%	PASS	kNm/m
SLS	145.3	363.0	40%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	226.1	869.0	26%	PASS
DA1,C2	220.1	873.5	25%	PASS
SLS	142.1	647.7	22%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	5%	19%	PASS

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6.5.4: Stage 2d - Summary of Results - Stability Checks

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	155.7	512.6	30%	PASS	kNm/m
DA1,C2	142.2	457.0	31%	PASS	kNm/m
SLS	98.1	348.2	28%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	252.2	765.2	33%	PASS	kNm/m
DA1,C2	229.8	674.2	34%	PASS	kNm/m
SLS	157.0	430.2	37%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	355.8	1143.3	31%	PASS
DA1,C2	315.6	1162.2	27%	PASS
SLS	228.5	936.4	24%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	26%	PASS
DA1,C2	8%	30%	PASS

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6.6: Critical Sections and Load Cases - Stage 3

Assumptions

1. 6N2 fill is assumed to be placed against all sides of the concrete structure. This influence of CBGM material surround the culvert structure has been ignored following cl.3.9.9. in HS2-HS2-BR-STD-000-000001 Technical Standard - Bridge Design Basis. The performance of the CBGM is expected to be superior to the 6N material.

2. 6N2 fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

3. Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

4. The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

5. The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

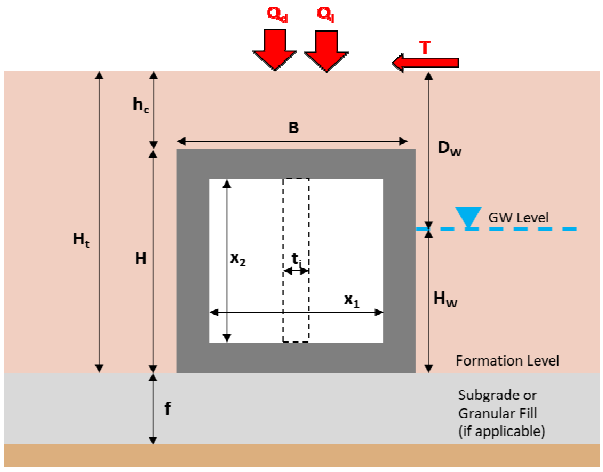
6. It is assumed to be no water in the structure for the Sliding and overturning assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

7. Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

1 .The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Input Data - Geometry, Loading and Material Properties



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Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickne	hc	3.40	m
Depth to culvert base	Ht	7.20	m
Depth of granular fill below founding leve	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	5.30	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	5.30	m

Loading

Unit weight of concrete - Inferior	γc	24.0	kN/m3	
Culvert Self Weight - Inferior	Wc	121.0	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25.0	kN/m3	
Culvert Self Weight - Superior	Wc	126.0	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and F	Qd	13.7	kN/m	
Permanent load due to Fill	Qo	224.4	kN/m	
Total Permanent Load	Qdt	238.1	kN/m	
Live Load Stress~	ql	77.5	kN/m2	
Live Load~	Ql	255.6	kN/m	
Traction and braking force at top of culv	T	0.0	kN/m	~Use dispersed value of T
Unit weight of water	γw	9.8	kN/m3	
Weight of Water - Full	Qw	73.6	kN/m	

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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6.6.1 Stage 3 - Summary of Results - Stability Checks - HS2 Up / Down Main Line

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	364.1	772.0	47%	PASS	kNm/m
DA1,C2	355.0	682.5	52%	PASS	kNm/m
SLS	230.6	392.9	59%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	597.9	1221.7	49%	PASS	kNm/m
DA1,C2	580.2	1073.8	54%	PASS	kNm/m
SLS	377.7	562.3	67%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	652.0	1528.0	43%	PASS
DA1,C2	638.5	1449.7	44%	PASS
SLS	413.8	907.8	46%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	32%	PASS
DA1,C2	6%	32%	PASS

HS2 Ltd - Code 1 - Accepted

7.1: Conclusions

- The settlement analysis is carried out using Plaxis2D_V21
- The calculated total settlement (137mm) below culvert is within the limits stated, i.e. 175mm.
- The calculated Horizontal separation (18.2mm) at the potential fault line is not within the limits stated, i.e. 10mm. Therefore, lining at the potential fault line zone has been proposed, refer to drawing no. 1MC06-CEK-CV-DDE-CS04_CL18-000069
- The analysis assumed that 2.0m of Excavation and replacement below the culvert founding level with Class 1A2 material.

7.2: Recommendations

- It is recommended that 2.0m of the ground will be excavated and replaced below the base of the culvert. The replacement fill shall be granular Class 1A2 material.
- It is assumed in the geotechnical calculations that the base of the culvert will be placed directly on soils with the stiffness value of 25MPa and undrained shear strength of 85kPa.
- To identify the location of the potential fault line, the trench of 11.3m x 105m size shall be conducted. A qualified and competent Geotechnical Engineer shall inspect the excavation of the trench to confirm the stratigraphy & location of the potential fault line.

7.3: Notes for STR drawings

1. A qualified and competent Geotechnical Engineer shall inspect the excavation to confirm the stratigraphy & location of the potential fault line. The site check report shall be submitted to the designer within 24hrs after the excavation reaches the formation level. If the potential fault line is confirmed the location, then the direction of the fault line, the properties of the soils within the fault line and the site photos etc. shall be included in the report.

2. The undrained stiffness (Eu ≥25MPa) or undrained shear strength (Cu ≥85kPa) shall be confirmed at the excavation base along the footprint of the culvert in cohesive soil. Confirmation of stiffness shall be completed by plate bearing test in accordance with BS 1377, with a minimum 600mm diameter plate. Confirmation of undrained shear strength shall be completed by hand vane shear tests. A qualified and competent geologist shall carry out field identification of rock as per Table 5 of BS EN ISO 14689 to identify rock as weak to medium strong rock.

3. As per the geotechnical design of the culvert considering structural requirements, a minimum 2.0m thick E&R with Class 1A2 material below the founding level of culvert is required. An angle of shearing resistance (φ') of minimum 40° is required for Class 1A2 fill in accordance with Table 6/1 of the Specification (Ref:1MC06-CEK-AU-SPE-CS04_CL18-000002).

4. Any spots with Eu ≤25MPa or Cu≤85kPa encountered at E&R formation level shall be excavated and replaced with Class 1A2 along the entire length of the culvert. An angle of shearing resistance (φ') of a minimum 40° is required for Class 1A2 fill in accordance with Table 6/1 of the Specification (Ref:1MC06-CEK-AU-SPE-CS04_CL18-000002).

5. Geotextile separator is required at the base of E&R for the full length of the culvert in accordance with Appendix 6/5 of Earthworks (Series 600) Specification (Ref: 1MC06-CEK-AU-SPE-CS04_CL18-000002).

6. For geotechnical risks, please see the Geotechnical Risk Register 1MC06-CEK-GT-REG-CS04_CL18-000005.

7. CDM risks have been added in Digital CDM and an extract is provided in IDR Package 207.1 – CDM Register (Ref: 1MC06-CEK-AU-REG-CS04_CL18-000042).

8. At landscape bund, for backfilling behind the headwall and wingwalls, sides of the culvert and top cover of the culvert Class 6N2 shall be used satisfying a minimum strength requirement of the angle of shearing resistance (φ') of 40° in accordance with Table 6/1 of the Specification (Ref: 1MC06-CEK-AU-SPE-CS04_CL18-000002). At the Mainline embankment, CBGM shall be used satisfying the minimum angle of shearing resistance (φ') of 40° and the requirements mentioned in Table 6/1 of the Specification (Ref: 1MC06-CEK-AU-SPE-CS04_CL18-000002).

9. A minimum cover of 0.8m shall be placed above the culvert before any construction vehicle movements cross the culvert. At Landscape bund, Class 6N2 shall be used satisfying a minimum strength requirement of the angle of shearing resistance (φ') of 40° in accordance with Table 6/1 of the Specification (Ref: 1MC06-CEK-AU-SPE-CS04_CL18-000002). At the Mainline embankment, CBGM shall be used satisfying the minimum angle of shearing resistance (φ') of 40° and the requirements mentioned in Table 6/1 of the Specification (Ref: 1MC06-CEK-AU-SPE-CS04_CL18-000002).

10. The tolerance for maximum difference in fill level to the opposite sides of the culvert is 250mm. Placement of fill with differential levels greater than 250mm on either side of the culvert requires a temporary works design. It has been assumed that earthworks above the culvert (mainline earthworks and landscape bunds) will be constructed concurrently. No allowance has been made in the design for sequencing the earthworks construction along the culvert.

11. HS2 mainline transition details are presented on Sedrup North Culvert – HS2 Mainline Transition Details drawing (Ref: 1MC06-CEK-GT-DDE-CS04_CL18-000003)

12. Groundwater is expected to be encountered during excavation; therefore, appropriate temporary drainage measures are to be employed. All excavations are to be dry before excavation is backfilled, and material compacted.

13. The design of the temporary excavation is not included in the permanent design. The Contractor shall ensure the stability of the side slopes of the excavation.

HS2 Ltd - Code 1 - Accepted

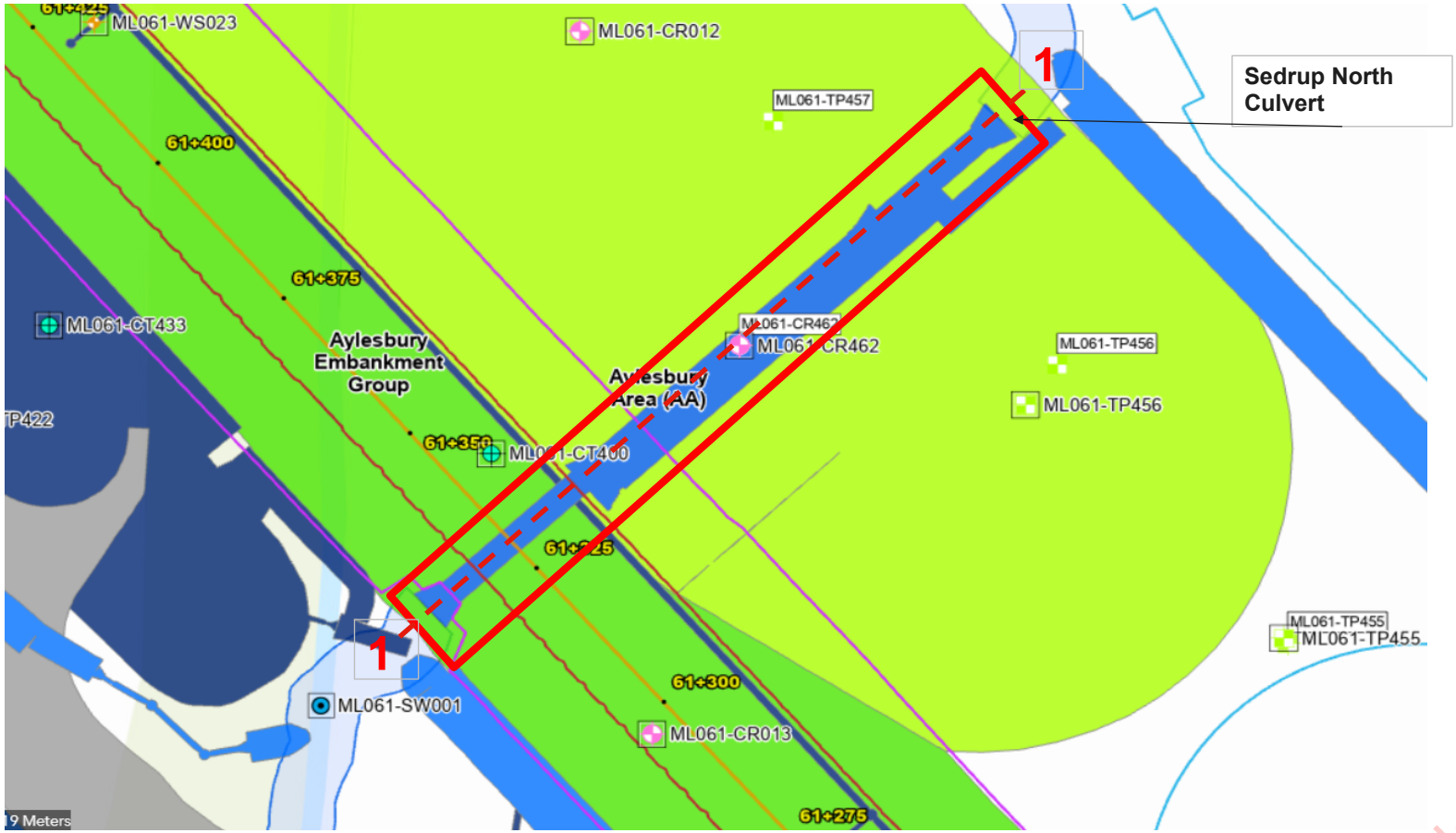
SUBJECT

Sedrup North Culvert

SHEET NO.

Appendix H-2 : Maps

HS2 Ltd - Code 1 - Accepted



SUBJECT

TITLE

Sedrup North Culvert	H1.1: Existing GI Location Map - Extract from ArcGIS online viewer

HS2 Ltd - Code 1 - Accepted

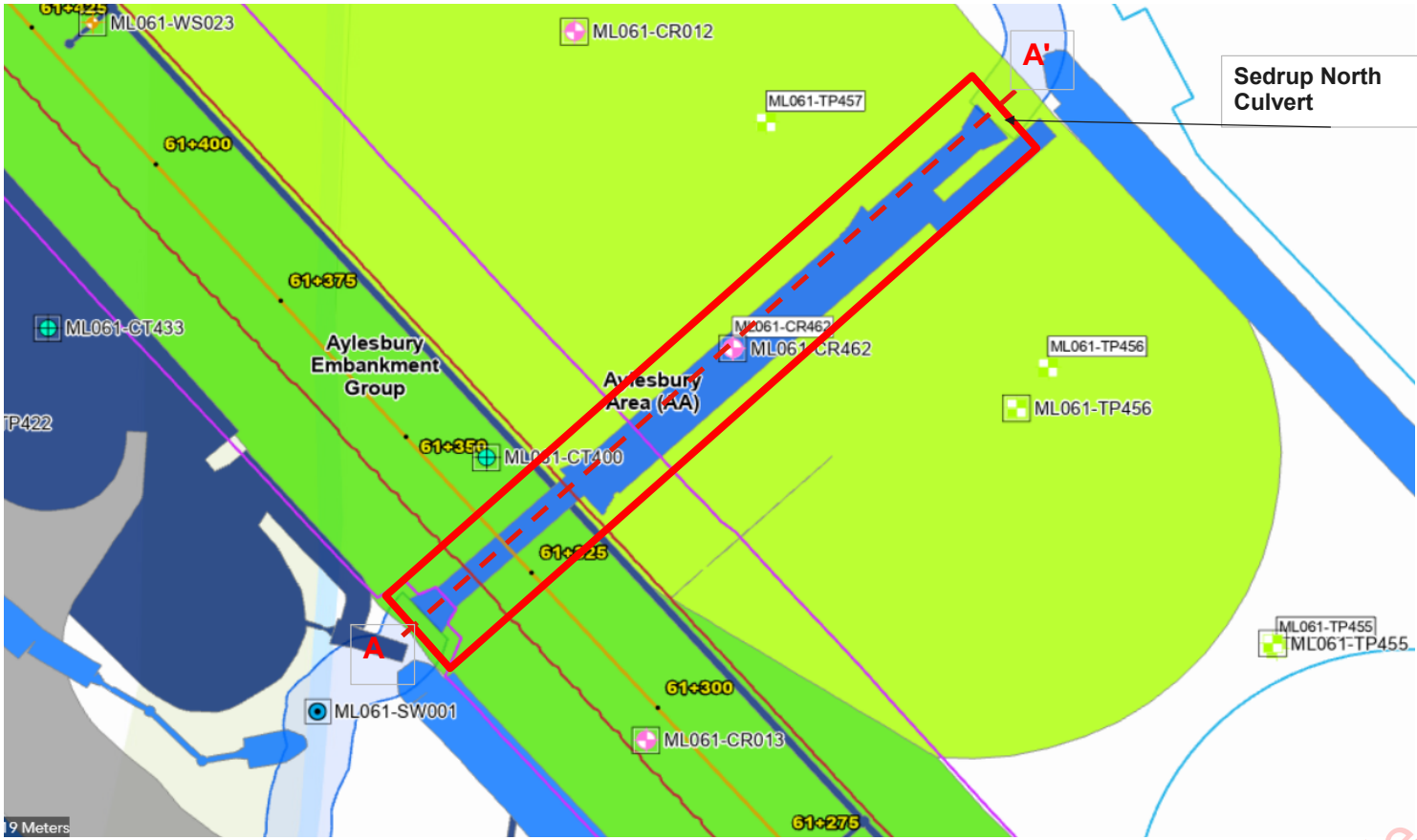
SUBJECT

Sedrup North Culvert

SHEET NO.

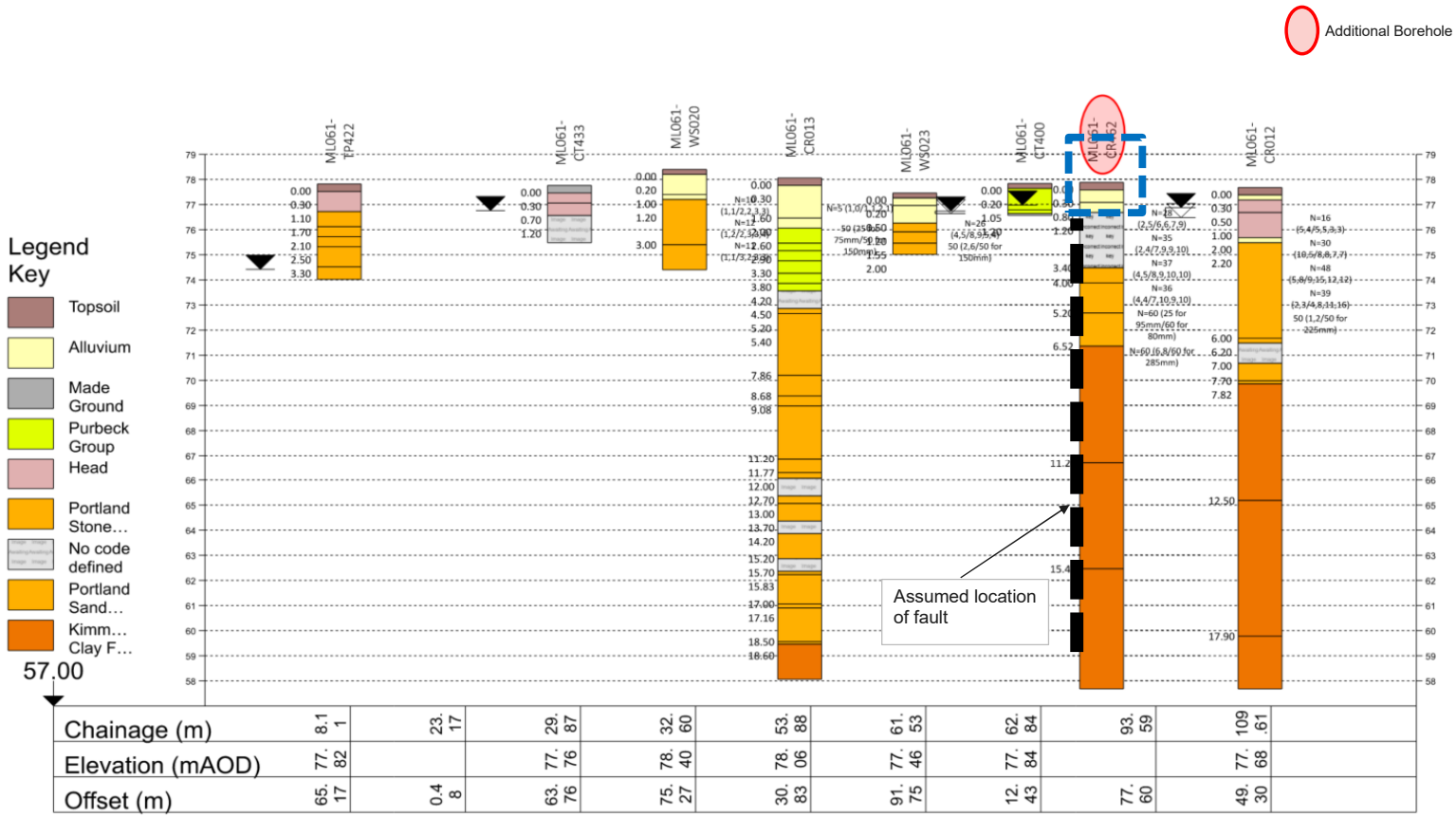
Appendix H-3 : Geological section

HS2 Ltd - Code 1 - Accepted



SUBJECT	TITLE
Sedrup North Culvert	**H2.1: Geological Cross-Section - plan**

HS2 Ltd - Code 1 - Accepted



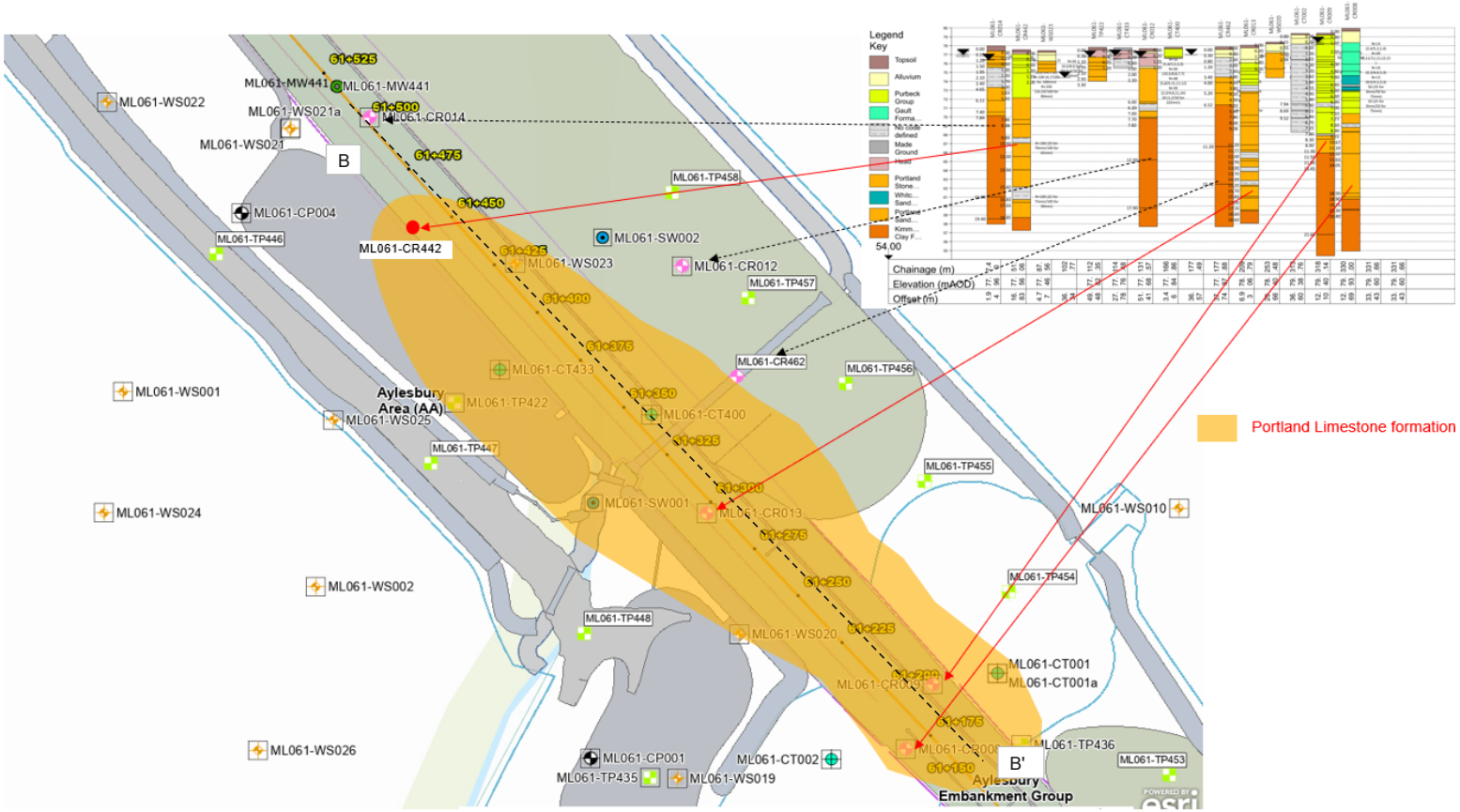
SUBJECT

Sedrup North Culvert

TITLE

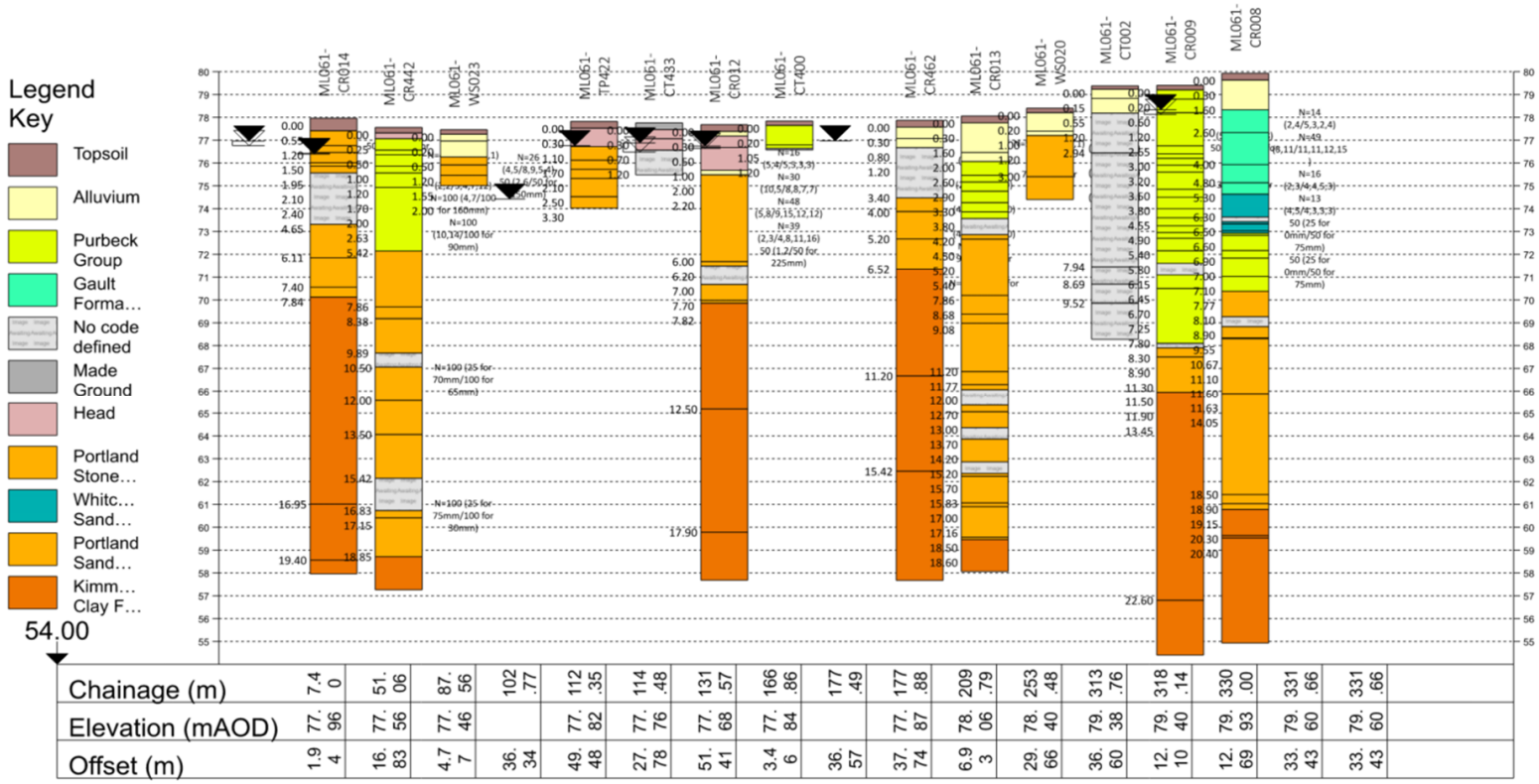
H2.2: Geological Longitudinal Section (section 1-1)

HS2 Ltd - Code 1 - Accepted



SUBJECT	TITLE
Sedrup North Culvert	**H2.3: GI Location Map - Highlighted the presence of limestone to identify the variation in strata / fault**

HS2 Ltd - Code 1 - Accepted



SUBJECT	TITLE
Sedrup North Culvert	**H2.4: GI Location Map - Longitudinal section along mainline (~200m upline and downline) to identify the presence of Limestone, Refer H2.3 for Plan**

HS2 Ltd - Code 1 - Accepted

SUBJECT

Sedrup North Culvert

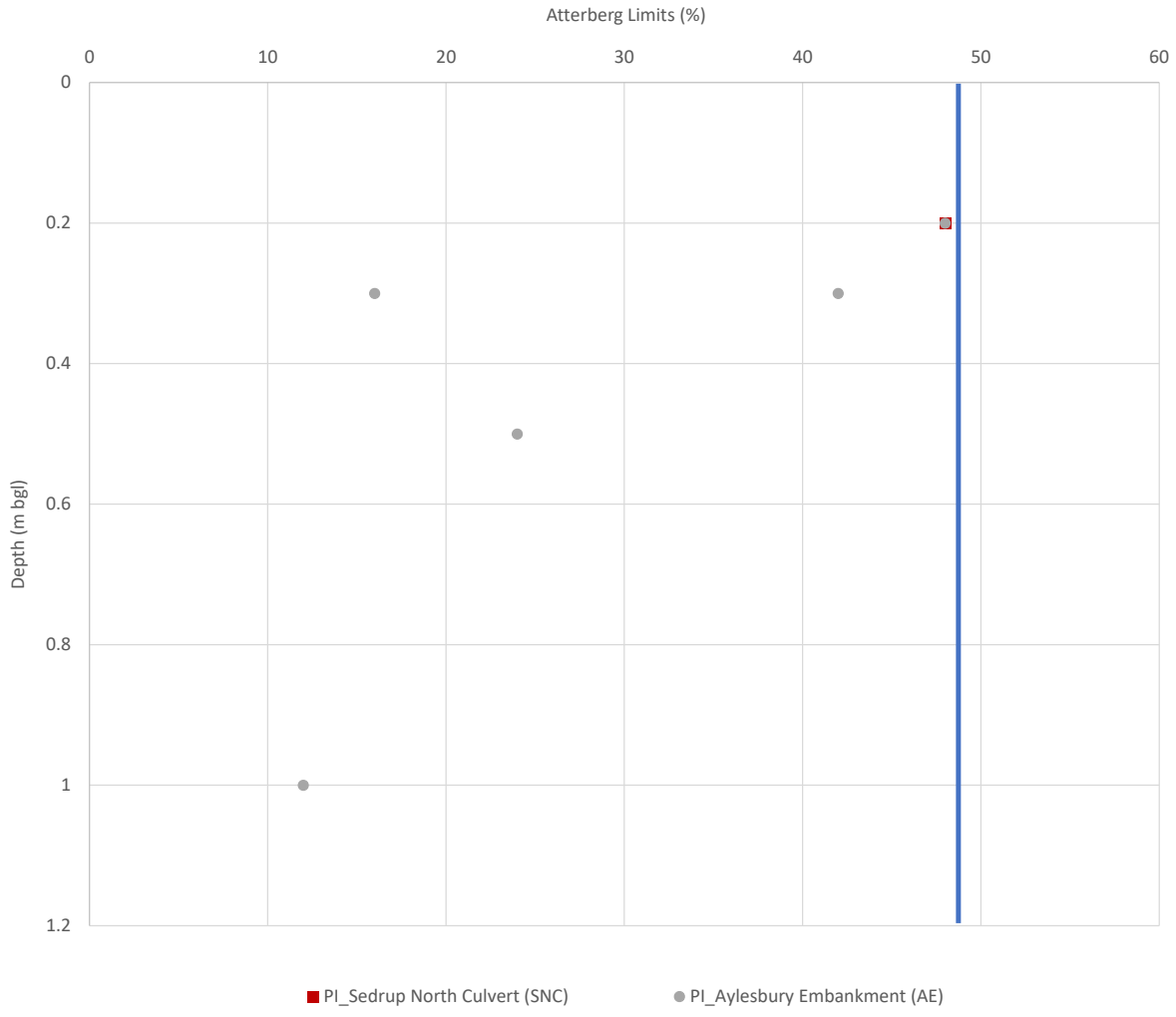
SHEET NO.

Appendix H-4 : Data Plots

HS2 Ltd - Code 1 - Accepted

Liquid limit, Plastic limit and Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	ALV-C Z	Figure:	H.4.6



Number of test	6

Notes

Number of test value denotes number of Atterberg Limits tests completed only.

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

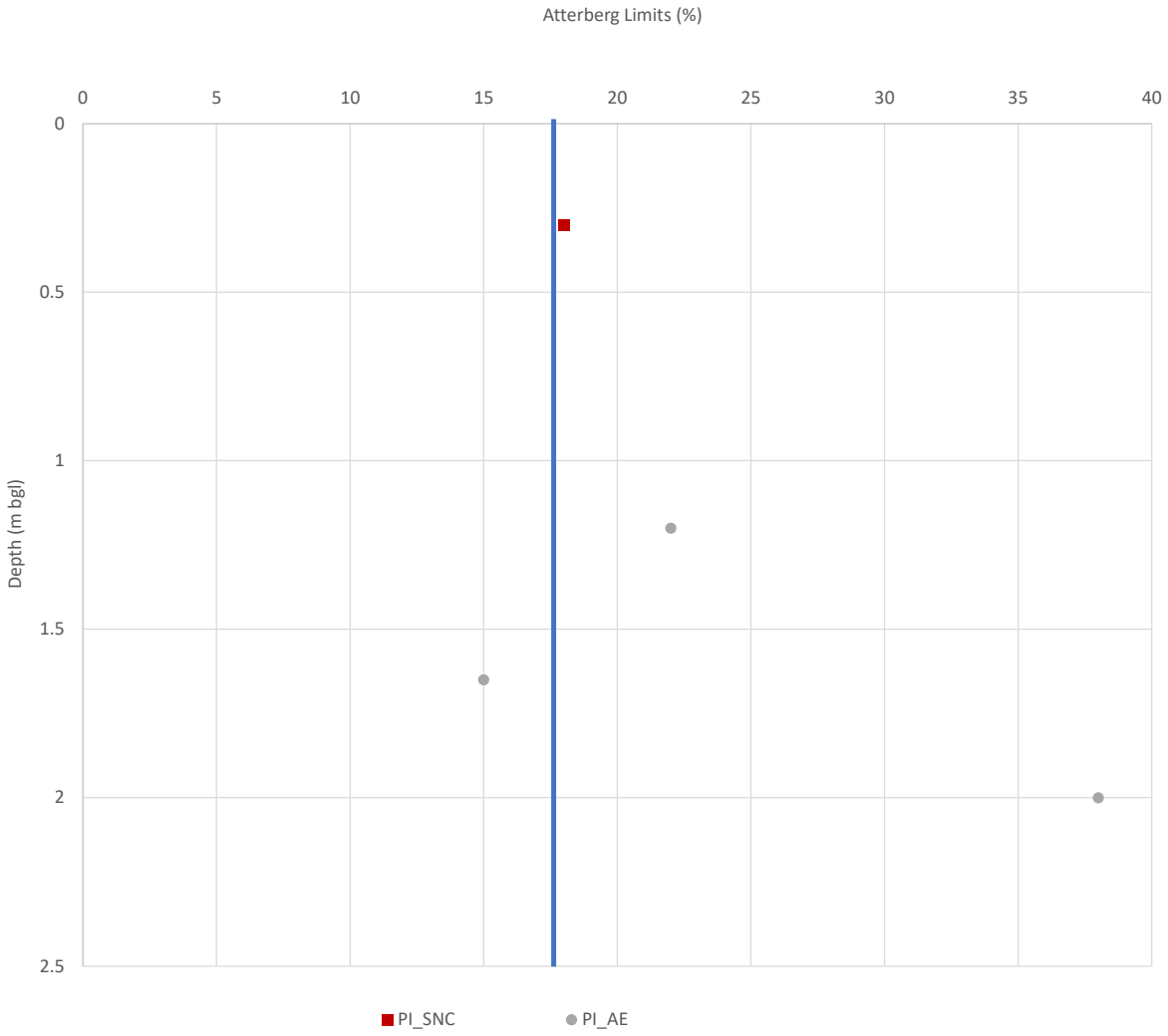
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Liquid limit, Plastic limit and Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	HEAD-S V SV	Figure:	H.4.7



Number of test	4

Notes

Number of test value denotes number of Atterberg Limits tests completed only.

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

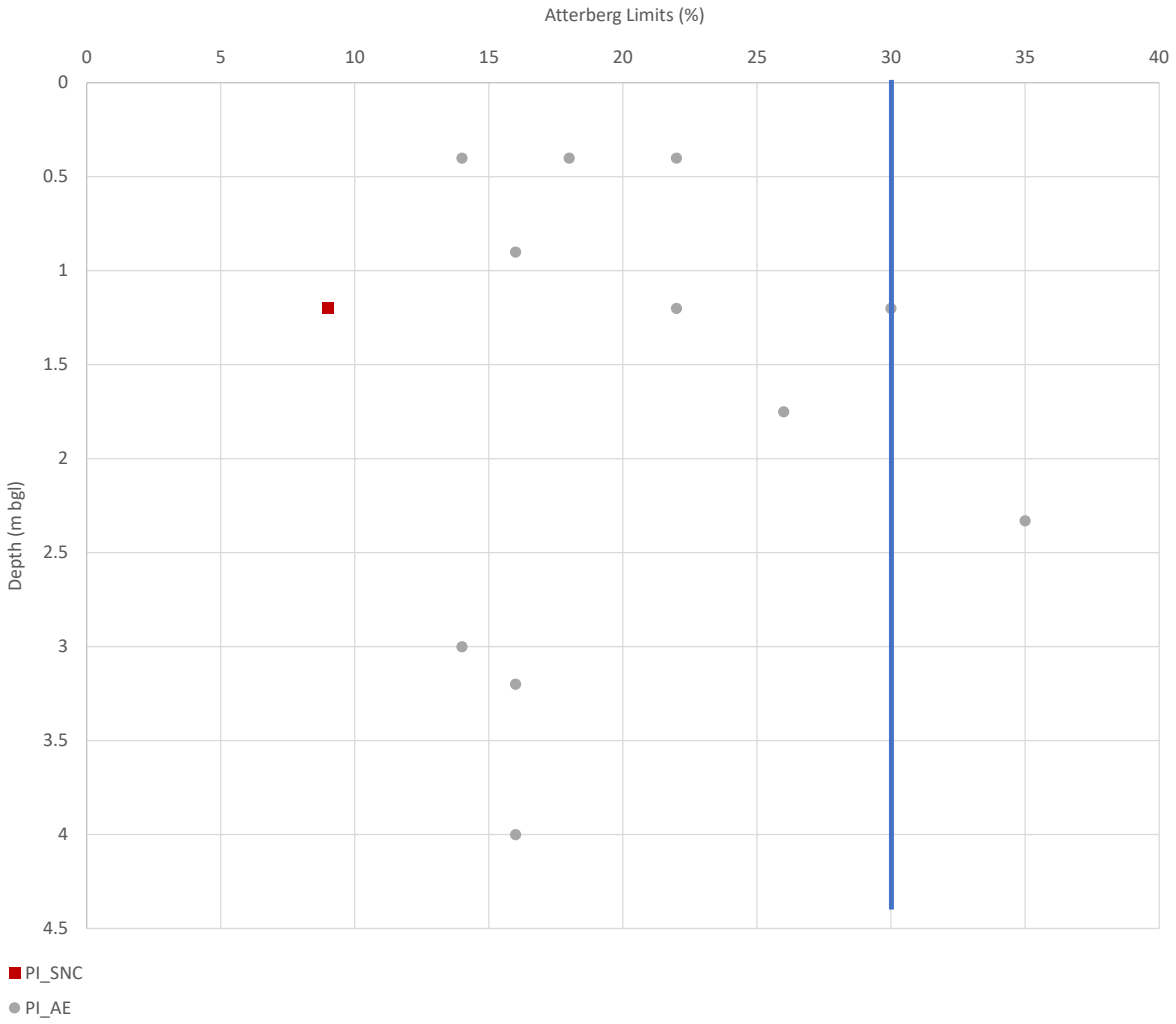
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Liquid limit, Plastic limit and Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	POST-C Z	Figure:	H.4.8



Number of test	12

Notes

Number of test value denotes number of Atterberg Limits tests completed only.

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

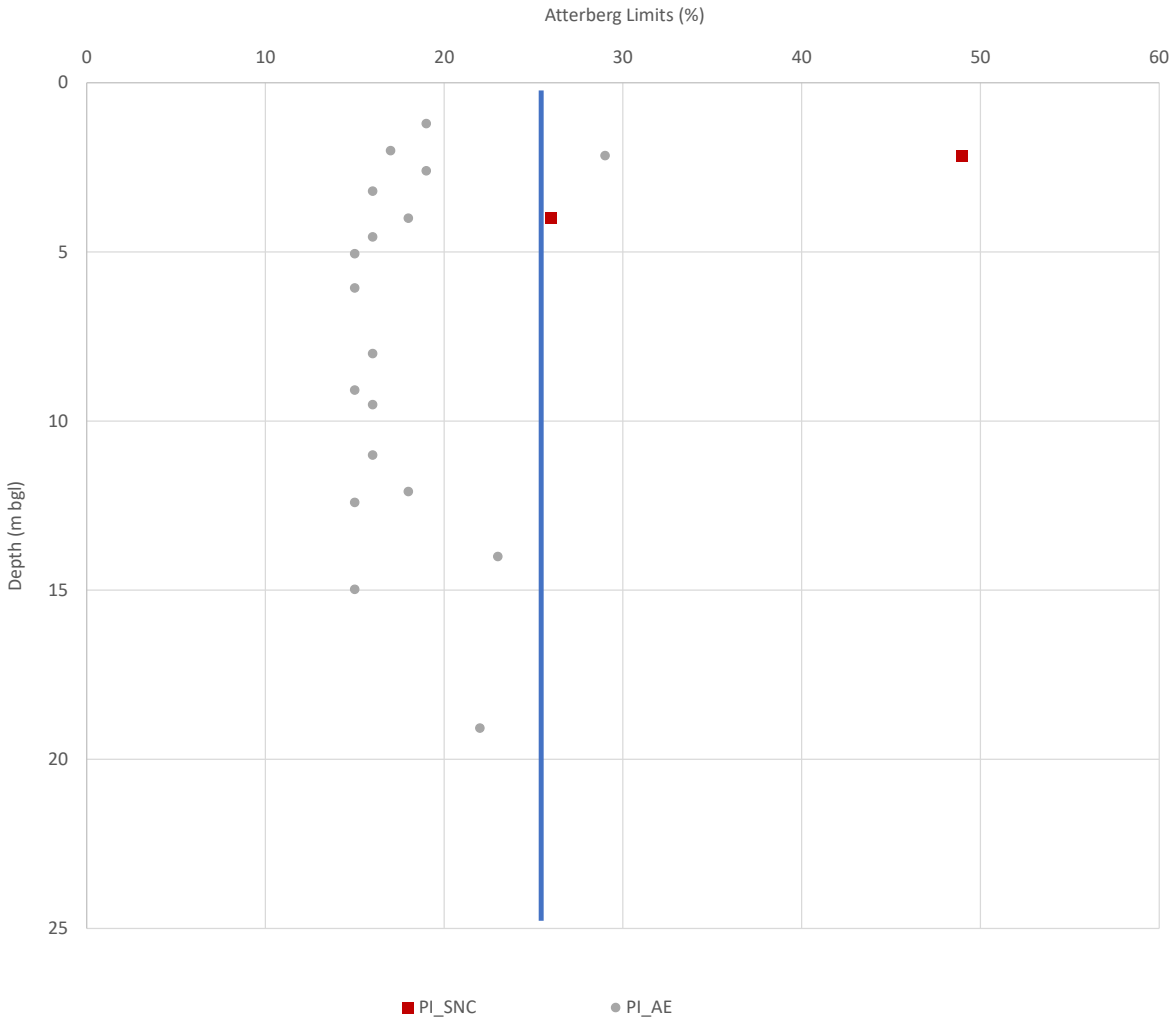
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Liquid limit, Plastic limit and Plasticity Index with depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	KC-C Z	Figure:	H.4.9



Number of test	20

Notes

Number of test value denotes number of Atterberg Limits tests completed only.

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

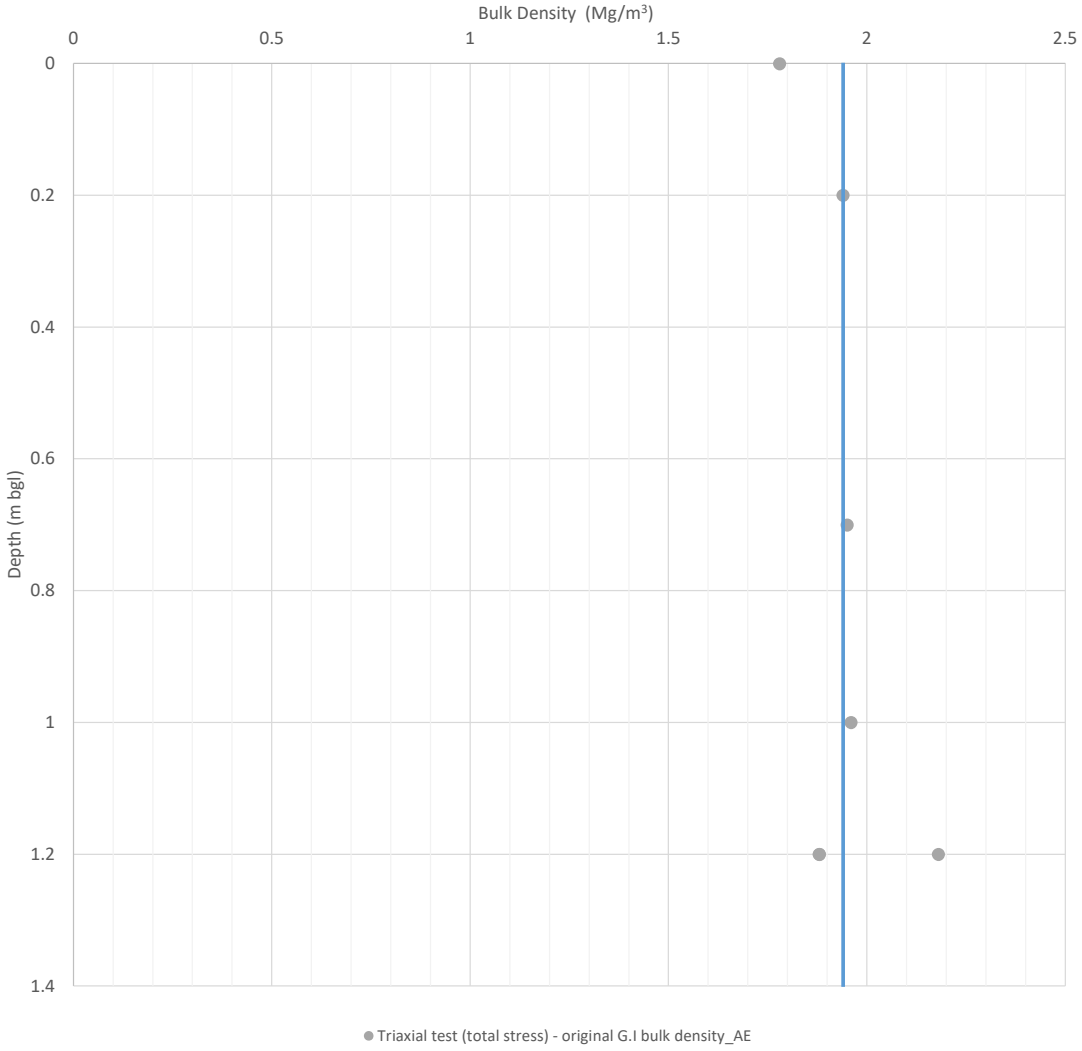
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Bulk Density vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	Head	Figure:	H.4.9



	Bulk Density
Number of Tests	7
Minimum	1.78
Maximum	2.18

Notes

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

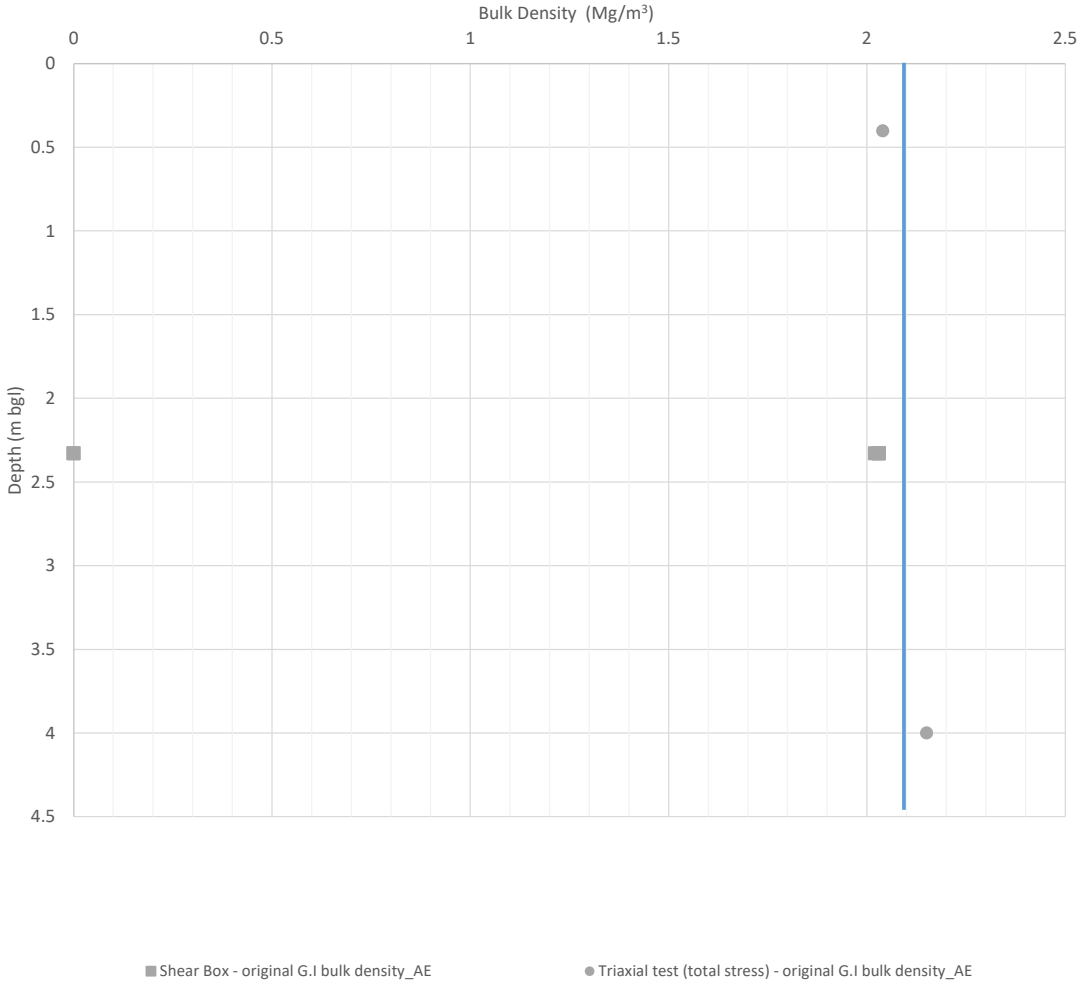
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Bulk Density vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	POST-C Z	Figure:	H.4.10



	Bulk Density
Number of Tests	6
Minimum	2.02
Maximum	2.15

Notes

Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

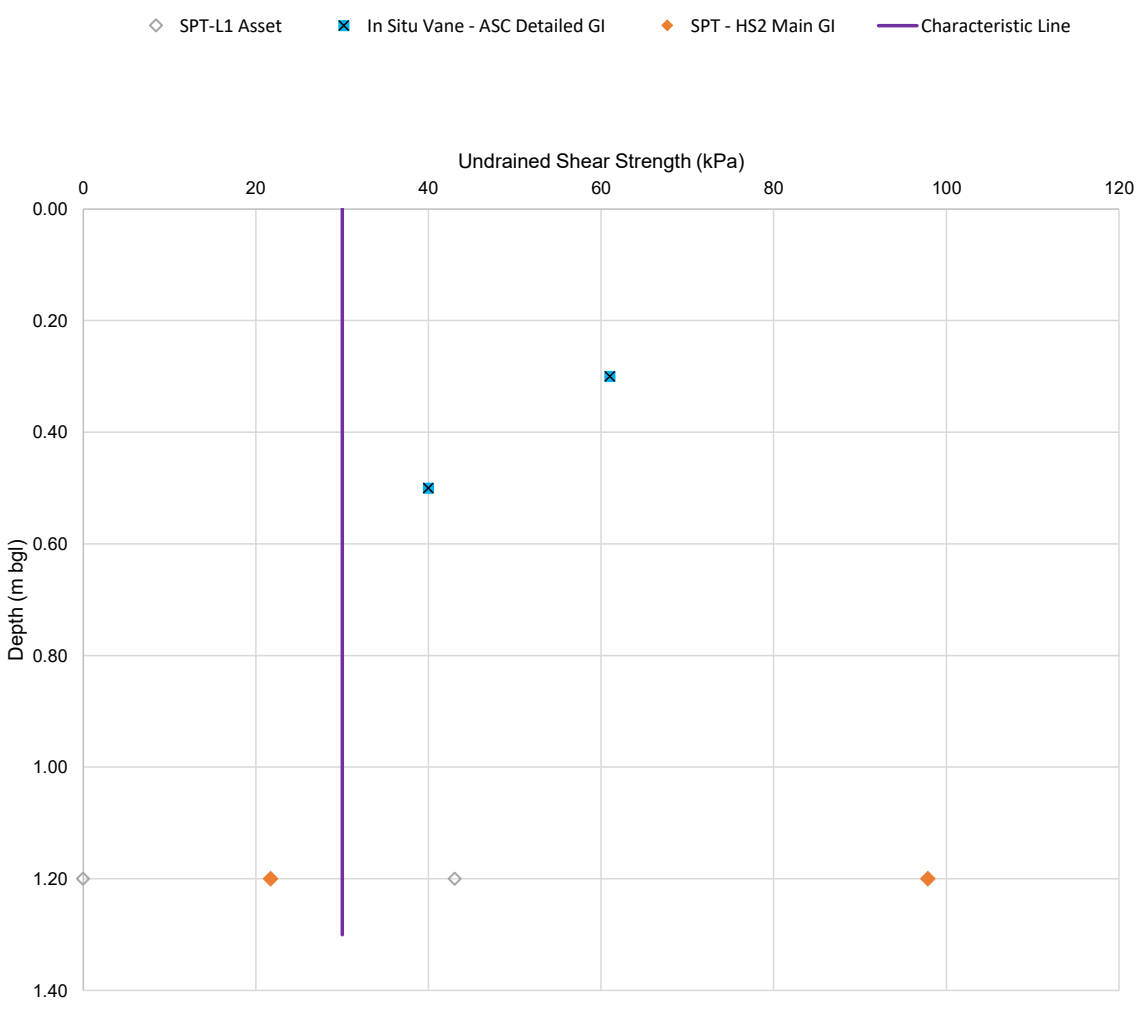
SNC: Sedrup North Culvert
AE: Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	ALV	Figure:	H.4.1



	Total
Number of Tests	-
Minimum	22
Maximum	98

Notes
Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

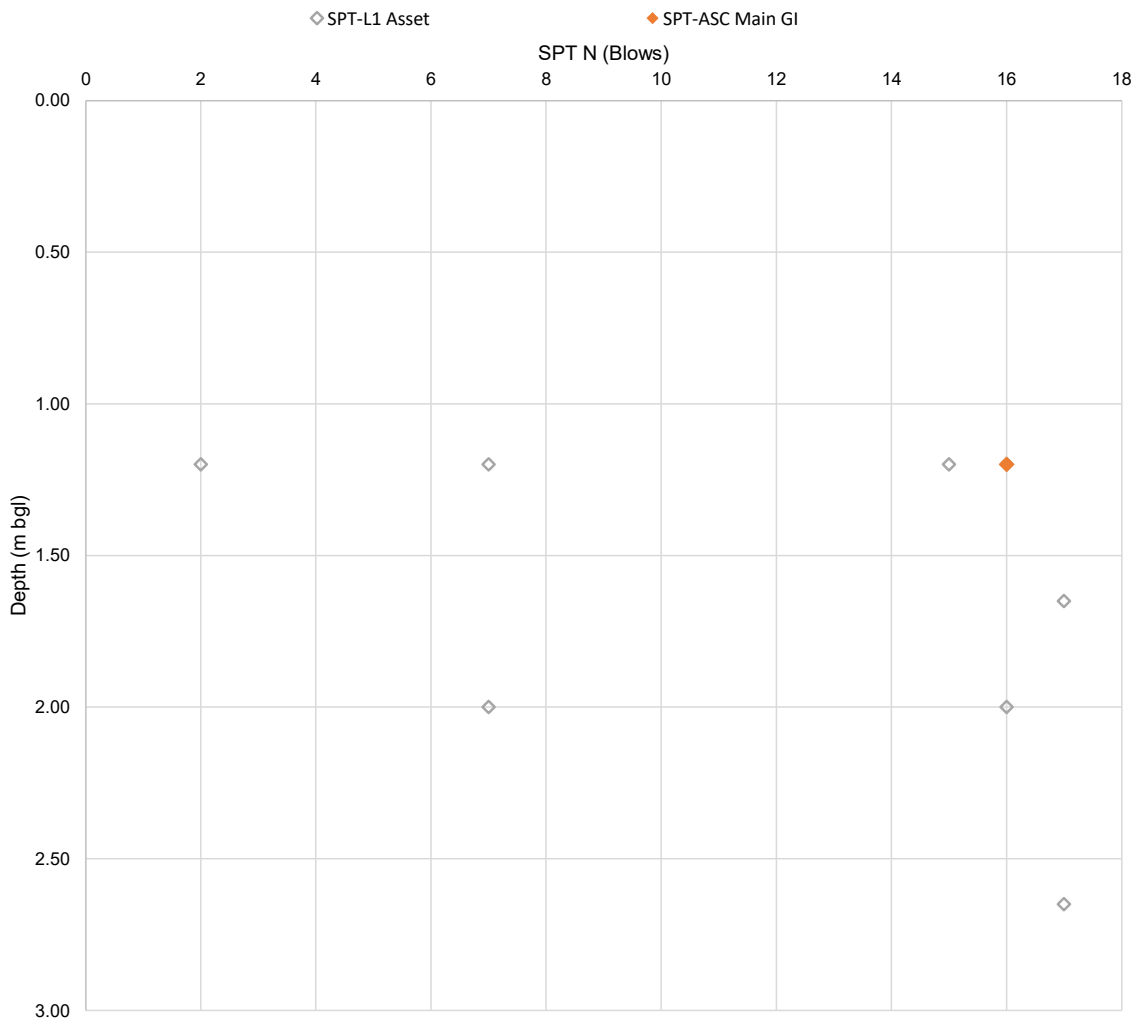
L1 Asset - Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	HEAD-S V SV	Figure:	H.4.2



	Total
Number of Tests	-
Minimum	-
Maximum	-

Notes
Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

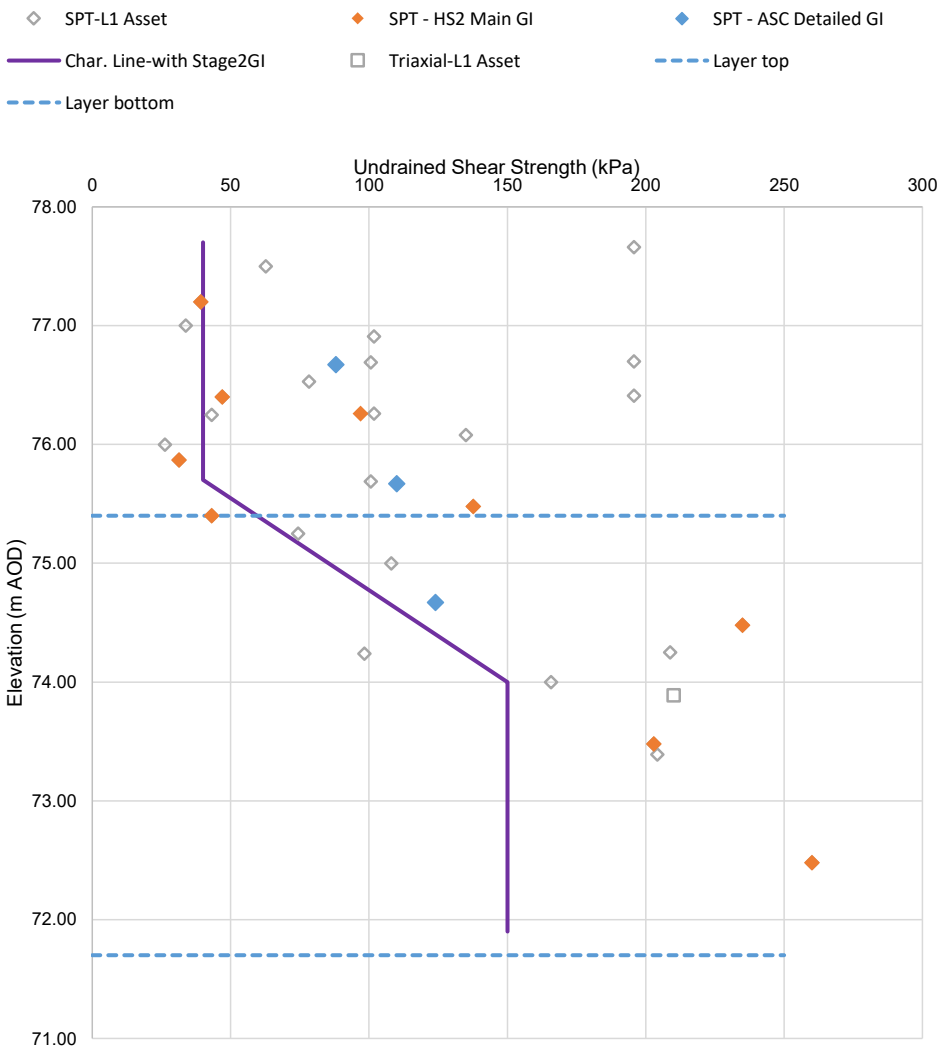
L1 Asset - Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	POST-C Z	Figure:	H.4.3



	Total
Number of Tests	-
Minimum	31
Maximum	260

Notes
Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

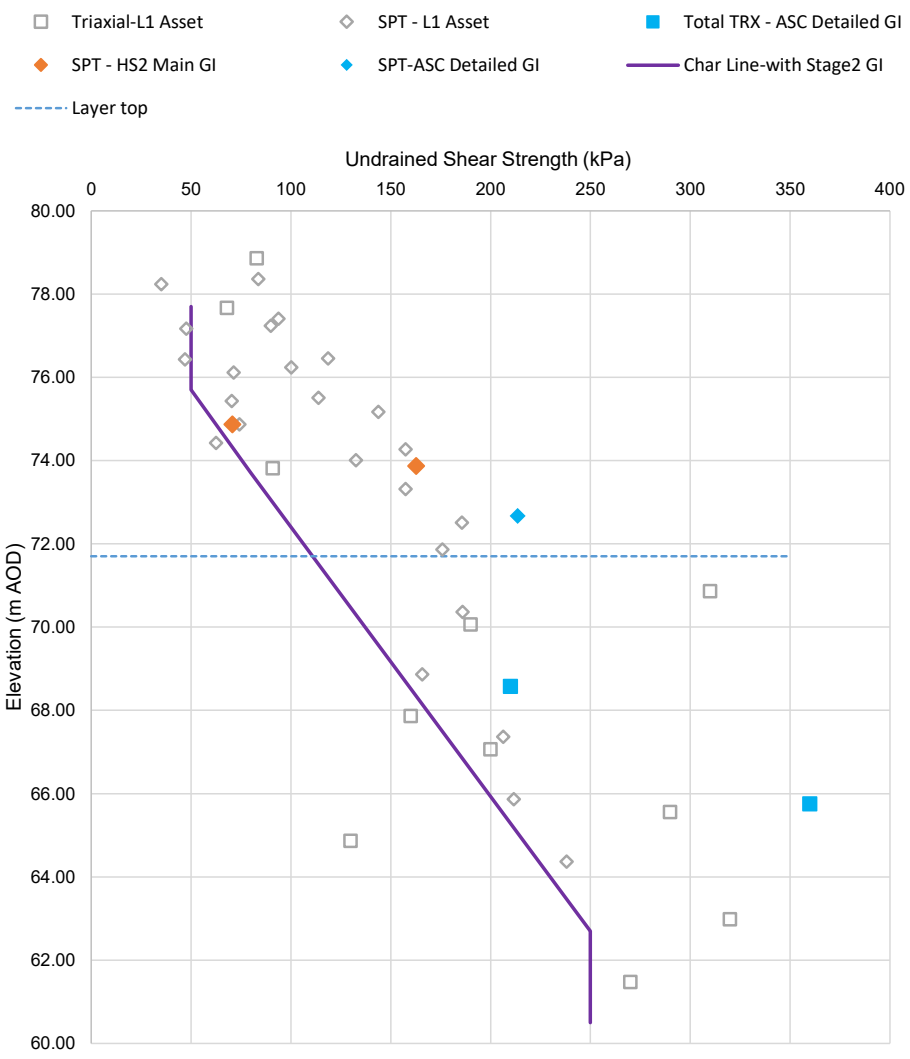
L1 Asset - Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	0	Figure:	H.4.4



	Total
Number of Tests	-
Minimum	71
Maximum	360

Notes
Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

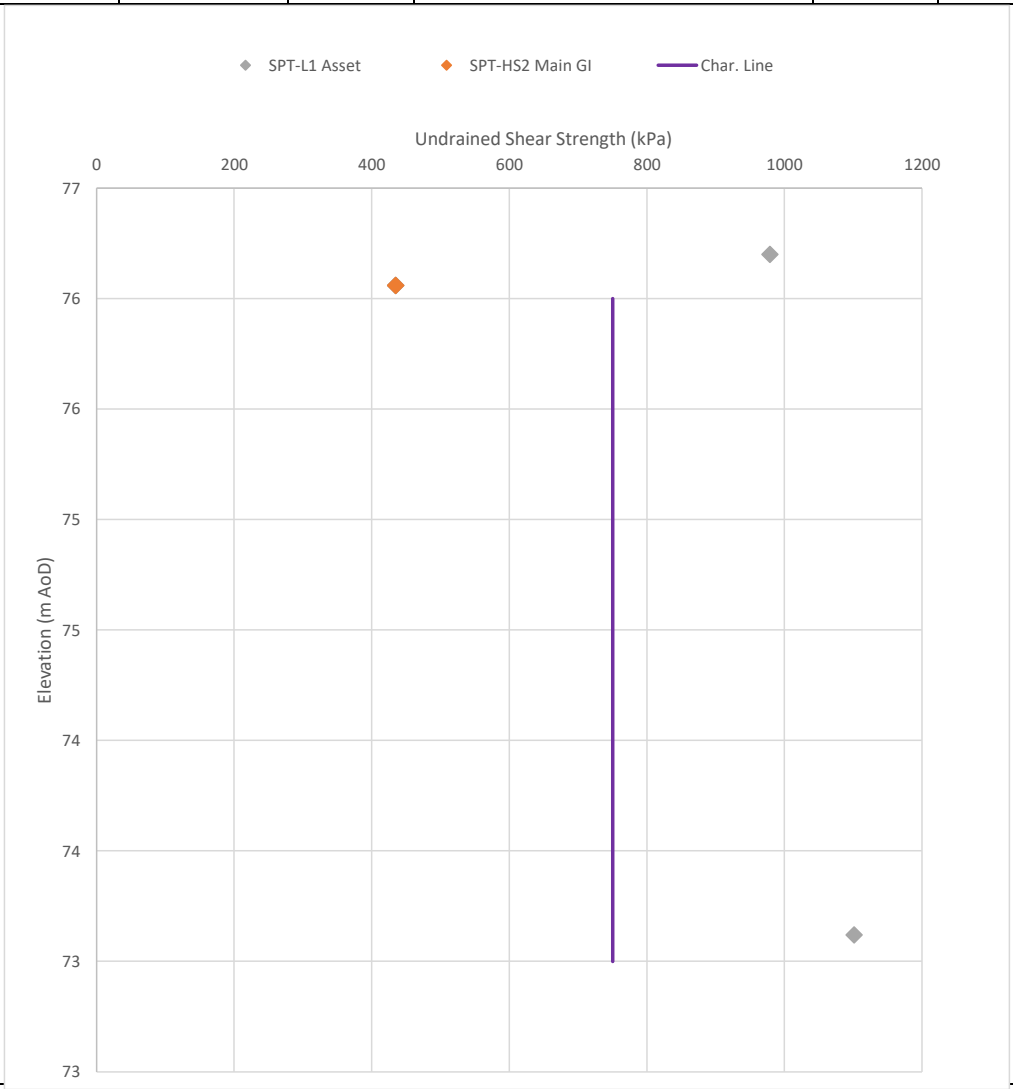
L1 Asset - Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

Undrained Shear Strength vs. Depth (m bgl)

Project ID:	10039367	Project Title:	1MC06 - Stage One C2 - MWCC - North Portal of Chiltern Tunnel to Brackley	Location:	Sedrup North Culvert
Client:	HS2	Geology :	PB-LMST	Figure:	H.4.5



	Total
Number of Tests	11
Minimum	435
Maximum	1101

Notes
Background C3 scheme wide parameters shown are based on holebase database geology codes dated 10/09/2021.

L1 Asset - Aylesbury Embankment

Plot Date:	24/04/2023
Database reference	C2-C3-CONT-ASC

HS2 Ltd - Code 1 - Accepted

SUBJECT

Sedrup North Culvert

SHEET NO.

Appendix H-5 : Stability checks

HS2 Ltd - Code 1 - Accepted

REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design. See Section 4.7.

The calculations presented in this section consider **STAGE 1.**

HS2 Ltd - Code 1 - Accepted

REF | CALCULATIONS | OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

The load spread from Class I foundation material to underlying natural material is assumed to be at 30 degrees (1H:1.7V spread load). Hence the width of load spread at natural ground level is 5.6 m.

HS2 Ltd - Code 1 - Accepted

REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

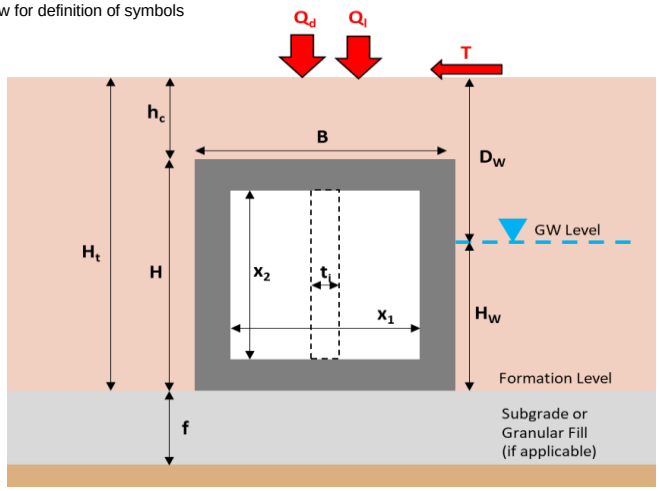
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

HS2 Ltd - Code 1 - Accepted

REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	0.00	m
Depth to culvert base	Ht	3.80	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	1.90	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	1.90	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	120.96	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	0	kN/m	
Permanent load due to Fill	Qo	0	kN/m	
Total Permanent Load	Qdt	0	kN/m	
Live Load Stress~	ql	10	kN/m2	
Live Load~	Ql	33	kN/m	
Traction and braking force at top of culvert*	T	0	kN/m	
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - Full	Qw	73.575	kN/m	

~Use dispersed value of T

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states			
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax
0.33	0.33	SLS	0.60
0.33	0.40	GEO (Comb 1)	0.72
0.41	0.49	GEO (Comb 2)	0.84

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

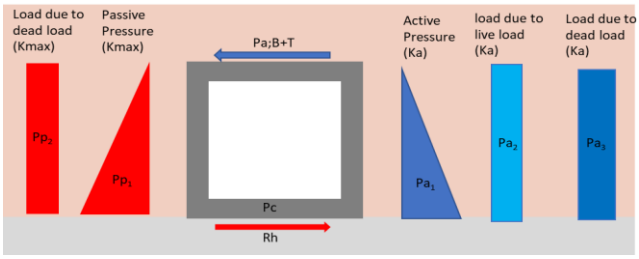
Interface Friction (δ) between fill and subgrade interface

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

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Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	0.00	kN/m
DA1, C2	0.00	kN/m
SLS	0.00	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	0.00	kN/m
DA1, C2	0.00	kN/m
SLS	0.00	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*Ka*H*γQ

	Pa2	
DA1, C1	18.81	kN/m
DA1, C2	20.25	kN/m
SLS	12.54	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	89.13	kN/m
DA1, C2	109.45	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	114.9	83.0	31.9	16.1	kN/m
DA1, C2	121.0	61.5	59.5	24.9	kN/m
SLS	121.0	61.5	59.5	30.0	kN/m

Results of Sliding Check - at Concrete/Subgrade Interface

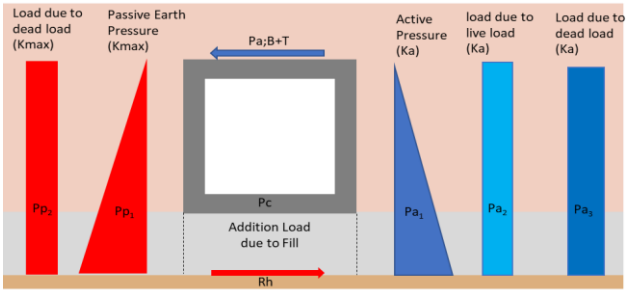
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	89.2	105.2	85%	PASS
DA1,C2	84.1	134.3	63%	PASS
SLS	55.5	108.2	51%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Sliding Check - Granular Fill / Subgrade Interface

H	=	3.80 m
f	=	2.00 m
H + f = H'	=	5.80 m
H_{wULS} + f	=	3.90 m
H_{wSLS} + f	=	3.90 m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	0.00	kN/m
DA1, C2	0.00	kN/m
SLS	0.00	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	0.00	kN/m
DA1, C2	0.00	kN/m
SLS	0.00	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*Ka*H'*γQ

	P'a2	
DA1, C1	28.71	kN/m
DA1, C2	30.91	kN/m
SLS	19.14	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	189.47	kN/m
DA1, C2	232.68	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	252.9	170.4	82.4	31.6	kN/m
DA1, C2	266.2	126.3	139.9	42.2	kN/m
SLS	266.2	126.3	139.9	53.7	kN/m

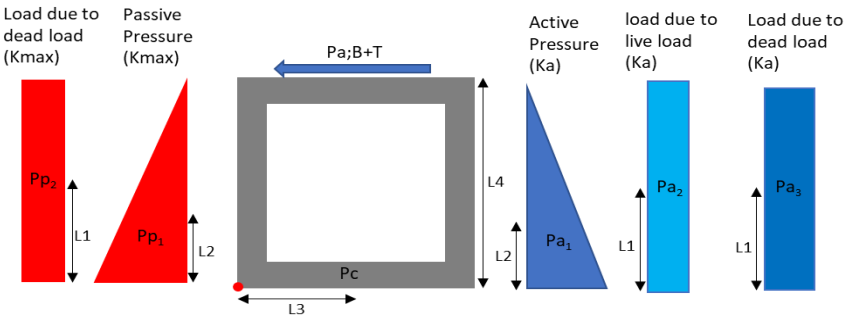
Results of Sliding Check - at Granular Fill/Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	178.3	221.1	81%	PASS
DA1,C2	166.6	274.9	61%	PASS
SLS	110.6	219.9	50%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

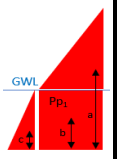
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	0.00	kNm/m
DA1, C2	0.00	kNm/m
SLS	0.00	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance



Lever Arm		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	119.00	kNm/m
DA1, C2	146.14	kNm/m
SLS	104.39	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	52.59	kNm/m
DA1, C2	98.09	kNm/m
SLS	98.09	kNm/m

Lever arm, L3 = 1.65 m

Lever arms:

a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	0.00	kNm/m
DA1, C2	0.00	kNm/m
SLS	0.00	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2

Ma2 = Pa2 * L1

	Pa2	
DA1, C1	35.74	kNm/m
DA1, C2	38.48	kNm/m
SLS	23.83	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	0	kNm/m
DA1, C2	0	kNm/m
SLS	0	kNm/m

Lever arm, L4 = 3.8 m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	129.7	171.6	76%	PASS	kNm/m
DA1,C2	123.7	244.2	51%	PASS	kNm/m
SLS	81.2	202.5	40%	PASS	kNm/m

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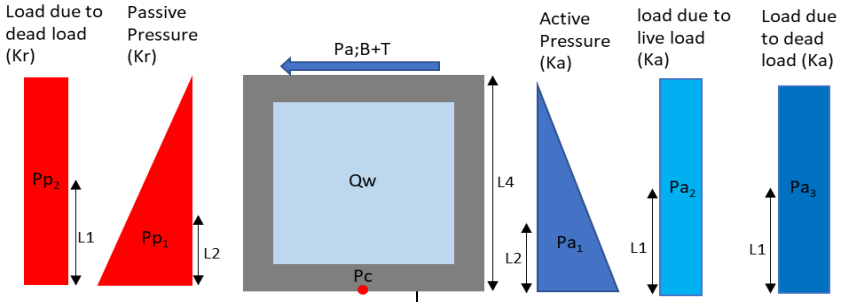
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r,DA1C1}$ = 0.72

Considering Passive Earth Pressure $K_{r,DA1C2}$ = 0.84

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	0.00	kN/m
DA1, C2	0.00	kN/m
SLS	0.00	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

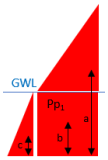
Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

Mp2 = Pp2 * L1

	Pp2	
DA1, C1	0.00	kNm/m
DA1, C2	0.00	kNm/m
SLS	0.00	kNm/m

Lever arm, L_1 = 1.9 m



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = (0.5*H$^{2)}$*γ*Kr*γG)

	Pp1	
DA1, C1	89.13	kN/m
DA1, C2	109.45	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	119.00	kNm/m
DA1, C2	146.14	kNm/m
SLS	104.39	kNm/m

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REF | CALCULATIONS | OUTPUT

1

Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

	Ma3 + Ma2 + Ma1+Mbt	Mp2 + Mp1 + Mpc	
	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	129.7	119.0	10.7
DA1,C2	123.7	146.1	0.0
SLS	81.2	104.4	0.0

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

	Pa3 + Pa2 + Pa1+Pb+t	Pp2 + Pp1	
	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	89.2	89.1	0.0
DA1,C2	84.1	109.5	0.0
SLS	55.5	78.2	0.0

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 126.0 kN /m run (unfactored)

Total Live Load for bearing = Ql = 33.0 kN /m run (unfactored)

Total Water Load for bearing = Qw = 73.6 kN /m run (unfactored)
(Assuming Culvert Full of Water)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	10.7	0.0
DA1,C2	0.0	0.0
SLS	0.0	0.0

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	2%	10%	PASS
DA1,C2	3%	12%	PASS

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SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	89.17	105.2	85%	PASS	kNm/m
DA1,C2	84.10	134.3	63%	PASS	kNm/m
SLS	55.54	108.2	51%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	178.29	221.1	81%	PASS	kNm/m
DA1,C2	166.65	274.9	61%	PASS	kNm/m
SLS	110.55	219.9	50%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	129.7	171.6	76%	PASS
DA1,C2	123.7	244.2	51%	PASS
SLS	81.2	202.5	40%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	2%	10%	PASS
DA1,C2	3%	12%	PASS

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REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design.

The calculations presented in this section consider **STAGE 2A.**

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REF | CALCULATIONS | OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

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REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

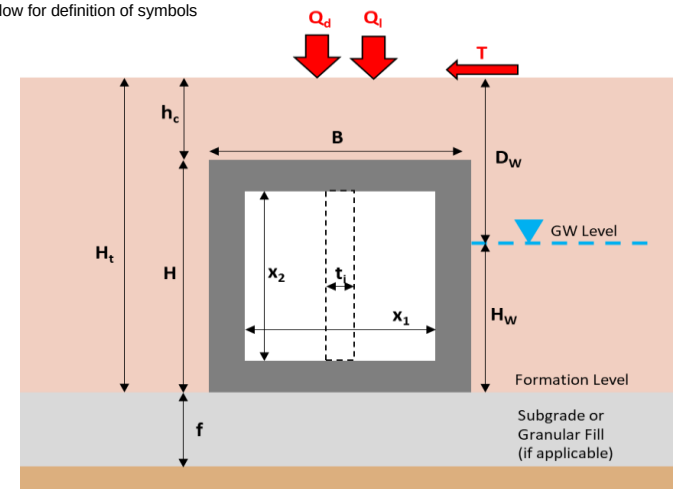
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

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REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	0.80	m
Depth to culvert base	Ht	4.60	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	2.70	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	2.70	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	121	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	0	kN/m	
Permanent load due to Fill	Qo	52.8	kN/m	
Total Permanent Load	Qdt	52.8	kN/m	
Live Load Stress~	ql	0	kN/m2	Added A60H load directly further on in cals
Live Load~	Ql	0	kN/m	
Traction and braking force at top of culvert*	T	0	kN/m	~Use dispersed value of T
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - ULS level	Qw	36.8	kN/m	

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states					
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax	Design - Earth Pressure Kr	Limiting Earth Pressure Kp*
0.33	0.33	SLS	0.60	0.60	0.6
0.33	0.40	GEO (Comb 1)	0.72	1.67	3.10
0.41	0.49	GEO (Comb 2)	0.84	1.38	2.40

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required
*Kr for ULS can approach the limiting passive earth pressure (Kp) but cannot exceed it. This is on the basis that there is no limit to displacements in the temporary case. See Note 3 of Table B-5 in PD6694-1:2011.

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

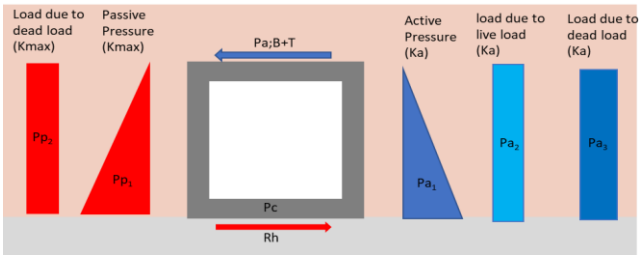
Interface Friction (δ) between fill and subgrade interface

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

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Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	96.46	kN/m
DA1, C2	83.90	kN/m
SLS	36.48	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	32.83	kN/m
DA1, C2	29.79	kN/m
SLS	20.06	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=qL*γQ

	Pa2	
DA1, C1	113.73	kN/m
DA1, C2	121.30	kN/m
SLS	75.82	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	206.73	kN/m
DA1, C2	179.82	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	165.1	83.0	82.0	41.4	kN/m
DA1, C2	173.8	61.5	112.3	46.9	kN/m
SLS	173.8	61.5	112.3	56.7	kN/m

Results of Sliding Check - at Concrete/Subgrade Interface

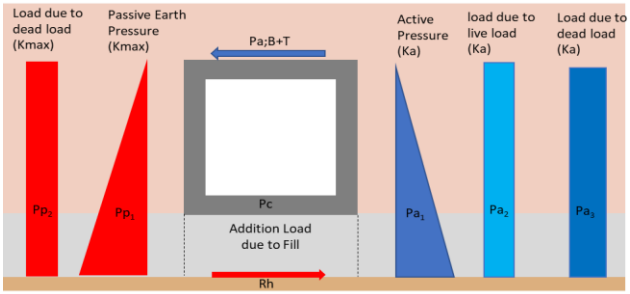
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	216.9	344.6	63%	PASS
DA1,C2	214.9	310.7	69%	PASS
SLS	138.9	171.4	81%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Sliding Check - Granular Fill / Subgrade Interface

H	=	3.80 m
f	=	2.00 m
H + f = H'	=	5.80 m
H_{wULS} + f	=	3.90 m
H_{wSLS} + f	=	3.90 m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	147.23	kN/m
DA1, C2	128.06	kN/m
SLS	55.68	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	50.11	kN/m
DA1, C2	45.47	kN/m
SLS	30.62	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*γQ

	P'a2	
DA1, C1	113.73	kN/m
DA1, C2	121.30	kN/m
SLS	75.82	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	439.47	kN/m
DA1, C2	382.27	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	303.0	170.4	132.6	50.9	kN/m
DA1, C2	319.0	126.3	192.7	58.2	kN/m
SLS	319.0	126.3	192.7	74.0	kN/m

Results of Sliding Check - at Granular Fill/Subgrade Interface

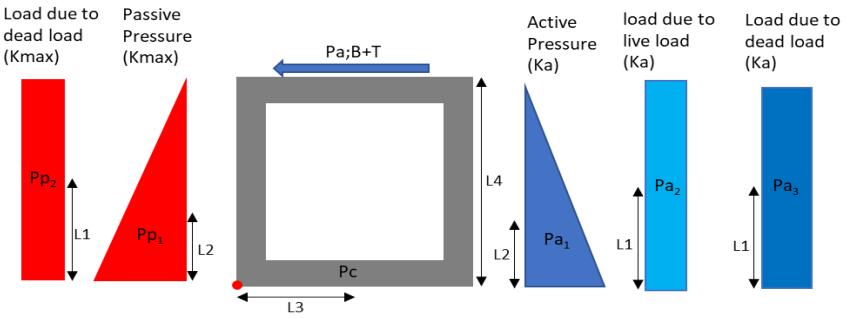
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	313.4	637.6	49%	PASS
DA1,C2	302.5	568.5	53%	PASS
SLS	197.9	295.9	67%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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REF | CALCULATIONS | OUTPUT

Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

Mp2 = Pp2 * L1

	Pp2	
DA1, C1	183.27	kNm/m
DA1, C2	159.42	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	62.38	kNm/m
DA1, C2	56.60	kNm/m
SLS	38.12	kNm/m

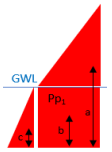
Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2

Ma2 = BM * partial factor

	Pa2	
DA1, C1	298.70	kNm/m
DA1, C2	323.24	kNm/m
SLS	199.13	kNm/m

Lever arm, L_1 = N/A m



Lever Arm		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	276.02	kNm/m
DA1, C2	240.09	kNm/m
SLS	104.39	kNm/m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	135.36	kNm/m
DA1, C2	185.21	kNm/m
SLS	185.21	kNm/m

Lever arm, L3 = 1.65 m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	0	kNm/m
DA1, C2	0	kNm/m
SLS	0	kNm/m

Lever arm, L4 = 3.8 m

Lever arms:		
a	2.5333	m
b	0.95	m
c	0.6333	m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	455.0	594.6	77%	PASS	kNm/m
DA1,C2	465.1	584.7	80%	PASS	kNm/m
SLS	294.7	358.9	82%	PASS	kNm/m

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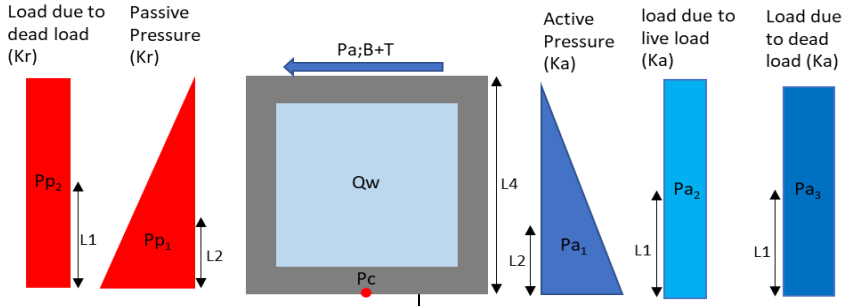
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r,DA1C1}$ = 0.88

Considering Passive Earth Pressure $K_{r,DA1C2}$ = 0.82

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	50.83	kN/m
DA1, C2	49.86	kN/m
SLS	36.48	kN/m

Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

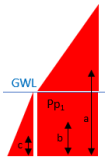
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	96.57	kNm/m
DA1, C2	94.73	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Horizontal load due to live load

Live load does not contribute to passive resistance



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = (0.5*H$^{2)}$*γ*Kr*γG)

	Pp1	
DA1, C1	108.93	kN/m
DA1, C2	114.67	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	145.45	kNm/m
DA1, C2	142.66	kNm/m
SLS	104.39	kNm/m

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Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

Ma3 + Ma2 + Ma1+Mbt Mp2 + Mp1 + Mpc

	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	455.0	242.0	213.0
DA1,C2	465.1	237.4	227.7
SLS	294.7	173.7	121.0

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

Pa3 + Pa2 + Pa1+Pb+t Pp2 + Pp1

	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	216.9	159.8	57.2
DA1,C2	214.9	164.5	50.4
SLS	138.9	114.7	24.2

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 178.8 kN /m run (unfactored)

Total Live Load for bearing = Ql = 0.0 kN /m run (unfactored)

Total Water Load for bearing = Qw = 36.8 kN /m run (unfactored)

(Assuming ULS water level in the Culvert)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	213.0	57.2
DA1,C2	227.7	50.4
SLS	121.0	24.2

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	5%	18%	PASS
DA1,C2	15%	28%	PASS

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REF | CALCULATIONS | OUTPUT

SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	216.92	344.6	63%	PASS	kNm/m
DA1,C2	214.94	310.7	69%	PASS	kNm/m
SLS	138.88	171.4	81%	PASS	kNm/m

OUTPUT: ok

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	313.42	637.6	49%	PASS	kNm/m
DA1,C2	302.51	568.5	53%	PASS	kNm/m
SLS	197.85	295.9	67%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	455.0	594.6	77%	PASS
DA1,C2	465.1	584.7	80%	PASS
SLS	294.7	358.9	82%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	5%	18%	PASS
DA1,C2	15%	28%	PASS

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REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design.

The calculations presented in this section consider **STAGE 2B.**

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REF | CALCULATIONS | OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

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REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

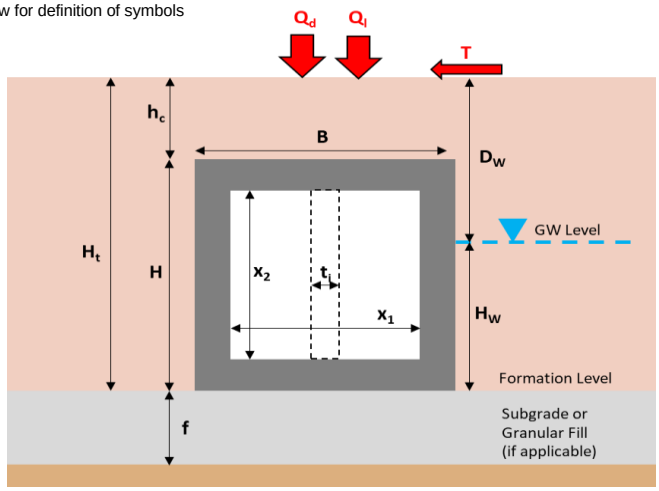
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

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REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	0.80	m
Depth to culvert base	Ht	4.60	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	2.70	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	2.70	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	121	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	0	kN/m	
Permanent load due to Fill	Qo	52.8	kN/m	
Total Permanent Load	Qdt	52.8	kN/m	
Live Load Stress~	ql	0	kN/m2	Added A60H load directly further on in cals
Live Load~	Ql	0	kN/m	
Traction and braking force at top of culvert*	T	17.5	kN/m	~Use dispersed value of T
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - ULS water Level	Qw	36.8	kN/m	

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

HS2 Ltd - Code 1 - Accepted

Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states					
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax	Design - Earth Pressure Kr	Limiting Earth Pressure Kp*
0.33	0.33	SLS	0.60	0.60	0.6
0.33	0.40	GEO (Comb 1)	0.72	1.67	3.10
0.41	0.49	GEO (Comb 2)	0.84	1.38	2.40

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required
*Kr for ULS can approach the limiting passive earth pressure (Kp) but cannot exceed it. This is on the basis that there is no limit to displacements in the temporary case. See Note 3 of Table B-5 in PD6694-1:2011.

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

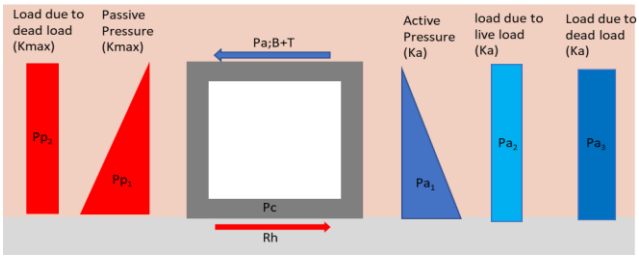
Interface Friction (δ) between fill and subgrade interface

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

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Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	96.46	kN/m
DA1, C2	83.90	kN/m
SLS	36.48	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	32.83	kN/m
DA1, C2	29.79	kN/m
SLS	20.06	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*γQ

	Pa2	
DA1, C1	44.84	kN/m
DA1, C2	52.24	kN/m
SLS	29.90	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	206.73	kN/m
DA1, C2	179.82	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

The vertical axis load is necessary to generate the braking forces and therefore if unfavourable braking forces are applied then so should the vertical live load as permanent load.

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	26.25	kN/m
DA1, C2	22.75	kN/m
SLS	17.5	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd*	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	331.3	83.0	248.3	125.4	kN/m
DA1, C2	348.8	61.5	287.3	120.1	kN/m
SLS	348.8	61.5	287.3	145.1	kN/m

* Added live axial load / width x favourable permanent partial factor over culvert

Results of Sliding Check - at Concrete/Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	174.3	428.6	41%	PASS
DA1,C2	168.6	383.9	44%	PASS
SLS	110.5	259.8	43%	PASS

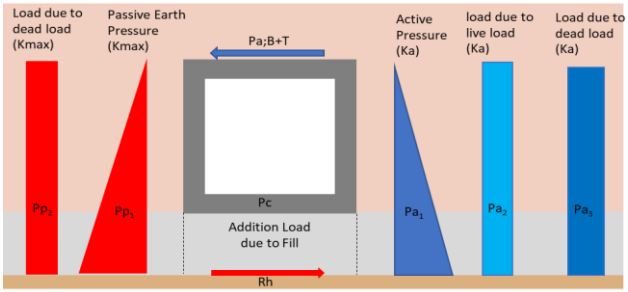
Use limiting Kp to reflect utilisation for ULS sliding.

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REFERENCE | CALCULATIONS | OUTPUT

Sliding Check - Granular Fill / Subgrade Interface

H =	3.80	m
f =	2.00	m
H + f = H' =	5.80	m
H_{wULS} + f =	3.90	m
H_{wSLS} + f =	3.90	m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	147.23	kN/m
DA1, C2	128.06	kN/m
SLS	55.68	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	50.11	kN/m
DA1, C2	45.47	kN/m
SLS	30.62	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*γQ

	P'a2	
DA1, C1	44.84	kN/m
DA1, C2	52.24	kN/m
SLS	29.90	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	439.47	kN/m
DA1, C2	382.27	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	26.25	kN/m
DA1, C2	22.75	kN/m
SLS	17.5	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd*	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	469.3	170.4	298.8	114.7	kN/m
DA1, C2	494.0	126.3	367.7	111.0	kN/m
SLS	494.0	126.3	367.7	141.1	kN/m

* Added live axial load / width x favourable permanent partial factor over culvert

Results of Sliding Check - at Granular Fill/Subgrade Interface

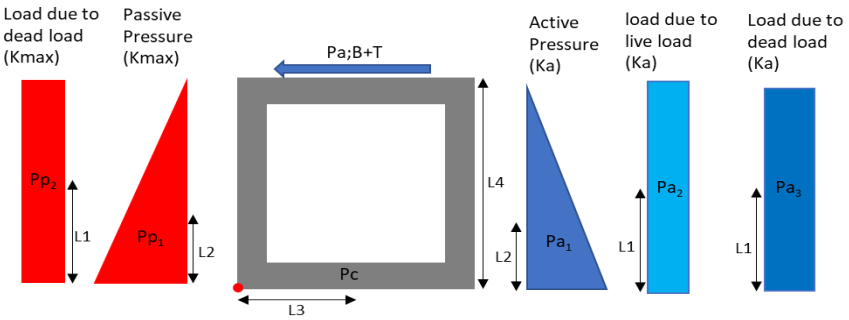
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	270.8	701.4	39%	PASS
DA1,C2	256.2	621.3	41%	PASS
SLS	169.4	363.0	47%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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REF | CALCULATIONS | OUTPUT

Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

Mp2 = Pp2 * L1

	Pp2	
DA1, C1	183.27	kNm/m
DA1, C2	159.42	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	62.38	kNm/m
DA1, C2	56.60	kNm/m
SLS	38.12	kNm/m

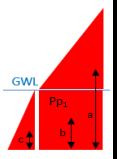
Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2 due to live load

Ma2 = BM x partial factor

	Pa2	
DA1, C1	78.35	kNm/m
DA1, C2	98.50	kNm/m
SLS	52.23	kNm/m

Lever arm, L_1 = N/A m



Lever Arm		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	276.02	kNm/m
DA1, C2	240.09	kNm/m
SLS	104.39	kNm/m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	409.67	kNm/m
DA1, C2	473.96	kNm/m
SLS	473.96	kNm/m

Lever arm, L3 = 1.65 m

Lever arms:		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	99.75	kNm/m
DA1, C2	86.45	kNm/m
SLS	66.5	kNm/m

Lever arm, L4 = 3.8 m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	334.4	869.0	38%	PASS	kNm/m
DA1,C2	326.8	873.5	37%	PASS	kNm/m
SLS	214.3	647.7	33%	PASS	kNm/m

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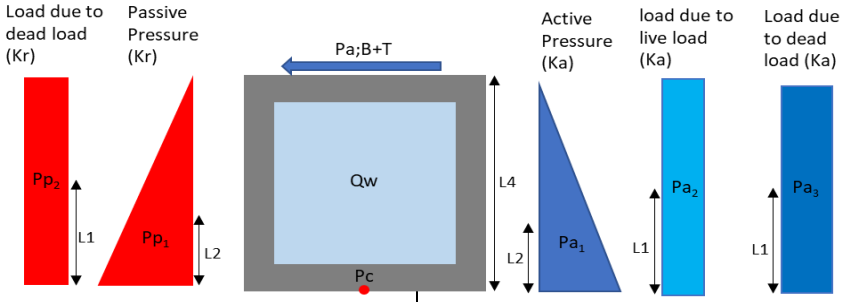
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r,DA1C1}$ = 0.88

Considering Passive Earth Pressure $K_{r,DA1C2}$ = 0.82

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	50.83	kN/m
DA1, C2	49.86	kN/m
SLS	36.48	kN/m

Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

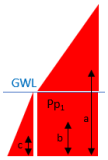
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	96.57	kNm/m
DA1, C2	94.73	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Horizontal load due to live load

Live load does not contribute to passive resistance



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = (0.5*H$^{2)}$*γ*Kr*γG)

	Pp1	
DA1, C1	108.93	kN/m
DA1, C2	114.67	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	145.45	kNm/m
DA1, C2	142.66	kNm/m
SLS	104.39	kNm/m

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Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

Ma3 + Ma2 + Ma1+Mbt Mp2 + Mp1 + Mpc

	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	334.4	242.0	92.4
DA1,C2	326.8	237.4	89.4
SLS	214.3	173.7	40.6

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

Pa3 + Pa2 + Pa1+Pb+t Pp2 + Pp1

	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	174.3	159.8	14.5
DA1,C2	168.6	164.5	4.1
SLS	110.5	114.7	0.0

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 178.8 kN /m run (unfactored)

Total Live Load for bearing = Ql = 165.0 kN /m run (unfactored)

Total Water Load for bearing = Qw = 36.8 kN /m run (unfactored)

(Assuming ULS level of water in the Culvert)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	92.4	14.5
DA1,C2	89.4	4.1
SLS	40.6	0.0

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	6%	20%	PASS

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REF | CALCULATIONS | OUTPUT

SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	174.29	428.6	41%	PASS	kNm/m
DA1,C2	168.63	383.9	44%	PASS	kNm/m
SLS	110.46	259.8	43%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	270.79	701.4	39%	PASS	kNm/m
DA1,C2	256.20	621.3	41%	PASS	kNm/m
SLS	169.43	363.0	47%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	334.4	869.0	38%	PASS
DA1,C2	326.8	873.5	37%	PASS
SLS	214.3	647.7	33%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	6%	20%	PASS

HS2 Ltd - Code 1 - Accepted

REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design.

The calculations presented in this section consider **STAGE 2C.**

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REF | CALCULATIONS | OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

HS2 Ltd - Code 1 - Accepted

REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

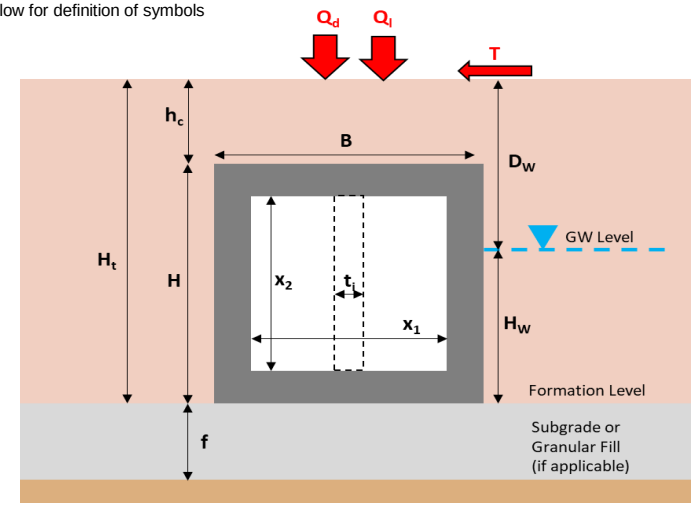
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

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REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	0.80	m
Depth to culvert base	Ht	4.60	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	2.70	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	2.70	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	121	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	0	kN/m	
Permanent load due to Fill	Qo	52.8	kN/m	
Total Permanent Load	Qdt	52.8	kN/m	
Live Load Stress~	ql	0	kN/m2	Added A60H load directly further on in cals
Live Load~	Ql	0	kN/m	
Traction and braking force at top of culvert*	T	0.0	kN/m	~Use dispersed value of T
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - ULS level	Qw	36.8	kN/m	

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states					
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax	Design - Earth Pressure Kr	Limiting Earth Pressure Kp*
0.33	0.33	SLS	0.60	0.60	0.6
0.33	0.40	GEO (Comb 1)	0.72	1.67	3.10
0.41	0.49	GEO (Comb 2)	0.84	1.38	2.40

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required
*Kr for ULS can approach the limiting passive earth pressure (Kp) but cannot exceed it. This is on the basis that there is no limit to displacements in the temporary case. See Note 3 of Table B-5 in PD6694-1:2011.

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

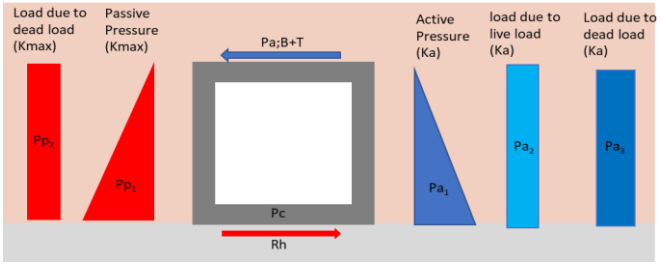
Interface Friction (δ) between fill and subgrade interface

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

HS2 Ltd - Code 1 - Accepted

Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	96.46	kN/m
DA1, C2	83.90	kN/m
SLS	36.48	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	32.83	kN/m
DA1, C2	29.79	kN/m
SLS	20.06	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=qL*γQ

	Pa2	
DA1, C1	34.84	kN/m
DA1, C2	38.14	kN/m
SLS	23.22	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	206.73	kN/m
DA1, C2	179.82	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	331.3	83.0	248.3	125.4	kN/m
DA1, C2	348.8	61.5	287.3	120.1	kN/m
SLS	348.8	61.5	287.3	145.1	kN/m

Added Live Load with Permanent Load Partial Factors

Results of Sliding Check - at Concrete/Subgrade Interface

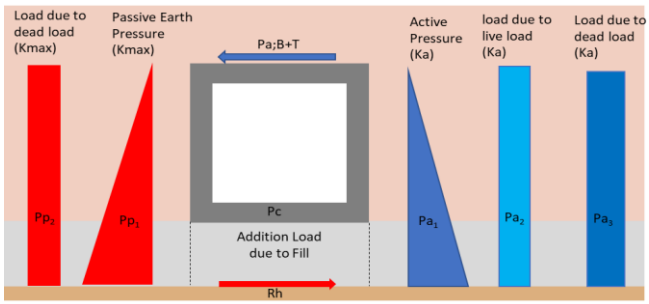
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	138.0	428.6	32%	PASS
DA1,C2	131.8	383.9	34%	PASS
SLS	86.3	259.8	33%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Sliding Check - Granular Fill / Subgrade Interface

H =	3.80	m
f =	2.00	m
H + f = H' =	5.80	m
H_{wULS} + f =	3.90	m
H_{wSLS} + f =	3.90	m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	147.23	kN/m
DA1, C2	128.06	kN/m
SLS	55.68	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	50.11	kN/m
DA1, C2	45.47	kN/m
SLS	30.62	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*γQ

	P'a2	
DA1, C1	34.84	kN/m
DA1, C2	38.14	kN/m
SLS	23.22	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	439.47	kN/m
DA1, C2	382.27	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	469.3	170.4	298.8	114.7	kN/m
DA1, C2	494.0	126.3	367.7	111.0	kN/m
SLS	494.0	126.3	367.7	141.1	kN/m

Results of Sliding Check - at Granular Fill/Subgrade Interface

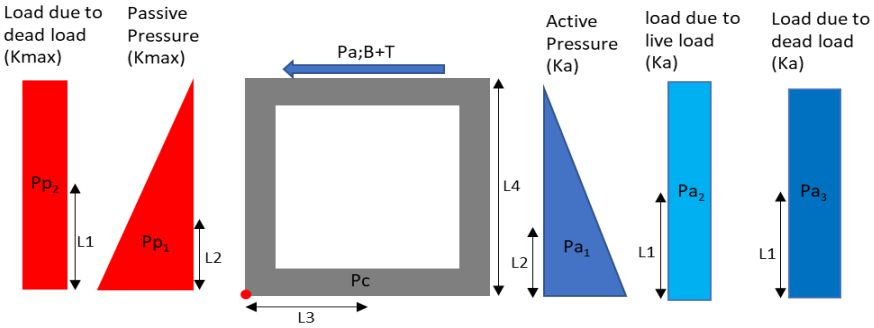
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	234.5	701.4	33%	PASS
DA1,C2	219.3	621.3	35%	PASS
SLS	145.3	363.0	40%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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REF | CALCULATIONS | OUTPUT

Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

Mp2 = Pp2 * L1

	Pp2	
DA1, C1	183.27	kNm/m
DA1, C2	159.42	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	62.38	kNm/m
DA1, C2	56.60	kNm/m
SLS	38.12	kNm/m

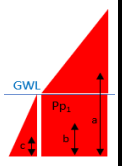
Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2

Ma2 = BM * partial factor

	Pa2	
DA1, C1	69.79	kNm/m
DA1, C2	78.26	kNm/m
SLS	46.53	kNm/m

Lever arm, L_1 = N/A m



	Lever Arm	
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	276.02	kNm/m
DA1, C2	240.09	kNm/m
SLS	104.39	kNm/m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	409.67	kNm/m
DA1, C2	473.96	kNm/m
SLS	473.96	kNm/m

Lever arm, L3 = 1.65 m

Lever arms:

a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	0	kNm/m
DA1, C2	0	kNm/m
SLS	0	kNm/m

Lever arm, L4 = 3.8 m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	226.1	869.0	26%	PASS	kNm/m
DA1,C2	220.1	873.5	25%	PASS	kNm/m
SLS	142.1	647.7	22%	PASS	kNm/m

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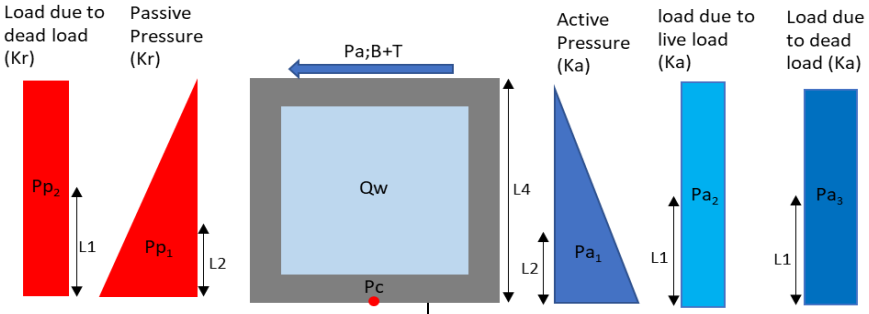
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r.DA1C1}$ = 0.88

Considering Passive Earth Pressure $K_{r.DA1C2}$ = 0.82

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	50.83	kN/m
DA1, C2	49.86	kN/m
SLS	36.48	kN/m

Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

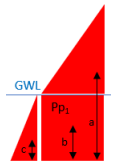
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	96.57	kNm/m
DA1, C2	94.73	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Horizontal load due to live load

Live load does not contribute to passive resistance



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = $(0.5*H^{2)}*\gamma*Kr*\gamma G)$

	Pp1	
DA1, C1	108.93	kN/m
DA1, C2	114.67	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	145.45	kNm/m
DA1, C2	142.66	kNm/m
SLS	104.39	kNm/m

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REF | CALCULATIONS | OUTPUT

Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

Ma3 + Ma2 + Ma1+Mbt Mp2 + Mp1 + Mpc

	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	226.1	242.0	0.0
DA1,C2	220.1	237.4	0.0
SLS	142.1	173.7	0.0

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

Pa3 + Pa2 + Pa1+Pb+t Pp2 + Pp1

	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	138.0	159.8	0.0
DA1,C2	131.8	164.5	0.0
SLS	86.3	114.7	0.0

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 178.8 kN /m run (unfactored)

Total Live Load for bearing = Ql = 175.0 kN /m run (unfactored)

Total Water Load for bearing = Qw = 36.8 kN /m run (unfactored)

(Assuming ULS water level in the Culvert)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	0.0	0.0
DA1,C2	0.0	0.0
SLS	0.0	0.0

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	5%	19%	PASS

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REF | CALCULATIONS | OUTPUT

SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	138.03	428.6	32%	PASS	kNm/m
DA1,C2	131.78	383.9	34%	PASS	kNm/m
SLS	86.29	259.8	33%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	234.53	701.4	33%	PASS	kNm/m
DA1,C2	219.35	621.3	35%	PASS	kNm/m
SLS	145.26	363.0	40%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	226.1	869.0	26%	PASS
DA1,C2	220.1	873.5	25%	PASS
SLS	142.1	647.7	22%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	3%	17%	PASS
DA1,C2	5%	19%	PASS

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REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design.

The calculations presented in this section consider **STAGE 2D.**

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REF	CALCULATIONS	OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

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REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

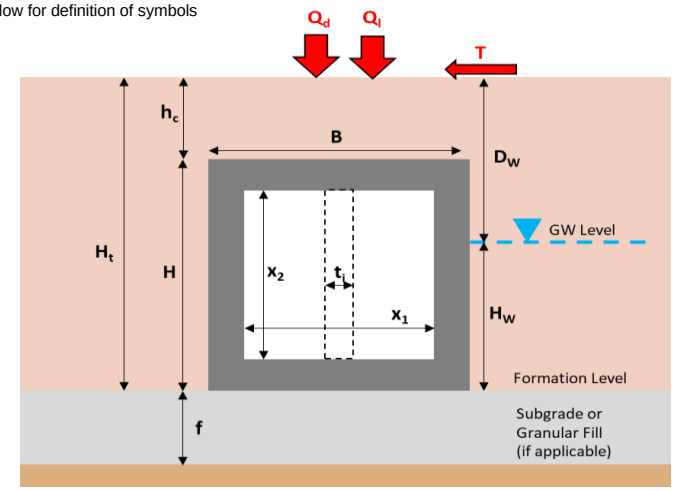
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

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REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	0.80	m
Depth to culvert base	Ht	4.60	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	2.70	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	2.70	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	121	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	0	kN/m	
Permanent load due to Fill	Qo	52.8	kN/m	
Total Permanent Load	Qdt	52.8	kN/m	
Live Load Stress~	ql	0	kN/m2	Added A60H load directly further on in cals
Live Load~	Ql	0	kN/m	
Traction and braking force at top of culvert*	T	35	kN/m	~Use dispersed value of T
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - ULS level	Qw	36.8	kN/m	

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states					
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax	Design - Earth Pressure Kr	Limiting Earth Pressure Kp*
0.33	0.33	SLS	0.60	0.60	0.6
0.33	0.40	GEO (Comb 1)	0.72	1.67	3.10
0.41	0.49	GEO (Comb 2)	0.84	1.38	2.40

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required
*Kr for ULS can approach the limiting passive earth pressure (Kp) but cannot exceed it. This is on the basis that there is no limit to displacements in the temporary case. See Note 3 of Table B-5 in PD6694-1:2011.

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

Interface Friction (δ) between fill and subgrade interface

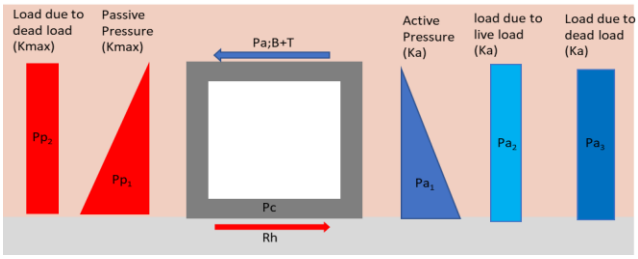
Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

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REF | CALCULATIONS | OUTPUT

Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	96.46	kN/m
DA1, C2	83.90	kN/m
SLS	36.48	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	32.83	kN/m
DA1, C2	29.79	kN/m
SLS	20.06	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=qL*γQ

	Pa2	
DA1, C1	0.00	kN/m
DA1, C2	3.11	kN/m
SLS	0.00	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	206.73	kN/m
DA1, C2	179.82	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	52.5	kN/m
DA1, C2	45.5	kN/m
SLS	35	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	497.6	83.0	414.5	209.4	kN/m
DA1, C2	523.8	61.5	462.3	193.3	kN/m
SLS	523.8	61.5	462.3	233.5	kN/m

Added Live Load with Permanent Load Partial Factors

Results of Sliding Check - at Concrete/Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	155.7	512.6	30%	PASS
DA1,C2	142.2	457.0	31%	PASS
SLS	98.1	348.2	28%	PASS

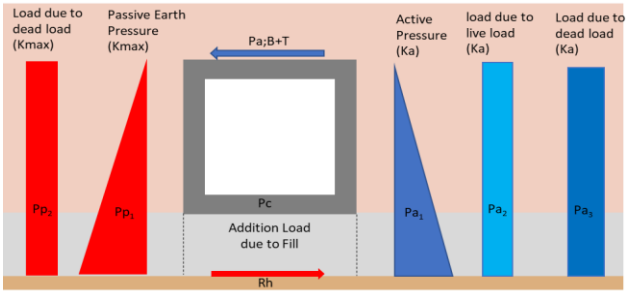
Use limiting Kp to reflect utilisation for ULS sliding.

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REFERENCE | CALCULATIONS | OUTPUT

Sliding Check - Granular Fill / Subgrade Interface

H	=	3.80 m
f	=	2.00 m
H + f = H'	=	5.80 m
H_{wULS} + f	=	3.90 m
H_{wSLS} + f	=	3.90 m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	147.23	kN/m
DA1, C2	128.06	kN/m
SLS	55.68	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	50.11	kN/m
DA1, C2	45.47	kN/m
SLS	30.62	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*γQ

	P'a2	
DA1, C1	0.00	kN/m
DA1, C2	3.11	kN/m
SLS	0.00	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	439.47	kN/m
DA1, C2	382.27	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	52.5	kN/m
DA1, C2	45.5	kN/m
SLS	35	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	635.5	170.4	465.1	178.5	kN/m
DA1, C2	669.0	126.3	542.7	163.9	kN/m
SLS	669.0	126.3	542.7	208.3	kN/m

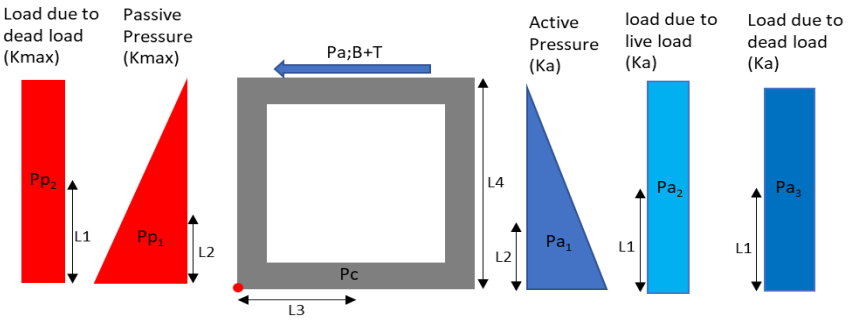
Results of Sliding Check - at Granular Fill/Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	252.2	765.2	33%	PASS
DA1,C2	229.8	674.2	34%	PASS
SLS	157.0	430.2	37%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

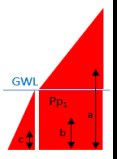
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	183.27	kNm/m
DA1, C2	159.42	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance



Lever Arm		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	276.02	kNm/m
DA1, C2	240.09	kNm/m
SLS	104.39	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	683.98	kNm/m
DA1, C2	762.71	kNm/m
SLS	762.71	kNm/m

Lever arm, L3 = 1.65 m

Lever arms:

a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	62.38	kNm/m
DA1, C2	56.60	kNm/m
SLS	38.12	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2

Ma2 = BM * partial factor

	Pa2	
DA1, C1	0.00	kNm/m
DA1, C2	0.85	kNm/m
SLS	0.00	kNm/m

Lever arm, L_1 = N/A m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	199.5	kNm/m
DA1, C2	172.9	kNm/m
SLS	133	kNm/m

Lever arm, L4 = 3.8 m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	355.8	1143.3	31%	PASS	kNm/m
DA1,C2	315.6	1162.2	27%	PASS	kNm/m
SLS	228.5	936.4	24%	PASS	kNm/m

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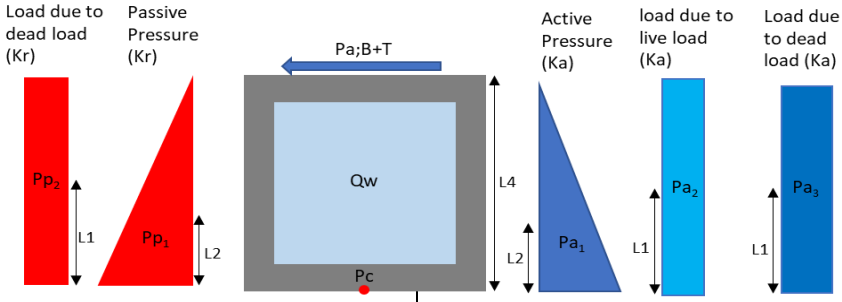
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r,DA1C1}$ = 0.88

Considering Passive Earth Pressure $K_{r,DA1C2}$ = 0.82

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	50.83	kN/m
DA1, C2	49.86	kN/m
SLS	36.48	kN/m

Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

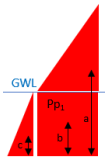
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	96.57	kNm/m
DA1, C2	94.73	kNm/m
SLS	69.31	kNm/m

Lever arm, L_1 = 1.9 m

Horizontal load due to live load

Live load does not contribute to passive resistance



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = (0.5*H$^{2)}$*γ*Kr*γG)

	Pp1	
DA1, C1	108.93	kN/m
DA1, C2	114.67	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	145.45	kNm/m
DA1, C2	142.66	kNm/m
SLS	104.39	kNm/m

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Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

Ma3 + Ma2 + Ma1+Mbt Mp2 + Mp1 + Mpc

	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	355.8	242.0	113.8
DA1,C2	315.6	237.4	78.2
SLS	228.5	173.7	54.8

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

Pa3 + Pa2 + Pa1+Pb+t Pp2 + Pp1

	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	155.7	159.8	0.0
DA1,C2	142.2	164.5	0.0
SLS	98.1	114.7	0.0

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 178.8 kN /m run (unfactored)

Total Live Load for bearing = Ql = 350.0 kN /m run (unfactored)

Total Water Load for bearing = Qw = 36.8 kN /m run (unfactored)

(Assuming ULS water level in the Culvert)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	113.8	0.0
DA1,C2	78.2	0.0
SLS	54.8	0.0

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	26%	PASS
DA1,C2	8%	30%	PASS

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REF | CALCULATIONS | OUTPUT

SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	155.70	512.6	30%	PASS	kNm/m
DA1,C2	142.25	457.0	31%	PASS	kNm/m
SLS	98.06	348.2	28%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	252.19	765.2	33%	PASS	kNm/m
DA1,C2	229.81	674.2	34%	PASS	kNm/m
SLS	157.04	430.2	37%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	355.8	1143.3	31%	PASS
DA1,C2	315.6	1162.2	27%	PASS
SLS	228.5	936.4	24%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	26%	PASS
DA1,C2	8%	30%	PASS

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REF	CALCULATIONS	OUTPUT

Purpose and Scope

The purpose of this spreadsheet is to automate the culvert design process. Design shall be carried out in accordance with the requirements of Eurocode 7 and PD6694-1:2011 to check that the sliding and overturning limit states are satisfied and that sufficient bearing resistance is available.

Input Data

Geometrical and loading input data shall be obtained from the Structures team.

Data regarding soil properties shall be obtained from a local ground model derived using available ground investigation information and published sources. Ground models should be approved prior to commencing culvert design.

Cells where user input is required are coloured yellow.

Self-calculating cells are coloured blue

Cells with fixed values are coloured grey

Assumptions

6N fill is assumed to be placed against all sides of the concrete structure. This influence of CBM material surround the culvert structure has been ignored following cl.3.9.9. in HS2-HS2-BR-STD-000-000001 Technical Standard - Bridge Design Basis. The performance of the CBM is expected to be superior to the 6N material.

6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)
Unsuitable founding material (weak, wet, deleterious soil, contaminated soil, made ground) is to be excavated and replaced to provide a suitable founding layer.

The ULS Groundwater level is defined by the engineer but is to lie no higher than the top of the structure.

The expected operational water level (measured relative to the culvert) should be used for the SLS calculations. Where the operational water level is not known it will be assumed that GWL lies 1m above with the base of the culvert.

It is assumed to be no water in the structure for the Sliding and overturing assessment. Whereas for the culvert is assumed to be full of water for the Bearing Capacity assessment.

Point loads are distributed from the ground level the top of culvert level. The live load pressure is not distributed any deeper than the top of the culvert.

Limitations

The calculation sheet considers no 'Negative' arching effects and down drag induced loading applied to the roof slab of the structure. Additional analysis may be required in accordance with PD6694.

Load Cases and Construction Stages

PD 6694-1:2011 Clause 10.6 requires the following stages to be analysed in the design

Stage 1 The completed structure is backfilled up to the top of the roof.

Stage 2 The structure is backfilled to an intermediate level between roof level and finished surface level, at which it is proposed to use the structure for construction traffic.

Stage 3 The structure is fully backfilled and in service.

Stage 1 was considered for Backfill to top of structure (no cover) with 10kPa load. Operational groundwater level is adopted 1m above the base of culvert.

Stage 2 was considered for Backfill required before A60H construction traffic allowed above culvert. Groundwater at 1m above the base of excavation.

Stage 3 - the structure in service - the structure team have provided details for two sections which they have assessed as the most critical load case, which were analysed for the design. See Section 4.7.

The calculations presented in this section consider **STAGE 3.**

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REF	CALCULATIONS	OUTPUT

Material Properties - Natural Subgrade

Unit weight above water table	γdry	21	kN/m3
Unit weight below water table	γsat	21	kN/m3
Submerged unit weight of subgrade	γsub	11.19	kN/m3
Peak angle of shearing resistance*	ϕ'p	22	°
Critical state angle of shearing resistance	ϕ'cv	21	°
DA1-C2 Design angle of shearing resistance	ϕ'd	16.8	
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	85	kPa

*Design value of angle of shearing resistance should be the lowest of:
- peak (factored) value; or
- critical state value.

17 deg

Material Properties - 6N2 Fill

Unit weight above water table	γdry	20	kN/m3
Unit weight below water table	γsat	22	kN/m3
Submerged unit weight of 6N fill	γsub	12.19	kN/m3
Peak angle of shearing resistance	ϕ'p	40	°
Critical state angle of shearing resistance	ϕ'cv	35	°
DA1-C2 Design angle of shearing resistance	ϕ'd	33.87	°
Effective cohesion	c'	0	kPa
Undrained shear strength	cu	0	kPa

~Design value of angle of shearing resistance for 6N is taken as 35deg (phi cv). Phi peak is taken as 40deg, falling to 34deg when factored.

34 deg

Subgrade material: Granular Fill

Define the ground conditions below the culvert. If this has not yet been defined, choose "Natural Ground" for conservatism.

Shear Strength Properties of granular fill Class 1A2

Critical state angle of shearing resistance	φ'cv	40	°
Peak angle of shearing resistance	φ'p	40	°
DA1-C2 Design angle of shearing resistance	φ'd	33.873	°

34 deg

Notes

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REF | CALCULATIONS | OUTPUT

Loading information received from STRs

The characteristic loading were provided by the structures team on the 25/01/2023 by email.

The characterisitic permanent loads are presented below:

Characteristic permanent loading

Load due to Rails & fixtures =	0.22	kN/m^2
Load due to Track Slab =	2.28	kN/m^2
Load due to HBL =	2.94	kN/m^2
Load due to Proctection Layer =	2.79	kN/m^2
Load due to prepared subgrade, crossfall & protection =	5.42	kN/m^2
Load due to soil fill =	64.00	kN/m^2
Load due to culvert SW =	44.12	kN/m^2

The characterisitic variable loads are presented below:

Characteristic variable loading

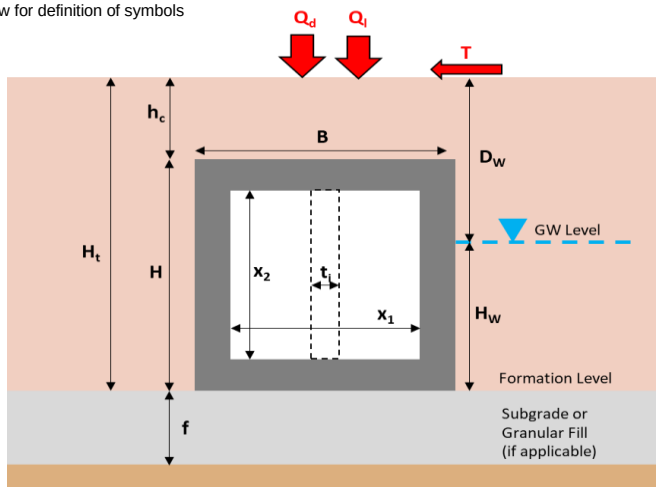
Load due to LM71 =	47.45	kN/m^2
Load due to Kirow Crane =	0.00	kN/m^2
Load due to water pressure =	30.00	kN/m^2

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REF | CALCULATIONS | OUTPUT

Input Data - Geometry, Loading and Material Properties

See tabled below for definition of symbols



Geometric Properties

Culvert Width	B	3.3	m
Culvert Height	H	3.8	m
Culvert Length	L	94.09	m
Exterior Concrete thickness	t0	0.4	m
Internal Wall thickness (if applicable)	ti	0	m
Clear width	x1	2.5	m
Clear height	x2	3	m
Cross-sectional area	A	5.04	m2
Height of fill above culvert (cover thickness)	hc	3.40	m
Depth to culvert base	Ht	7.20	m
Depth of granular fill below founding level	f	2.00	m

Groundwater Levels

ULS - groundwater level	Hw	1.90	m
ULS - groundwater level	Dw	5.30	m
SLS - operational level	Hw	1.90	m
SLS - operational level	Dw	5.30	m

Loading

Unit weight of concrete - Inferior	γc	24	kN/m3	
Culvert Self Weight - Inferior	Wc	120.96	kN/m	*Used in Sliding, Overturning Check
Unit weight of concrete - Superior	γc	25	kN/m3	
Culvert Self Weight - Superior	Wc	126	kN/m	*Used in Bearing Check
Permanent Load excl. Culvert SW and Fill	Qd	13.65	kN/m	
Permanent load due to Fill	Qo	224.4	kN/m	
Total Permanent Load	Qdt	238.05	kN/m	
Live Load Stress~	ql	77.45	kN/m2	
Live Load~	Ql	255.59	kN/m	
Traction and braking force at top of culvert*	T	0	kN/m	
Unit weight of water	γw	9.81	kN/m3	
Weight of Water - Full	Qw	73.575	kN/m	

~Use dispersed value of T

*if this value is not provided by the structural engineer calculate using Section 10.2.8.2 of PD6694

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Partial Factors

The partial factors on actions presented below are taken from the National Annex to BS EN 1990 Table NA.A2.4(B) & Table NA.A2.4 (C). The partial factors on materials presented below are taken from the National Annex to BS EN 1997 Table A.NA.4.

Actions - "Bridges"

			SLS	DA1,C1	DA1,C2
Permanent	Unfavourable	γG	1.00	1.35	1.00
	Favourable		1.00	0.95	1.00
Variable	Unfavourable	γQ	1.00	1.50	1.30
	Favourable		1.00	0.00	0.00

Materials

Soil Parameter	Symbol	SLS	DA1 ,C1	DA1, C2
Angle of Shearing Resistance	γϕ'	1	1	1.25
Effective Cohesion	γc'	1	1	1.25
Undrained Shear Strength	γcu	1	1	1.4
Unconfined strength	γqu	1	1	1.4
Density	γγ	1	1	1

Coefficient of Earth Pressure, K

The K values presented in the table below are taken from Table B.5 of PD6694-1:2011, Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004. These represent the most onerous loading conditions which the culvert structures are likely to experience. Note, these values of K include the partial factor γm, and (for permanent earth pressure at ULS) the model factor γsd;k.

Values of K that may be applied to various limit states					
Horizontal Traffic Surcharge, Ka	Earth Pressure Ka	Limit State	Earth Pressure Kr/Kmax	Design - Earth Pressure Kr	Limiting Earth Pressure Kp*
0.33	0.33	SLS	0.60	0.60	0.60
0.33	0.40	GEO (Comb 1)	0.72	1.67	3.10
0.41	0.49	GEO (Comb 2)	0.84	1.38	2.40

*Kr for SLS should not exceed Kmax, otherwise a separate assessment of movement is required
*Kr for ULS can approach the limiting passive earth pressure (Kp) but cannot exceed it. This is on the basis that there is no limit to displacements in the temporary case. See Note 3 of Table B-5 in PD6694-1:2011.

Interface Friction (δ) between concrete and subgrade or (where applicable) fill

Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 0.67ϕ'.

	ϕ'd	δ
DA1, C1	40	26.8
DA1, C2	33.873	22.695
SLS	40	26.8

Interface Friction (δ) between fill and subgrade interface

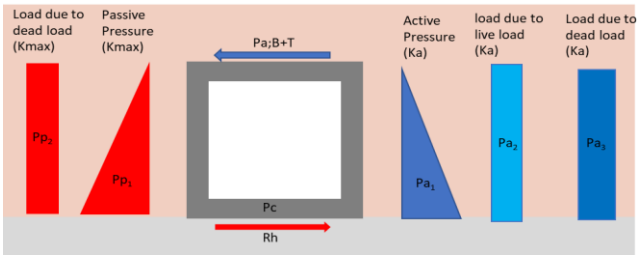
Design angle of shearing resistance for subgrade soils is presented below. δ is taken as 1.0ϕ'.

	ϕ'd	δ
DA1, C1	21	21
DA1, C2	16.8	16.8
SLS	21	21

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REF | CALCULATIONS | OUTPUT

Sliding Check -Culvert / Granular Fill or Subgrade Interface



Horizontal load due to dead vertical load (Pp2)

Pp2 = H*(Qdt/B)*Kmax*γG

	Pp2	
DA1, C1	434.89	kN/m
DA1, C2	378.28	kN/m
SLS	164.47	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H*γG

	Pa3	
DA1, C1	148.02	kN/m
DA1, C2	134.32	kN/m
SLS	90.46	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*Ka*H*γQ

	Pa2	
DA1, C1	145.68	kN/m
DA1, C2	156.87	kN/m
SLS	97.12	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pp1	
DA1, C1	206.73	kN/m
DA1, C2	179.82	kN/m
SLS	78.18	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	Pa1	
DA1, C1	70.36	kN/m
DA1, C2	63.85	kN/m
SLS	43.00	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Culvert Foundation Level

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	341.1	83.0	258.0	130.3	kN/m
DA1, C2	359.0	61.5	297.5	124.4	kN/m
SLS	359.0	61.5	297.5	150.3	kN/m

Results of Sliding Check - at Concrete/Subgrade Interface

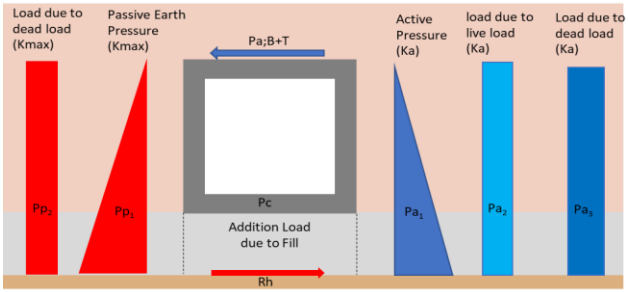
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	364.1	772.0	47%	PASS
DA1,C2	355.0	682.5	52%	PASS
SLS	230.6	392.9	59%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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Sliding Check - Granular Fill / Subgrade Interface

H	=	3.80 m
f	=	2.00 m
H + f = H'	=	5.80 m
H_{wULS} + f	=	3.90 m
H_{wSLS} + f	=	3.90 m



Horizontal load due to dead vertical load (Pp2)

Pp2 = H'*(Qdt/B)*Kmax*γG

	P'p2	
DA1, C1	663.78	kN/m
DA1, C2	577.38	kN/m
SLS	251.03	kN/m

Horizontal load due to dead vertical load (Pa3)

Pa3 = (Qdt/B)*Ka*H'*γG

	P'a3	
DA1, C1	225.93	kN/m
DA1, C2	205.01	kN/m
SLS	138.07	kN/m

Horizontal load due to live load

Live load does not contribute to passive resistance

Horizontal load due to live load (Pa2)

Pa2=ql*Ka*H'*γQ

	P'a2	
DA1, C1	222.36	kN/m
DA1, C2	239.43	kN/m
SLS	148.24	kN/m

Passive Resistance (Pp1) - ULS Case

Calculation allowing for $H_{w\text{-}ULS}$' and $H_{W\text{-}SLS}$'

	P'p1	
DA1, C1	439.47	kN/m
DA1, C2	382.27	kN/m
SLS	166.20	kN/m

Active Pressure (Pa1) - ULS Case

Calculation allowing for Hw-ULS and HW-SLS

	P'a1	
DA1, C1	149.58	kN/m
DA1, C2	135.73	kN/m
SLS	91.41	kN/m

Lateral Force due to Braking & Traction (Pa;B+T)

	Pa;fill	
DA1, C1	0	kN/m
DA1, C2	0	kN/m
SLS	0	kN/m

Lateral Resistance Due to Friction (R_h) at Granular Fill / Subgrade Interface

Upward force due to water displacement is taken as γ_w*Hw*B*γG

	Total Vertical Load Vd	Water pressure Ud	Net Vertical Rvd = Vd - Ud	Rh = Rvd tan δ	
DA1, C1	479.0	170.4	308.6	118.4	kN/m
DA1, C2	504.2	126.3	378.0	114.1	kN/m
SLS	504.2	126.3	378.0	145.1	kN/m

Results of Sliding Check - at Granular Fill/Subgrade Interface

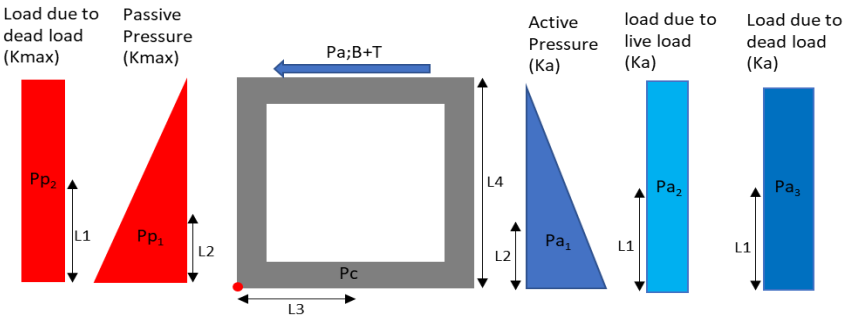
	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?
DA1,C1	597.9	1221.7	49%	PASS
DA1,C2	580.2	1073.8	54%	PASS
SLS	377.7	562.3	67%	PASS

Use limiting Kp to reflect utilisation for ULS sliding.

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REF | CALCULATIONS | OUTPUT

Overturning Check - Moments about Corner



Moment due to Passive Pressure Pp2

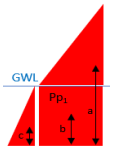
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	826.29	kNm/m
DA1, C2	718.74	kNm/m
SLS	312.49	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to live load

Live load does not contribute to passive resistance



Lever Arm		
a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	276.02	kNm/m
DA1, C2	240.09	kNm/m
SLS	104.39	kNm/m

Moment due to Culvert Weight (Pc)

M,pc = (Vertical Load - GW uplift)*L3

	Pc	
DA1, C1	425.74	kNm/m
DA1, C2	490.88	kNm/m
SLS	490.88	kNm/m

Lever arm, L3 = 1.65 m

Lever arms:

a	2.5333	m
b	0.95	m
c	0.6333	m

Moment due to Active Pressure Pa3

Ma3 = Pa3 * L1

	Pa3	
DA1, C1	281.25	kNm/m
DA1, C2	255.20	kNm/m
SLS	171.87	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa2

Ma2 = Pa2 * L1

	Pa2	
DA1, C1	276.80	kNm/m
DA1, C2	298.05	kNm/m
SLS	184.53	kNm/m

Lever arm, L_1 = 1.9 m

Moment due to Active Pressure Pa1 - ULS

Ma1 = Pa1 * L2

	Pa1	
DA1, C1	93.95	kNm/m
DA1, C2	85.25	kNm/m
SLS	57.41	kNm/m

Moment due to Braking & Traction Pa;B+T

Ma;B+T = (Pa;fill)*L4

	Pa;B+T	
DA1, C1	0	kNm/m
DA1, C2	0	kNm/m
SLS	0	kNm/m

Lever arm, L4 = 3.8 m

Summary of Overturning and Sliding Analysis

The check considers the out of horizontal and moment equilibriums about the corner of the culvert considering at the limiting passive earth pressure Kr.

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail	
DA1,C1	652.0	1528.0	43%	PASS	kNm/m
DA1,C2	638.5	1449.7	44%	PASS	kNm/m
SLS	413.8	907.8	46%	PASS	kNm/m

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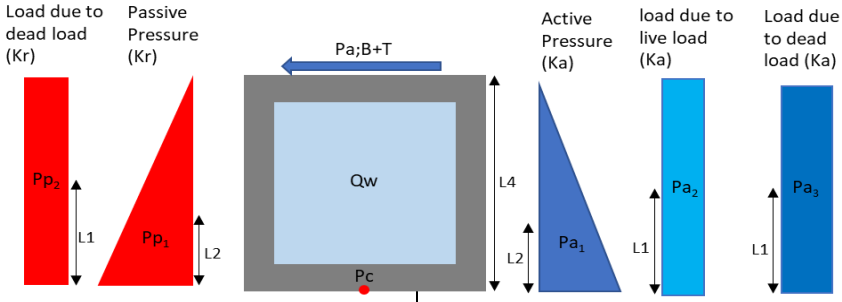
REF | CALCULATIONS | OUTPUT

Bearing - Net Forces and Moments about Centreline

Considering Passive Earth Pressure $K_{r,DA1C1}$ = 0.88

Considering Passive Earth Pressure $K_{r,DA1C2}$ = 0.82

Passive Earth pressure is limited to 20% of limiting passive earth pressures (K_P) to negate a strain compatibility check.



Moments about centreline of Culverts

Horizontal load due to dead vertical load (Pp2)

Pp2 = Qdt/B*Kr*H*γG

	Pp2	
DA1, C1	229.16	kN/m
DA1, C2	224.78	kN/m
SLS	164.47	kN/m

Moments about centreline of Culverts

Moment due to dead vertical load (Pp2)

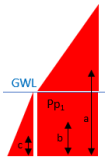
Mp2 = Pp2 * L1

	Pp2	
DA1, C1	435.41	kNm/m
DA1, C2	427.08	kNm/m
SLS	312.49	kNm/m

Lever arm, L_1 = 1.9 m

Horizontal load due to live load

Live load does not contribute to passive resistance



ULS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

SLS Lever Arm

a	2.5333	m
b	0.95	m
c	0.6333	m

Passive Resistance (Pp1) - ULS Case

Pp1 = (0.5*H$^{2)}$*γ*Kr*γG)

	Pp1	
DA1, C1	108.93	kN/m
DA1, C2	114.67	kN/m
SLS	78.18	kN/m

Moment due to Passive Pressure Pp1 - ULS

Mp1 = Pp1 * L2

	Pp1	
DA1, C1	145.45	kNm/m
DA1, C2	142.66	kNm/m
SLS	104.39	kNm/m

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Summary of Bearing Check (strain-compatibility-limited passive resistance)

The check considers the net out of balance horizontal force and moment about culvert centreline using limited passive resistance, negating a strain compatibility check.

Moment Equilibrium about centreline (Limited Movement)

Ma3 + Ma2 + Ma1+Mbt Mp2 + Mp1 + Mpc

	Total Destabilising Moment Σactive (kNm/m)	Total Stabilizing Moment Σpassive (kNm/m)	Resultant Moment (kNm/m)
DA1,C1	652.0	580.9	71.1
DA1,C2	638.5	569.7	68.8
SLS	413.8	416.9	0.0

Horizontal Force Equilibrium (Limited Movement, Limited Sliding Resistance)

Pa3 + Pa2 + Pa1+Pb+t Pp2 + Pp1

	Total Destabilising Force Σactive (kN/m)	Total Stabilizing Force Σpassive (kN/m)	Resultant Force (kN/m)
DA1,C1	364.1	338.1	26.0
DA1,C2	355.0	339.4	15.6
SLS	230.6	242.7	0.0

Vertical Forces - For Bearing Sheet

Total vertical dead load for bearing = Qd = 364.1 kN /m run (unfactored)

Total Live Load for bearing = Ql = 255.6 kN /m run (unfactored)

Total Water Load for bearing = Qw = 73.6 kN /m run (unfactored)
(Assuming Culvert Full of Water)

Net Moments/Force for bearing check

	Resultant Moment (kNm/m)	Resultant Horizontal Force (kN/m)
DA1,C1	71.1	26.0
DA1,C2	68.8	15.6
SLS	0.0	0.0

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	32%	PASS
DA1,C2	6%	32%	PASS

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REF | CALCULATIONS | OUTPUT

SUMMARY OF CHECKS

Results of Sliding Check -Culvert / Granular Fill or Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	364.07	772.0	47%	PASS	kNm/m
DA1,C2	355.03	682.5	52%	PASS	kNm/m
SLS	230.58	392.9	59%	PASS	kNm/m

Results of Sliding Check - Granular Fill / Subgrade Interface

	Total Destabilising Force ΣActive	Total Stabilizing Force ΣPassive	Utilisation (%)	okay or not?	
DA1,C1	597.87	1221.7	49%	PASS	kNm/m
DA1,C2	580.17	1073.8	54%	PASS	kNm/m
SLS	377.72	562.3	67%	PASS	kNm/m

Results of Overturning Check

	Total Destabilising Moment ΣActive	Total Stabilizing Moment ΣPassive	Utilisation (%)	Pass/Fail
DA1,C1	652.0	1528.0	43%	PASS
DA1,C2	638.5	1449.7	44%	PASS
SLS	413.8	907.8	46%	PASS

Results of Bearing Check

Calculated Utilisation for Bearing Check, ignoring favourable uplift from buoyancy.

	Granular Subgrade	Natural Subgrade	
DA1,C1	4%	32%	PASS
DA1,C2	6%	32%	PASS

HS2 Ltd - Code 1 - Accepted

SUBJECT

Sedrup North Culvert

SHEET NO.

Appendix H-6 : Culvert Headwall Design

HS2 Ltd - Code 1 - Accepted

ASSET SUMMARY SHEET

PROJECT TITLE

HS2

DESIGN OFFICE

Bristol

Asset ID & Name

Culvert Headwall Design

SHEET NO.

1 of **44**

Rev	TOTAL SHEETS	AUTHOR	DATE	CHECKED BY	DATE	APPROVED BY	DATE	COMMENTS
1		AJ	11/05/21	DG	18/05/21	HDS	09/07/21	
2		AJ	17/09/21	DG	17/09/21	HDS	17/09/21	

SUPERSEDES DOC No.

HS2 Ltd - Code 1 - Accepted

SUBJECT

Culvert Headwall Design

DOCUMENT NO.

1MC06-CEK-GT-REP-CS04_CL18-000037
C05

1 DESIGN BASIS

1.1 REMIT

Calculation present design of Standard Headwall details for Culverts on HS2

1.2 INTRODUCTION

This calculation package presents the standard design for a headwall to resist bearing, sliding, overturning and global stability against earth pressuse and eccentric weight of the wall unit. Generic Moderately conservative ground conditions are assumed for the purpsoes of validaing the desings for standard details. Total settlement checks are presented and the forward tilt and 'pull-out' of the headwall from the culvert is considered.

Calculation package contains sections:

2) Headwall Sizes

3) Materal Parameters

4) Loads

5) 'Negative' Load Allowance for Culvert for stability Calculation

5b) Allowance for Culvert - Construction phase 20kPa Surcharge

6) Greta Results

6b) Greta Results - Construction Phase 20kPa Surcharge

7) Punching Shear Check

8) Pdisp Settlement results, Derivation of sub-grade moduli for Structures

9) Pressure for Civils

1.3 ASSUMPTIONS

a Spread footing is assumed to be at plane locations and Sloping effects of the ground are modelled by surcharges or β angles as appropiate.

b Ground water level conservatively assessed to be at ground level for the purposes of standard detail design, and that typically, culverts are placed in areas of local high ground water (carrying water bodies).

c 6N/Class 1 fill is assumed to be placed behind and below concrete structure. This assumption has been made for mainline structures crossing the HS2 trace where CBM transition material will surround the culvert structure. This influence of CBM material surround the culvert structure has been ignored following cl.3.9.9. in HS2-HS2-BR-STD-000-000001 Technical Standard - Bridge Design Basis. The performance of the CBM is expected to be superior to the 6N material.

d 6N fill properties as per Materials Reuse Report (1MC06-CEK-REP-C002-600003)

e The proposed foundation type is a trapezoidal pad. For design purpose equivallent rectangular geometry is estimated from the trapezoidal shape.

f Failure modes considered for this calculation package are: Bearing resistance failure, Overturning (GEO), Resistance to Sliding (GEO), vertical movement and checked for rotational pullout (SLS)

g A minimum 10kPa surcharge is considered appropiate for the retention function of the Headwall to standard details. Should the design require a variable surcahrge of 10kPa or higher the Headwall must be examined. it is considered that a 5kPa variable surcharge (pedestrian) should be sufficiently covered by the 10kPa minimum permanent surcharge.

h Assumptions for Global Stability are given in relevent subsequent section

i The head walls are discreet retaining units and the material around them will be free draining compacted granular material, water pressure behind the wall has been conservatively assumed to be at 1/3 of the retained height of fill behind the headwall.

k Earth Pressure missing due to the culvert behind the wall is accountered for via a equivelent load acting in front of the wall, as well as the missing water pressure due to the culvert void (See Section 7).

1.4 STANDARDS AND REFERENCES

1 BS EN 1997-1:2004 Geotechnical Design

2 NA to BS EN 1997-1:2004

3 NA to BS EN 1997-2:2007

4 BS8004: 2015 - Code of practice for Foundations

5 Foundation Analysis and Design - J. E. Bowles 5th Edition

6 Foundation Design and Construction - M J Tomlinson 7th Edition

7 HS2-HS2-BR-STD-000-000005 HS2 Technical Standard - Culverts and Underpasses

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2. Headwall Sizes

This sheet documents the inputs for the headwall sizes, calculation of 'equivelent' square pad from the trapozoidal base, and earth pressure heights. 6 No. sizes of headwall were provided by structures, with dimensions titled 'A'-'E' in the table below. The total weight, and the vertical & horizontal dimenstions of the centre of gravity were also supplied.

To demonstrate stability and appropiately simplify the calculation approach, an 'equivelent' pad foundation was calculated from the trapozoidal base of each Headwall size. The width of the headwalls as shown in diagram 2.3 is one of two values depending on the case (3550mm or 3650mm as wall thickness varies for headwall sizes 1-2 and 3-6.

The 'Equivelent' Length is calculated as per section 2.3 and shown in the final column of table 2.1. Retained heights have been calculated in section 2.2. Retained heights consist of the culvert inner height (provided), 2*culvert slab, max retained height of soil above culvert, and top protection slab thickness.

2.1 HS2 Mainline Culverts - Headwall/ Wingwall Sizes

	Provided by Structures										
Headwall No.	HS2 Culvert Size/ Ref	A (max height of	B (min height of	C (headwall & wingwall thickness)	D (width of apron)	E (internal width of	Total Weight (unfactore	Vertical distance to Centre of Gravity VCG	Horizontal distance to Centre of Gravity HCG	Back width of headwall	effective length of foundatio
		mm	**mm**	**mm**	**mm**	**mm**	**kN**	**mm**	**mm**	**mm**	**m**
HW 1	**3.5 x 2**	4550	2500	400	7210	3500	673	1133	2116	4300	5.76
HW 2	**2.5 x 3**	5550	3500	400	6210	2500	681	1636	2106	3300	4.76
HW 3	**2 x 2**	4450	2400	300	5645	2000	464	1002	1998	2800	4.22
HW 4	**2 x 1.35**	3800	1750	300	5645	2000	425	738	1994	2800	4.22
HW 5	**2 x 1.5**	3950	1900	300	5645	2000	434	795	1992	2800	4.22
HW 6	**1.5 x 1.5**	3950	1900	300	5195	1550	405	808	1982	2350	3.77

** Excludes the mammal ledge and natural bed within the headwall. Includes the 1.25m upstand case only. Based on 25kN/m^3 density

Vertical dimension taken from the top of the apron slab. Horizontal dimension taken from the furthest extent of the shear key.

(Refer to sketches below for description of A, B, C, D & E)

Diagram 2.1

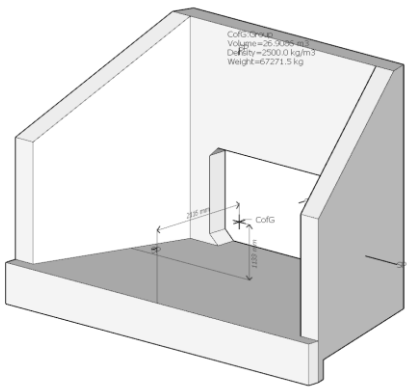
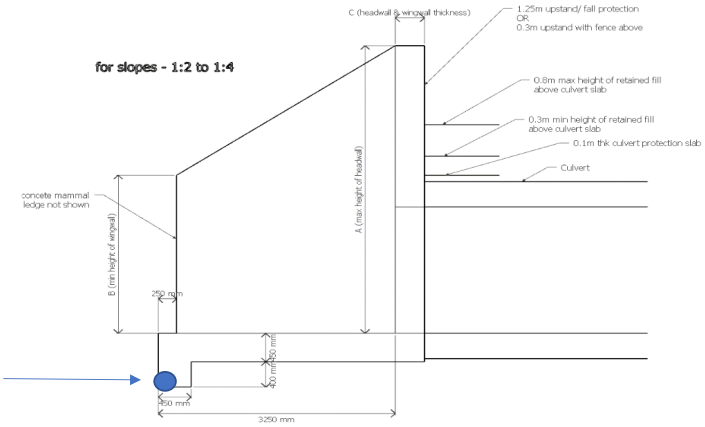


Diagram 2.2



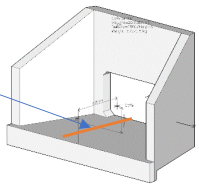
2.2

	Culvert Size (m x m)	Inner Height (mm)	Roof slab thickness (mm)	Total Height of culvert	Slab + shear key (mm)	retain slab above culvert (mm)	min retained soil (mm)	max retained soil (mm)	Active retained height	Passive retained height
									h_A maximum (mm)	h_P (mm) (unused)
HW 1	**3.5 x 2**	2000	400	2800	950	100	300	800	4650	950
HW 2	**2.5 x 3**	3000	400	3800	950	100	300	800	5650	950
HW 3	**2 x 2**	2000	300	2600	950	100	300	800	4450	950
HW 4	**2 x 1.35**	1350	300	1950	950	100	300	800	3800	950
HW 5	**2 x 1.5**	1500	300	2100	950	100	300	800	3950	950
HW 6	**1.5 x 1.5**	1500	300	2100	950	100	300	800	3950	950

height as per bottom right diagram 2.5

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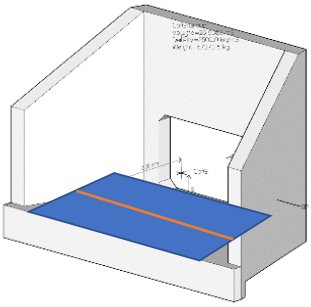
Diagram 2.3

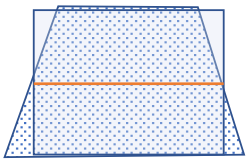


2.3 **"width" of headwall including back wall and shear key**

Internal width	3250		mm	
wall thickness	300 or 400		mm	
W total	3550	or	3650	mm
	HW 3-6		HW 1-2	

Diagram 2.4





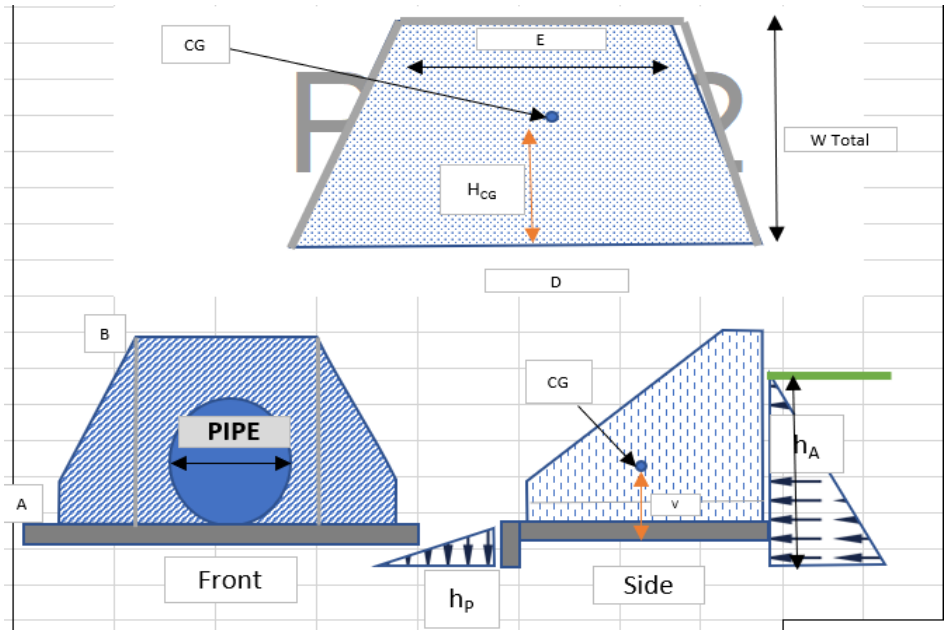
2.3 **"Effective Length" of foundation**

= Average of front width and back width of trapezium base of headwall

e.g. **HW 1**

effective length = $\frac{7.21+4.3}{2}$ = 5.76

Diagram 2.5



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3. Material Parameters

This standard design covers a generic, moderately conservative ground scenario. Below shows the parameters for a 'poor' cohesive soil with Class 6N fill used as specified in the Standard Headwall Drawings and the project specification.

A conservatively "poor" ground condition has been assumed whose minimum properties will be communicated on the standard earthwork drawings to be communicated to the contractor and verified on site. Ground Conditions that do not meet this generalised condition will require further bespoke calculation to determine minimum amount of excavate and replacement.

"Poor" is defined here as a *Cu of 30kPa* and a medium to high plasticity with a φ_{cv}' *of 22 degrees.*

	Parameter	Value	Units		Notes Ref
Class 6N Fill (behind wall and excavat and replace below base)	γ_s =	22	kN/m^3		1
	γ_b =	20	kN/m^3		1
	ν' =	0.3	Drained		
	ϕ' cv,k =	35	degrees		1
	ϕ' cv,d =	35	degrees	DA1-C2	
	ϕ' pk,k =	42	degrees		
	ϕ' pk,d =	35.8	degrees	DA1-C2	
	E'=	50000	kPa		

	Parameter	Value	Units		Notes Ref
Cohesive Deposite (Soil Below Foundation)	γ_s =	18	kN/m^3		
	γ_b =	18	kN/m^4		
	ϕ' cv,k =	22	degrees		2
	ϕ' cv,d =	17.9	degrees	DA1-C2	
	ϕ' pk,k =	22	degrees		
	ϕ' pk,d =	17.9	degrees	DA1-C2	
	Cu =	30	kPa		
	C' =	0	kPa		
	Eu=	9000	kPa		3
	E'=	5400	kPa		4
	ν =	0.5	Undrained		4
	ν' =	0.15	Drained		
	OCR=	1			5

1 Class 6N Parameters from HS2 material reuse document: 1MC06 – Stage One C2 – MWCC – North Portal of Chiltern Tunnels to Brackley. Definition of Scheme Design - Earthwork Material Re-Use. Document No. 1MC06-CEK-GT-REP C002-000029, Rev. C01

2 φ' = 22 assuming medium - high plasticity clay

3 Jamiolkowski, Eu / Cu = 300 for lightly overconsolidated, high plasticity cohesive material. Eu = 9MPa.

4 E' / Eu = 0.6 for shallow foundations
E' = 5400

5 OCR assumed 1-2, normally consolidated to lightly overconsolidated

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4 Loads This sheet covers the calculation of the resultant loads for use in greta

1. The Headwall Selfwight loads,moment and centre of gravity from centroid;

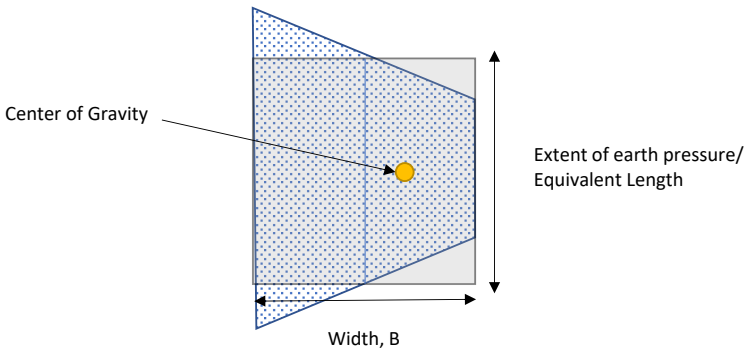
4.1 Equivelent Foundation Sizes

FOUNDATION DETAILS	Symbol	Units	HW 1	HW 2	HW 3	HW 4	HW 5	HW 6
Width	B	m	3.65	3.65	3.55	3.55	3.55	3.55
Equivelent Length	L	m	5.76	4.76	4.22	4.22	4.22	3.77
Construction type	Pre-cast concrete		Smooth	Smooth	Smooth	Smooth	Smooth	Smooth

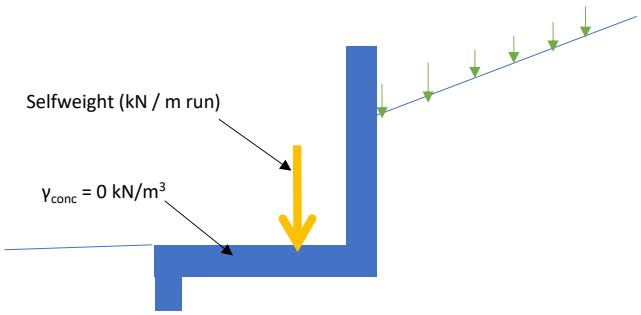
4.2 The following table summarises the Self Weight of Headwall, its Vertical Load and Centre of Gravity, and the Moments about both the center and front edge of the foundation. The Weight of the Headwall for each size was provided by structures. Fz is the weight as a force. The centre of gravity was provided by structures from the front edge of the headwall, this is then calculated as from the cente of the equivelent pad size for each Headwall size. For Greta input, a force per m run is calculated off the previously derived 'equivalent width'.

Headwall Self-Weight						
Effect	HW 1	HW 2	HW 3	HW 4	HW 5	HW 6
Weight (kg)	68603	69419	47299	43323	44241	41284
Fz_1 (kN)	673.0	681.0	464.0	425.0	434.0	405.0
CG position (d) (m)	2.12	2.11	2.11	2.00	1.99	1.99
e_x (m) from centre	0.29	0.28	0.33	0.22	0.22	0.22
Extent of EP (L) (m)	5.76	4.76	4.22	4.22	4.22	3.77
Selfweight (kN / m run) input to Greta	116.9	143.2	109.9	100.7	102.8	107.4

Extent of Equivelent Pad, See Diagram 2.4
= Fz/'equivelent' foundation length



For the Greta input, a unit weight of the wall material has been set to zero. Instead the Selfweight of the wall has been calcualted and directly inputted as a Vertical force (kN / m run) across the equivalent length.

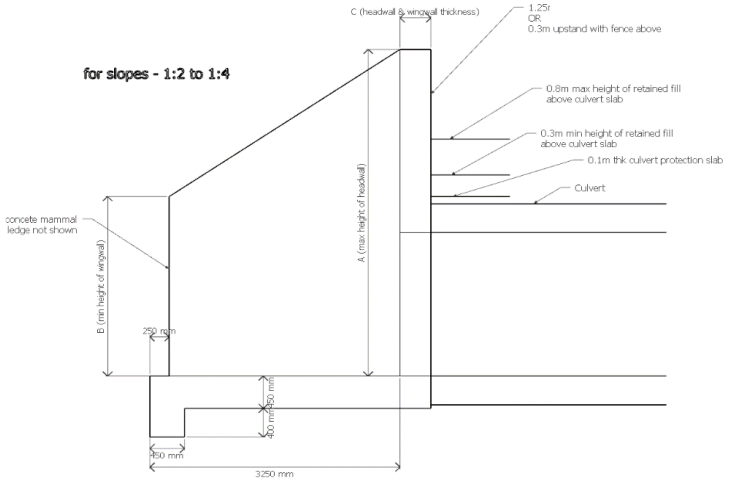


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Reduced Upstand - 0.3m

In some cases, rather than 1.25m upstand above the retained material, there will be only 0.3m with a fence above. Due to the reduced weight of a shorter headwall backwall, this case is also analysed for bearing, sliding and overturning. See Section 6a. HW2 is the only case that fails due to sliding with the reduced weight of a 0.3m upstand therefore requires a thicker base to increase the weight of the headwall in alternative place, see below.

Effect	HW 1	HW 2	HW 3	HW 4	HW 5	HW 6
Weight (kg)	68603	69419	47299	43323	44241	41284
Fz_1 (kN)	597.6	639.5	412.0	374.0	381.9	356.4
CG position (d) (m)	2.12	1.96	2.11	2.00	1.99	1.99
e_x (m) from centre	0.29	0.14	0.33	0.22	0.22	0.22
Extent of EP (L) (m)	5.76	4.76	4.22	4.22	4.22	3.77
Selfweight (kN / m run) input to Greta	103.8	134.5	97.6	88.6	90.4	94.5
Offset from centre of back wall	1.33	1.49	1.29	1.40	1.41	1.41
Vertical COG	1.133	1.14	1.002	0.738	0.795	0.808

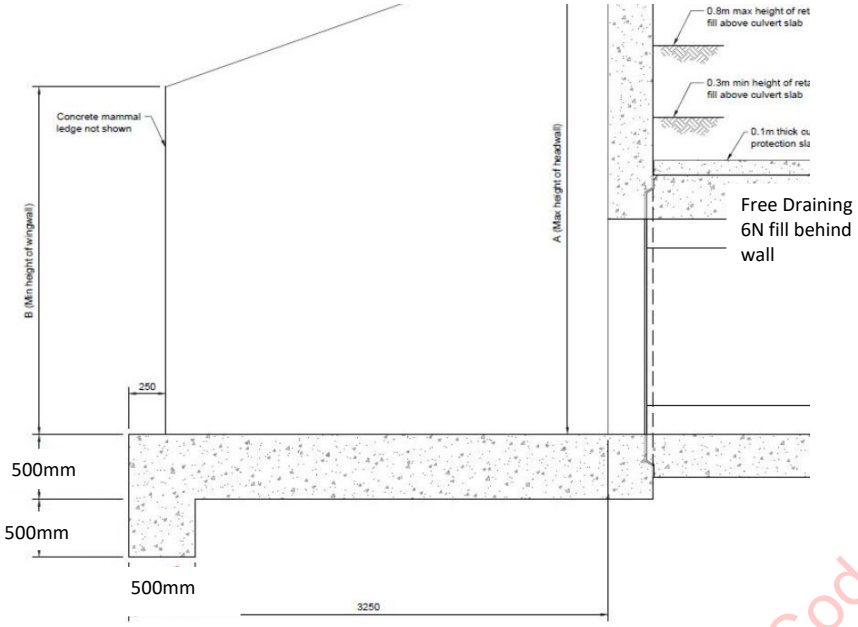


HW2 weight increase solution

1 Base thickess increased from 450mm to 500mm

2 Shear Key thickness increased from 400mm to 500mm, and width increased from 450mm to 500mm

3 To achieve enough bearing capacity, free draining fill behind the wall must be specified in order to lower the design groundwater level behind the wall to top of base slab. 0% extra free draining, fines content 6N/6P fill. This fill requirement is just for HW2.



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5a Allowance for culvert "void" behind wall in earth pressure calculation

Unlike a classic retaining wall, part of the earth pressures acting behind the wall of the headwall are not present due to the culvert. The magnitude of these 'missing' forces is determined and applied in the stability programme Greta as a '-ve' force to determine the stability of the system. This section estimates the earth pressure, surcharge and water pressure that is not acting on the wall due to the culvert. Because these 'missing' pressures are limited to the extent of the culvert width and not across the entire extent of the headwall they must be decreased proportially to the equivalent width for input to the Stability Model (Greta).

Diagram 5.2 illustrates the loads removed due to the culvert. Section 5.2 to 5.5 shows the complete process representatively for headwall size No. 1 (HW 1). The Earth Pressure removed is calculated in section 5.2. Surcharges in section 5.3 and 5.4 is the combining of earth selfweight and surcharge. Section 5.5 is the water pressure calculation for HW 1.
This sheet describes the post construction situation where sloping ground is present and a 10kPa surcharge is modelled and assumed to include any pedestrian loads. Section 5b goes on to calculate the 'missing' pressures for the mid-construction phase.

Diagram 5.1 *Plan of Head wall with Earth Pressure Arrows showing width of culvert with no earth pressure/surcharge/water pressure acting.*

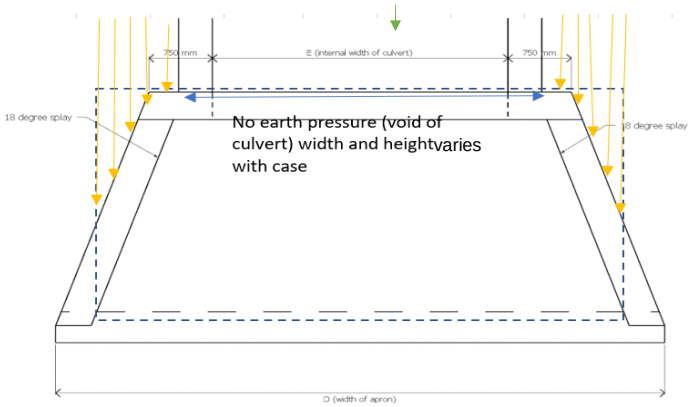
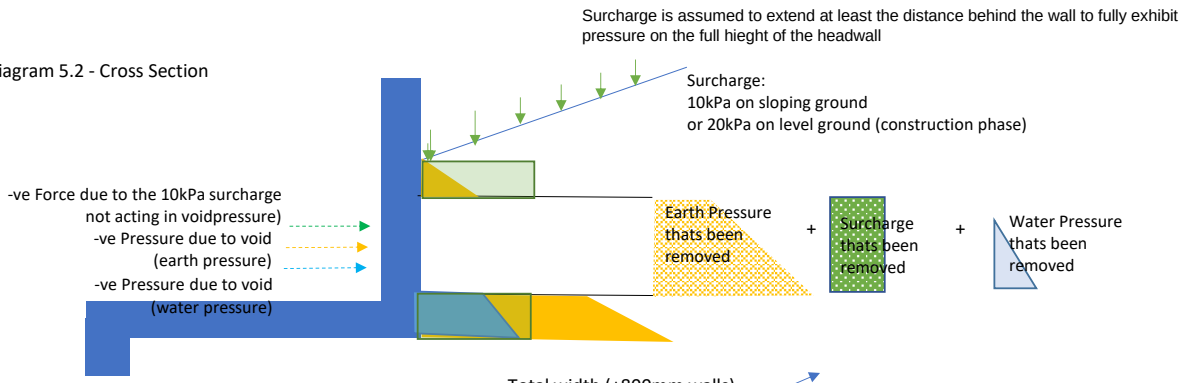
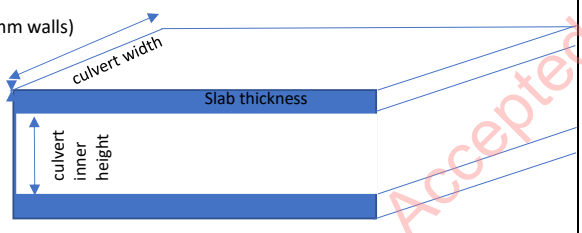


Diagram 5.2 - Cross Section



Total width (+800mm walls)



5.1 Headwall Size needed for calculation

Headwall Number	Culvert Size (m x m)	Inner Height (mm)	Roof slab thickness (mm)	Total Height of culvert (mm)	Inner Culvert width (mm)	Total retained height (mm)
HW 1	**3.5 x 2**	2000	400	2800	3500	3700
HW 2	**2.5 x 3**	3000	400	3800	2500	4700
HW 3	**2 x 2**	2000	300	2600	2000	3500
HW 4	**2 x 1.35**	1350	300	1950	2000	2850
HW 5	**2 x 1.5**	1500	300	2100	2000	3000
HW 6	**1.5 x 1.5**	1500	300	2100	1500	3000

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Example for HW 1 (Permanent works, sloping ground)

for Headwall 1

5.2 Earth Pressure per m run across headwall width

DA1C2 h1= 0.8+0.1= 0.90 m; ka= 0.38 ***Coulomb Theory***

$= \gamma_b h_1 k_a h_2 + \frac{1}{2}\gamma_b k_a h_2^2$

= 20 x 0.90 x 2.80 x 0.38 + 0.5 x 20 x 0.38 x 2.8²

= 19.0 + 29.50

= 48.5 kN/m of earth pressure (applied at 1/3 culvert height approx. @0.93m) 0.9333 m

DA1C1 ka= 0.26

= 20.0 x 0.9 x 0.26 x 2.8 + 0.5 x 20 x 0.26 x 2.8²

13.3 + 20.714

34.0 kN characteristic

45.9 kN/m design of earth pressure (applied at 1/3 culvert height approx. @0.93m)

SLS

34.0 kN/m unfactored DA1C1 load

5.3 Surcharge per m run across headwall width

DA1C2

$= Q \times k_a \times h_2 \times$ Partial Factor

= 10 0.38 2.8

= 10.537 kN/m surcharge pressure (applied at half culvert height 1.4 m

Partial Factor = 1

DA1C1

$Q \times k_a \times h_2 \times$ Partial Factor

10 0.26 2.8

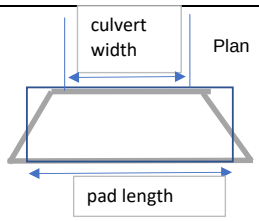
7.4 kN/m surcharge pressure (applied at half culvert height 1.4 m

10.0 kN/m design value

SLS

7.4 kN unfactored DA1C1 load

5.4 For Culvert Wdith



DA1C2 59.0 kN/m run

253.7 kN total on headwall system

Over equivelent length of foundation (to input into greta)

= **44.1** kN/m run across culvert

DA1C1 55.9 kN/m run

240.5 kN total on headwall system

41.8 kN/m run across culvert

SLS 41.4

178.1 total on headwall system

33.2 across culvert

1.3

5.5 water pressure = 0.5 x h² x 10 h=1/3*retained height= 1.23 m

7.6 kN/m across headwall width

SLS/DA1C2 5.7 kN/m across culvert width acts at 0.41 m

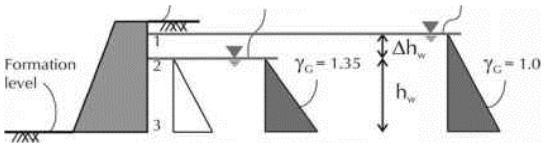
1.23 m Height of GW modelled SLS/DA1C2 (m)

10.2 kN/m across headwall width

DA1C1 5.7 kN/m across culvert width

1.4 kN/m Height of GW modelled DA1C1 (m)

acts at 0.48 m



Design values of groundwater pressures shall represent the most unfavourable values that could occur during the design lifetime of the structure. A third of the retained height is not the worst credible GWL, therefore it is an acceptable method for the water pressure is treated as a permanent action and factored by yg = 1.35 in DA1C1 ULS

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5.6 **For all Headwalls:** Sloping ground 26.6°

Couloumb theory DA1 C2 ka = 0.38 γ 20 kN/m^3

DA1 C1 ka = 0.26 h1= 0.9 m

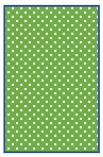
Earth Pressure removed due to culvert (1 : 2 sloping ground)



$\gamma_b h_1 k_a h_2 + \frac{1}{2}\gamma_b k_a h_2^2$

	h_2	DA1 C2	DA1 C1	SLS
	m	kN	kN	kN
HW 1	2.8	48.5	45.9	34.0
HW 2	3.8	80.1	75.9	56.2
HW 3	2.6	43.1	40.8	30.2
HW 4	2.0	27.5	26.1	19.3
HW 5	2.1	30.8	29.2	21.6
HW 6	2.1	30.8	29.2	21.6

10kPa Surcharge removed due to culvert (1 : 2 sloping ground)



$Q \times k_a \times h_2$

	DA1C2	DA1C1	SLS
	kN	kN	kN
HW 1	10.54	9.99	7.40
HW 2	14.30	13.55	10.04
HW 3	9.78	9.27	6.87
HW 4	7.34	6.96	5.15
HW 5	7.90	7.49	5.55
HW 6	7.90	7.49	5.55

Combined for sloping ground (1:2 slope)

	DA1C2			DA1C1			SLS		
	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)
HW 1	59.0	253.7	44.1	55.9	240.5	41.8	41.4	178.1	31.0
HW 2	94.4	311.5	65.5	89.5	295.2	62.1	66.3	218.7	46.0
HW 3	52.8	147.9	35.0	50.1	140.2	33.2	37.1	103.9	24.6
HW 4	34.9	97.6	23.1	33.0	92.5	21.9	24.5	68.5	16.2
HW 5	38.7	108.4	25.7	36.7	102.8	24.3	27.2	76.1	18.0
HW 6	38.7	89.1	23.6	36.7	84.4	22.4	27.2	62.5	16.6

Water Pressure Removed due to culvert



	Total retained height (mm)	Height of GW modelled (m)	Water Pressure unfactored per m run (kN/m) (SLS/DA1C2)	Water Pressure unfactoredtotal over headwall (kN/m) (SLS/DA1C2)	Acting height (m)	Height of GW modelled DA1C1 (m)	Water Pressure unfactored per m run (kN/m) (DA1C1)	Water Pressure unfactoredtotal over headwall (kN/m) (DA1C1)	Acting height (m)
HW 1	3700	1.23	7.6	5.68	0.41	1.43	10.3	7.7	0.48
HW 2*	4700	0.50	1.3	0.87	0.17	0.58	1.7	1.2	0.19
HW 3	3500	1.2	6.8	4.51	0.39	1.36	9.2	6.1	0.45
HW 4	2850	1.0	4.5	2.99	0.32	1.10	6.1	4.0	0.37
HW 5	3000	1.0	5.0	3.32	0.33	1.16	6.8	4.5	0.39
HW 6	3000	1.0	5.0	3.05	0.33	1.16	6.8	4.1	0.39

*HW2 models reduced height of groundwater behind the wall due to specifying a free draining granular fill in order to meet the requirements for bearing capacity

Parameter		Symbol	Combination 1			Combination 2		
			A1	M1	R1	A2	M2	R1
Permanent Action	Unfavourable	$\gamma_{G;dst,\ unfav}$	1.35			1.00		
Variable Action	Unfavourable	γ_Q	1.50			1.30		
	Favourable		0			0		
Accidental Action	Unfavourable	γ_{acc}	1.00			1.00		
	Favourable		0			0		
Bearing Resistance (tan		$\gamma_{\phi'}$		1.00			1.3	
Effective cohesion (c')		$\gamma_{c'}$		1.00			1.3	
Drained Shear Strength		γ_{cu}		1.00			1.4	
Weight Density (γ)		γ_γ		1.00			1.0	
Sliding Resistance (R_V) - B		γ_{Rv}			1.00			1.00
Sliding Resistance (R_H) - S		γ_{RH}			1.00			1.00

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5b Construction Phase 20kPa - Allowance for culvert "void" behind wall in earth pressure calculation

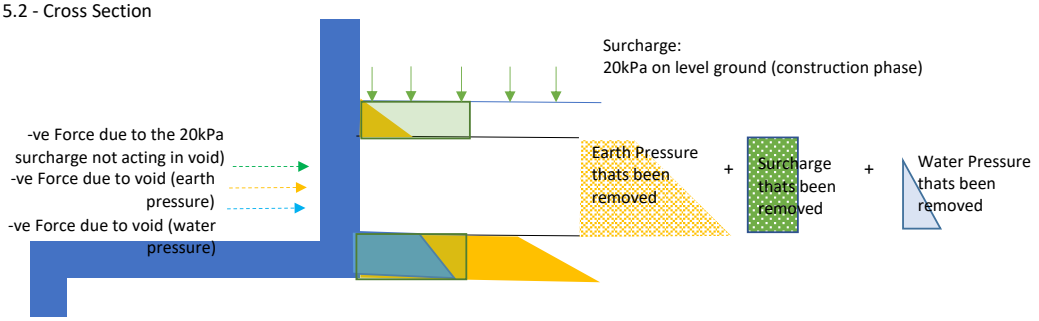
Section 5 calculates the missing loads due to the culvert for use in post constuction stability analysis. This Section uses the same methodoloy as the previous Section 5a, but with the construction phase loading and backfill conditions where compaction vehicles are considered to act directly behind the headwall. For this assessment it is assumed that the most onerous condition for the construction stage is with the maximum retained height and flat ground behind the headwall. As this is a short-term variable load for the construction condition, only the undrained case is analysed.

As before, the magnitude of these 'missing' forces are determined and applied in the stability programme Greta as a '-ve' force to determine the stability of the system. This section estimates the earth pressure, surcharge and water pressure that is not acting on the wall due to the culvert. Here the surcharge is changed to a variable 20kPa rather than a 10kPa used in the post construction assessment. The Greta results are shown in Section 6b.

Rankine Theory Ka is used due to the angle behind the wall being zero. During this phase of construction it is assumed that the minimum 0.5m thickness of 6N fill below the headwall is present providing bearing resistance as specified in the standard detail drawings.

Surcharge is assumed to extend at least the distance behind the wall to fully exhibit pressure on the full hieght of the headwall

Diagram 5.2 - Cross Section



5.1 Headwall Size needed for calculation

Headwall Number	Culvert Size (m x m)	Inner Height (mm)	Roof slab thickness (mm)	Total Height of culvert (mm)	Inner Culvert width (mm)	Total retained height (mm)
HW 1	**3.5 x 2**	2000	400	2800	3500	3700
HW 2	**2.5 x 3**	3000	400	3800	2500	4700
HW 3	**2 x 2**	2000	300	2600	2000	3500
HW 4	**2 x 1.35**	1350	300	1950	2000	2850
HW 5	**2 x 1.5**	1500	300	2100	2000	3000
HW 6	**1.5 x 1.5**	1500	300	2100	1500	3000



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5.6 **For all Headwalls:** Rankine theory (beta=0)

DA1 C2 ka = 0.20 γ 20 kN/m^3

DA1 C1 ka = 0.26 h1= 0.9 m

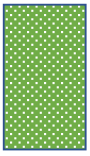
Earth Pressure removed due to culvert (flat ground)



$\gamma_b h_1 k_a h_2 + \frac{1}{2}\gamma_b k_a h_2^2$

	h_2	DA1 C2	DA1 C1	SLS
	m	kN	kN	kN
HW 1	2.8	25.5	45.6	33.8
HW 2	3.8	42.2	75.3	55.8
HW 3	2.6	22.7	40.5	30.0
HW 4	2.0	14.5	25.9	19.2
HW 5	2.1	16.2	29.0	21.5
HW 6	2.1	16.2	29.0	21.5

Surcharge for construction phase (20kPa) removed for culvert



$Q \times k_a \times h_2$

Variable Load so use 1.5 partial factor for unfavourable variable load

	DA1C2	DA1C1	SLS
	kN	kN	kN
HW 1	14.43	22.03	14.69
HW 2	19.58	29.90	19.93
HW 3	13.40	20.46	13.64
HW 4	10.05	15.34	10.23
HW 5	10.82	16.52	11.01
HW 6	10.82	16.52	11.01

Combined for flat ground constuction phase (with 20kN)

	DA1C2			DA1C1			SLS		
	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)	Sum per m run (kN/m)	Total over headwall (kN)	Per m run of equiv foundation (kN/m)
HW 1	40.0	171.8	29.9	67.6	290.8	50.5	48.5	208.4	36.2
HW 2	61.8	203.8	42.9	105.2	347.3	73.0	75.7	249.9	52.6
HW 3	36.1	101.0	23.9	61.0	170.7	40.4	43.6	122.2	28.9
HW 4	24.5	68.7	16.3	41.2	115.4	27.3	29.4	82.3	19.5
HW 5	27.1	75.8	17.9	45.5	127.4	30.2	32.5	91.0	21.5
HW 6	27.1	62.2	16.5	45.5	104.7	27.8	32.5	74.7	19.8

Water Pressure Removed due to culvert



	Total retained height (mm)	Height of GW modelled (m)	greta input GW level (m)	Water Pressure unfactored (kN/m)
HW 1	3700	1.2	0.8	7.6
HW 2	4700	1.6	1.1	12.3
HW 3	3500	1.2	0.7	6.8
HW 4	2850	1.0	0.5	4.5
HW 5	3000	1.0	0.6	5.0
HW 6	3000	1.0	0.6	5.0

Partial Factors

Parameter		Symbol	Combination 1			Combination 2		
			A1	M1	R1	A2	M2	R1
Permanent Action	Infavourable	$\gamma_{G;dst,\ unfav}$	1.35			1.00		
Variable Action	Infavourable	γ_Q	1.50			1.30		
	Favourable		0			0		
Accidental Action	Infavourable	γ_{acc}	1.00			1.00		
	Favourable		0			0		
earing Resistance (tan		$\gamma_{\phi'}$		1.00			1.3	
Effective cohesion (c')		$\gamma_{c'}$		1.00			1.3	
rained Shear Strength		γ_{cu}		1.00			1.4	
Weight Density (γ)		γ_γ		1.00			1.0	
ng Resistance (R_v) - B		γ_{Rv}			1.00			1.00
ng Resistance (R_H) - S		γ_{RH}			1.00			1.00

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6a. Greta Outputs - Post Construction

Method:

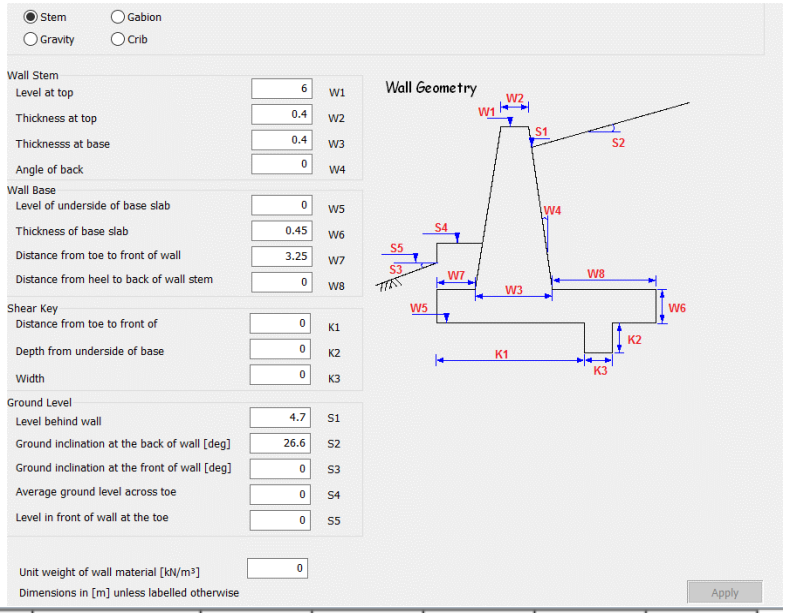
The following sheet provides the Greta Outputs for Bearing, Sliding and Overturning checks. All Degree of utilisation are below 100%.The design does not rely on passive resistance created from the shear key. It is instead assumed that the shear key is there to aid erosion protection.

Undrained and drained analysis is shown for all cases, the undrained results for sliding and overturning are comparable to the drained case. The bearing capacity has a higher factor of safety in undrained than drained. A Punching Shear Check is perfomed in Section 6 for the 2 worst cases in bearing - HW 2 & HW 3 - the bearing capacity was inadequate where the headwall was bearing directly onto cohesive soil so 0.5m of excavate and replace is proposed. Sloping ground is accounted for using Greta's angle input in geometry - the soil was input at 26.6 degrees to the horizontal. Cw/C ratio is assumed to be 1 for both undrained and drained case.

The sum of restoring forces as output by greta shown in the tables includes all input restoring forces as calculated in Section 5. To come to the actual factor of safety the net disturbing force is calculated as the difference between the sum of disturbing forces and the input restoring forces. Then a ratio of the restoring to disturbing can be accurately calculated, this gives a higher FOS than Greta calculates. Both have been provided here for completeness.

Wall Geometry Greta input

Example for 1 HW



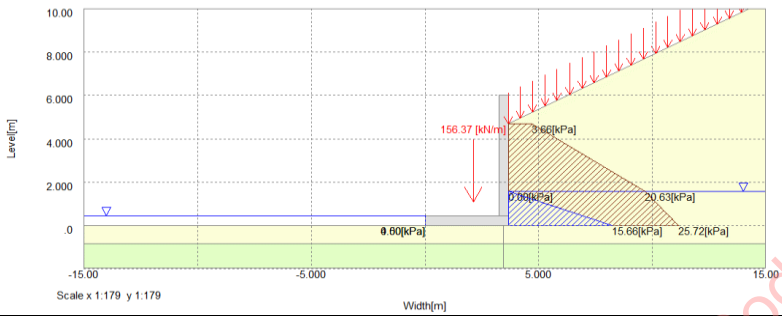
Material Parameters Input

Undrained

Material	A Description	B Earth pressure coefficients	C Drained /Undrained	D Bulk unit weight Above GWL [kN/m3]	E Bulk unit weight Below GWL [kN/m3]	F Strength parameters Phi [°]	G Strength parameters Cohesion [kN/m2]
Defaults		User Specified	Drained				
1	class 6N	Calculated (EC7)	Drained	20.00	22.00	42.00	0.00
2	cohesive	Calculated (EC7)	Undrained	18.00	18.00	0.00	30.00
3							

Drained

Material	Description	Earth pressure coefficients	Drained /Undrained	Bulk unit weight Above GWL [kN/m3]	Bulk unit weight Below GWL [kN/m3]	Strength parameters Phi [°]	Strength parameters Cohesion [kN/m2]
Defaults		User Specified	Drained				
1	class 6N	Calculated (EC7)	Drained	20.00	22.00	42.00	0.00
2	cohesive	Calculated (EC7)	Drained	18.00	18.00	22.00	0.00
3							



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HW 1 Drained			
Set up			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	38.9	111.7	115.6
Average vertical bearing stress [kPa]	27.5	36.8	25.9
Greta Output FOS	1.4	3.0	4.5
Degree of Utilization (%)	70.7	33.0	22.4
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	29.2	35.2	36.2
Sum of disturbing forces [kN]	-73.5	-70.8	-53.0
Sum of restoring forces [kN]	83.3	88.8	78.1
Input Forces (restoring)]kN]	54.1	53.6	41.9
Net disturbing force [kN]	-19.4	-17.2	-11.1
Ratio of Net disturbing to sliding resistance	1.5	2.0	3.3
Sliding factor of safety (Greta)	1.1	1.3	1.5
Calculated Degree of Utilization (%)	66.5	48.9	30.7
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-159.9	-163.7	-131.3
Sum of restoring moments [kNm/m]	308.1	307.3	286.5
Greta Output FOS	1.9	1.9	2.2
Degree of Utilization (%)	51.9	53.3	45.8

HW 1 Undrained			
Set up			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	101.1	147.0	150.4
Average vertical bearing stress [kPa]	28.2	38.1	28.8
Greta Output FOS	3.6	3.9	5.2
Degree of Utilization (%)	27.9	25.9	19.2
Sliding Check			
Sliding resistance [kN]	68.5	98.6	101.9
Sum of disturbing forces [kN]	-73.5	-68.9	-49.5
Sum of restoring forces [kN]	121.2	149.0	141.6
Input Forces (restoring)]kN]	52.7	50.4	39.6
Net disturbing force [kN]	-20.8	-18.5	-9.9
Ratio of Net disturbing to sliding resistance	3.3	5.3	10.3
Sliding factor of safety (Greta)	1.7	2.2	2.9
Calculated Degree of Utilization (%)	30.3	18.8	9.7
Overturning Check			
Sum of disturbing moments [kNm/m]	-159.9	147.0	-109.7
Sum of restoring moments [kNm/m]	304.3	307.1	276.0
Greta Output FOS	1.9	2.0	2.5
Degree of Utilization (%)	52.5	50.0	39.7

HW 2 Drained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	31.1	88.7	98.4
Average vertical bearing stress [kPa]	31.2	44.5	32.0
Greta Output FOS	1.0	2.0	3.1
Degree of Utilization (%)	100.2	50.1	32.5
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	36.0	43.3	44.7
Sum of disturbing forces [kN]	-113.6	-109.7	-82.2
Sum of restoring forces [kN]	117.1	121.0	108.2
Input Forces (restoring)]kN]	81.1	77.6	63.5
Net disturbing force [kN]	-32.6	-32.0	-18.7
Ratio of Net disturbing to sliding resistance	1.1	1.4	2.4
Sliding factor of safety (Greta)	1.0	1.1	1.3
Calculated Degree of Utilization (%)	90.4	73.9	41.8
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-263.2	-264.9	-208.2
Sum of restoring moments [kNm/m]	462.6	454.5	420.7
Greta Output FOS	1.8	1.7	2.0
Degree of Utilization (%)	56.9	58.3	49.5

HW 2 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	92.8	141.9	145.3
Average vertical bearing stress [kPa]	36.4	45.9	31.8
Greta Output FOS	2.6	3.1	4.6
Degree of Utilization (%)	39.2	32.4	21.9
Sliding Check			
Sliding resistance [kN]	65.7	96.5	104.4
Sum of disturbing forces [kN]	-113.6	-106.6	-82.2
Sum of restoring forces [kN]	144.5	171.9	163.7
Input Forces (restoring)]kN]	78.8	75.4	59.3
Net disturbing force [kN]	-34.8	-31.3	-22.9
Ratio of Net disturbing to sliding resistance	1.9	3.1	4.6
Sliding factor of safety (Greta)	1.3	1.6	2.0
Calculated Degree of Utilization (%)	53.0	32.4	21.9
Overturning Check			
Sum of disturbing moments [kNm/m]	-263.2	-251.0	-208.2
Sum of restoring moments [kNm/m]	434.1	431.1	400.6
Greta Output FOS	1.6	1.7	1.9
Degree of Utilization (%)	60.6	58.2	52.0

Punching Shear Check for HW2 was completed (Section 7)

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HW 3 Drained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	23.5	90.1	43.7
Average vertical bearing stress [kPa]	27.1	39.4	27.8
Greta Output FOS	0.9	2.3	1.6
Degree of Utilization (%)	115.5	43.7	63.8
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	24.2	34.2	30.0
Sum of disturbing forces [kN]	-66.5	-62.4	-48.0
Sum of restoring forces [kN]	68.5	76.2	86.9
Input Forces (restoring)]kN]	44.2	42.0	56.9
Net disturbing force [kN]	-22.3	-20.4	8.9
Ratio of Net disturbing to sliding resistance	1.1	1.7	3.4
Sliding factor of safety (Greta)	1.0	1.2	1.2
Calculated Degree of Utilization (%)	92.1	59.7	29.6
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-144.0	-138.5	-119.3
Sum of restoring moments [kNm/m]	247.6	261.8	263.9
Greta Output FOS	1.7	1.9	1.6
Degree of Utilization (%)	58.1	52.9	61.5

HW 3 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	97.2	145.7	147.5
Average vertical bearing stress [kPa]	28.2	39.4	28.0
Greta Output FOS	3.4	3.7	5.3
Degree of Utilization (%)	29.0	27.0	19.0
Sliding Check	36.0		
Sliding resistance [kN]	57.0	87.3	90.6
Sum of disturbing forces [kN]	-66.5	-62.4	-47.5
Sum of restoring forces [kN]	99.8	130.1	123.0
Input Forces (restoring)]kN]	42.8	42.7	32.4
Net disturbing force [kN]	-23.7	-19.7	-15.1
Ratio of Net disturbing to sliding resistance	2.4	4.4	6.0
Sliding factor of safety (Greta)	1.5	2.1	2.6
Calculated Degree of Utilization (%)	41.6	22.5	16.7
Overturning Check			
Sum of disturbing moments [kNm/m]	-144.0	-138.5	-116.7
Sum of restoring moments [kNm/m]	243.7	261.7	244.3
Greta Output FOS	1.7	1.9	2.1
Degree of Utilization (%)	59.1	52.9	47.8

Punching Shear Check for HW3 was completed (Section 7)

HW 4 Drained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	36.4	104.7	105.7
Average vertical bearing stress [kPa]	25.8	38.8	24.4
Greta Output FOS	1.4	2.7	4.3
Degree of Utilization (%)	70.8	37.0	23.0
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	25.3	34.9	31.4
Sum of disturbing forces [kN]	-46.2	-44.4	-33.3
Sum of restoring forces [kN]	54.9	62.7	54.9
Input Forces (restoring)]kN]	29.5	27.8	23.4
Net disturbing force [kN]	-16.6	-16.6	-9.8
Ratio of Net disturbing to sliding resistance	1.5	2.1	3.2
Sliding factor of safety (Greta)	1.2	1.4	1.6
Calculated Degree of Utilization (%)	65.7	47.5	31.3
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-99.4	-103.5	-85.2
Sum of restoring moments [kNm/m]	218.6	236.2	209.5
Greta Output FOS	2.2	2.3	2.5
Degree of Utilization (%)	45.5	43.8	40.7

HW 4 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	102.0	147.6	149.4
Average vertical bearing stress [kPa]	26.2	35.3	24.7
Greta Output FOS	3.9	4.2	6.1
Degree of Utilization (%)	25.7	23.9	16.5
Sliding Check			
Sliding resistance [kN]	64.1	91.9	94.6
Sum of disturbing forces [kN]	-46.2	-43.3	-33.3
Sum of restoring forces [kN]	92.8	119.3	116.3
Input Forces (restoring)]kN]	28.6	27.4	21.7
Net disturbing force [kN]	-17.6	-15.9	-11.5
Ratio of Net disturbing to sliding resistance	3.7	5.8	8.2
Sliding factor of safety (Greta)	2.0	2.8	3.5
Calculated Degree of Utilization (%)	27.4	17.3	12.2
Overturning Check			
Sum of disturbing moments [kNm/m]	-99.4	-96.3	-85.2
Sum of restoring moments [kNm/m]	216.7	216.8	207.8
Greta Output FOS	2.2	2.3	2.4
Degree of Utilization (%)	45.9	44.4	41.0

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HW 5 Drained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	34.1	93.7	101.9
Average vertical bearing stress [kPa]	26.7	35.964	25.1
Greta Output FOS	1.3	2.6	4.1
Degree of Utilization (%)	78.3	38.4	24.7
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	25.8	31.1	32.0
Sum of disturbing forces [kN]	-50.6	-48.6	-36.4
Sum of restoring forces [kN]	58.5	62.1	57.9
Input Forces (restoring)]kN]	32.7	31.0	25.9
Net disturbing force [kN]	-17.9	-17.7	-10.5
Ratio of Net disturbing to sliding resistance	1.4	1.8	3.0
Sliding factor of safety (Greta)	1.2	1.3	1.6
Calculated Degree of Utilization (%)	69.3	56.7	32.9
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-108.6	-112.7	-92.3
Sum of restoring moments [kNm/m]	227.9	226.0	216.8
Greta Output FOS	2.1	2.0	2.3
Degree of Utilization (%)	47.7	49.9	42.6

HW 5 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	101.2	147.0	148.9
Average vertical bearing stress [kPa]	27.3	36.5	25.5
Greta Output FOS	3.7	4.0	5.8
Degree of Utilization (%)	26.9	24.8	17.1
Sliding Check			
Sliding resistance [kN]	62.7	89.9	93.0
Sum of disturbing forces [kN]	-50.6	-47.4	-36.4
Sum of restoring forces [kN]	94.4	120.3	117.0
Input Forces (restoring)]kN]	31.7	30.3	24.0
Net disturbing force [kN]	-18.8	-17.1	-12.4
Ratio of Net disturbing to sliding resistance	3.3	5.3	7.5
Sliding factor of safety (Greta)	1.9	2.5	3.2
Calculated Degree of Utilization (%)	30.1	19.0	13.3
Overturning Check			
Sum of disturbing moments [kNm/m]	-108.6	-104.9	-92.3
Sum of restoring moments [kNm/m]	225.1	225.0	214.8
Greta Output FOS	2.1	2.1	2.3
Degree of Utilization (%)	48.2	46.6	42.9

HW 6 Drained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	31.8	98.2	97.4
Average vertical bearing stress [kPa]	28.7	38.6	26.9
Minimum FOS bearing	1.0	1.0	1.0
Greta Output FOS	1.1	2.5	3.6
Degree of Utilization (%)	90.2	39.3	27.6
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	27.3	33.0	33.8
Sum of disturbing forces [kN]	-50.6	-48.6	-36.4
Sum of restoring forces [kN]	57.7	61.8	58.3
Input Forces (restoring)]kN]	30.5	28.8	24.5
Net disturbing force [kN]	-20.1	-19.8	-11.9
Ratio of Net disturbing to sliding resistance	1.4	1.7	2.8
Sliding factor of safety (Greta)	1.1	1.3	1.6
Calculated Degree of Utilization (%)	73.7	60.0	35.2
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-108.6	-112.7	-92.3
Sum of restoring moments [kNm/m]	232.4	231.2	222.5
Greta Output FOS	2.1	2.1	2.4
Degree of Utilization (%)	46.7	48.8	41.5

HW 6 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	99.9	146.0	148.2
Average vertical bearing stress [kPa]	29.2	38.9	27.3
Greta Output FOS	3.4	3.8	5.4
Degree of Utilization (%)	29.3	26.6	18.4
Sliding Check			
Sliding resistance [kN]	61.8	88.7	91.9
Sum of disturbing forces [kN]	-50.6	-47.4	-36.4
Sum of restoring forces [kN]	91.4	117.1	114.5
Input Forces (restoring)]kN]	29.6	28.4	22.6
Net disturbing force [kN]	-20.9	-19.0	-13.8
Ratio of Net disturbing to sliding resistance	3.0	4.7	6.7
Sliding factor of safety (Greta)	1.8	2.5	3.1
Calculated Degree of Utilization (%)	33.9	21.4	15.0
Overturning Check			
Sum of disturbing moments [kNm/m]	-108.6	-104.9	-92.3
Sum of restoring moments [kNm/m]	230.1	230.2	220.5
Greta Output FOS	2.1	2.2	2.4
Degree of Utilization (%)	47.2	45.6	41.8

HS2 Ltd - Code 1 - Accepted

SUBJECT: **Culvert Headwall Design**

DOCUMENT NO.: 1MC06-CEK-GT-REP-CS04_CL18-000037 C05

ASSET SUMMARY SHEET

updated

Effect	HW 1	HW 2	HW 3	HW 4	HW 5	HW 6
Weight (kg)	68603	69419	47299	43323	44241	41284
Fz_1 (kN)	597.6	639.5	412.0	374.0	381.9	356.4
CG position (d) (m)	2.12	1.96	2.11	2.00	1.99	1.99
e_x (m) from centre	0.29	0.14	0.33	0.22	0.22	0.22
Extent of EP (L) (m)	5.76	4.76	4.22	4.22	4.22	3.77
Selfweight (kN / m run) input to Greta	103.8	134.5	97.6	88.6	90.4	94.5
Offset from centre of back wall	1.33	1.49	1.29	1.40	1.41	1.41

With Upstand 0.3m

Reduced Self Weight

Retained material height stay same

HW2 amended with 500mm base,500mm shear key

HW2 with new centre of gravity, other cases with orignal

HW2 with lowered groundwater level (free draining fill) in long term drained

Vertical COG	HW 1	HW 2	HW 3	HW 4	HW 5	HW 6
	1.133	1.14	1.002	0.738	0.795	0.808

HW 2 Drained

Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	33.4	88.6	101.0
Average vertical bearing stress [kPa]	35.7	48.7	33.6
Greta Output FOS	0.9	1.8	3.0
Degree of Utilization (%) *	106.9	55.0	33.3
Sliding Check	DA1C2	DA1C1	DA1C1
Sliding resistance [kN]	39.561	50.1	49.4
Sum of disturbing forces [kN]	-106.1	-98.8	-73.5
Sum of restoring forces [kN]	113.2	114.4	101.5
Input Forces (restoring)]kN]	73.6	64.3	52.1
Net disturbing force [kN]	-32.4	-34.5	-21.5
Ratio of Net disturbing to sliding resistance	1.22	1.5	2.3
Sliding factor of safety (Greta)	1.07	1.2	1.4
Calculated Degree of Utilization (%)	82.0	68.9	43.5
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-212.7	-199.3	-156.0
Sum of restoring moments [kNm/m]	422.6	407.3	378.0
Greta Output FOS	2.0	2.0	2.4
Degree of Utilization (%)	50.3	48.9	41.3

HW 2 Undrained

Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	88.2	143.7	144.9
Average vertical bearing stress [kPa]	40.4	34.042	28.2
Greta Output FOS	2.2	4.2	5.1
Degree of Utilization (%)	45.7	23.7	19.5
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	54.6	89.8	100.5
Sum of disturbing forces [kN]	-113.6	-82.2	-82.2
Sum of restoring forces [kN]	133.4	149.1	159.8
Input Forces (restoring)]kN]	78.8	59.3	59.3
Net disturbing force [kN]	-34.8	-22.9	-22.9
Ratio of Net disturbing to sliding resistance	1.6	3.9	4.4
Sliding factor of safety (Greta)	1.2	1.8	1.9
Calculated Degree of Utilization (%)	63.8	25.5	22.8
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-263.2	-208.2	-208.2
Sum of restoring moments [kNm/m]	394.1	360.6	366.6
Greta Output FOS	1.5	1.7	1.8
Degree of Utilization (%)	66.8	57.7	56.8

HW 3 Drained

Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	28.9	87.9	53.1
Average vertical bearing stress [kPa]	20.0	31.743	17.4
Greta Output FOS	1.4	2.8	3.1
Degree of Utilization (%)	69.2	36.1	32.8
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	23.2	29.2	25.0
Sum of disturbing forces [kN]	-66.5	-62.4	-71.1
Sum of restoring forces [kN]	67.5	71.3	81.9
Input Forces (restoring)]kN]	44.2	42.0	56.9
Net disturbing force [kN]	-22.3	-20.4	-14.2
Ratio of Net disturbing to sliding resistance	1.0	1.4	1.8
Sliding factor of safety (Greta)	1.0	1.1	1.2
Calculated Degree of Utilization (%)	96.0	69.8	56.9
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-144.0	-138.5	-162.2
Sum of restoring moments [kNm/m]	277.2	274.8	276.9
Greta Output FOS	1.9	2.0	1.7
Degree of Utilization (%)	51.9	50.4	58.6

HW 3 Undrained

Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	96.9	145.5	147.3
Average vertical bearing stress [kPa]	27.5	34.882	24.7
Greta Output FOS	3.5	4.2	6.0
Degree of Utilization (%)	28.3	24.0	16.8
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	56.1	83.8	87.6
Sum of disturbing forces [kN]	-66.5	-62.4	-47.5
Sum of restoring forces [kN]	98.9	126.6	120.0
Input Forces (restoring)]kN]	42.8	42.7	32.4
Net disturbing force [kN]	-23.7	-19.7	-15.1
Ratio of Net disturbing to sliding resistance	2.4	4.3	5.8
Sliding factor of safety (Greta)	1.5	2.0	2.5
Calculated Degree of Utilization (%)	42.3	23.5	17.3
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-144.0	-138.5	-116.7
Sum of restoring moments [kNm/m]	238.1	239.5	222.1
Greta Output FOS	1.7	1.7	1.9
Degree of Utilization (%)	60.5	57.8	52.5

* 0.5m of fill is under the base of the headwall up to the depth of the shearkey, Greta models the headwall as bearing onto the cohesive CWF. Bearing Capacity is sufficient shown through the punching shear check in section 7.

HW 1,4,5,6 were also analysed but less critical than HW2,3 so only the 2 worst cases are reported here.

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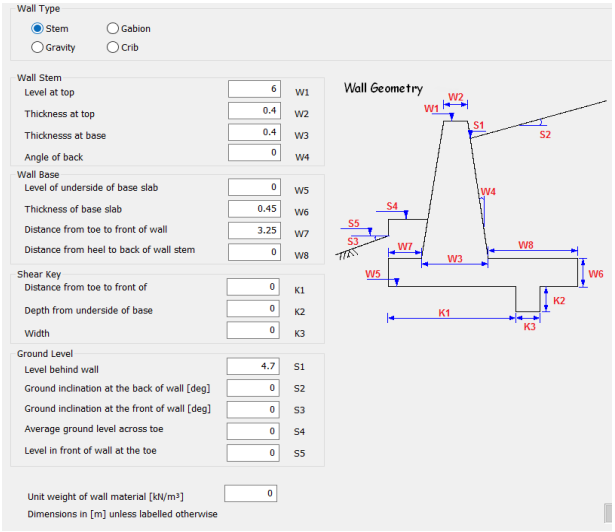
6b. Greta Outputs - Construction Phase

The following sheet provides the Greta Outputs for Bearing, Sliding and Overturning checks for a mid construction phase where compaction vehicles are present behind the wall at the top of the retained material, hence a 20kN/m3 surcharge is used. The design does not rely on passive resistance created from the shear key, as it is too optimistic to assume or specify a full passive wedge of granular material in front of the wall where erosion protection of unspecified properties may be placed. It is instead assumed that the shear key is there to aid erosion protection.

Only an undrained analysis is performed for each Headwall, it is assumed that the compaction loading is a short-term response. DA1C1, DA1C2 and SLS are all run for each headwall size. The Cw/C ratio in every case is assumed to be 1 for adhesion between granular fill and cohseive soil. The retained material is modelled at flat ground as it is not considered reasonable to have a 20kPa construction surcharge applied to the sloping ground (running construction traffic along this).

The sum of restoring forces as output by greta shown in the tables includes all input restoring forces as calculated in Section 5. To come to the actual factor of safety the net disturbing force is calculated as the difference between the sum of disturbing forces and the input restoring forces. Then a ratio of the restoring to disturbing can be accurately calculated, this gives a higher FOS than Greta calculates. Both have been provided here for completeness.

Wall Geometry Greta input
Example for HW 1

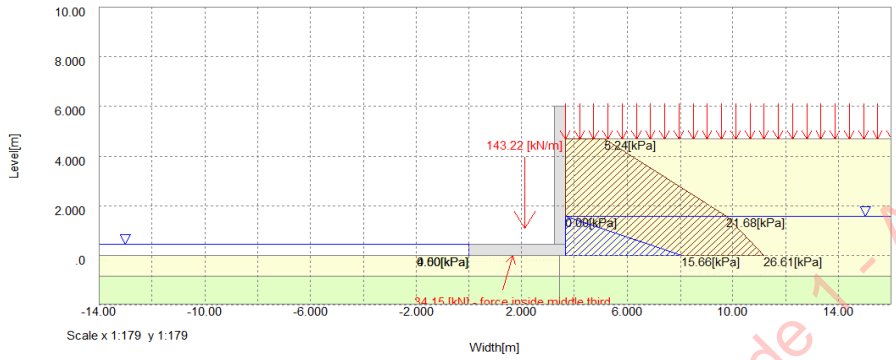


Material Parameters Input
Undrained

Material	A Description	B Earth pressure coefficients	C Drained /Undrained	D Bulk unit weight Above GWL [kN/m3]	E Bulk unit weight Below GWL [kN/m3]	F Strength parameters Phi [°]	G Strength parameters Cohesion [kN/m2]
Defaults		User Specified	Drained				
1	class 6N	Calculated (EC7)	Drained	20.00	22.00	42.00	0.00
2	cohesive	Calculated (EC7)	Undrained	18.00	18.00	0.00	30.00
3							

Drained

Material	Description	Earth pressure coefficients	Drained /Undrained	Bulk unit weight Above GWL [kN/m3]	Bulk unit weight Below GWL [kN/m3]	Strength parameters Phi [°]	Strength parameters Cohesion [kN/m2]
Defaults		User Specified	Drained				
1	class 6N	Calculated (EC7)	Drained	20.00	22.00	42.00	0.00
2	cohesive	Calculated (EC7)	Drained	18.00	18.00	22.00	0.00
3							



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HW 1 Undrained			
Set up			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	99.3278	153.075	154.25
Average vertical bearing stress [kPa]	29.86	36.3122	29.61
Greta Output FOS	3.33	4.21552	5.21
Degree of Utilization (%)	30.06	23.72	19.20
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	64.11	108.85	107.36
Sum of disturbing forces [kN]	-61.29	-62.38	-44.39
Sum of restoring forces [kN]	102.63	167.97	152.18
Input Forces (restoring)]kN]	38.52	59.12	44.82
Net disturbing force [kN]	-22.77	-3.27	0.43
Ratio of Net disturbing to sliding resistance	2.82	33.34	249.21
Sliding factor of safety (Greta)	1.67	2.69	3.43
Calculated Degree of Utilization (%)	35.51	3.00	0.40
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-147.24	-148.87	-106.06
Sum of restoring moments [kNm/m]	280.90	312.16	295.66
Greta Output FOS	1.91	2.10	2.79
Degree of Utilization (%)	52.42	47.69	35.87

HW 2 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	81.122	151.308	151.66
Average vertical bearing stress [kPa]	42.2636	47.4206	31.73
Greta Output FOS	1.91943	3.19077	4.78
Degree of Utilization (%)	63.47	31.34	20.92
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	55.98	106.51	104.37
Sum of disturbing forces [kN]	-99.67	-93.91	-64.57
Sum of restoring forces [kN]	112.17	192.79	171.06
Input Forces (restoring)]kN]	56.18	86.28	66.69
Net disturbing force [kN]	-43.48	-7.62	2.12
Ratio of Net disturbing to sliding resistance	1.29	13.97	49.23
Sliding factor of safety (Greta)	1.13	2.05	2.65
Calculated Degree of Utilization (%)	77.67	7.16	2.03
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-250.659	-236.29	-149.78
Sum of restoring moments [kNm/m]	394.90	444.40	316.05
Greta Output FOS	1.58	1.88	2.11
Degree of Utilization (%)	63.47	53.17	47.39

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HW 3 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	89.2048	150.765	151.00
Average vertical bearing stress [kPa]	33.08	37.3705	29.58
Greta Output FOS	2.70	4.03432	5.10
Degree of Utilization (%)	37.09	24.79	19.59
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	48.14	93.80	93.49
Sum of disturbing forces [kN]	-61.40	-56.89	-43.41
Sum of restoring forces [kN]	79.86	142.02	129.20
Input Forces (restoring)]kN]	31.72	48.22	35.71
Net disturbing force [kN]	-29.68	-8.68	-7.70
Ratio of Net disturbing to sliding resistance	1.62	10.81	12.14
Sliding factor of safety (Greta)	1.30	2.50	2.98
Calculated Degree of Utilization (%)	61.65	9.25	8.24
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-143.3	-135.08	-104.17
Sum of restoring moments [kNm/m]	226.80	266.30	247.84
Greta Output FOS	1.58	1.97	2.38
Degree of Utilization (%)	63.18	50.72	42.03

HW 4 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	101.524	151.221	151.75
Average vertical bearing stress [kPa]	26.79	38.2032	24.00
Greta Output FOS	3.79	3.95834	6.32
Degree of Utilization (%)	26.39	25.26	15.82
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	62.14	96.97	96.79
Sum of disturbing forces [kN]	-39.81	-40.54	-31.20
Sum of restoring forces [kN]	83.97	129.79	121.82
Input Forces (restoring)]kN]	21.83	32.83	25.03
Net disturbing force [kN]	-17.99	-7.71	-6.17
Ratio of Net disturbing to sliding resistance	3.45	12.58	15.68
Sliding factor of safety (Greta)	2.11	3.20	3.90
Calculated Degree of Utilization (%)	28.95	7.95	6.38
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-94.85	-95.69	-84.78
Sum of restoring moments [kNm/m]	207.50	239.06	209.70
Greta Output FOS	2.19	2.50	2.47
Degree of Utilization (%)	45.71	40.03	40.43

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HW 5 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	96.9043	150.325	151.61
Average vertical bearing stress [kPa]	29.85	35.6617	24.67
Greta Output FOS	3.25	4.21532	6.15
Degree of Utilization (%)	30.81	23.72	16.27
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	56.85	92.69	95.72
Sum of disturbing forces [kN]	-48.01	-45.85	-33.94
Sum of restoring forces [kN]	80.77	128.90	123.23
Input Forces (restoring)]kN]	23.91	36.21	27.51
Net disturbing force [kN]	-24.10	-9.64	-6.43
Ratio of Net disturbing to sliding resistance	2.36	9.61	14.89
Sliding factor of safety (Greta)	1.68	2.81	3.63
Calculated Degree of Utilization (%)	42.38	10.40	6.72
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-110.00	-106.56	-91.48
Sum of restoring moments [kNm/m]	215.07	229.59	217.05
Greta Output FOS	1.96	2.15	2.37
Degree of Utilization (%)	51.15	46.41	42.15

HW 6 Undrained			
Bearing Capacity	DA1C2	DA1C1	SLS
Ultimate bearing capacity of soil [kPa]	95.939	149.239	150.85
Average vertical bearing stress [kPa]	31.70	38.1411	26.50
Greta Output FOS	3.03	3.91282	5.69
Degree of Utilization (%)	33.05	25.56	17.57
Sliding Check	DA1C2	DA1C1	SLS
Sliding resistance [kN]	56.64	90.93	94.30
Sum of disturbing forces [kN]	-48.01	-45.85	-33.94
Sum of restoring forces [kN]	79.16	124.75	120.12
Input Forces (restoring)]kN]	22.51	33.81	25.81
Net disturbing force [kN]	-25.50	-12.04	-8.13
Ratio of Net disturbing to sliding resistance	2.22	7.55	11.60
Sliding factor of safety (Greta)	1.65	2.72	3.54
Calculated Degree of Utilization (%)	45.01	13.24	8.62
Overturning Check	DA1C2	DA1C1	SLS
Sum of disturbing moments [kNm/m]	-110.00	-106.56	-91.48
Sum of restoring moments [kNm/m]	220.76	234.23	222.43
Greta Output FOS	2.01	2.20	2.43
Degree of Utilization (%)	49.83	45.49	41.13

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7. Punching Shear Calculation

This sheet provides a punching shear bearing check for Headwalls 2 & 3, where the Greta anaylsis showed they were the critical cases in bearing. This method takes the output component forces from Greta and calculates a bearing resistance based on a 30 degree load spread from 0.5m of granular fill on top of poor cohesive soil, as shown in Diagram 7.1. The bearing assessment for the headwall has been undertaken using the Arcadis Bearing Capacity Spreadsheet. The total foundation load has been calculated considering the outputs from the Oasys Greta analysis undertaken for the sliding and overturning assessment. Additional load due to the self weight of the replacement fill has been included.

HW 2 **1.25m upstand, original base sizing**

Vertical load on Foundation, V =	530.1	kN	
Horizontal load on foundation, H =	-154.8	kN	
Moment loading on foundation, M =	19.4	kNm (About underside of centre)	
Effective cohesion of weak layer, c_2 =	0	kPa	
Effective friction angle of weak layer, ϕ_2 =	17.9	Degrees	DA1C2
Effective cohesion of strong layer, c_1 =	0	kPa	
Effective friction angle of strong layer, ϕ_1 =	35.77	Degrees	DA1C2
Unit weight of strong layer, γ_1 =	20	kN/m³	
Unit weight of weak layer, γ_2 =	18	kN/m³	
Width of foundation, B =	3.65	m	
Length of foundation, L =	4.76	m	
Depth of foundation, D =	0	m (Founding depth)	
Thickness of strong layer below foundation, H =	0.5	m	

Calculations

Eccentricity of load, e = M/V = 0.04 m
Effective foundation width, B' = B-2e = 3.58 m
Load inclination angle, α (relative to the vertical) = tan⁻¹(H/V) = -16.2801 degrees
Effective Width of lower at weaker soil interface assuming 30 degree load spread $B'_{weak\ soil}$ 4.08 m
(assuming no load spread in L dimension)
=B+ 2*0.5*tan(26.6) = 2.77 + 0.5

The additional load due to the self-weight of the replacement fill below the base of the headwall is calculated below:

Effective unit weight of Class 6N Fill	=	10	kN/m³
Depth of fill	=	0.5	m
Length of fill	=	4.76	m
Width of fill	=	3.65	m
Load due to weight of fill	**=**	**86.77875**	**kN**

Increased Foundation Loads

The total foundation forces, including the increase in applied load due to the self weight of granular fill (unit weight = 20kN/m3) is presented below.

Increase normal force = Effective length x effective width x depth of fill x unit weight of fill

The effective foundation loads, considering the spread of load through the underlying Class 6N Fill, are presented in *Table 10-13* below.

HeadWall - Effective Foundation Loads

	Total force on foundation			Effective Width, B' (m)
	Normal Force (kN/m)	Parallel Force (kN/m)	Moment (kNm/m)	
DA1 C2 (Drained)	616.9	-154.8	19.4	4.08

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Moment about centre calculation

	Moment about edge (kNm)	counter moment (kNm)	moment around centre (kNm)
per m run	199.40	203.47	4.07
total	948.13	967.50	19.37

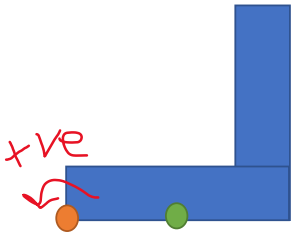
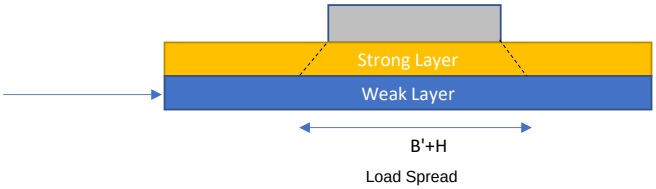


Diagram 7.1



Output from Greta DA1C2 HW2 analysis

Component forces

Component	Normal Force [kN/m]	Parallel Force [kN/m]	Moment [kNm/m]
Weight of wall	0.00000	0.00000	0.00000
Weight of soil	0.00000	0.00000	0.00000
Weight of water	0.00000	0.00000	0.00000
Load on wall	143.22000	80.03800	443.96655
Load behind wall	0.00000	-18.45631	-43.37232
Load in front of wall	0.00000	0.00000	0.00000
Shear on virtual back (excluding base)	0.00000	0.00000	0.00000
Shear on back of base	5.06158	0.00000	18.47478
Shear on virtual front (excluding base)	0.00000	0.00000	0.00000
Shear on front of base	0.00000	0.00000	0.00000
Soil pressure on virtual back	0.00000	-82.89260	-133.88917
Soil pressure on virtual front	0.00000	0.00000	0.00000
Horizontal surchage on back	0.00000	0.00000	0.00000
Water pressure on back	0.00000	-12.26178	-6.40065
Water pressure on front	0.00000	1.01250	0.15187
Water pressure under base	-36.79200	0.00000	-79.53532
Anchor forces on wall	0.00000	0.00000	0.00000
Resultant	111.48958	-32.56018	199.39575

	Bearing capacity (kPa)	Peak Bearing stress (kPa)	Factor of Safety	Utilisation
DA1 C2 (Drained)	100.7	44.8	2.25	45%

The static design indicates the Headwall is stable when founded on 0.5m excavate and replacement of Clay with Class 6N Fill.

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HW 3

Vertical load on Foundation, V =	316.7	kN
Horizontal load on foundation, H =	-84.5	kN
Moment loading on foundation, M =	120.7	kNm (About underside of centre)
Effective cohesion of weak layer, c_2 =	0	kPa
Effective friction angle of weak layer, ϕ_2 =	17.9	Degrees
Effective cohesion of strong layer, c_1 =	0	kPa
Effective friction angle of strong layer, ϕ_1 =	35.77	Degrees
Unit weight of strong layer, γ_1 =	20	kN/m^3
Unit weight of weak layer, γ_2 =	18	kN/m^3
Width of foundation, B =	3.55	m
Length of foundation, L =	4.22	m
Depth of foundation, D =	0	m (Founding depth)
Thickness of strong layer below foundation, H =	0.5	m

Calculations

Eccentricity of load, e = M/V = 0.38 m Effective foundation width, B' = B 2.7881 m

Load inclination angle, α (relative to the vertical) = $\tan^{-1}(H/V)$ = -14.944 degrees

Effective Width of lower at weaker soil interface assuming 30 degree load spread $B'_{weak\ soil}$ 3.29 m (assuming no load spread in L dimension)

The additional load due to the self-weight of the replacement fill below the base of the headwall is calculated below:

Effective unit weight of Class 6N Fill	=	10	kN/m^3
Depth of fill	=	0.5	m
Length of fill	=	4.22	m
Width of fill	=	3.55	m
Load due to weight of fil	**=**	**74.949**	**kN**

Increased Foundation Loads

The total foundation forces, including the increase in applied load due to the self weight of granular fill (unit weight = 20kN/m3) is presented below.

Increase normal force = Effective length x effective width x depth of fill x unit weight of fill

The effective foundation loads, considering the spread of load through the underlying Class 6N Fill, are presented in *Table 10-13* below.

HeadWall - Effective Foundation Loads

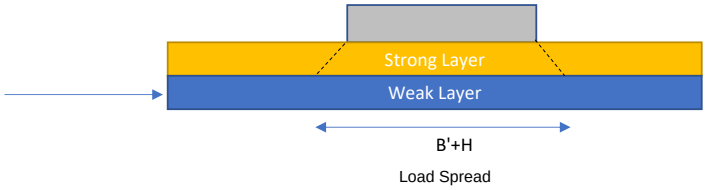
	Total force on foundation			
	Normal Force (kN/m)	Parallel Force (kN/m)	Moment (kNm/m)	Effective Width, B' (m
DA1 C2 (Drained)	391.7	-84.5	120.7	3.29

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Moment about centre calculation

	Moment about edge (kNm)	counter moment (kNm)	moment around centre (kNm)
per m run	104.57	133.14	28.57
total	441.53	562.18	120.65

Diagram 7.1



Output from Greta DA1C2 HW3 analysis

Component forces

Component	Normal Force [kN/m]	Parallel Force [kN/m]	Moment [kNm/m]
Weight of wall	0.00000	0.00000	0.00000
Weight of soil	0.00000	0.00000	0.00000
Weight of water	0.00000	0.00000	0.00000
Load on wall	100.70000	45.50900	234.22824
Load behind wall	0.00000	-13.74406	-24.05210
Load in front of wall	0.00000	0.00000	0.00000
Shear on virtual back (excluding base)	0.00000	0.00000	0.00000
Shear on back of base	3.87379	0.00000	14.13933
Shear on virtual front (excluding base)	0.00000	0.00000	0.00000
Shear on front of base	0.00000	0.00000	0.00000
Soil pressure on virtual back	0.00000	-45.95400	-55.28299
Soil pressure on virtual front	0.00000	0.00000	0.00000
Horizontal surchage on back	0.00000	0.00000	0.00000
Water pressure on back	0.00000	-6.84450	-2.66935
Water pressure on front	0.00000	1.01250	0.15187
Water pressure under base	-29.56500	0.00000	-61.94962
Anchor forces on wall	0.00000	0.00000	0.00000
Resultant	75.00879	-20.02106	104.56538

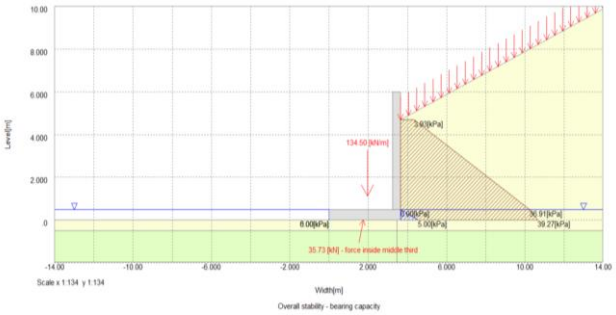
	Bearing capacity (kPa)	Peak Bearing stress (kPa)	Factor of Safety	Utilisation
DA1 C2 (Drained)	82.8	46.8	1.77	56%

The static design indicates the Headwall is stable when founded on 0.5m excavate and replacement of Clay with Class 6N Fill.

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0.3m upstand HW2 increased base thickness to increase self weight.

Ground water level behind the wall is lowered to the front of wall level of top of base slab. To achieve this a 6N fill with 0% fines content allowing drainage behind the wall is specified. Water pressure removed due to allowance for culvert is also reduced because of this. See section 5a, HW2 only has 0.5m Water Pressue



HW 2

Vertical load on Foundation, V =	582.0	kN	
Horizontal load on foundation, H =	-182.7	kN	
Moment loading on foundation, M =	120.0	kNm (About underside of centre)	
Effective cohesion of weak layer, c_2 =	0	kPa	
Effective friction angle of weak layer, ϕ_2 =	17.9	Degrees	DA1C2
Effective cohesion of strong layer, c_1 =	0	kPa	
Effective friction angle of strong layer, ϕ_1 =	35.77	Degrees	DA1C2
Unit weight of strong layer, γ_1 =	20	kN/m³	
Unit weight of weak layer, γ_2 =	18	kN/m³	
Width of foundation, B =	3.65	m	
Length of foundation, L =	4.76	m	
Depth of foundation, D =	0	m (Founding depth)	
Thickness of strong layer below foundation, H =	0.5	m	

Calculations

Eccentricity of load, e = M/V = 0.21 m Effective foundation width, B' = B-2e = 3.2376 m

Load inclination angle, α (relative to the vertical) = $\tan^{-1}$(H/V) = -17.4272 degrees

Effective Width of lower at weaker soil interface assuming 30 degree load spread $B'_{weak\ soil}$ 3.7376 m (assuming no load spread in L dimension)

The additional load due to the self-weight of the replacement fill below the base of the headwall is calculated below:

Effective unit weight of Class 6N Fill	=	10	kN/m³
Depth of fill	=	0.5	m
Length of fill	=	4.76	m
Width of fill	=	3.65	m
Load due to weight of fil	**=**	**86.77875**	**kN**

Increased Foundation Loads

The total foundation forces, including the increase in applied load due to the self weight of granular fill (unit weight = 20kN/m3) is presented below.

Increase normal force = Effective length x effective width x depth of fill x unit weight of fill

The effective foundation loads, considering the spread of load through the underlying Class 6N Fill, are presented in *Table 10-13* below.

HeadWall - Effective Foundation Loads

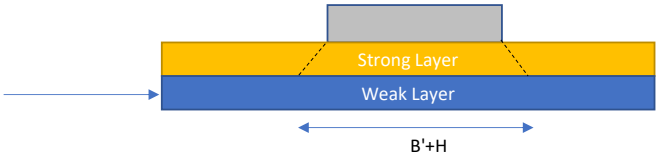
	Total force on foundation			Effective Width, B' (m)
	Normal Force (kN/m)	Parallel Force (kN/m)	Moment (kNm/m)	
DA1 C2 (Drained)	668.8	-182.7	120.0	3.74

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Moment about centre calculation

	Moment about edge (kNm)	counter moment (kNm)	moment around centre (kNm)
per m run	198.14	223.37	25.24
total	942.13	1062.13	120.00

Diagram 7.1



Component forces

Component	Normal Force [kN/m]	Parallel Force [kN/m]	Moment [kNm/m]
Weight of wall	0.00000	0.00000	0.00000
Weight of soil	0.00000	0.00000	0.00000
Weight of water	0.00000	0.00000	0.00000
Load on wall	134.50000	66.37000	388.21790
Load behind wall	0.00000	-18.45631	-43.37232
Load in front of wall	0.00000	0.00000	0.00000
Shear on virtual back (excluding base)	0.00000	0.00000	0.00000
Shear on back of base	6.14525	0.00000	22.43017
Shear on virtual front (excluding base)	0.00000	0.00000	0.00000
Shear on front of base	0.00000	0.00000	0.00000
Soil pressure on virtual back	0.00000	-86.35195	-135.83448
Soil pressure on virtual front	0.00000	0.00000	0.00000
Horizontal surchage on back	0.00000	0.00000	0.00000
Water pressure on back	0.00000	-1.25000	-0.20833
Water pressure on front	0.00000	1.25000	0.20833
Water pressure under base	-18.25000	0.00000	-33.30625
Anchor forces on wall	0.00000	0.00000	0.00000
Resultant	122.39525	-38.43825	198.13501

	Bearing capacity (kPa)	Peak Bearing stress (kPa)	Factor of Safety	Utilisation
DA1 C2 (Drained)	99.0	54.0	1.83	55%

The static design indicates the Headwall is stable when founded on 0.5m excavate and replacement of Clay with Class 6N Fill.

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8 Settlement

Pdisp has been used to carry out a settlement calculation for the equivelent pad foundation. The bearing pressure and applied area (B' * L) were derived by the Greta analysis.

PDisp Analysis

Settlement has been estimated at the base of the foundation due to the self weight of the structure and the foundation.

Assumptions:

a) Boussinesq method is used.

b) PDisp ver 20.0 has been used.

c) Undrained and Drained soil parameters used

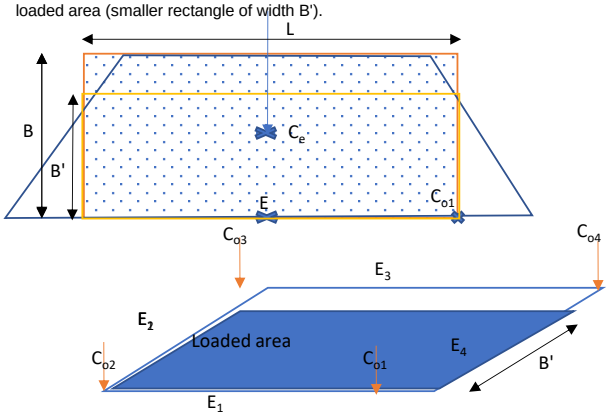
d) SLS approach is followed.

e) Rigid Foundation - settlement is average of corner, edge, middle

f) Pitch forward has been checked against the limits for 'pull-out' of the Culvert segments (10mm)

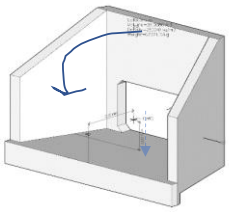
The rigid boundary has been adopted as the level at which the stress increase from loads applied on the culvert reduces to 20% of the effective existing overburden. The rigid boundary derivation can be found in Section 8.2

Diagram showing input to Pdisp - simplified headwall to rectangle and effective loaded area (smaller rectangle of width B').



The Pdisp model used the full foundation area with a non uniform loaded, calculated as B' width

"Pitch forward" due to differential



Results: It is concluded that the calculated settlements are within the tolerance for the headwall/culvert connection. The worst case is HW 2, with a total settlement of 15mm with 6mm of post construction settlement. The post construction tolerance is 100mm as specified by the structures team. However, this is a nominal check on the stand alone headwall and it is deemed that the 100mm is for the headwall system and takes into account the adjacent earthwork causing settlement to the headwall. It is anticpated that the culvert headwall will move in alignment with the culvert and earthwork adjacent.

	Immediate		Total		Post Construction		
	Undrained		Drained Settlement		"Drained - Undrained"		
	averag	maxim	averag	maximum	averag	maximum	
HW 1	5	8	8	13	3	5	largest headwall
HW 2	6	9	10	15	4	6	highest Pave
HW 3	-	-	-	-	-	-	
HW 4	6	6	7	10	1	4	lowest Pave
HW 5	-	-	-	-	-	-	
HW 6	5	8	9	13	4	5	smallest headwall
lower limit	5	6	7	10	1	4	
upper limit	6	9	10	15	4	6	

8.1 Rotation forward

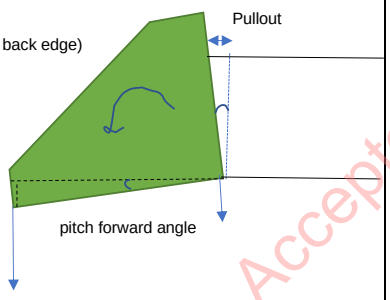
HW 2

The critical headwall size for pitch forward is assessed to be HW 2. The differetial settlement of the front and back of the headwall will result in pull out at the top of the culvert of 6mm and should be taken into account in the culvert design. Top of the culvert pullout tolerance is limited to 10mm by the sealant design as provided by structures.

Largest difference in settlement front and back = Max(front corner, front edge) - Min(back corner, back edge)
= 5.4 mm

pitch forward, angle = $\tan^{-1}(5.4/3650)$
= 0.085 °

pullout at top of culvert (3800mm height) = 5.6 mm
Round up to 6mm



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SUBJECT

Culvert Headwall Design

DOCUMENT NO.

1MC06-CEK-GT-REP-CS04_CL18-000037
C05

8.2 Pdisp Inputs

As taken from greta output

For whole length of headwall

Calculation for Moment about centre

Difference between self wieght moment about the front and vertical force moment about the front to give net moment.

	B	Pad L	A'	Pave	Normal force (V)/kN	Parallel force (H)	Moment	Self Weight moment about centre from vertical force (kNm)	V	H	Moment around front (kNm)	moment around centre (kNm)	e	B'	A'
1	3.65	5.76	21.0	27.4	90.31	17.47	149.00	948.56	519.8	100.6	857.5	91.0	0.2	3.3	19.0
2	3.65	4.76	17.4	30.9	111.49	28.81	201.36	967.49	530.1	137.0	957.5	10.0	0.0	3.6	17.2
3	3.55	4.22	15.0	26.9	75.01	20.02	104.57	562.18	316.7	84.5	441.5	120.7	0.4	2.8	11.8
4	3.55	4.22	15.0	25.7	78.38	15.12	119.67	587.47	331.0	63.9	505.3	82.2	0.2	3.1	12.9
5	3.55	4.22	15.0	26.6	79.72	16.17	119.38	597.49	336.6	68.3	504.1	93.4	0.3	3.0	12.6
6	3.55	3.77	13.4	28.6	84.32	18.13	124.49	564.61	318.1	68.4	469.6	95.0	0.3	3.0	11.1

8.3 Rigid Boundary Calculation

HW 1 - Centre point - as example

Z[Level]	Depth Below EGL (m bgl)	[a] Effective Vertical Stress	[b] Effective Overburden Stress (kN/m2)	Eff. Vertical Stress:Overburden (%) [a]/[b] * 100
0	0.0	30.0	0.0	0.00
-0.5	0.5	26.9	5.1	527.62
-1	1.0	23.3	10.2	228.83
-1.5	1.5	20.1	15.3	131.42
-2	2.0	16.7	20.4	82.06
-2.5	2.5	14.3	25.5	55.94
-3	3.0	12.2	30.6	39.82
-3.5	3.5	10.2	35.7	28.56
-4	4.0	8.8	40.8	21.61
-4.5	4.5	7.7	45.9	16.72
-5	5.0	6.6	51.0	12.86
-5.5	5.5	5.8	56.0	10.32
-6	6.0	5.1	61.1	8.40
-6.5	6.5	4.5	66.2	6.76
-7	7.0	4.0	71.3	5.65
-7.5	7.5	3.6	76.4	4.76
-8	8.0	3.2	81.5	3.96
-8.5	8.5	2.9	86.6	3.40
-9	9.0	2.7	91.7	2.95
-9.5	9.5	2.4	96.8	2.50
-10	10.0	2.2	101.9	2.20
-10.5	10.5	2.1	107.0	1.95
-11	11.0	1.9	112.1	1.68
-11.5	11.5	1.8	117.2	1.51
-12	12.0	1.7	122.3	1.35
-12.5	12.5	1.5	127.4	1.18
-13	13.0	1.4	132.5	1.08
-13.5	13.5	1.4	137.6	0.99
-14	14.0	1.2	142.7	0.87
-14.5	14.5	1.2	147.8	0.82

8.4 HW 1

	drained		undrained	
Rigid Boundary	4.2 mbgl	for centre	4.1	mbgl
	3.8 mbgl	for front edge	3.8	mbgl
	2.8 mbgl	for front left corner	2.8	mbgl

HW 1 rigid boundary set at 4.5m for all positions

Drained		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	12.2	0.0	30.0	71.3	0.0
Edge 1		0.0	0.0	0.0	7.4	0.0	15.0	35.8	0.0
Edge 2		-2.9	1.8	0.0	6.5	0.0	15.0	35.7	0.0
Edge 4		2.9	1.8	0.0	6.5	0.0	15.0	35.7	0.0
Edge 3		0.0	3.7	0.0	5.4	0.0	0.0	1.4	0.0
Corner 1		2.9	0.0	0.0	4.0	0.0	7.5	17.9	0.0
Corner 2		-2.9	0.0	0.0	4.0	0.0	7.5	17.9	0.0
Corner 3		-2.9	3.7	0.0	3.0	0.0	0.0	0.7	0.0
Corner 4		2.9	3.7	0.0	3.0	0.0	0.0	0.7	0.0

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Pdisp Output
Undrained

		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	7.4	0.0	30.0	71.3	0.0
Edge 1		0.0	0.0	0.0	4.5	0.0	15.0	35.8	0.0
Edge 2		-2.9	1.8	0.0	3.9	0.0	15.0	35.7	0.0
Edge 3		0.0	3.7	0.0	3.3	0.0	0.0	1.4	0.0
Edge 4		2.9	1.8	0.0	3.9	0.0	15.0	35.7	0.0
Corner 1		2.9	0.0	0.0	2.4	0.0	7.5	17.9	0.0
Corner 2		-2.9	0.0	0.0	2.4	0.0	7.5	17.9	0.0
Corner 3		-2.9	3.7	0.0	1.8	0.0	0.0	0.7	0.0
Corner 4		2.9	3.7	0.0	1.8	0.0	0.0	0.7	0.0

7.4 Settlement Results HW 1

At the level of foundation:

Average Settlement		
	Undrained	Drained
SLS (mm)	5	8

Maximum Settlement		
	Undrained	Drained
SLS (mm)	8	13

	drained			undrained	
Rigid Boundary	3.9	mbgl	for centre	3.9	mbgl
	3.5	mbgl	for front edge	3.5	mbgl
	2.8	mbgl	for front left corner	2.8	mbgl

HW 2 rigid boundary set at 4m for all positions
Drained

		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	14.5	0.0	40.0	95.0	0.0
Edge 1		0.0	0.0	0.0	8.6	0.0	20.0	47.6	0.0
Edge 2		-2.4	1.8	0.0	7.8	0.0	20.0	47.5	0.0
Edge 4		2.4	1.8	0.0	7.8	0.0	20.0	47.5	0.0
Edge 3		0.0	3.7	0.0	5.7	0.0	0.0	1.4	0.0
Corner 1		2.4	0.0	0.0	4.8	0.0	10.0	23.9	0.0
Corner 2		-2.4	0.0	0.0	4.8	0.0	10.0	23.9	0.0
Corner 3		-2.4	3.7	0.0	3.2	0.0	0.0	0.8	0.0
Corner 4		2.4	3.7	0.0	3.2	0.0	0.0	0.8	0.0

Pdisp Output
Undrained

		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	8.8	0.0	40.0	95.0	0.0
Edge 1		0.0	0.0	0.0	5.2	0.0	20.0	47.6	0.0
Edge 2		-2.4	1.8	0.0	4.7	0.0	20.0	47.5	0.0
Edge 3		0.0	3.7	0.0	3.4	0.0	0.0	1.4	0.0
Edge 4		2.4	1.8	0.0	4.7	0.0	20.0	47.5	0.0
Corner 1		2.4	0.0	0.0	2.9	0.0	10.0	23.9	0.0
Corner 2		-2.4	0.0	0.0	2.9	0.0	10.0	23.9	0.0
Corner 3		-2.4	3.7	0.0	1.9	0.0	0.0	0.8	0.0
Corner 4		2.4	3.7	0.0	1.9	0.0	0.0	0.8	0.0

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7.4 Settlement Results HW 2

At the level of foundation:

Average Settlement		
	Undrained	Drained
SLS (mm)	6	10

Maximum Settlement		
	Undrained	Drained
SLS (mm)	9	15

Rigid Boundary	
3.6 mbgl	for centre
3.2 mbgl	for front edge
2.5 mbgl	for front left corner

HW 4 rigid boundary set at 4m for all positions

Drained		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	9.5	0.0	26.6	63.1	0.0
Edge 1		0.0	0.0	0.0	5.6	0.0	13.3	31.7	0.0
Edge 2		-2.1	1.8	0.0	5.2	0.0	13.3	31.6	0.0
Edge 4		2.1	1.8	0.0	5.2	0.0	13.3	31.6	0.0
Edge 3		0.0	3.7	0.0	3.9	0.0	0.0	1.2	0.0
Corner 1		2.1	0.0	0.0	3.1	0.0	6.7	15.9	0.0
Corner 2		-2.1	0.0	0.0	3.1	0.0	6.7	15.9	0.0
Corner 3		-2.1	3.7	0.0	2.3	0.0	0.0	0.6	0.0
Corner 4		2.1	3.7	0.0	2.3	0.0	0.0	0.6	0.0

Pdisp Output Undrained		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	5.8	0.0	26.6	63.1	0.0
Edge 1		0.0	0.0	0.0	3.4	0.0	13.3	31.7	0.0
Edge 2		-2.1	1.8	0.0	3.2	0.0	13.3	31.6	0.0
Edge 3		0.0	3.7	0.0	2.3	0.0	0.0	1.2	0.0
Edge 4		2.1	1.8	0.0	3.2	0.0	13.3	31.6	0.0
Corner 1		2.1	0.0	0.0	1.9	0.0	6.7	15.9	0.0
Corner 2		-2.1	0.0	0.0	1.9	0.0	6.7	15.9	0.0
Corner 3		-2.1	3.7	0.0	1.4	0.0	0.0	0.6	0.0
Corner 4		2.1	3.7	0.0	1.4	0.0	0.0	0.6	0.0

7.4 Settlement Results HW 4

At the level of foundation:

Average Settlement		
	Undrained	Drained
SLS (mm)	4	7

Maximum Settlement		
	Undrained	Drained
SLS (mm)	6	10

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Rigid Boundary	5.4 mbgl
	4.6 mbgl
	3.6 mbgl

HW 6 rigid boundary set at 5.5m for all positions

Drained		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	12.2	0.0	31.0	73.5	0.0
Edge 1		0.0	0.0	0.0	7.6	0.0	15.5	36.9	0.0
Edge 2		-1.9	1.8	0.0	7.2	0.0	15.1	36.3	0.0
Edge 4		1.9	1.8	0.0	7.2	0.0	15.1	36.3	0.0
Edge 3		0.0	3.7	0.0	4.8	0.0	0.0	0.8	0.0
Corner 1		1.9	0.0	0.0	4.6	0.0	7.5	18.2	0.0
Corner 2		-1.9	0.0	0.0	4.6	0.0	7.5	18.2	0.0
Corner 3		-1.9	3.7	0.0	3.1	0.0	0.0	0.5	0.0
Corner 4		1.9	3.7	0.0	3.1	0.0	0.0	0.5	0.0

Pdisp Output

Undrained		x	y	z	dz	Stress: Calc. Level	Stress: Vertical	Stress: Sum Princ.	Vert. Strain
		[m]	[m]	[mOD]	[mm]	[mOD]	[kN/m²]	[kN/m²]	[ε]
Centre		0.0	1.8	0.0	7.4	0.0	31.0	73.5	0.0
Edge 1		0.0	0.0	0.0	4.6	0.0	15.5	36.9	0.0
Edge 2		-1.9	1.8	0.0	4.3	0.0	15.1	36.3	0.0
Edge 4		1.9	1.8	0.0	4.3	0.0	15.1	36.3	0.0
Edge 3		0.0	3.7	0.0	2.9	0.0	0.0	0.8	0.0
Corner 1		1.9	0.0	0.0	2.8	0.0	7.5	18.2	0.0
Corner 2		-1.9	0.0	0.0	2.8	0.0	7.5	18.2	0.0
Corner 3		-1.9	3.7	0.0	1.9	0.0	0.0	0.5	0.0
Corner 4		1.9	3.7	0.0	1.9	0.0	0.0	0.5	0.0

7.4 Settlement Results HW 6

At the level of foundation:

Average Settlement		
	Undrained	Drained
SLS (mm)	5	9

Maximum Settlement		
	Undrained	Drained
SLS (mm)	8	13

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9. Derivation of Subgrade Moduli & Structural Earth Pressures

Pdisp software is used with a nominal load of 100kPa with a simplified rectangle load (a trapezium was modelled for case 1 and shown to make negligable difference, see 9.2). 100kPa nominal load is used. 3 No. analyses were completed for each headwall size: a nominally assessed 'upperbound' ground conditions (drained only) and a 'lowerbound' stiffness ground condition (with both undrained and drained).

The 'upperbound' condition is considered as a dense granular material of 50MPa of sufficient thickness. The lower bound corresponds to the strength and stiffness of the material defined in section 3 (0.5m of granular fill material overlying undrained cohesive soil with cu=30kPa and Eu = 9 and E' = 5.4 MPa).

Both Short-term and long-term sub-grade moduli are provided for the lowerbound ground conditions.

As it is assessed that the foundation will act as a single rigid base, an average sub-grade modulus is given of the 3 geometric extremes (centre, corner edge).

Ground Model

Material 1	cu=30 kPa	Soft cohesive material		
lower bound		Eu=	9 MPa	typical clay over consolidated, high plasticity
		E'=	5.4 Mpa	350*Cu relationship undrained and drained
Material 2		Granular fill / weak bedrock		
upper bound		E'=	50 Mpa	assuming E+R with granular fill
		drained only		stiff/hard material below

Ground Model 1 (analysis 1 and 2)		
	0 to 0.5m E+R	material 2
	0.5 to base	material 1
Ground model 2 (analysis 3)		
	all depth	material 2

	L/B	rigid/flexible
HW 1	1.58	Rigid
HW 2	1.30	Rigid
HW 3	1.19	Rigid
HW 4	1.19	Rigid
HW 5	1.19	Rigid
HW 6	1.06	Rigid

9.1 Subgrade modulus

3 No. Conditions have been examined to give upper and lower bound of typical values to structures for standard details:

1. 0.5m 6N fill over soft cohesive (undrained)
2. 0.5m 6N fill over soft cohesive (drained)
3. Signifigant thickness of dense granular material (Class 6N)

See settlement calc sheet for rigid boundary calculation

HW 1 — rigid boundary for subgrade modulus using 100kPa, = 7m

	Cohesive drained	Cohesive undrained	Granular/Stiff	units	
corner	19.5	11.7	2.3	mm	largest headwall
centre	52.6	31.8	6.7	mm	
side	24.8	15.0	4.1	mm	
average	33.5	20.2	5.0	mm	
subrage modulus	2986	4945	20000	kN/m3	

HW 2 — rigid boundary for subgrade modulus using 100kPa, = 7m

	Cohseive drained	Cohseive undrained	Granular/Stiff	units	
corner	19.0	11.4	2.3	mm	highest Pave
centre	49.7	30.1	6.4	mm	
side	31.2	18.9	3.9	mm	
average	34.0	21.0	5.0	mm	
subrage modulus	2941	4762	20000	kN/m3	

HW 4 — rigid boundary for subgrade modulus using 100kPa, = 7m

	cohseive drained	cohseive undrained	Granular/Stiff	units	
corner	18.5	11.2	2.2	mm	lowest Pave
centre	47.6	28.8	6.2	mm	
side	29.7	18.0	3.7	mm	
average	32.0	20.0	5.0	mm	
subrage modulus	3125	5000	20000	kN/m3	

HW 6 — rigid boundary for subgrade modulus using 100kPa, = 10m

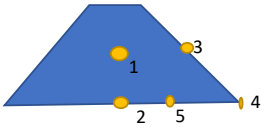
	cohseive drained	cohseive undrained	Granular/Stiff	units	
corner	21.6	13.0	2.6	mm	smallest headwall
centre	50.1	30.3	6.5	mm	
side	32.2	19.5	4.0	mm	
average	35.0	21.0	5.0	mm	
subrage modulus	2857	4762	20000	kN/m3	

HS2 Ltd - Code 1 - Accepted

9.2 Trapezium Comparison

A comparison was conducted for the springs anaylsis in Pdisp for HW 1 against the actual trapezium shape of the headwall base to demonstrate that rectangle simplification was adequate in the derivation of the provided sub-grade moduli values.

Trapezium



Points on diagram above	Cohseive drained	Cohseive undrained	Granular	units
1	52.3	31.6	6.7	mm
2	29.5	17.9	3.7	mm
3	15.7	9.5	1.9	mm
4	32.7	18.4	4.0	mm
5	33.8	20.4	4.1	mm
average	**39.0**	**23.0**	**5.0**	mm
subrage modulus	2564	4348	20000	kN/m3

The worst case settlement of the cente point could be taken which is reliant on the foundation acting as a flexible structure. However given the completely rigid base, the settlement will be conservatively be the average of the settlement of points at approximately even geometric distribution.

Subgrade Modulus	Longterm Cohesive soil	Shortterm Cohesive soil	Stiff soil/granular fill.	units	
Lower bound	2857	4762	20000	kN/m3	
Upper bound	3125	5000	20000	kN/m3	
Since little change	3000	4900	20000	kN/m3	for 100kPa

1 value for each is taken

In a linear elastic analysis then there is no difference due to soil, only due to the depth of influence which is dependent on the foundation size and stress increase.

HS2 Ltd - Code 1 - Accepted

Pressures

For use by Structures for ULS case
k0 acting on back of wall - at rest pressure
Assuming an OCR of 3 for granular fill

1-sin(42)*SQRT(3)*(1+sin(26.6)) **0.83** = k0 **for DA1C1 or setB**
1-sin(35.8)*SQRT(3)*(1+sin(26.6)) **1.04** = k0 **for DA1C2 or setC**

The Earth Pressure Coefficient K0 is given in Eurocode 7 as k0= (10sin(ϕ') x √(OCR) x (1+sin(β)), where β is the sloping ground angle (26.6 degrees) - Meherehof/Kezdi

At rest Earth Pressure - Sloping Ground - Post Construction

0.5*height of reatined material^2*k0*unit weight of soil
e.g. HW 1
force behind wall = 113.58 kN
pressure at base = H*k0*unit weight
61.397 kPa

vertical stress * k0= pressure at any place on the wall

earth pressure

	SetB	SetC				SetB			SetC		
	k0 ratio	k0 ratio	height	unit weight	Surcharge (kN/m2)	pressure at top (kPa)	pressure at base (kPa)	force behind wall (kN/mRun), outside of culvert	pressure at top (kPa)	pressure at base (kPa)	force behind wall (kN/mRun), outside of culvert
HW 1	**0.83**	**1.04**	3.70	20.0	10	11.2	94.1	194.8	10.4	87.4	181.0
HW 2	**0.83**	**1.04**	4.70	20.0	10	11.2	116.5	300.1	10.4	108.2	278.8
HW 3	**0.83**	**1.04**	3.50	20.0	10	11.2	89.6	176.4	10.4	83.3	163.9
HW 4	**0.83**	**1.04**	2.85	20.0	10	11.2	75.0	122.9	10.4	69.7	114.2
HW 5	**0.83**	**1.04**	3.00	20.0	10	11.2	78.4	134.4	10.4	72.9	124.9
HW 6	**0.83**	**1.04**	3.00	20.0	10	11.2	78.4	134.4	10.4	72.9	124.9

At rest Earth Pressure - Flat Ground - During Construction

1-sin(42)*SQRT(3)*(1+sin(0)) **0.57** = k0 **for DA1C1 or setB**
1-sin(35.8)*SQRT(3)*(1+sin(0)) **0.72** = k0 **for DA1C2 or setC**

earth pressure

	SetB	SetC				SetB			SetC		
	k0 ratio	k0 ratio	height	unit weight	Surcharge (kN/m2)	pressure at top (kPa)	pressure at base (kPa)	force behind wall (kN/mRun), outside of culvert	pressure at top (kPa)	pressure at base (kPa)	force behind wall (kN/mRun), outside of culvert
HW 1	**0.57**	**0.72**	3.70	20.0	20	17.2	74.4	169.5	18.7	71.9	167.6
HW 2	**0.57**	**0.72**	4.70	20.0	20	17.2	89.9	251.7	18.7	86.3	246.6
HW 3	**0.57**	**0.72**	3.50	20.0	20	17.2	71.3	154.9	18.7	69.0	153.5
HW 4	**0.57**	**0.72**	2.85	20.0	20	17.2	61.3	111.8	18.7	59.7	111.7
HW 5	**0.57**	**0.72**	3.00	20.0	20	17.2	63.6	121.2	18.7	61.8	120.8
HW 6	**0.57**	**0.72**	3.00	20.0	20	17.2	63.6	121.2	18.7	61.8	120.8

Water Pressures on Headwall

		SLS/DA1C2		DA1C1	
	Total retained height (mm)	Height of GW modelled (DA1C2/SLS) (m)	Water Pressure unfactored per m run (kN/m) (SLS/DA1C2)	Height of GW modelled DA1C1 (m)	Water Pressure unfactored per m run (kN/m) (DA1C1)
HW 1	3700	1.2	7.6	1.4	10.3
HW 2	4700	1.6	12.3	1.8	16.6
HW 3	3500	1.2	6.8	1.4	9.2
HW 4	2850	1.0	4.5	1.1	6.1
HW 5	3000	1.0	5.0	1.2	6.8
HW 6	3000	1.0	5.0	1.2	6.8

HS2 Ltd - Code 1 - Accepted

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue		Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	1 of 3

CHARACTERISTIC SOIL PARAMETERS	Symbol	Units	INPUT
Bulk density (overburden soil)	γ_b	kN/m^3	20.0
Saturated density (overburden soil)	γ_{sat}	kN/m^3	22.0
Bulk density (foundation soil)	γ_b	kN/m^3	18.0
Saturated density (foundation soil)	γ_{sat}	kN/m^3	18.0
Density of water	γ_w	kN/m^3	10.0
Undrained shear strength	$c_{u.k}$	kN/m^2	0.0
Effective cohesion	c'_k	kN/m^2	0.0
Angle of shearing resistance	ϕ'_k	degree	22.0

FOUNDATION DETAILS	Symbol	Units	INPUT
Unit weight of concrete	γ_{conc}	kN/m^3	25.0
Width	B	m	4.1
Length	L	m	4.8
Base thickness	D_B	m	0.5
Depth (ground to founding level)	D	m	0.5

ADDITIONAL DESIGN DETAILS	Symbol	Units	INPUT
Depth of groundwater (from ground level)	D_w	m	0.00
Slope gradient	β	degree	0.00
Foundation inclination	α	degree	0.00

DESIGN ACTIONS	Input loads			Total load		
	DA1 C1	DA1 C2	BS	DA1 C1	DA1 C2	BS
Vertical		616.91		327.14	859.24	242.32
Horizontal		-154.82		0.00	-154.82	0.00
Moment		19.37		0.00	19.37	0.00

DESIGN SUMMARY

Method	DA1C1	DA1C2	BS8004
Base	Compression	Compression	Compression
R_d (Drained)	OK	OK	OK
R_d (Undrained)	N/A	N/A	N/A
Utilisation Factor, Δ (%)	13%	45%	5%

RESULTS

R_d/A' (Drained)= **100.7 kN/m^2**

R_d/A' (Undrained)= **N/A kN/m^2**

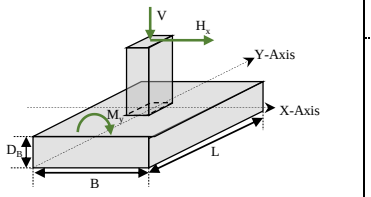
DESIGN ASSUMPTIONS

- Ground condition below foundation extends to depth > B
- Does not calculate bearing resistance of;
  - stronger soil overlying weaker soil
  - weaker soil overlying stronger soil
  - two way eccentrically loaded foundation
- BS8004 method, FOS 1.00 on P_{max}
- Allow Depth Factor ? NO
- Design groundwater level taken as ground level

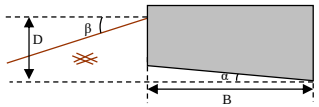
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GEOMETRY

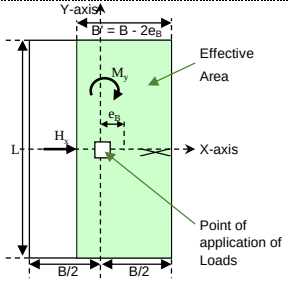
3D VIEW



SECTION



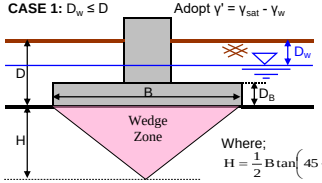
PLAN



EFFECT OF GROUNDWATER ON BEARING RESISTANCE

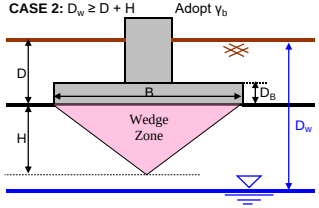
Unit weight to be adopted in bearing resistance calculations are as follow;

CASE 1: $D_w \le D$ Adopt $\gamma' = \gamma_{sat} - \gamma_w$

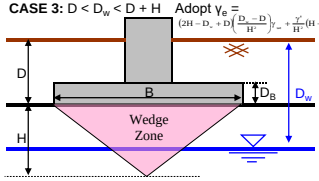


Where; $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$

CASE 2: $D_w \ge D + H$ Adopt γ_b



CASE 3: $D < D_w < D + H$ Adopt γ_e = $\frac{(2H - D_w + D)\left(\frac{D_w - D}{H'}\right)\gamma'_w + \frac{\gamma'}{H'}(H - D_w + D)^2}{2H - D_w + D}$



Code ref	DESIGN SOIL PARAMETERS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.4	$\phi'_{des} = \tan^{-1}\left[\frac{\tan\phi'_k}{\gamma_{\phi'}}\right]$	$\phi'_{des} = \tan^{-1}\left[\frac{\tan 22}{1.00}\right] =$ 22.0 degree	$\phi'_{des} = \tan^{-1}\left[\frac{\tan 22}{1.25}\right] =$ 17.9 degree	$\phi'_{des} = \phi'_p =$ 22 degree
	$c'_{des} = \frac{c'_k}{\gamma_{c'}}$	$c'_{des} = \frac{0.0}{1.00} =$ 0.0 kN/m^2	$c'_{des} = \frac{0.0}{1.25} =$ 0.0 kN/m^2	$c'_{des} = c' =$ 0 kN/m^2
	$c_{u.des} = \frac{c_{u.k}}{\gamma_{cu}}$	$c_{u.des} = \frac{0.0}{1.00} =$ 0.0 kN/m^2	$c_{u.des} = \frac{0.0}{1.40} =$ 0.0 kN/m^2	$c_{u.des} = c_u =$ 0 kN/m^2
	$\gamma_{b.des} = \frac{\gamma_b}{\gamma_\gamma}$	$\gamma_{b.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{b.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{b.des} = \gamma_b =$ 18.0 kN/m^3
	$\gamma_{sat.des} = \frac{\gamma_{sat}}{\gamma_\gamma}$	$\gamma_{sat.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{sat.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{sat.des} = \gamma_{sat} =$ 18.0 kN/m^3
	$\gamma_{w.des} = \frac{\gamma_w}{\gamma_\gamma}$	$\gamma_{w.des} = \frac{10.0}{1.00} =$ 10.0 kN/m^3	$\gamma_{w.des} = \frac{10.0}{1.00} =$ 10.0 kN/m^3	$\gamma_{w.des} = \gamma_w =$ 10.0 kN/m^3
	Height of Wedge Zone $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$	$H = \frac{1}{2}\left[4.08\right] \tan\left[45 + \frac{22.0}{2}\right] =$ 3.022 m	$H = \frac{1}{2}\left[4.08\right] \tan\left[45 + \frac{17.9}{2}\right] =$ 2.801 m	$H = \frac{1}{2}\left[4.08\right] \tan\left[45 + \frac{22.0}{2}\right] =$ 3.022 m

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS	Project: Culvert Headwall Design	Job ref	Calculations							
	Part of structure/scheme and status: Headwall - DA1C2 Drained	Office of issue: Bristol	Telephone no	Division	Calculations by: AJ	Checked by: DG	Date: 06/09/2021	Rev.: A	Design Status: VALID	Calc sheet no: 2 of 3

Code ref	DESIGN ACTIONS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.3 Buildings	$V_{des.tot} = \sum_{unfav}^{fav} V_{des}$	$V_{des.tot}$ = 327.1 kN	$V_{des.tot}$ = 859.2 kN	$V_{des.tot}$ = 242.3 kN
	$H_{x.des.tot} = \sum_{unfav}^{fav} H_{x.des}$	$H_{x.des.tot}$ = 0.00 kN	$H_{x.des.tot}$ = -154.823 kN	$H_{x.des.tot}$ = 0 kN
	$M_{y.des.tot} = \sum_{unfav}^{fav} M_{y.des}$	$M_{y.des.tot}$ = 0 kNm	$M_{y.des.tot}$ = 19.36949 kNm	$M_{y.des.tot}$ = 0 kNm
	Eccentricity in B-Direction $e_B = \frac{M_{y.des.tot}}{V_{des.tot}} \leq \frac{B}{6}$ (Compression) $> \frac{B}{6}$ (Tension)	Eccentricity in B-Direction $e_B = \frac{0}{327.1351} = 0.000$ m $\leq \frac{4.08}{6} = 0.679$ m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{19.36949}{859.236} = 0.023$ m $\leq \frac{4.08}{6} = 0.679$ m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{0}{242.3223} = 0.000$ m $\leq \frac{4.08}{6} = 0.679$ m Base ir Compression
	Effective Width, B' = B -2e_B	B' = 4.08 — 2 ×0.000 = 4.077 m	B' = 4.08 — 2 ×0.023 = 4.031 m	B' = 4.08 — 2 0.000 = 4.077 m
	For Foundation in Compression Maximum Bearing Stress $P_{max} = \frac{V_{des.tot}}{BL}\left(1+\frac{6e_B}{B}\right)$	For Foundation in Compression $P_{max} = \frac{327.1}{4.08 \times 4.76}\left[1+\frac{6\times0.000}{4.08}\right]$ = 16.875 kN/m²	For Foundation in Compression $P_{max} = \frac{859.2}{4.08 \times 4.76}\left[1+\frac{6\times0.023}{4.08}\right]$ = 45.823 kN/m²	For Foundation in Compression $P_{max} = \frac{242.3}{4.08 \times 4.76}\left[1+\frac{6\times0.000}{4.08}\right]$ = 12.500 kN/m²
	For Foundation in Tension Maximum Bearing Stress $P_{max} = \frac{4V_{des.tot}}{3(B-2e_B)L}$	For Foundation in Tension $P_{max} = \frac{4 \times 327.1}{3\left(4.08 - 2\times0.000\right)4.76}$ = 22.500 kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 859.2}{3\left(4.08 - 2\times0.023\right)4.76}$ = 59.772 kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 242.3}{3\left(4.08 - 2\times0.000\right)4.76}$ = 16.667 kN/m²
		Foundation is Compression	**Foundation is Compression**	**Foundation is Compression**
	Effective Width, B'	B' = 4.077 m	B' = 4.031 m	B' = 4.077 m
	Effective Length, L' = L	L' = 4.755 m	L' = 4.755 m	L' = 4.755 m
	Effective Area, A' = B' x L'	A' = 4.077 × 4.755 = 19.386 m²	A' = 4.031 × 4.755 = 19.167 m²	A' = 4.077 × 4.755 = 19.386 m²
	Average Bearing Stress $P_{ave} = \frac{V_{des.tot}}{B' \times L'}$	$P_{ave} = \frac{327.1}{4.077 \times 4.755}$ = 16.875 kN/m²	$P_{ave} = \frac{859.2}{4.031 \times 4.755}$ = 44.828 kN/m²	$P_{ave} = \frac{242.3}{4.077 \times 4.755}$ = 12.500 kN/m²
	Maximum Bearing Stress, P_{max}	P_{max} = 16.875 kN/m²	P_{max} = 45.823 kN/m²	P_{max} = 12.500 kN/m²

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue	Telephone no	Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	3 of 3

Code ref	DESIGN BEARING RESISTANCE DRAINED CONDITION	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004 DRAINED BEARING RESISTANCE
		DRAINED BEARING RESISTANCE	DRAINED BEARING RESISTANCE	DRAINED BEARING RESISTANCE
EC7 Annex D Sec. D.4	Bearing Capacity Factors $N_q = e^{\pi \tan\phi'} \tan^2(45+\phi'/2)$ $N_c = (N_q - 1)\cot\phi'$ $N_\gamma = 2(N_q - 1)\tan\phi'$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1\right)\tan 22.0 = 5.51$	$N_q = e^{\pi \tan 17.9} an^2 \left[45 + \frac{17.9}{2}\right] = 5.21$ $N_c = \frac{5.21 - 1}{\tan 17.9} = 13.03$ $N_\gamma = 2\left(5.21 - 1\right)\tan 17.9 = 2.72$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1 \tan 22.0\right) = 5.51$
	$m = \left[\frac{2+\frac{B'}{L'}}{1+\frac{B'}{L'}}\right]$	$m = \frac{2 + \frac{4.077}{4.76}}{1 + \frac{4.077}{4.76}} = 1.538$ m = 1.538, m+1 = 2.538	$m = \frac{2 + \frac{4.031}{4.76}}{1 + \frac{4.031}{4.76}} = 1.541$ m = 1.541, m+1 = 2.541	$m = \frac{2 + \frac{4.077}{4.76}}{1 + \frac{4.077}{4.76}} = 1.538$ m = 1.538, m+1 = 2.538
	Load Inclination Factors $i_q = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^m$ $i_\gamma = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^{m+1}$ $i_c = \left[i_q - \frac{1-i_q}{N_c \tan\phi'}\right]$	$i_q = \left[1 - \frac{0.00}{327 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{1.538} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{327 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{2.538} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$	$i_q = \left[1 - \frac{-154.82}{859 + \frac{\#\#\#\#\# \times 0.00}{\tan 17.9}}\right]^{1.541} = 1.291$ $i_\gamma = \left[1 - \frac{-154.82}{859 + \frac{\#\#\#\#\# \times 0.00}{\tan 17.9}}\right]^{1.541} = 1.523$ $i_c = 1.291 - \left[\frac{1 - 1.291}{13.03 \tan 17.9}\right] = 1.360$	$i_q = \left[1 - \frac{0.00}{242 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{1.538} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{242 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{2.538} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$
	Base Inclination Factors $b_q = b_\gamma = (1 - \alpha.\tan\phi')^2$ $b_c = b_q - \frac{1-b_q}{N_c \tan\phi'}$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 17.9\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{13.03 \tan 17.9} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$
	Shape Factors $s_q = 1 + \left(\frac{B'}{L'}\right)\sin\phi'$ $s_\gamma = 1 - 0.3\left(\frac{B'}{L'}\right)$ $s_c = \frac{s_q N_q - 1}{N_q - 1}$	$s_q = 1 + \left[\frac{4.077}{4.755}\right]\sin 22.0 = 1.321$ $s_\gamma = 1 - 0.3\left[\frac{4.077}{4.755}\right] = 0.743$ $s_c = \frac{1.321 \times 7.82 - 1}{7.82 - 1} = 1.368$	$s_q = 1 + \left[\frac{4.031}{4.755}\right]\sin 17.9 = 1.261$ $s_\gamma = 1 - 0.3\left[\frac{4.031}{4.755}\right] = 0.746$ $s_c = \frac{1.261 \times 5.21 - 1}{5.21 - 1} = 1.323$	$s_q = 1 + \left[\frac{4.077}{4.755}\right]\sin 22.0 = 1.321$ $s_\gamma = 1 - 0.3\left[\frac{4.077}{4.755}\right] = 0.743$ $s_c = \frac{1.321 \times 7.82 - 1}{7.82 - 1} = 1.368$
Bowles 4th Edition Table 4.5	Depth Factors $d_c = 1 + 0.4k$ $d_q = 1 + 2\tan\phi'(1-\sin\phi')^2 k$ $d_\gamma = 1.0$ Where; $k = \frac{D}{B}$, for $\frac{D}{B} \le 1$ $k = \tan^{-1}\left(\frac{D}{B}\right)$, for $\frac{D}{B} > 1$	$\frac{D}{B} = \frac{0.50}{4.077} = 0.123 \le 1$ $d_c = 1 + 0.4 \times 0.123 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.123 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{4.077} = 0.123 \le 1$ $d_c = 1 + 0.4 \times 0.123 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 17.9 \times \left(1 - \sin 17.9\right)^2 \times 0.123 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{4.077} = 0.123 \le 1$ $d_c = 1 + 0.4 \times 0.123 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.123 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$
	Ground Inclination Factors $g_q = g_\gamma = (1 - \tan\beta)^2$ $g_c = 1 - (\beta/147°)$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$
	Effective overburden, q'	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²
	Vertical Bearing Resistance, R/A'	$R/A' = c'N_c b_c s_c i_c d_c g_c + q'N_q b_q s_q i_q d_q g_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma$ R/A' = 0.0 ×16.88 ×1.000 ×1.368 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.321 ×1.000 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×4.077 ×5.51 ×1.000 ×0.743 ×1.000 ×1.000 ×1.000 R/A' = 128.745 kN/m²	$R/A' = c'N_c b_c s_c i_c d_c g_c + q'N_q b_q s_q i_q d_q g_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma$ R/A' = 0.0 ×13.03 ×1.000 ×1.323 ×1.360 ×1.000 ×1.000 + 6.0 ×5.21 ×1.000 ×1.261 ×1.291 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×4.031 ×2.72 ×1.000 ×0.746 ×1.523 ×1.000 ×1.000 R/A' = 100.719 kN/m²	$R/A' = c'N_c b_c s_c i_c d_c g_c + q'N_q b_q s_q i_q d_q g_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma$ R/A' = 0.0 ×16.88 ×1.000 ×1.368 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.321 ×1.000 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×4.077 ×5.51 ×1.000 ×0.743 ×1.000 ×1.000 ×1.000 R/A' = 128.745 kN/m² R/A'$_{net}$ = 128.7 - 6.000 = ##### kN/m²
EC7 Annex A Table A.5	Design Vertical Bearing Resistance, $R_d/A' = \frac{R/A'}{\gamma_{R;v}}$	$R_d/A' = \frac{128.745}{1.00} = \mathbf{128.745}$ kN/m² $P_{ave} = 16.9$ kN/m² ≤ R_d/A' Bearing Resistance OK $\Delta = \frac{16.875}{128.7} = 13\%$	$R_d/A' = \frac{100.719}{1.00} = \mathbf{100.719}$ kN/m² $P_{ave} = 44.8$ kN/m² ≤ R_d/A' Bearing Resistance OK $\Delta = \frac{44.828}{100.7} = 45\%$	$R_d/A' = \frac{122.745}{1.00} = \mathbf{122.745}$ kN/m² $P_{max.net} = 12.5 - 6.000 = 6.500$ kN/m² ≤ R_d/A'_{net} Bearing Resistance OK $\Delta = \frac{6.500}{122.7} = 5\%$
	Utilisation Factor, Δ (%) $\Delta = \frac{P_{ave}}{R_d/A'}$			

Version 3

HS2 Ltd - Code 1 - Accepted

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue		Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	1 of 3

CHARACTERISTIC SOIL PARAMETERS	Symbol	Units	INPUT
Bulk density (overburden soil)	γ_b	kN/m^3	20.0
Saturated density (overburden soil)	γ_{sat}	kN/m^3	22.0
Bulk density (foundation soil)	γ_b	kN/m^3	18.0
Saturated density (foundation soil)	γ_{sat}	kN/m^3	18.0
Density of water	γ_w	kN/m^3	10.0
Undrained shear strength	$c_{u.k}$	kN/m^2	0.0
Effective cohesion	c'_k	kN/m^2	0.0
Angle of shearing resistance	ϕ'_k	degree	22.0

FOUNDATION DETAILS	Symbol	Units	INPUT
Unit weight of concrete	γ_{conc}	kN/m^3	25.0
Width	B	m	3.3
Length	L	m	4.2
Base thickness	D_B	m	0.5
Depth (ground to founding level)	D	m	0.5

ADDITIONAL DESIGN DETAILS	Symbol	Units	INPUT
Depth of groundwater (from ground level)	D_w	m	0.00
Slope gradient	β	degree	0.00
Foundation inclination	α	degree	0.00

DESIGN ACTIONS	Input loads			Total load		
	DA1 C1	DA1 C2	BS	DA1 C1	DA1 C2	BS
Vertical		391.67		234.29	565.22	173.55
Horizontal		-84.53		0.00	-84.53	0.00
Moment		120.65		0.00	120.65	0.00

DESIGN SUMMARY

Method	DA1C1	DA1C2	BS8004
Base	Compression	Compression	Compression
R_d (Drained)	OK	OK	OK
R_d (Undrained)	N/A	N/A	N/A
Utilisation Factor, Δ (%)	15%	56%	6%

RESULTS

R_d/A' (Drained)= **82.8 kN/m^2**

R_d/A' (Undrained)= **N/A kN/m^2**

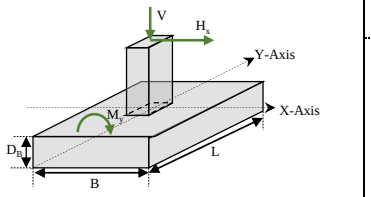
DESIGN ASSUMPTIONS

- Ground condition below foundation extends to depth > B
- Does not calculate bearing resistance of;
  - stronger soil overlying weaker soil
  - weaker soil overlying stronger soil
  - two way eccentrically loaded foundation
- BS8004 method, FOS 1.00 on P_{max}
- Allow Depth Factor ? NO
- Design groundwater level taken as ground level

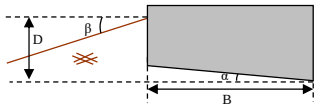
1

GEOMETRY

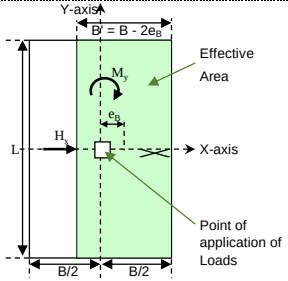
3D VIEW



SECTION



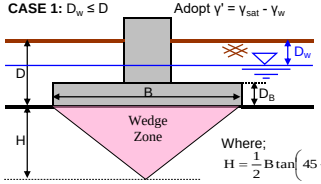
PLAN



EFFECT OF GROUNDWATER ON BEARING RESISTANCE

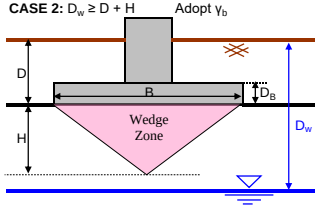
Unit weight to be adopted in bearing resistance calculations are as follow;

CASE 1: $D_w \le D$ Adopt $\gamma' = \gamma_{sat} - \gamma_w$

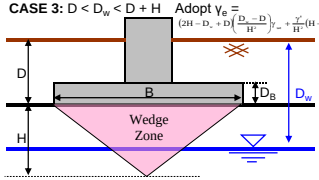


Where; $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$

CASE 2: $D_w \ge D + H$ Adopt γ_b



CASE 3: $D < D_w < D + H$ Adopt γ_e = $\frac{(2H - D_w + D)\left(\frac{D_w - D}{H}\right)\gamma'_w + \frac{\gamma'}{H}(H - D_w + D)^2}{2H - D}$



Code ref	DESIGN SOIL PARAMETERS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.4	$\phi'_{des} = \tan^{-1}\left[\frac{\tan\phi'_k}{\gamma_{\phi'}}\right]$	ϕ'_{des} = $\tan^{-1}\left[\frac{\tan 22}{1.00}\right]$ = 22.0 degree	ϕ'_{des} = $\tan^{-1}\left[\frac{\tan 22}{1.25}\right]$ = 17.9 degree	ϕ'_{des} = ϕ'_p = 22 degree
	$c'_{des} = \frac{c'_k}{\gamma_{c'}}$	c'_{des} = $\frac{0.0}{1.00}$ = 0.0 kN/m^2	c'_{des} = $\frac{0.0}{1.25}$ = 0.0 kN/m^2	c'_{des} = c' = 0 kN/m^2
	$c_{u.des} = \frac{c_{u.k}}{\gamma_{c_u}}$	$c_{u.des}$ = $\frac{0.0}{1.00}$ = 0.0 kN/m^2	$c_{u.des}$ = $\frac{0.0}{1.40}$ = 0.0 kN/m^2	$c_{u.des}$ = c_u = 0 kN/m^2
	$\gamma_{b.des} = \frac{\gamma_b}{\gamma_\gamma}$	$\gamma_{b.des}$ = $\frac{18.0}{1.00}$ = 18.0 kN/m^3	$\gamma_{b.des}$ = $\frac{18.0}{1.00}$ = 18.0 kN/m^3	$\gamma_{b.des}$ = γ_b = 18.0 kN/m^3
	$\gamma_{sat.des} = \frac{\gamma_{sat}}{\gamma_\gamma}$	$\gamma_{sat.des}$ = $\frac{18.0}{1.00}$ = 18.0 kN/m^3	$\gamma_{sat.des}$ = $\frac{18.0}{1.00}$ = 18.0 kN/m^3	$\gamma_{sat.des}$ = γ_{sat} = 18.0 kN/m^3
	$\gamma_{w.des} = \frac{\gamma_w}{\gamma_\gamma}$	$\gamma_{w.des}$ = $\frac{10.0}{1.00}$ = 10.0 kN/m^3	$\gamma_{w.des}$ = $\frac{10.0}{1.00}$ = 10.0 kN/m^3	$\gamma_{w.des}$ = γ_w = 10.0 kN/m^3
	Height of Wedge Zone $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$	$H = \frac{1}{2}\left[3.29\right] \tan\left[45 + \frac{22.0}{2}\right]$ = 2.437 m	$H = \frac{1}{2}\left[3.29\right] \tan\left[45 + \frac{17.9}{2}\right]$ = 2.259 m	$H = \frac{1}{2}\left[3.29\right] \tan\left[45 + \frac{22.0}{2}\right]$ = 2.437 m

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS	Project: Culvert Headwall Design	Job ref	Calculations							
	Part of structure/scheme and status: Headwall - DA1C2 Drained	Office of issue: Bristol	Telephone no	Division	Calculations by: AJ	Checked by: DG	Date: 06/09/2021	Rev.: A	Design Status: VALID	Calc sheet no: 2 of 3

Code ref	DESIGN ACTIONS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.3 Buildings	$V_{des.tot} = \sum_{unfav}^{fav} V_{des}$	$V_{des.tot}$ = 234.3 kN	$V_{des.tot}$ = 565.2 kN	$V_{des.tot}$ = 173.6 kN
	$H_{x.des.tot} = \sum_{unfav}^{fav} H_{x.des}$	$H_{x.des.tot}$ = 0.00 kN	$H_{x.des.tot}$ = -84.5345 kN	$H_{x.des.tot}$ = 0 kN
	$M_{y.des.tot} = \sum_{unfav}^{fav} M_{y.des}$	$M_{y.des.tot}$ = 0 kNm	$M_{y.des.tot}$ = 120.6546 kNm	$M_{y.des.tot}$ = 0 kNm
	Eccentricity in B-Direction $e_B = \frac{M_{y.des.tot}}{V_{des.tot}} \leq \frac{B}{6}$ (Compression) $> \frac{B}{6}$ (Tension)	Eccentricity in B-Direction $e_B = \frac{0}{234.2927}$ = 0.000 m ≤ $\frac{3.29}{6}$ = 0.548 m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{120.6546}{565.2208}$ = 0.213 m ≤ $\frac{3.29}{6}$ = 0.548 m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{0}{173.5502}$ = 0.000 m ≤ $\frac{3.29}{6}$ = 0.548 m Base ir Compression
	Effective Width, B' = B -2e_B	B' = 3.29 — 2 ×0.000 = 3.288 m	B' = 3.29 — 2 ×0.213 = 2.862 m	B' = 3.29 — 2 0.000 = 3.288 m
	For Foundation in Compression Maximum Bearing Stress $P_{max} = \frac{V_{des.tot}}{BL}\left(1+\frac{6e_B}{B}\right)$	For Foundation in Compression $P_{max} = \frac{234.3}{3.29 \times 4.22}\left[1+\frac{6 \times 0.000}{3.29}\right]$ = 16.875 kN/m²	For Foundation in Compression $P_{max} = \frac{565.2}{3.29 \times 4.22}\left[1+\frac{6 \times 0.213}{3.29}\right]$ = 56.533 kN/m²	For Foundation in Compression $P_{max} = \frac{173.6}{3.29 \times 4.22}\left[1+\frac{6 \times 0.000}{3.29}\right]$ = 12.500 kN/m²
	For Foundation in Tension Maximum Bearing Stress $P_{max} = \frac{4V_{des.tot}}{3(B-2e_B)L}$	For Foundation in Tension $P_{max} = \frac{4 \times 234.3}{3\left(3.29 - 2 \times 0.000\right)4.22}$ = 22.500 kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 565.2}{3\left(3.29 - 2 \times 0.213\right)4.22}$ = 62.359 kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 173.6}{3\left(3.29 - 2 \times 0.000\right)4.22}$ = 16.667 kN/m²
		Foundation is Compression	**Foundation is Compression**	**Foundation is Compression**
	Effective Width, B'	B' = 3.288 m	B' = 2.862 m	B' = 3.288 m
	Effective Length, L' = L	L' = 4.223 m	L' = 4.223 m	L' = 4.223 m
	Effective Area, A' = B' x L'	A' = 3.288 × 4.223 = 13.884 m²	A' = 2.862 × 4.223 = 12.085 m²	A' = 3.288 × 4.223 = 13.884 m²
	Average Bearing Stress $P_{ave} = \frac{V_{des.tot}}{B' \times L'}$	$P_{ave} = \frac{234.3}{3.288 \times 4.223}$ = 16.876 kN/m²	$P_{ave} = \frac{565.2}{2.862 \times 4.223}$ = 46.771 kN/m²	$P_{ave} = \frac{173.6}{3.288 \times 4.223}$ = 12.500 kN/m²
	Maximum Bearing Stress, P_{max}	P_{max} = 16.875 kN/m²	P_{max} = 56.533 kN/m²	P_{max} = 12.500 kN/m²

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue	Telephone no	Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	3 of 3

Code ref		STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
	DESIGN BEARING RESISTANCE DRAINED CONDITION	**DRAINED BEARING RESISTANCE**	**DRAINED BEARING RESISTANCE**	**DRAINED BEARING RESISTANCE**
EC7 Annex D Sec. D.4	Bearing Capacity Factors $N_q = e^{\pi \tan\phi'} \tan^2(45+\phi'/2)$ $N_c = (N_q - 1)\cot\phi'$ $N_\gamma = 2(N_q - 1)\tan\phi'$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1\right)\tan 22.0 = 5.51$	$N_q = e^{\pi \tan 17.9} an^2 \left[45 + \frac{17.9}{2}\right] = 5.21$ $N_c = \frac{5.21 - 1}{\tan 17.9} = 13.03$ $N_\gamma = 2\left(5.21 - 1\right)\tan 17.9 = 2.72$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1 \tan 22.0\right) = 5.51$
	$m = \left[\frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}}\right]$	$m = \frac{2 + \frac{3.288}{4.22}}{1 + \frac{3.288}{4.22}} = 1.562$ m = 1.562, m+1 = 2.562	$m = \frac{2 + \frac{2.862}{4.22}}{1 + \frac{2.862}{4.22}} = 1.596$ m = 1.596, m+1 = 2.596	$m = \frac{2 + \frac{3.288}{4.22}}{1 + \frac{3.288}{4.22}} = 1.562$ m = 1.562, m+1 = 2.562
	Load Inclination Factors $i_q = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^m$ $i_\gamma = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^{m+1}$ $i_c = \left[i_q - \frac{1 - i_q}{N_c \tan\phi'}\right]$	$i_q = \left[1 - \frac{0.00}{234 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{1.562} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{234 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{2.562} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$	$i_q = \left[1 - \frac{-84.53}{565 + \frac{\#\#\#\#\# \times 0.00}{\tan 17.9}}\right]^{1.596} = 1.249$ $i_\gamma = \left[1 - \frac{-84.53}{565 + \frac{\#\#\#\#\# \times 0.00}{\tan 17.9}}\right]^{1.596} = 1.436$ $i_c = 1.249 - \left[\frac{1 - 1.249}{13.03 \tan 17.9}\right] = 1.308$	$i_q = \left[1 - \frac{0.00}{174 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{1.562} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{174 + \frac{\#\#\#\#\# \times 0.00}{\tan 22.0}}\right]^{2.562} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$
	Base Inclination Factors $b_q = b_\gamma = (1 - \alpha.\tan\phi')^2$ $b_c = b_q - \frac{1 - b_q}{N_c \tan\phi'}$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 17.9\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{13.03 \tan 17.9} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$
	Shape Factors $s_q = 1 + \left(\frac{B'}{L'}\right)\sin\phi'$ $s_\gamma = 1 - 0.3\left(\frac{B'}{L'}\right)$ $s_c = \frac{s_q N_q - 1}{N_q - 1}$	$s_q = 1 + \left[\frac{3.288}{4.223}\right]\sin 22.0 = 1.292$ $s_\gamma = 1 - 0.3\left[\frac{3.288}{4.223}\right] = 0.766$ $s_c = \frac{1.292 \times 7.82 - 1}{7.82 - 1} = 1.335$	$s_q = 1 + \left[\frac{2.862}{4.223}\right]\sin 17.9 = 1.208$ $s_\gamma = 1 - 0.3\left[\frac{2.862}{4.223}\right] = 0.797$ $s_c = \frac{1.208 \times 5.21 - 1}{5.21 - 1} = 1.257$	$s_q = 1 + \left[\frac{3.288}{4.223}\right]\sin 22.0 = 1.292$ $s_\gamma = 1 - 0.3\left[\frac{3.288}{4.223}\right] = 0.766$ $s_c = \frac{1.292 \times 7.82 - 1}{7.82 - 1} = 1.335$
Bowles 4th Edition Table 4.5	Depth Factors $d_c = 1 + 0.4k$ $d_q = 1 + 2\tan\phi'(1 - \sin\phi')^2 k$ $d_\gamma = 1.0$ Where; $k = \frac{D}{B}$, for $\frac{D}{B} \le 1$ $k = \tan^{-1}\left(\frac{D}{B}\right)$, for $\frac{D}{B} > 1$	$\frac{D}{B} = \frac{0.50}{3.288} = 0.152 \le 1$ $d_c = 1 + 0.4 \times 0.152 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.152 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{3.288} = 0.152 \le 1$ $d_c = 1 + 0.4 \times 0.152 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 17.9 \times \left(1 - \sin 17.9\right)^2 \times 0.152 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{3.288} = 0.152 \le 1$ $d_c = 1 + 0.4 \times 0.152 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.152 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$
	Ground Inclination Factors $g_q = g_\gamma = (1 - \tan\beta)^2$ $g_c = 1 - (\beta/147°)$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$
	Effective overburden, q'	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²
	Vertical Bearing Resistance, R/A' $R/A' = c'N_c b_c s_c i_c d_c g_c + q'N_q b_q s_q i_q d_q g_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma$	R/A' = 0.0 ×16.88 ×1.000 ×1.335 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.292 ×1.000 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×3.288 ×5.51 ×1.000 ×0.766 ×1.000 ×1.000 ×1.000 R/A' = 116.131 kN/m²	R/A' = 0.0 ×13.03 ×1.000 ×1.257 ×1.308 ×1.000 ×1.000 + 6.0 ×5.21 ×1.000 ×1.208 ×1.249 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×2.862 ×2.72 ×1.000 ×0.797 ×1.436 ×1.000 ×1.000 R/A' = 82.803 kN/m²	R/A' = 0.0 ×16.88 ×1.000 ×1.335 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.292 ×1.000 ×1.000 ×1.000 $+\frac{1}{2}\times$ 8.0 ×3.288 ×5.51 ×1.000 ×0.766 ×1.000 ×1.000 ×1.000 R/A' = 116.131 kN/m² R/A'$_{net}$ = 116.1 - 6.000 = ##### kN/m²
EC7 Annex A Table A.5	Design Vertical Bearing Resistance, $R_d/A' = \frac{R/A'}{\gamma_{R;v}}$	$R_d/A' = \frac{116.131}{1.00} =$ **116.131 kN/m²**	$R_d/A' = \frac{82.803}{1.00} =$ **82.803 kN/m²**	$R_d/A' = \frac{110.131}{1.00} =$ **110.131 kN/m²**
		$P_{ave} = 16.9$ kN/m² ≤ R_d/A' Bearing Resistance OK	$P_{ave} = 46.8$ kN/m² ≤ R_d/A' Bearing Resistance OK	$P_{max.net} = 12.5 - 6.000 = 6.500$ kN/m² ≤ R_d/A'_{net} Bearing Resistance OK
	Utilisation Factor, Δ (%) $\Delta = \frac{P_{ave}}{R_d/A'}$	$\Delta = \frac{16.876}{116.1} = 15\%$	$\Delta = \frac{46.771}{82.8} = 56\%$	$\Delta = \frac{6.500}{110.1} = 6\%$

Version 3

HS2 Ltd - Code 1 - Accepted

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue		Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	1 of 3

CHARACTERISTIC SOIL PARAMETERS	Symbol	Units	INPUT
Bulk density (overburden soil)	γ_b	kN/m^3	20.0
Saturated density (overburden soil)	γ_{sat}	kN/m^3	22.0
Bulk density (foundation soil)	γ_b	kN/m^3	18.0
Saturated density (foundation soil)	γ_{sat}	kN/m^3	18.0
Density of water	γ_w	kN/m^3	10.0
Undrained shear strength	$c_{u.k}$	kN/m^2	0.0
Effective cohesion	c'_k	kN/m^2	0.0
Angle of shearing resistance	ϕ'_k	degree	22.0

FOUNDATION DETAILS	Symbol	Units	INPUT
Unit weight of concrete	γ_{conc}	kN/m^3	25.0
Width	B	m	3.7
Length	L	m	4.8
Base thickness	D_B	m	0.5
Depth (ground to founding level)	D	m	0.5

ADDITIONAL DESIGN DETAILS	Symbol	Units	INPUT
Depth of groundwater (from ground level)	D_w	m	0.00
Slope gradient	β	degree	0.00
Foundation inclination	α	degree	0.00

DESIGN ACTIONS	Input loads			Total load		
	DA1 C1	DA1 C2	BS	DA1 C1	DA1 C2	BS
Vertical		668.77		299.91	890.92	222.16
Horizontal		-182.69		0.00	-182.69	0.00
Moment		120.00		0.00	120.00	0.00

DESIGN SUMMARY

Method	DA1C1	DA1C2	BS8004
Base	Compression	Compression	Compression
R_d (Drained)	OK	OK	OK
R_d (Undrained)	N/A	N/A	N/A
Utilisation Factor, Δ (%)	14%	55%	6%

RESULTS

R_d/A' (Drained)= **98.9 kN/m^2**

R_d/A' (Undrained)= **N/A kN/m^2**

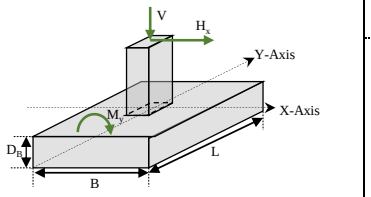
DESIGN ASSUMPTIONS

- Ground condition below foundation extends to depth > B
- Does not calculate bearing resistance of;
  - stronger soil overlying weaker soil
  - weaker soil overlying stronger soil
  - two way eccentrically loaded foundation
- BS8004 method, FOS 1.00 on P_{max}
- Allow Depth Factor ? NO
- Design groundwater level taken as ground level

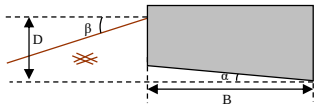
1

GEOMETRY

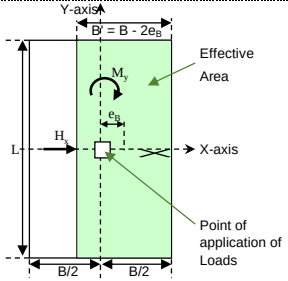
3D VIEW



SECTION



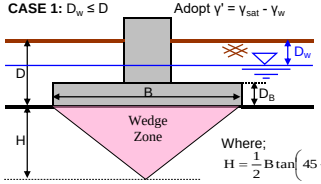
PLAN



EFFECT OF GROUNDWATER ON BEARING RESISTANCE

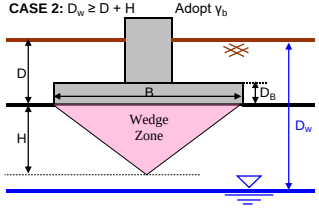
Unit weight to be adopted in bearing resistance calculations are as follow;

CASE 1: $D_w \le D$ Adopt $\gamma' = \gamma_{sat} - \gamma_w$

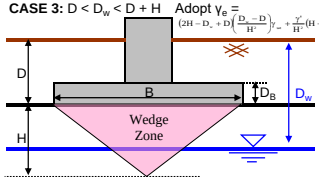


Where; $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$

CASE 2: $D_w \ge D + H$ Adopt γ_b



CASE 3: $D < D_w < D + H$ Adopt $\gamma_e = \frac{(2H - D_w + D)\left(\frac{D_w - D}{H}\right)\gamma'_w + \frac{\gamma'}{H}(H - D_w + D)^2}{(2H - D_w + D)\left(\frac{D_w - D}{H}\right) + \frac{1}{H}(H - D_w + D)^2}$



Code ref	DESIGN SOIL PARAMETERS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.4	$\phi'_{des} = \tan^{-1}\left[\frac{\tan\phi'_k}{\gamma_{\phi'}}\right]$	$\phi'_{des} = \tan^{-1}\left[\frac{\tan 22}{1.00}\right] =$ 22.0 degree	$\phi'_{des} = \tan^{-1}\left[\frac{\tan 22}{1.25}\right] =$ 17.9 degree	$\phi'_{des} = \phi'_p =$ 22 degree
	$c'_{des} = \frac{c'_k}{\gamma_{c'}}$	$c'_{des} = \frac{0.0}{1.00} =$ 0.0 kN/m^2	$c'_{des} = \frac{0.0}{1.25} =$ 0.0 kN/m^2	$c'_{des} = c' =$ 0 kN/m^2
	$c_{u.des} = \frac{c_{u.k}}{\gamma_{c_u}}$	$c_{u.des} = \frac{0.0}{1.00} =$ 0.0 kN/m^2	$c_{u.des} = \frac{0.0}{1.40} =$ 0.0 kN/m^2	$c_{u.des} = c_u =$ 0 kN/m^2
	$\gamma_{b.des} = \frac{\gamma_b}{\gamma_\gamma}$	$\gamma_{b.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{b.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{b.des} = \gamma_b =$ 18.0 kN/m^3
	$\gamma_{sat.des} = \frac{\gamma_{sat}}{\gamma_\gamma}$	$\gamma_{sat.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{sat.des} = \frac{18.0}{1.00} =$ 18.0 kN/m^3	$\gamma_{sat.des} = \gamma_{sat} =$ 18.0 kN/m^3
	$\gamma_{w.des} = \frac{\gamma_w}{\gamma_\gamma}$	$\gamma_{w.des} = \frac{10.0}{1.00} =$ 10.0 kN/m^3	$\gamma_{w.des} = \frac{10.0}{1.00} =$ 10.0 kN/m^3	$\gamma_{w.des} = \gamma_w =$ 10.0 kN/m^3
	Height of Wedge Zone $H = \frac{1}{2} B \tan\left(45 + \frac{\phi'}{2}\right)$	$H = \frac{1}{2}\left[3.74\right] \tan\left[45 + \frac{22.0}{2}\right] =$ 2.771 m	$H = \frac{1}{2}\left[3.74\right] \tan\left[45 + \frac{17.9}{2}\right] =$ 2.568 m	$H = \frac{1}{2}\left[3.74\right] \tan\left[45 + \frac{22.0}{2}\right] =$ 2.771 m

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS	Project: Culvert Headwall Design	Job ref	Calculations							
	Part of structure/scheme and status: Headwall - DA1C2 Drained	Office of issue: Bristol	Telephone no	Division	Calculations by: AJ	Checked by: DG	Date: 06/09/2021	Rev.: A	Design Status: VALID	Calc sheet no: 2 of 3

Code ref	DESIGN ACTIONS	STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
NA to BS EN 1997-1:2004 Table A.NA.3 Buildings	$V_{des.tot} = \sum_{unfav}^{fav} V_{des}$	$V_{des.tot}$ = 299.9 kN	$V_{des.tot}$ = 890.9 kN	$V_{des.tot}$ = 222.2 kN
	$H_{x.des.tot} = \sum_{unfav}^{fav} H_{x.des}$	$H_{x.des.tot}$ = 0.00 kN	$H_{x.des.tot}$ = -182.687 kN	$H_{x.des.tot}$ = 0 kN
	$M_{y.des.tot} = \sum_{unfav}^{fav} M_{y.des}$	$M_{y.des.tot}$ = 0 kNm	$M_{y.des.tot}$ = 119.9966 kNm	$M_{y.des.tot}$ = 0 kNm
	Eccentricity in B-Direction $e_B = \frac{M_{y.des.tot}}{V_{des.tot}} \leq \frac{B}{6}$ (Compression) $> \frac{B}{6}$ (Tension)	Eccentricity in B-Direction $e_B = \frac{0}{299.9099} = 0.000$ m $\leq \frac{3.74}{6} = 0.623$ m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{119.9966}{890.9225} = 0.135$ m $\leq \frac{3.74}{6} = 0.623$ m Base ir Compression	Eccentricity in B-Direction $e_B = \frac{0}{222.1555} = 0.000$ m $\leq \frac{3.74}{6} = 0.623$ m Base ir Compression
	Effective Width, $B' = B - 2e_B$	$B' = 3.74 - 2 \times 0.000 = 3.738$ m	$B' = 3.74 - 2 \times 0.135 = 3.468$ m	$B' = 3.74 - 2 \times 0.000 = 3.738$ m
	For Foundation in Compression Maximum Bearing Stress $P_{max} = \frac{V_{des.tot}}{BL}\left(1 + \frac{6e_B}{B}\right)$	For Foundation in Compression $P_{max} = \frac{299.9}{3.74 \times 4.76}\left[1 + \frac{6 \times 0.000}{3.74}\right] = 16.875$ kN/m²	For Foundation in Compression $P_{max} = \frac{890.9}{3.74 \times 4.76}\left[1 + \frac{6 \times 0.135}{3.74}\right] = 60.993$ kN/m²	For Foundation in Compression $P_{max} = \frac{222.2}{3.74 \times 4.76}\left[1 + \frac{6 \times 0.000}{3.74}\right] = 12.500$ kN/m²
	For Foundation in Tension Maximum Bearing Stress $P_{max} = \frac{4V_{des.tot}}{3(B - 2e_B)L}$	For Foundation in Tension $P_{max} = \frac{4 \times 299.9}{3\,(3.74 - 2 \times 0.000)\,4.76} = 22.500$ kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 890.9}{3\,(3.74 - 2 \times 0.135)\,4.76} = 72.044$ kN/m²	For Foundation in Tension $P_{max} = \frac{4 \times 222.2}{3\,(3.74 - 2 \times 0.000)\,4.76} = 16.667$ kN/m²
		Foundation is Compression	**Foundation is Compression**	**Foundation is Compression**
	Effective Width, B'	B' = 3.738 m	B' = 3.468 m	B' = 3.738 m
	Effective Length, L' = L	L' = 4.755 m	L' = 4.755 m	L' = 4.755 m
	Effective Area, A' = B' x L'	A' = 3.738 × 4.755 = 17.774 m²	A' = 3.468 × 4.755 = 16.490 m²	A' = 3.738 × 4.755 = 17.774 m²
	Average Bearing Stress $P_{ave} = \frac{V_{des.tot}}{B' \times L'}$	$P_{ave} = \frac{299.9}{3.738 \times 4.755} = 16.873$ kN/m²	$P_{ave} = \frac{890.9}{3.468 \times 4.755} = 54.027$ kN/m²	$P_{ave} = \frac{222.2}{3.738 \times 4.755} = 12.499$ kN/m²
	Maximum Bearing Stress, P_{max}	$P_{max} = 16.875$ kN/m²	$P_{max} = 60.993$ kN/m²	$P_{max} = 12.500$ kN/m²

Version 3

HS2 Ltd - Code 1 - Accepted

ARCADIS

Project	Job ref	Calculations							
Culvert Headwall Design									
Part of structure/scheme and status	Office of issue	Telephone no	Division	Calculations by	Checked by	Date	Rev.	Design Status	Calc sheet no
Headwall - DA1C2 Drained	Bristol			AJ	DG	06/09/2021	A	VALID	3 of 3

Code ref		STR - Design Approach 1 Combination 1 - A1 "+" M1 "+" R	GEO - Design Approach 1 Combination 2 - A2 "+" M2 "+" R	BS 8004
	DESIGN BEARING RESISTANCE DRAINED CONDITION	**DRAINED BEARING RESISTANCE**	**DRAINED BEARING RESISTANCE**	**DRAINED BEARING RESISTANCE**
EC7 Annex D Sec. D.4	Bearing Capacity Factors $N_q = e^{\pi \tan\phi'} \tan^2(45+\phi'/2)$ $N_c = (N_q - 1)\cot\phi'$ $N_\gamma = 2(N_q - 1)\tan\phi'$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1\right)\tan 22.0 = 5.51$	$N_q = e^{\pi \tan 17.9} an^2 \left[45 + \frac{17.9}{2}\right] = 5.21$ $N_c = \frac{5.21 - 1}{\tan 17.9} = 13.03$ $N_\gamma = 2\left(5.21 - 1\right)\tan 17.9 = 2.72$	$N_q = e^{\pi \tan 22.0} an^2 \left[45 + \frac{22.0}{2}\right] = 7.82$ $N_c = \frac{7.82 - 1}{\tan 22.0} = 16.88$ $N_\gamma = 2\left(7.82 - 1 \tan 22.0\right) = 5.51$
	$m = \left[\frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}}\right]$	$m = \frac{2 + \frac{3.738}{4.76}}{1 + \frac{3.738}{4.76}} = 1.56$ m = 1.56, m+1 = 2.56	$m = \frac{2 + \frac{3.468}{4.76}}{1 + \frac{3.468}{4.76}} = 1.578$ m = 1.578, m+1 = 2.578	$m = \frac{2 + \frac{3.738}{4.76}}{1 + \frac{3.738}{4.76}} = 1.56$ m = 1.56, m+1 = 2.56
	Load Inclination Factors $i_q = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^m$ $i_\gamma = \left[1 - \frac{H}{V + A'c'\cot\phi'}\right]^{m+1}$ $i_c = \left[i_q - \frac{1 - i_q}{N_c \tan\phi'}\right]$	$i_q = \left[1 - \frac{0.00}{300 + \#\#\#\#\# \times 0.00}\right]^{1.560} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{300 + \#\#\#\#\# \times 0.00}\right]^{2.560} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$	$i_q = \left[1 - \frac{-182.69}{891 + \#\#\#\#\# \times 0.00}\right]^{1.578} = 1.342$ $i_\gamma = \left[1 - \frac{-182.69}{891 + \#\#\#\#\# \times 0.00}\right]^{1.578} = 1.617$ $i_c = 1.342 - \left[\frac{1 - 1.342}{13.03 \tan 17.9}\right] = 1.423$	$i_q = \left[1 - \frac{0.00}{222 + \#\#\#\#\# \times 0.00}\right]^{1.560} = 1.000$ $i_\gamma = \left[1 - \frac{0.00}{222 + \#\#\#\#\# \times 0.00}\right]^{2.560} = 1.000$ $i_c = 1.000 - \left[\frac{1 - 1.000}{16.88 \tan 22.0}\right] = 1.000$
	Base Inclination Factors $b_q = b_\gamma = (1 - \alpha.\tan\phi')^2$ $b_c = b_q - \frac{1 - b_q}{N_c \tan\phi'}$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 17.9\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{13.03 \tan 17.9} = 1.000$	$b_q = b_\gamma = \left(1 - 0 \tan 22.0\right)^2 = 1.000$ $b_c = 1.000 - \frac{1 - 1.000}{16.88 \tan 22.0} = 1.000$
	Shape Factors $s_q = 1 + \left(\frac{B'}{L'}\right)\sin\phi'$ $s_\gamma = 1 - 0.3\left(\frac{B'}{L'}\right)$ $s_c = \frac{s_q N_q - 1}{N_q - 1}$	$s_q = 1 + \left[\frac{3.738}{4.755}\right]\sin 22.0 = 1.294$ $s_\gamma = 1 - 0.3\left[\frac{3.738}{4.755}\right] = 0.764$ $s_c = \frac{1.294 \times 7.82 - 1}{7.82 - 1} = 1.337$	$s_q = 1 + \left[\frac{3.468}{4.755}\right]\sin 17.9 = 1.224$ $s_\gamma = 1 - 0.3\left[\frac{3.468}{4.755}\right] = 0.781$ $s_c = \frac{1.224 \times 5.21 - 1}{5.21 - 1} = 1.277$	$s_q = 1 + \left[\frac{3.738}{4.755}\right]\sin 22.0 = 1.294$ $s_\gamma = 1 - 0.3\left[\frac{3.738}{4.755}\right] = 0.764$ $s_c = \frac{1.294 \times 7.82 - 1}{7.82 - 1} = 1.337$
Bowles 4th Edition Table 4.5	Depth Factors $d_c = 1 + 0.4k$ $d_q = 1 + 2\tan\phi'(1 - \sin\phi')^2 k$ $d_\gamma = 1.0$ Where; $k = \frac{D}{B}$, for $\frac{D}{B} \le 1$ $k = \tan^{-1}\left(\frac{D}{B}\right)$, for $\frac{D}{B} > 1$	$\frac{D}{B} = \frac{0.50}{3.738} = 0.134 \le 1$ $d_c = 1 + 0.4 \times 0.134 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.134 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{3.738} = 0.134 \le 1$ $d_c = 1 + 0.4 \times 0.134 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 17.9 \times \left(1 - \sin 17.9\right)^2 \times 0.134 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$	$\frac{D}{B} = \frac{0.50}{3.738} = 0.134 \le 1$ $d_c = 1 + 0.4 \times 0.134 = 1.000$ *LIMIT* $d_q = 1 + 2\tan 22.0 \times \left(1 - \sin 22.0\right)^2 \times 0.134 = 1.000$ *LIMIT* $d_\gamma = 1.00 = 1.000$
	Ground Inclination Factors $g_q = g_\gamma = (1 - \tan\beta)^2$ $g_c = 1 - (\beta/147°)$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$	$g_\gamma = g_q = \left(1 - \tan 0.0\right)^2 = 1.000$ $g_c = 1 - \frac{0}{147} = 1.000$
	Effective overburden, q'	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²	$q' = 20.0 \times 0 + 12.0 \times 0.5 = 6.000$ kN/m²
	Vertical Bearing Resistance, R/A' $R/A' = c' N_c b_c s_c i_c d_c g_c + q' N_q b_q s_q i_q d_q g_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma$	R/A' = 0.0 ×16.88 ×1.000 ×1.337 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.294 ×1.000 ×1.000 ×1.000 + ½ × 8.0 ×3.738 ×5.51 ×1.000 ×0.764 ×1.000 ×1.000 ×1.000 R/A' = 123.657 kN/m²	R/A' = 0.0 ×13.03 ×1.000 ×1.277 ×1.423 ×1.000 ×1.000 + 6.0 ×5.21 ×1.000 ×1.224 ×1.342 ×1.000 ×1.000 + ½ × 8.0 ×3.468 ×2.72 ×1.000 ×0.781 ×1.617 ×1.000 ×1.000 R/A' = 98.999 kN/m²	R/A' = 0.0 ×16.88 ×1.000 ×1.337 ×1.000 ×1.000 ×1.000 + 6.0 ×7.82 ×1.000 ×1.294 ×1.000 ×1.000 ×1.000 + ½ × 8.0 ×3.738 ×5.51 ×1.000 ×0.764 ×1.000 ×1.000 ×1.000 R/A' = 123.657 kN/m² R/A'$_{net}$ = 123.7 - 6.000 = ##### kN/m²
EC7 Annex A Table A.5	Design Vertical Bearing Resistance, $R_d/A' = \frac{R/A'}{\gamma_{R;v}}$	$R_d/A' = \frac{123.657}{1.00} = \mathbf{123.657}$ kN/m²	$R_d/A' = \frac{98.999}{1.00} = \mathbf{98.999}$ kN/m²	$R_d/A' = \frac{117.657}{1.00} = \mathbf{117.657}$ kN/m²
		$P_{ave} = 16.9$ kN/m² ≤ R_d/A' Bearing Resistance OK	$P_{ave} = 54.0$ kN/m² ≤ R_d/A' Bearing Resistance OK	$P_{max.net} = 12.5 - 6.000 = 6.500$ kN/m² ≤ R_d/A'_{net} Bearing Resistance OK
	Utilisation Factor, Δ (%) $\Delta = \frac{P_{ave}}{R_d/A'}$	$\Delta = \frac{16.873}{123.7} = 14\%$	$\Delta = \frac{54.027}{99.0} = 55\%$	$\Delta = \frac{6.500}{117.7} = 6\%$

Version 3

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Appendix I

Summary of Surcharge Loadings for Mainline HS2 Embankment

Number of Pages: 01

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Analyses Case		Minimum Surcharge on High-Speed Rail (Slab track) Embankment (kPa)	Comments
Stability Calculations	Short term case, during construction	Variable surcharge of 20kPa applied across the crest of embankments to represent potential construction surcharge loading.	This is representative of the temporary stability condition and applies either drained (effective stress) or undrained (total stress) geotechnical parameters as appropriate to loading condition (BS 6031, Ref.[62]). Variable surcharge may be taken as zero where beneficial to stability
	Long term design life case, operation loading conditions	Permanent surcharge of 15kPa for self-weight of slab track applied at finished embankment level over 3m track widths. Variable surcharge of 10kPa applied from the edge of the slab track to the crest of the embankment.	This is representative of the permanent stability condition and applies drained (effective stress) geotechnical parameters. Variable surcharge may be taken as zero where beneficial to stability
	Long term design life case, transient loading conditions	Permanent surcharge of 15kPa for self-weight of slab track and variable surcharge of 57kPa for LM71 train loading applied at finished embankment level over 3m track widths. Variable surcharge loading of 10kPa applied from the edge of the slab track to the crest of the embankment.	Train loads to LM71 are in accordance with BS EN 1991-2: Traffic loads on bridges (Ref.[70]) including an alpha factor of 1.1. For transient loading, undrained (total stress) geotechnical parameters may be applied to natural soils where appropriate as BS 6031, Ref [62]. Variable surcharge may be taken as zero where beneficial to stability
Settlement Calculations	Short term case, settlement during construction / pre-handover	Design preload surcharge applied as permanent load as appropriate across the crest of embankments as required to achieve settlement criteria.	Immediate elastic compression of the embankment fill also added, to calculate the total immediate settlement
	Long term design life case, permanent loading conditions	Permanent surcharge of 15kPa for self-weight of slab track applied at finished embankment level over 3m track widths. Additional component of variable surcharge may be applied as a permanent load as assessed for design to represent potential equivalent applied loading.	This is representative of the long-term consolidation settlement and applies drained geotechnical parameters

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Appendix J

First Pass Rapid Impact Assessment for Bridleway SBH/19 Footbridge

Number of Pages: 19

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Part A – Current Design and Construction Information

Asset Name	Bridleway SBH/19 Overbridge
Asset ID	UAID HS2-000001293
Design Status	In Progress – Pre-IDR
Construction Status	Not Started
Uncertainty in Ground Conditions Classification	3
Agreed Mitigation for Uncertainty in Ground Conditions	Proceed with Design - Review GI Data pre-IDR. Proceed with developing an **efficient design** using characteristic values/ground conditions as a cautious estimate of the value governing a limit state. Review GI Data as well as detailed design assumptions before L3

Part B – New GI and Field Trial Data

Current GI Database ID	Data Drop 01 (HS2 Main GI) Data Drop 19 Jan 2022 (Stage 2 GI)
New GI Database ID	Data Drop 24 but not assured until Data Drop 30 / Feb 2023 (Refer to Reference List)

GI Location ID	GI Location type	Final Depth (m)	Ground Level (m AOD)	Notes
ML060-RC432	RC	25.00	80.95	All exploratory holes in this list are designed to provide confirmation of the ground model and groundwater mode used during detailed design at the location of the bridge abutments. Furthermore, these exploratory holes are designed to validate design assumptions for the substructure design.
ML060-RC433	RC	25.45	81.58	

Field Trial ID/Type	Location	Notes
Pressuremeter tests / HPD3	ML060-RC432 and ML060-RC433	Tests carried out on the Purbeck Group Formation (medium strong grey LIMESTONE and stiff CLAY) and the Portland Stone Formation (weak to medium strong grey LIMESTONE and cemented dark grey silty SAND)

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Insitu/Lab Testing	Count (Min Average Max)	Notes
Standard penetration test N (SPT)	1 (9, 9, 9)	*Depth 1.2mbgl*
In Situ Vane Shear (kPa)	6 (81, 117, 171)	*Depth 0.1 – 1mbgl*

Groundwater Monitoring Location	Type of installation	Duration of Additional Monitoring	Notes
ML060-RC433	SP	5 months (Oct 22 to Feb 23)	5 (0.55, 1.34, 1.80) as *Count (Min Average Max)*

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Part C – First Pass SWOT Impact Assessment

Strengths	Weakness
• The additional Stage II GI includes 2No. 25.0m deep boreholes. This GI has the potential to decrease the level of uncertainty in ground conditions. • The entirety of scoped GI has now been completed on-site. • The boreholes of the additional Stage II GI validate that the Purbeck Group Formation (medium strong grey LIMESTONE and MUDSTONE) and the Portland Stone Formation (weak to medium strong grey LIMESTONE/SILTSTONE and cemented dark grey silty SAND) are present at the bridge location. These two formations were assumed to be a rigid boundary for settlement calculation during detailed design stage. The boreholes of the additional Stage II GI validate this assumption. • Groundwater is now better due to the monitoring results validates the groundwater taken during detailed design.	• The new GI does not agree with the previous GI data, in particular with the ground model for the Gault Formation: o At detailed design, the depth of the Gault Formation (described as firm to very stiff CLAY) was 5.5m at Abutment A02 (East Abutment). However, the boreholes of the additional Stage II GI indicates that the depth of the Gault Formation has increased to approximate 10.0m at Abutment A02 (East Abutment), almost double to that originally assumed during detailed design based on the available GI. o At detailed design stage, the depth of the Gault Formation (described as firm to very stiff CLAY) was 16.35m at Abutment A01 (West Abutment). However, the additional Stage II GI indicates that the depth of the Gault Formation has reduced to 2.0m at Abutment A01 (West Abutment). This is much less than original assumed during detailed design based on the available GI. • At detailed design, the Whitchurch Sand Formation was descried as very stiff SILT / extremely weak SANDSTONE (~1.48m - 1.5m deep). This formation was conservatively treated as very stiff SILT for the settlement calculations. Results of the additional Stage II GI indicates that the Whitchurch Sand Formation is described as firm sandy gravelly CLAY (1.62m deep found in one borehole of the additional GI only).

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Opportunities	Threats

• The results of the GI has the potential to reduce the estimated total settlements of the bridge:
  ○ The reduction in the depth of the Gault Formation from 16.35m to 2.0m at Abutment A01 gives the opportunity to reduce the total estimated settlements at Abutment A01. Since total settlements of the bridge were the largest at Abutment A01, a reduction in the depth of the Gault Formation will reduce the total settlements of the bridge, increasing the vertical bridge clearance between the track and the bridge deck after considering post-construction settlements.
	• The variation in thickness of the Gault Formation could increase the estimated differential settlements of the bridge and exceed the maximum differential settlement used in detailed design and stated in the AiP:
	  ○ An increase in the depth of the Gault Formation from 5.5m to 10.0m at Abutment A02 will increase the total settlements at Abutment A02. An increase in the depth of the Gault Formation at Abutment A02 combined with a reduction in the depth of the Gault Formation at Abutment A01 from 16.35m to 2.0m has the potential to increase the differential settlements between abutments. Therefore, the maximum differential settlements stated in the AiP and used in the detailed design of the bridge structure could be exceeded.
	  ○ Furthermore, it should be noted that at Abutment A02, the landscaping bund is the highest (8.6m high or equivalent UDL of 175kPa at existing ground level) where the landscaping bund Abutment A01 is 6.2m high or equivalent UDL of 125kPa at existing ground level. The difference in loading at each abutment has the potential to increase further the differential settlement between supports.

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Attachment 1 - GI Data Reference List

In situ hole ID	Borehole Type	Ground Level (mAOD)	Final Depth (m)	Distance From Asset (m)	Easting (Snake Grid)	Northing (Snake Grid)	Contract Area
ML060-CP002	CP	82.62	10.0	110	244041.17	318193.32	ACD
ML060-CR004	RC	81.63	20.0	170	244286.23	318117.26	ACD
ML060-CR011	RC	81.64	20.0	70	244075.81	318248.80	ACD
ML060-WS010	WS	83.19	4.0	110	244082.90	318141.56	ACD
ML060-TP427	TP	80.40	2.0	135	244178.80	318279.95	Stage II GI
ML060-CT425	CT	81.44	7.2	30	244164.28	318212.50	Stage II GI
ML060-CR424	RC	81.53	25	65	244187.03	318180.80	Stage II GI
ML060-RC432	**RC**	**80.95**	**25.00**	**0**	**211427.37**	**480974.40**	**Stage II GI**
ML060-RC433	**RC**	**81.58**	**25.45**	**0**	**211456.74**	**481004.11**	**Stage II GI**
	CP – Cable Percussion; **RC – Rotary Coring**; WS – Window sample, TP- Trial Pit, CT – Cone Penetration						

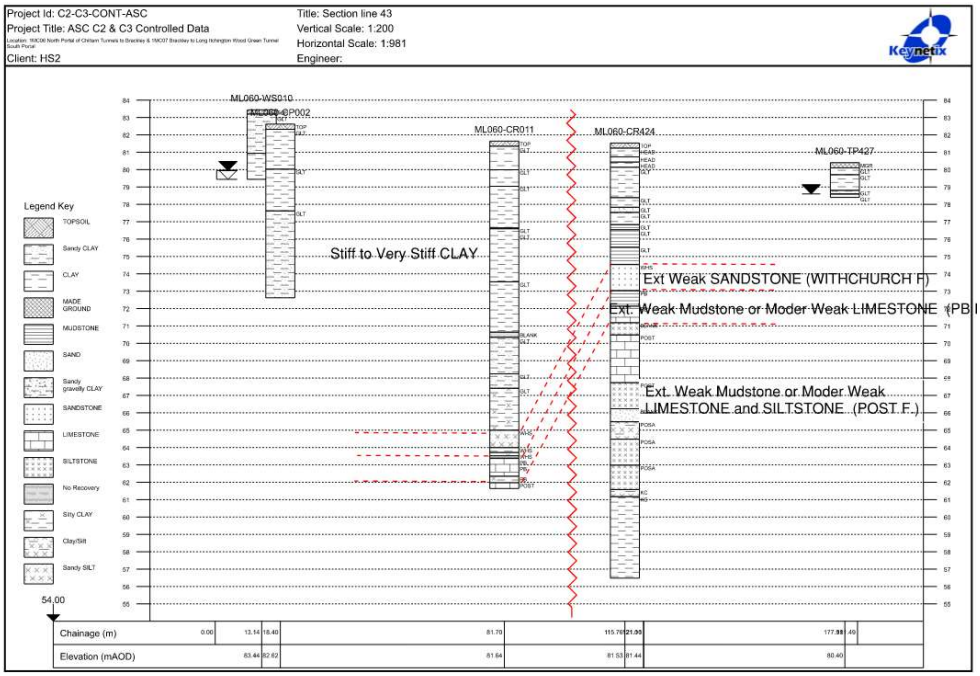
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Attachment 2 - GI Location Plan

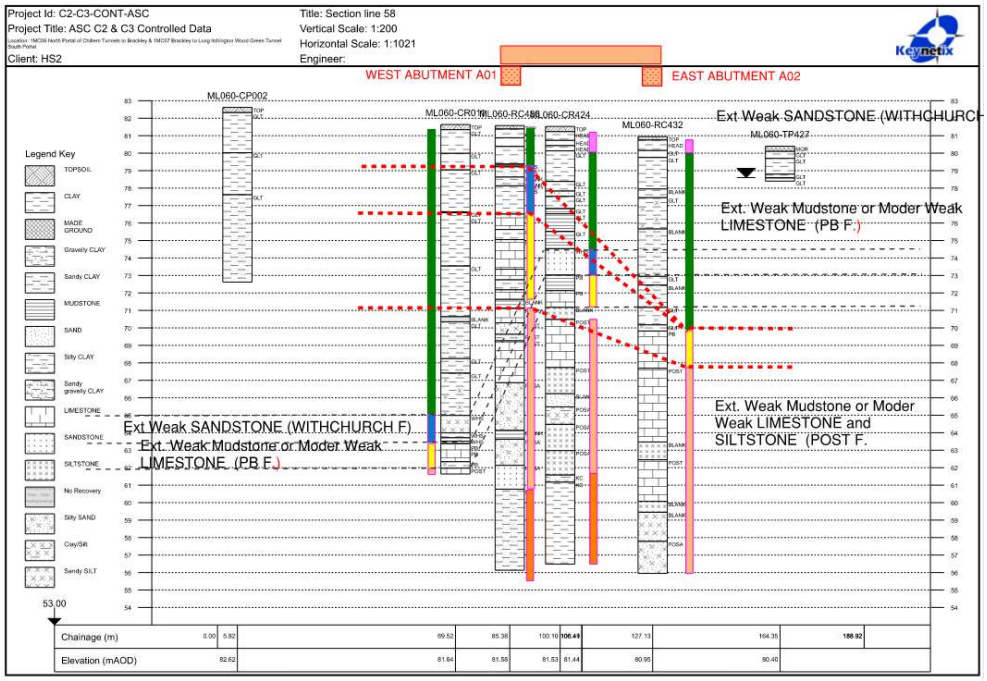


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Attachment 3 – Ground Model – Geological Cross Section (detailed design without new GI data)

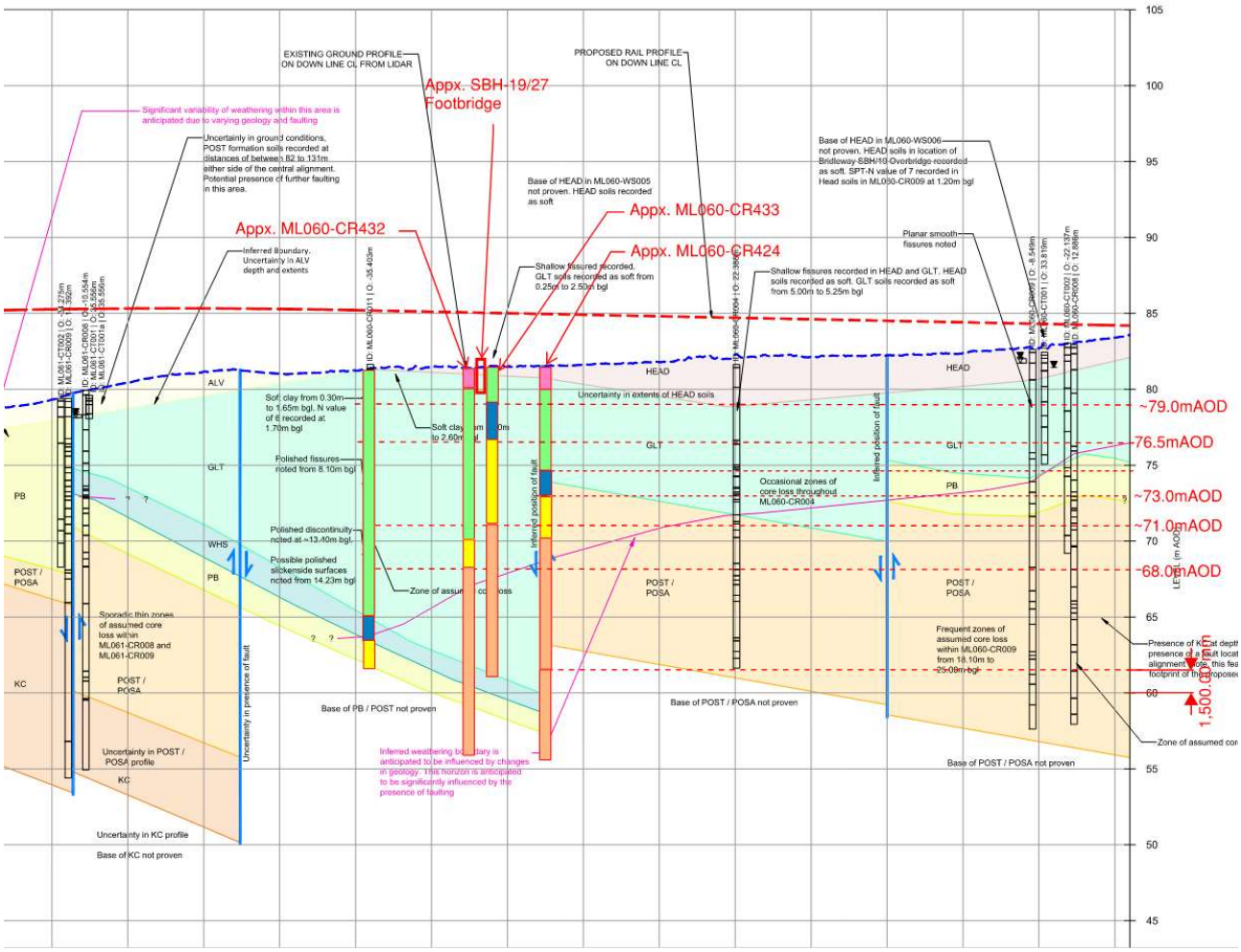


Attachment 4 – Ground Model – Geological Cross Section (with new GI data)



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Attachment 4 – Ground Model (longitudinal Geological Section)



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Part D – Specific comments to the content of this GDR

Section	Heading	Comments
1	Introduction	Not applicable.
1.1	Background	Not applicable.
1.2	Scope and Objective of the Report	The original scope of the previous revision of the GDR is complemented with this Appendix corresponding to the Validation of the Design as anticipated by the GEDBS-SMUGC.
1.3	Report Structure	Not applicable.
1.4	Limitations	Not applicable.
1.5	Description of the Works	Not applicable.
2	Earthworks (Mainline)	Not applicable.
3	Earthworks (Non-mainline)	Not applicable.
4	Structures	Not applicable.
4.1 to 4.5	Structures Sub-headings	Not applicable.
4.6	Bridleway SBH19 Overbridge	Not applicable.
4.6.1 to 4.6.3	Scope, Asset Description and Geotechnical Category	Not applicable.

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<table>
<tr><th>Section</th><th>Heading</th><th>Comments</th></tr>
<tr><td>4.7.4</td><td>Uncertainty in Ground Conditions Classification</td><td>The Level of Uncertainty in Ground Conditions – Classification has been updated.
Pre validation class: 3
Post validation class: 1
Sufficient GI data at the bridge location is now available and has been used together with all other boreholes within 150m from the site to produce the Rapid Impact Assessment. Due to the available supplementary GI:
1.50m excavation and replace is still required to remove soft deposits at shallow surface.
6-month hold period is no longer required.</td></tr>
<tr><td>4.6.5</td><td>Construction sequence</td><td>Not applicable.</td></tr>
<tr><td>4.6.6</td><td>Ground Investigation</td><td>All Stage II GI in this section as listed as final and available in Data Drop 30 Feb 2023.
GI Data Reference List<table>
<tr><th>In situ hole ID</th><th>Borehole Type</th><th>Ground Level (mAOD)</th><th>Final Depth (m)</th><th>Distance From Asset (m)</th><th>Contract Area</th></tr>
<tr><td>ML060-CP002</td><td>CP</td><td>82.62</td><td>10.0</td><td>110</td><td>ACD</td></tr>
<tr><td>ML060-CR004</td><td>RC</td><td>81.63</td><td>20.0</td><td>170</td><td>ACD</td></tr>
<tr><td>ML060- CR011</td><td>RC</td><td>81.64</td><td>20.0</td><td>70</td><td>ACD</td></tr>
<tr><td>ML060- WS010</td><td>WS</td><td>83.19</td><td>4.0</td><td>110</td><td>ACD</td></tr>
<tr><td>ML060-TP427</td><td>TP</td><td>80.40</td><td>2.0</td><td>135</td><td>Stage II GI</td></tr>
</table></td></tr>
</table>

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<table>
<tr><th>Section</th><th>Heading</th><th>Comments</th></tr>
<tr><td></td><td></td><td>
<table>
<tr><td>ML060-CT425</td><td>CT</td><td>81.44</td><td>7.2</td><td>30</td><td>Stage II GI</td></tr>
<tr><td>ML060-CR424</td><td>RC</td><td>81.53</td><td>25</td><td>65</td><td>Stage II GI</td></tr>
<tr><td>ML060-RC432</td><td>RC</td><td>80.95</td><td>25.00</td><td>0</td><td>Stage II GI</td></tr>
<tr><td>ML060-RC433</td><td>RC</td><td>81.58</td><td>25.45</td><td>0</td><td>Stage II GI</td></tr>
</table>
</td></tr>
<tr><td>4.6.7</td><td>Ground Conditions</td><td>The ground conditions have changed based on the new Stage II GI listed in tis RIA (Data Drop 30 Feb 2023).
<table>
<tr><th>Geological Unit</th><th>Top of Stratum Range Level (mAOD)</th><th>Thickness Range (m)</th></tr>
<tr><td>Topsoil</td><td>83.44 – 80.95</td><td>0.20 – 0.30</td></tr>
<tr><td>Soft to Firm CLAY (HEAD)</td><td>81.43 – 80.75</td><td>0.60 – 1.30</td></tr>
<tr><td>Firm to very Stiff CLAY (Gault Formation - GLT)</td><td>83.38 – 80.13</td><td>2.00 – 16.35</td></tr>
<tr><td>Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS) (1) (2)</td><td>79.38 – 63.99</td><td>1.28 – 2.87</td></tr>
<tr><td>(Purbeck Group - PB)</td><td>76.51 – 63.52</td><td>1.55 – 4.88</td></tr>
<tr><td>(Portland Stone Formation - POST)</td><td>71.73 – 61.97</td><td>4.25 – 11.74</td></tr>
</table>
</td></tr>
<tr><td>4.6.8</td><td>Groundwater Conditions</td><td>Not applicable.</td></tr>
</table>

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<table>
<tr><th>Section</th><th>Heading</th><th>Comments</th></tr>
<tr><td>4.6.9.1</td><td>Design Geotechnical Model</td><td>The additional Stage II GI indicated similar ground conditions than originally anticipated with a reduction of the HEAD at the bridge location. However, the new Stage II GI will not have a significant impact on the Design Geotechnical Model and the design.
Geotechnical Design Profile for East Abutment of Bridleway SBH/19 Overbridge
<table>
<tr><th>Geological Unit</th><th>Design Top of Stratum (mAOD)</th><th>Design Thickness (m)</th><th>Notes</th></tr>
<tr><td>Class 1A4/Class 1B4/Class 2A and 6N/6P</td><td colspan="2">Variable depending on abutment and founding level of earthworks blister.</td><td>Non-HSR General Fill for the "Dig & Replace" mitigation measure and the approach embankment. Selected granular fill (Class 6N/6P) for the earthworks blister approach embankment.</td></tr>
<tr><td>Topsoil</td><td>81.0</td><td>0.2</td><td>Topsoil will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.</td></tr>
<tr><td>Soft to Firm CLAY (HEAD)</td><td>80.8</td><td>1.2</td><td>HEAD will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth (removed under bridge abutments only).</td></tr>
<tr><td>Firm to very Stiff CLAY (Gault Formation - GLT)</td><td>80.1</td><td>10.3</td><td></td></tr>
</table></td></tr>
</table>

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Section	Heading	Comments

Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS)	69.8	>20	Portland Stone (POST) and Portland Sand (POSA) Formations underlie the Gault Formation. POST and POSA Formations assumed to be a rigid stratum (E'=75MPa)

Geotechnical Design Profile for West Abutment of Bridleway SBH/19 Overbridge

Geological Unit	Design Top of Stratum (mAOD)	Design Thickness (m)	Notes
Class 1A4/Class 1B4//Class 2A and 6N/6P	Variable depending on abutment and founding level of earthworks blister.		Non-HSR General Fill for the "Dig & Replace" mitigation measure and the approach embankment. Selective granular fill (6N/6P) for the earthworks blister approach embankment.
Topsoil	81.6	0.2	Topsoil will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.
Firm to very Stiff CLAY (Gault Formation – GLT)	81.4	2.0	Top 1.2m will be removed as part of the "Dig & Replace" mitigation measure to 1.5m depth.
Firm sandy to gravelly CLAY (The Purbeck (PB)	81.4	2.8	

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Section	Heading	Comments			
		Very Stiff SILT / Extremely Weak SANDSTONE (Whitchurch Sand Formation - WHS)	79.4	>20	Portland Stone (POST) and Portland Sand (POSA) Formations underlie the WSH Formation. PB, POST and POSA Formations assumed to be a rigid stratum (E'=75MPa)
4.6.9.2	Design Geotechnical Parameters	Not applicable. New GI was carried out to confirm the ground models. No testing was scheduled.			
4.6.9.3	Design Groundwater model	Not applicable			
4.6.10	Design Approach and Partial Factors for the Abutment Design	Not applicable			
4.6.11	Design of Bridleway SBH/19 Overbridge	Not applicable			
4.6.11.1 to 4.6.11.3	Seismic Design; Loading Acting on Abutment and Design Check Sections	Not applicable			
4.6.11.4	Settlement analysis	The ground model for each abutment has been adjusted as per the ground models in Section 4.6.9.1 of this RIA. Post construction settlements for Bridleway SBH19 Overbridge are presented in the table below:			

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Section	Heading	Comments

Case	Location (1, 2)	Landscape bund Load	Immediate Settlement	Consolidation Settlement	Total Settlement	Post-Construction Settlement	Allowable Post–Construction Settlement (3)
		kPa	mm	mm	mm	mm	mm
East Abutmt.A02 (10.3m thick Gault Clay West Abutmt.A01 (2.0m thick Gault Clay)	East Abutment	175	54	81	135	81	85
	East Embank.		12	18	30	18	20
	West Abutment	125	32	48	80	48	85
	West Embank.		6	9	15	9	20

(1) Settlements at the abutment were estimated at the vertical of the centre of the abutment base.

(2) Settlements at the embankment were estimated at the toe of the embankment.

(3) Allowable limit of 85mm set for the bridge abutments by the bridge designer

The maximum calculated post-construction settlement of 81mm at the East abutment.

The maximum calculated differential post-construction settlement of 33mm between the East and West abutments is within the allowable limit of 35mm (AIP report number 1MC06-CEK-BR-REP-CS04_CL18-000048 Rev C04).

The maximum calculated post-construction settlement of 81mm at the East Abutment. Results in settlement table indicates that post construction settlements are within the allowable limits of 85mm for the footbridge. Therefore, no additional remediation measures are required this asset.

A hold period of 6 months between the installation of the abutments and wingwalls and the installation of the bridge deck will not need to be introduced in order to reduce post construction settlement.

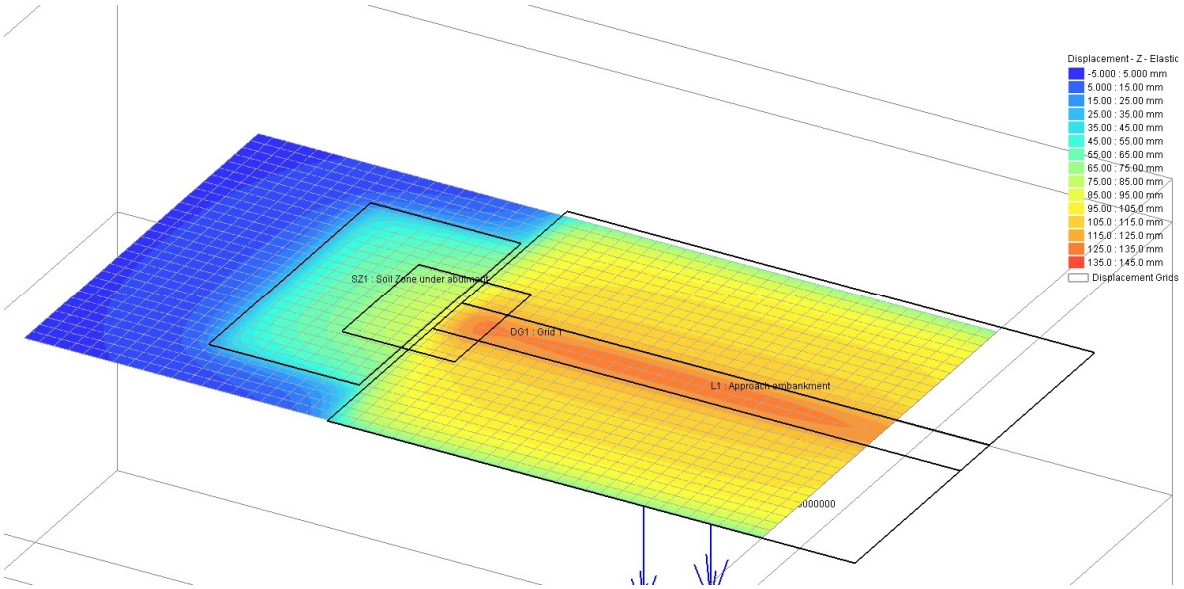
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Section	Heading	Comments
4.7 to 4.9	Ancillary Culvert 060-AC1; AC3 and AC4	Not applicable
5.0	Special Geotechnical Measures	Not applicable
6.0	Drainage	Not applicable
7.0	Assessment of Potential Contamination	Not applicable
8.0	Geotechnical Risk Register	Not applicable
9.0	Specification Appendices	Not applicable
10.0	Inspection, Testing and Verification Plan	Not applicable
11.0	Specification Appendices	Not applicable
12.0	References	Not applicable
Appendix A to Appendix C		Not applicable
Appendix D		Risks associated to Bridleway SBH-19 Overbridge have been updated in line with the results of the additional GI and this RIA.
Appendix E to Appendix I		Not applicable

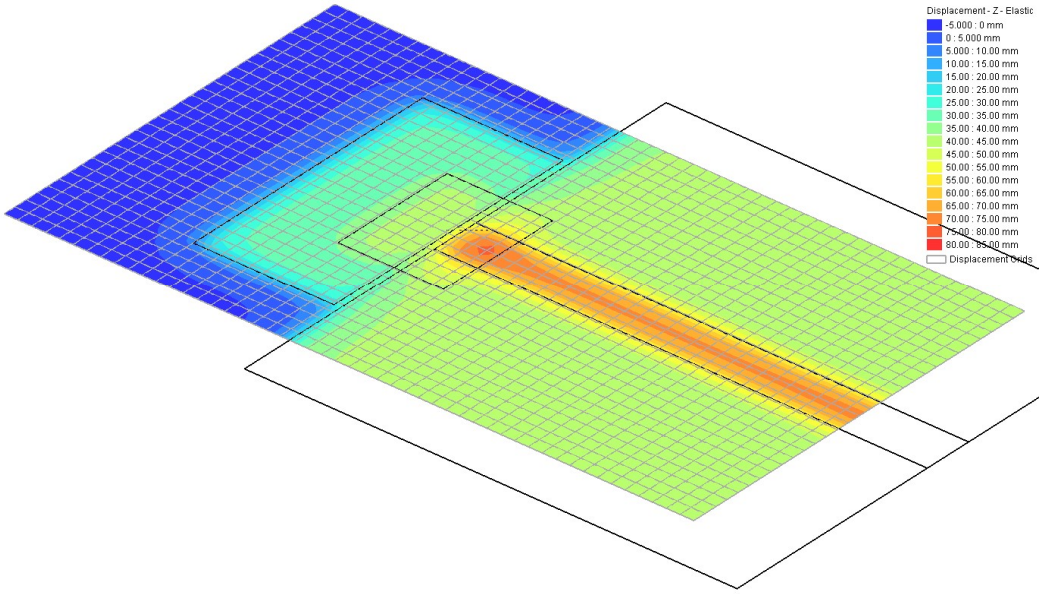
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PDisp Run

East Abutment - Total Settlement



West Abutment - Total Settlement



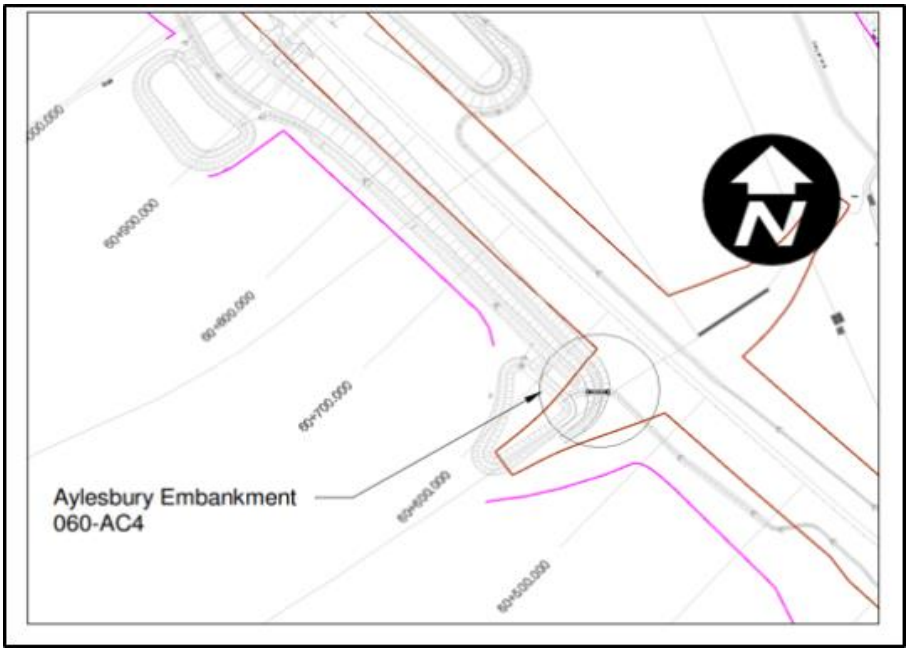
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Appendix K

Summary of Design – Aylesbury Embankment 060-AC4 Ancillary Culvert

Number of Pages: 06

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Ref	Introduction to Geotechnical Design	Output
PowerBI & GIS Maps	**Introduction** The purpose of this spreadsheet is to undertake the geotechnical design for the Pipe Culvert 060-AC4 (asset ID: HS2-00002N7VP) as part of IDR 207.2. The asset is located offline at the approximate HS2 chainage of 60+510 to 60+580 within the Aylesbury Embankment Level 1 Asset Group. The asset sits beneath the highway asset Oat Close Bridleway (SBH/19) Realignment (asset ID: HS2-000020RKL, part of IDR 207.2) and allows land drainage to flow beneath the Bridleway (SBH/19) Realignment. The following geotechnical assessments are presented in this calculation pack for the asset: * Assessment of ground conditions and geotechnical risks; * Concrete Durability; * Settlement of culvert and headwalls along with Damage Assessment i.e., evaluation of Joint Rotations and Opening, Spring Stiffness values where necessary; * Bearing Capacity, Overturning and Sliding check of the headwalls.	Tabs 5 to 8
Drawings provided by Ancillary team	**Input Data** The culvert geometry and details are based on the following drawings and should be read in conjunction with this calculation pack: 1MC06-CEK-CV-DSE-CS04_CL18-000072 1MC06-CEK-CV-DGA-CS04_CL18-000246 1MC06-CEK-CV-DGA-CS04_CL18-000261 1MC06-CEK-CV-DGA-CS04_CL18-000262	https://arcadiso365.sharepoint.com/:f:/r/teams/HS2ASC-AA/AA_Documents/04%20-
	Location Plan  	
GDR	**Submission** This calculation pack includes all details for the geotechnical design inputs applicable to the culvert, however, this is not a formally submitted deliverable. Instead a summary of this calculation pack is included within the Aylesbury Embankment GDR link provided below. 1MC06-CEK-GT-REP-CS04_CL18-000037	-

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Ref	Assumptions	Output
1	Ground conditions considered as part of the design will be validated on site prior to construction of the culvert and headwalls. The minimum requirements for site testing will be defined on the culvert drawings and in the IDR Specification.	
2	A reinforced concrete density of 25kN/m^3 has been assumed for the culvert and headwalls where concrete self weight is unfavourable but an inferior concrete density of 24kN/m^3 should be taken where the self weight is favourable eg sliding.	
3	Any changes to the culvert, headwall structure's geometry and/or loading will result in a further geotechnical verification check being required. Culvert and headwall dimensions and levels are provided by the Ancillary/Watercourse/Drainage teams and are assumed to be final.	
4	Due to limited GI data available at the location of the 060-AC4 Culvert, the ground model and parameters in Section 2.4.6 and Appendix B of the GDR (C02), related to Mainline Embankment have been considered for the design of Culvert and Headwalls. It is assumed that the ground model and geotechnical parameters presented in the Aylesbury Embankment GDR (C02) have been through a full PCRA.	
5	A groundwater level of 0.6m bgl, as per ML060-CR009 borehole which is the nearest to 060-AC4 culvert can been used in the design of the culvert. Conservatively, the groundwater level along the length of the culvert pipe is assumed to be at the ground level (83.168 mAOD). However, the groundwater level for the Headwall has been assumed to be at top of the Headwall baseslab level (81.813 m AOD).	
6	For durability assessment and concrete classifications in Head deposits, a mobile groundwater and brownfield site (accounting for any existing/disused roads) were conservatively assumed.	
7	The partial factors for actions for civil structures (buildings) were used for the Headwall stability and foundation calculations as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013. Reference is made to Set B and Set C partial factors (Tables NA.A2.4(B) and NA.A2.4(C)) of NA to BS EN 1990:2002+A1:2005.	
8	As the Culvert top level is above the existing ground level, the net pressure of self weight of culvert and the soil excavated for placing the pipe culvert below EGL has been considered in settlement analysis.	
9	The proposed foundation for the Headwall is cast-in-situ, so the friction at the interface between the foundation and the Class 6N6 material below , δ, can be considered ($\delta = \varphi_{cv}$).	
10	Where founding strata, i.e., base of Class 6N6, has an undrained shear strength of <40kPa, material shall be excavated in depth increments of at least 200mm intervals to comply with the requirement of undrained shear strength of >40kPa. The excavated area is to be replaced as soon as practicable by Class 6N6 materials to the base of the proposed foundation level.	
11	Pipe culvert is assumed to be stable while it is burried in the embankment, therefore no stability checks performed for the culvert segments.	
12	Backfill properties of class 6N6 material has been assumed to have peak friction angle of 40deg and constant volume (critical state) friction angle of 34deg with dry and saturated unit weight of 20 and 22kN/m^3 respectively.	

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Ref	Ground Model and Geotechnical Parameters	Output

Introduction

The ground conditions have be been assesed considering the following sources:
Aylesbury Embankment GIR Projectwise link provided below
1MC06-CEK-GT-REP-CS04_CL18-000011
Aylesbury Embankment GDR Projectwise link provided below
1MC06-CEK-GT-REP-CS04_CL18-000037
* Available GI from OpenGround
* GIS Viewer

Overview

Section 2.4.6 of GDR C02

There is no ground model and parameters for the non-mainline Earthworks which are adjacent to the culvert in the GDR (C02) as those are also part of IDR 207.2, hence, mainline Earthworks Ground model and Parameters in Section 2.4.6 and Appendix B of the GDR (C02) have been considered for the 060-AC4 Culvert.

Table 2-3 of Section 2.4.6 of GDR C02

Table 2-3 Summary of Ground Conditions at Different Chainages.

Ground Model (GM)	Strata	Depth Range
Ground Model 1 (Ch. 60+490 – Ch. 61+000)	Head	0.00 – 3.80
	Gault Clay Formation	3.80 – 9.05
	Portland Stone/Sand Formation	9.05 – 20.00+
Ground Model 2 (Ch. 61+000 – Ch. 61+200)	Alluvium	0.00 – 1.60
	Gault Clay Formation	1.60 – 5.30
	Whitchurch Formation	5.30 – 6.30

Table B-1 of Appendix B of GDR C02

Table B-1 Characteristic Geotechnical Parameters of In-situ Material

Geological Strata	γ_b (Mg/m³)	SPT (N)[1]	c_u [1][5] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	v_s	ν'	E_u (MPa)	E' (MPa)
Alluvium	1.8	0-22	0 → 2.0 = 20	0-2	21	23	0.5	0.2	6.0	4.8
Head	1.8	7 + 0.3z	0 → 2.0 = 30 2.0+= 40	0-2	21	23	0.5	0.2	0 → 2.0 = 9.0 2.0+= 12.0	0 → 2.0 = 7.2 2.0+= 9.6
Gault Clay Formation (GLT)	1.9	6 + 8.6z	50 + 10z → 150	8	21	23	0.5	0.2	15 + 3z → 45	12 + 2.4z → 36
Soft Portland Stone/Sand Formation (POST/POSA)	2.0	0 – 2 m 5 + 11z	20	0-2	23	25	0.5	0.2	6.0	4.8
Portland Stone/Sand Formation (POST/POSA)	2.0	2.0 m + 9 + 13.5z	50 + 10z → 130	8	26	28	0.5	0.2	15 + 3z → 39	12 + 2.4z → 31

Notes and Justifications:

1. z is the depth (m) measured from top of the individual stratum.

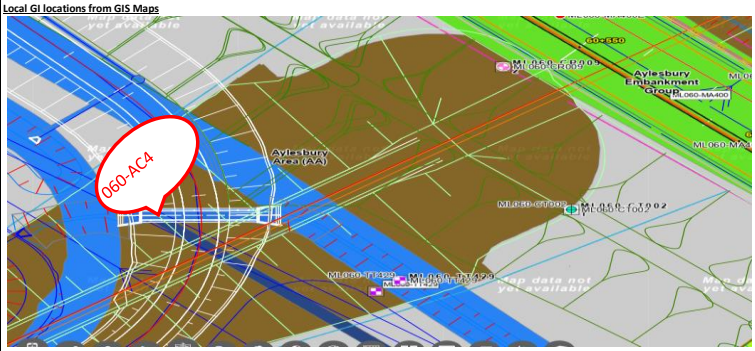
Available GI

The data set corresponds to ASC Detailed Design Data Drop 40 (Oct 2023) from Open Ground and no further GI is planned around this asset to inform detail design.

The figure and table below summarise the exploratory holes which have been considered when assessing the ground conditions local to the asset.

Local GI locations from GIS Maps

GIS Maps



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Ref	Ground Model and Geotechnical Parameters	Output

Geology

The BGS maps accessed through the GIS mapping does not indicate the presence of superficial geologies. However, the presence of GAULT FORMATION is shown as the bedrock geology.



OpenGround Data

Localised data within approximately 50m radius of the Culvert were checked for this location to confirm the ground model from the GDR (C02).
The following exploratory holes were checked, The borelogs of the below borlogs can be found below:
ML060-TT429
ML060-CT002
ML060-CR009
ML060-WS006

Borelog

Based on the borelog data the bottom level of the Head deposits is considered conservatively at 80.62 mAOD. Hence, the maximum thickness of the head deposit is 2.55m bgl. Hence, the thickness of Head deposits below culvert foundation level is 0.88m bFL.

Design Ground Model for Culvert foundation design

S.No	Geological Unit	Top of the Stratum (m AOD)	Bottom of the Stratum (m AOD)	Thickness, m	Culvert Foundation Level, m AOD	Thickness, m, bfL
1	Head Deposit	83.17	80.62	2.55	81.503	0.883
2	Gault Formation	80.62	70.37	10.25		
3	Very Strong POST/ POSA-CZ/LMST	70.37	59.22	11.15		

Invert level of culvert	81.968	m AOD

Head deposits were considered as the geology within the zone of influence of foundation design for bearing/sliding/punching checks. Whereas for settlement checks both Head deposits & Gault Formation have been used in the analysis.

Parameters adjustment for Head Deposits

The parameters from the Table B-1 of Appendix B of GDR (C02) for the Head Deposits suggest a upper limit of 30 kPa for Cu value for a depth upto 2m bgl. Whereas the earthwork specifications recommends that soil with Cu <40kPa, material shall be excavated in depth increments of at least 200mm intervals to comply with the requirement of undrained shear strength of >40kPa.
Hence for Head deposits a Cu value of 40 kPa is assumed for the foundation design.

Head deposits Cu of 40kPa was used for this foundation design

As Upper Bound Cu, Eu and E' values for Gault Formation mentioned in Table B-1 of Appendix B of GDR (C02) is higher than that of Portland Stone/Sand Formation (POST/POSA) which is at deeper depths than Gault formation as per our ground model. Hence to be conservative the Upper bound Cu, Eu and E' for Gault Formation is also restricted to 130 kPa, 39 MPa and 31 MPa respectively.

Parameters considered for the Design of Culvert and Headwall

Geological Unit	γ_b (kN/m^3)	c_u (kPa)	c' (kPa)	φ'_{cv} (°)	φ'_{peak} (°)	E_u (MPa)	E' (MPa)	v'	vu
Head Deposits	18	40	0	21	23	9	7.2	0.2	0.5
Gault Formation	19	50+10z restrict to 130	8	21	23	15+3z restrict to 39	12+2.4z restrict to 31	0.2	0.5
Portland Stone/ Sand Formation POST/POSA	20	50+10z restrict to 130	8	26	28	15+3z restrict to 39	12+2.4z restrict to 31	0.2	0.5
Class 6N6	20	-	-	34	40	-	60	0.2	-

Note: z is the depth (m) measured from top of individual stratum

Class 6N6 (granular fill) parameters

The Following parameters were adopted for the Class 6N6 in the design:

Table 6-1 of Contract-specific tables to 600 Series Appendices (1MC06-CEK-GT-REP-C002-000154)

6	N	6	Selected well graded granular material	High Strength Fill to non-HSR structures	Natural gravel, natural sand, crushed gravel, crushed rock, crushed concrete, slag, well- burnt colliery spoil or any combination thereof. None of these constituents shall include any argillaceous rock. Recycled aggregate except recycled asphalt. Where material is imported onto site which is not 'as dug' it shall be aggregate conforming to BS EN 13242 from one or more of the following source codes, see Notes 8, 9 and 10: P (natural aggregates – except shale, siltstone or slate, see Note 7); A2 (crushed concrete); A3 (crushed bricks, masonry); G1 (red coal shale)	(i) grading	BS 1377: Part 2 (on-site) BS EN 933-2 (Imported onto site)	Tab 6/2 Tab 6/5	Tab 6/2 Tab 6/5	End product 95% of maximum dry density of BS 1377: Part 4 (vibrating hammer method)	6	N	6
						(ii) uniformity coefficient	See Note 3	10	-				
						(iii) Los Angeles coefficient	Clause 635	-	40				
						(iv) undrained shear parameters (c and ϕ)	Clause 633	Not required					
						(v) effective friction angle (ϕ') and effective cohesion (c')	Clause 636	c' = 0kPa ϕ' = 40°	-				
						(vi) permeability	Clause 640	Note 20	-				
						(vii) mc	BS 1377:Part 2 See Note 4	Note 15	Note 15				
						(viii) slope stability test (where required in Appendix 6/6)	Clause 610	App 6/6					

A minimum Ev2 of 60MPa was assumed for Class 6N6 for the settlement calculations.

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Ref	Durability Assessment	Output
OpenGround BRE Special Digest 1	**Overview** Due to limited GI and chemical testing data available for the specific location of the 060-AC4 pipe culvert, all the latest chemical testing data (at the time of production of this calculation package, i.e., December 2023), for the entire Aylesbury Embankment Level 1 Asset Group were considered for this assessment (See tab 5a). As the Chemical testing data is available only for the top 2m of the strata. The Chemical assessment results for other soil types below 2m is taken from the section 2.4.17 & Table 2-21 of the Aylesbury Embankment GDR (C02). Table 2-21 of GDR (C02) shows DS-3 & AC-1s design sulphate class and ACEC Class respectively for Gault Clay which seems incorrect, hence for this asset we are using DS-3 & AC-3 as the classification for the Gault Clay. **Assumptions** Due to the presence of existing road which is to be realigned as Oat Close Bridleway (SBH/19) Realignment, brownfield locations were selected for the assessment. Conservatively, mobile water was assumed.	

Table C2 Aggressive Chemical Environment for Concrete (ACEC) classification for brownfield locations[a]

Sulfate and magnesium						Groundwater		
Design Sulfate Class for location	2:1 water/soil extract[b]		Groundwater		Total potential sulfate[c]	Static water	Mobile water	ACEC Class for location
1	2 (SO_4 mg/l)	3 (Mg mg/l)	4 (SO_4 mg/l)	5 (Mg mg/l)	6 (SO_4 %)	7 (pH)[d]	8 (pH)[d]	9
DS-1	< 500		< 400		< 0.24	≥ 2.5	> 6.5[d]	AC-1s
							> 6.5	AC-1
							5.5–6.5	AC-2z
							4.5–5.5	AC-3z
							2.5–4.5	AC-4z
DS-2	500–1500		400–1400		0.24–0.6	> 5.5		AC-1s
							> 6.5	AC-2
						2.5–5.5		AC-2s
							5.5–6.5	AC-3z
							4.5–5.5	AC-4z
							2.5-4.5	AC-5z
DS-3	1600–3000		1500–3000		0.7–1.2	> 5.5		AC-2s
							> 6.5	AC-3
						2.5–5.5		AC-3s
							5.5–6.5	AC-4
							2.5–5.5	AC-5
DS-4	3100–6000	≤ 1200	3100–6000	≤ 1000	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4
						2.5–5.5		AC-4s
							2.5–6.5	AC-5
DS-4m	3100–6000	> 1200[e]	3100–6000	> 1000[e]	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4m
						2.5–5.5		AC-4ms
							2.5–6.5	AC-5m
DS-5	> 6000	≤ 1200	> 6000	≤ 1000	> 2.4	> 5.5		AC-4s
						2.5–5.5	≥ 2.5	AC-5
DS-5m	> 6000	> 1200[e]	> 6000	> 1000[e]	> 2.4	> 5.5		AC-4ms
						2.5–5.5	≥ 2.5	AC-5m

Notes
a Brownfield locations are those sites, or parts of sites, that might contain chemical residues produced by or associated with industrial production (Section C5.1.3).

Summary of data

PH	7.41		Min, considered for 16 samples within 2m depth of influence
Total Potential Sulphate (TPS)	0.03	%	Considered for 1 sample within 2m depth of influence
SO_4 water soluble	179.27	mg/l	Mean of highest 20%, considered for 15 samples within 2m depth of influence

Results

Design Sulphate Classification for Head Deposits: DS-1

ACEC Class for Head Deposits: AC-1

Design Sulphate Classification for Gault Formation: DS-3

ACEC Class for Gault Formation: AC-3

Send via cover Email to Structural Engineer/Ancillary team

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Ref	Summary of foundation design Results	Output

Culvert

Settlement

	Min	Max	Tolerance	
Settlement of culvert	3.53	17.14	NA	mm
Rotation of segments	-0.19	0.22	NA	degrees
Relative displacement	-5.97	6.91	10.00	mm

Case 1 - Construction loading assuming β=0 with 20kpa surcharge applied to the headwall

Case 1a - Construction loading β=14 with 10kpa variable surcharge applied to the headwall

Case 2 - In service condition β=14 with 10kpa permanent surcharge applied to the headwall

Headwall

Sliding

	Foundation on natural ground			Foundation on 0.6m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	117%	91%	77%	96%	90%	89%
DA1,C2	90%	69%	58%	74%	68%	66%
SLS	75%	58%	52%	64%	60%	60%

Overturning

	Foundation on natural ground			Foundation on 0.6m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	84%	75%	73%	84%	75%	73%
DA1,C2	66%	57%	55%	66%	57%	55%
SLS	49%	44%	44%	49%	44%	44%

Bearing

	Natural Ground (considering E&R below the Foundation)			Foundation on 0.6m Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	30%	23%	23%	6%	5%	5%
DA1,C2	31%	23%	23%	11%	8%	8%

Recommendation: Utilisation <100%, however, **0.6m** of Class 6N6 fill required under Headwall to ensure stability.

Classifications for concrete

Design Sulphate Classification for Head Deposits: DS-1
ACEC Class for Head Deposits: AC-1
Design Sulphate Classification for Gault Formation: DS-3
ACEC Class for Gault Formation: AC-3

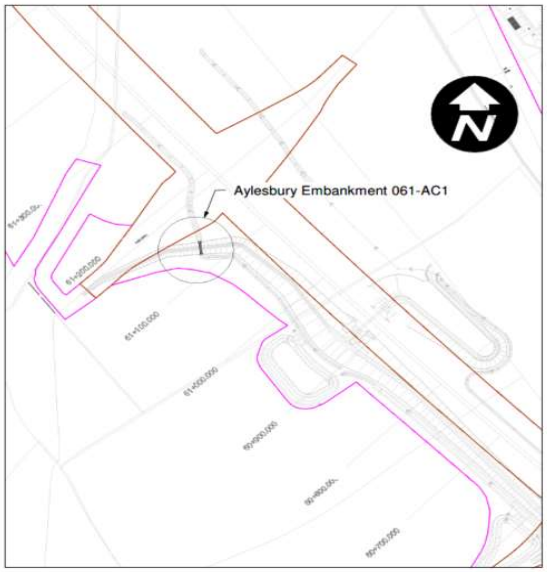
HS2 Ltd - Code 1 - Accepted

Appendix L

Summary of Design – Aylesbury Embankment 061-AC1 Ancillary Culvert

Number of Pages: 06

HS2 Ltd - Code 1 - Accepted

Ref	Introduction to Geotechnical Design	Output
PowerBI & GIS Maps	**Introduction** The purpose of this spreadsheet is to undertake the geotechnical design for the Pipe Culvert 061-AC1 (asset ID: HS2-00002N7VQ) as part of IDR 207.2. The asset is located offline at the approximate HS2 chainage 61+128 within the Aylesbury Embankment Level 1 Asset Group. The asset sits beneath the highway asset Oat Close Bridleway, SBH/19 Realignment (asset ID: HS2-000020RKL, part of IDR 207.2) and allows land drainage to flow beneath the highway. The following geotechnical assessments are presented in this calculation pack for the asset: * Assessment of ground conditions and geotechnical risks; * Concrete Durability; * Settlement of culvert and headwalls along with Damage Assessment i.e., evaluation of Joint Rotations and Opening, Spring Stiffness values where necessary; * Bearing Capacity, Overturning and Sliding check of the headwalls.	Tabs 5 to 8
Drawings provided by Ancillary team	**Input Data** The culvert geometry and details are based on the following drawings and should be read in conjunction with this calculation pack: 1MC06-CEK-CV-DGA-CS04_CL18-000247 P01 1MC06-CEK-CV-DGA-CS04_CL18-000263 P01 1MC06-CEK-CV-DGA-CS04_CL18-000264 P01 1MC06-CEK-CV-DSE-CS04_CL18-000073 P01 Drawings used at the time of this geotechnical design can be found here: HS2 ASC Aylesbury Area - Issue 03 - 30-11-2023 **Location Plan**  Location Plan 061-AC1 HS2-00002N7VQ at Chainage 61+128	
GDR	**Submission** This calculation pack includes all details for the geotechnical design inputs applicable to the culvert, however, this is not a formally submitted deliverable. Instead a summary of this calculation pack is included within the Aylesbury Embankment GDR (1MC06-CEK-GT-REP-CS04_CL18-000037 C02).	-

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Ref	Assumptions	Output
1	Ground conditions considered as part of the design shall be validated on site prior to construction of the culvert and headwalls. The minimum requirements for site testing are specificed on the culvert drawings and in the IDR Specification.	
2	A reinforced concrete density of 25kN/m^3 has been assumed for the culvert and headwalls.	
3	Any changes to the culvert, headwall structure's geometry and/or loading will result in a further geotechnical verification check being required. Culvert and headwall dimensions and levels are provided by the Ancillary/ Watercourse/ Drainage teams and are assumed to be final.	
4	Due to limited GI data available at the location of the Culvert 061-AC1, the parameters in Section 2.4 of the GDR (C02), related to Aylesbury Embankment (asset ID: HS2- HS2-000001027) have been considered most relevant to the culvert and were adopted. It is understood that the ground model and geotechnical parameters presented in the Aylesbery Embankment GDR (C02) have been through a full PCRA.	
5	As per table 2-5, section 2.4.7 GDR (C02) and neraest borehole (ML061-WS018), the groundwater monitoring recorded depths between 1.0 mbgl and 2.5mbgl. For culvert design, ground water level conservatively considered at the existing gound level on both sides.	
6	For durability assessment and concrete classifications in the natural ground, a mobile groundwater and brownfield site (accounting for any existing/disused roads) were conservatively assumed.	
7	The partial factors for actions for civil structures (buildings) were used for the foundation design of the headwalls as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013. Reference is made to Set B and Set C partial factors (Tables NA.A2.4(B) and NA.A2.4(C)) of NA to BS EN 1990:2002+A1:2005, whereas, the partial factors for actions for bridges were used for the design of the culvert section.	
8	As the Culvert top level is approx. at the existing ground level, self-weight of the Culvert is assumed to be same as the weight of the soil excavated below the existing ground and hence et load of Culvert has been set to zero for settlement calculations.	
9	In the sliding checks, as the headwalls are to be cast-in-situ, the design friction angle of shearing resistance δ between headwall base slab and the subgrade is taken as 1.0 ϕ' (delta=1.0*Phi').	
10	Where founding strata, i.e., base of Class 6N6, has an undrained shear strength of <40kPa, the existing soil shall be excavated in depths of at least 200mm increments to comply with the requirement of undrained shear strength of >40kPa. The excavated area is to be replaced as soon as practicable by Class 6N6 material to the proposed foundation level.	
11	Pipe culvert is assumed to be stable while it is buried in the embankment, therefore no stability checks performed for the culvert segments.	
12	Backfill properties of class 6N6 material has been assumed to have peak friction angle of 40deg and critical friction angle of 34deg with dry and saturated unit weight of 20 and 22kN/m^3 respectively.	

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Ref	Ground Model and Geotechnical Parameters	Output

Extracted from section 2.4 of GDR (C02)

Introduction

The ground conditions has been assesed considering the following sources:
* Aylesbury Embankment Supplementary GIR (1MC06-CEK-GT-REP-CS04_CL18-000011 C02)
* Aylesbury Embankment GDR (1MC06-CEK-GT-REP-CS04_CL18-000037 C02)
* Available GI from OpenGround
* GIS Viewer

Table 2-3: Summary of Ground Conditions at Different Chainages.

Ground Model (GM)	Strata	Depth Range
Ground Model 2 (Ch. 61+000 – Ch. 61+200)	Alluvium	0.00 – 1.60
	Gault Clay Formation	1.60 – 5.30
	Whitchurch Formation	5.30 – 6.30
	Purbeck Formation	6.30 – 8.10-
	Portland Stone/Sand Formation	8.10 – 21.50

Table B-1 Characteristic Geotechnical Parameters of In-situ Material

Geological Strata	γ_k (Mg/m³)	SPT (N)'	c_u [13] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	ν_k	ν'	E_u (MPa)	E' (MPa)	E_{unrel} (MPa)
Alluvium	1.8	0-22	0 → 2.0 z = 20	0-2	21	23	0.5	0.2	6.0	4.8	5.28
Gault Clay Formation (GLT)	1.9	6 + 8.6z	50 + 10z → 150	8	21	23	0.5	0.2	15 + 3z → 45	12 + 2.4z → 36	13.3 + 2.64z → 40
Portland Stone/Sand Formation (POST/POSA)	2.0	2.0 m + 9 + 13.5z	50 + 10z → 130	8	26	28	0.5	0.2	15 + 3z → 39	12 + 2.4z → 31	13.3 + 2.64z → 34.1

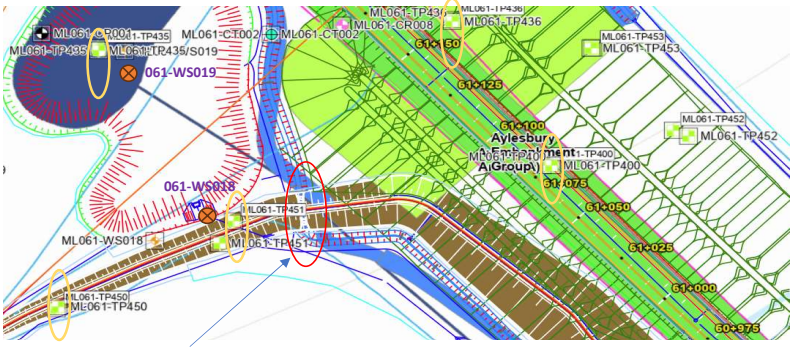
Available GI

The figure and table below summarise the exploratory holes which have been considered when assessing the ground conditions local to the asset.

The data set corresponds ASC Detailed Design Data Drop 40 (October 2023) from Open Ground and no further GI is planned around this asset to inform detail design.

Ref	Ground Model and Geotechnical Parameters	Output

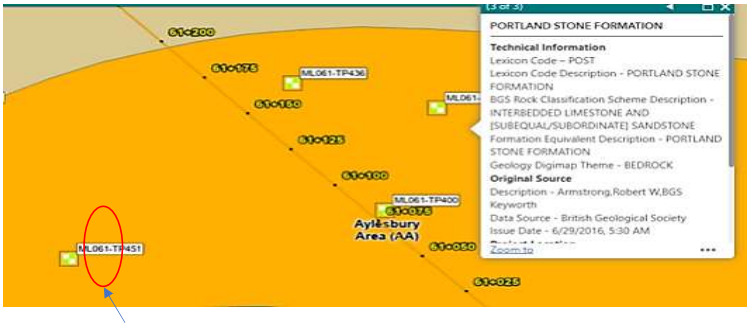
Local GI locations from GIS Maps



061-AC1

Geology

The BGS maps accessed through the GIS mapping does indicate the absence of superficial geologies. However, the presence of PORTLAND STONE FORMATION is shown as the bedrock geology.

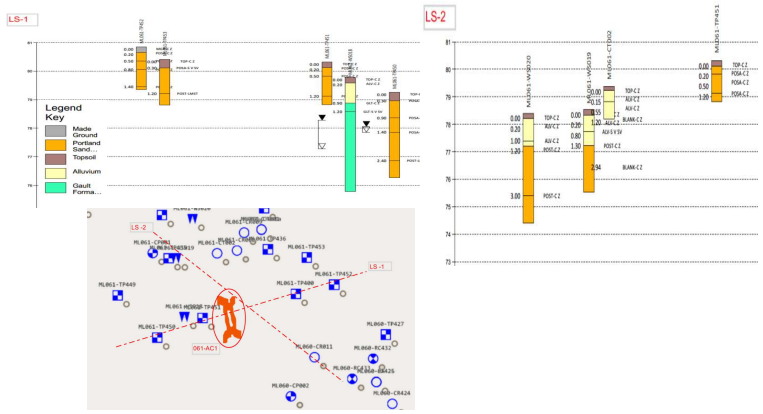


061-AC1

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OpenGround Data

"Localised data within approximately 100m radius of the Culvert were checked for this location to confirm the ground model from the GDR (C02).
The following exploratory holes were checked:
ML061-TP451
ML061-WS018
ML061-WS019
ML061-TP435
ML061-TP436
ML061-TP400
ML061-TP450
The localised data confirm the presence of Alluvium deposits to maximum depths to 1.3m bgl (ML061-WS019), described as soft light brown slightly sandy CLAY. Underlying Gault Formation (GLT) is present to a depth of 4 m bgl (base not proven) (ML061-WS018), described as soft light grey mottled brown slightly sandy CLAY.



Design Ground Model for Culvert foundation design (Ground Model 2 of Mainline Earthworks)

S.No	Geological Unit	Top of the Stratum (m AOD)	Bottom of the Stratum (m AOD)	Thickness, m
1	Alluvium	80.791	79.191	1.6
2	Gault Formation (GLT)	79.191	72.691	6.5
3	Portland Stone POSA CZ	72.691	Not proven	-

Culvert invert level	79.406	mAOD
Culvert founding level	78.941	mAOD

Innvert level of culvert, which is the base interior level of a pipe is 79.406 m AOD . By considering 155mm pipe thickness and 310mm thick granular bedding, the culvert is to be founded on GLT formation at 78.941 mAOD . GLT stratum was considered as the geology within the zone of influence of foundation design for bearing/ sliding/ bearing and settlement checks.

Parameters adjustment for Superficial strata

No parameter adjustment has been proposed for ALV/GLT.

Class 6N6 (granular fill) parameters

The Following parameters were adopted for the Class 6N6 in the design:

Table 6-1 of Contract-specific tables to 600 Series Appendices (1MC06-CEK-GT-REP-C002-000154)

6	N	6	Selected well graded granular material	High Strength Fill to non-HSR structures	Natural gravel, natural sand, crushed gravel, crushed rock, crushed concrete, slag, well- burnt colliery spoil or any combination thereof. None of these constituents shall include any argillaceous rock. Recycled aggregate except recycled asphalt. Where material is imported onto site which is not 'as dug' it shall be aggregate conforming to BS EN 13242 from one or more of the following source codes, see Notes 8, 9 and 10: P (natural aggregates – except shale, siltstone or slate, see Note 7); A2 (crushed concrete); A3 (crushed bricks, masonry); G1 (red coal shale)	(i) grading	BS 1377: Part 2 (on-site)	Tab 6/2	Tab 6/2	End product 95% of maximum dry density of BS 1377: Part 4 (vibrating hammer method)	6	N	6
						BS EN 933-2 (Imported onto site)	Tab 6/5	Tab 6/5				
					(ii) uniformity coefficient	See Note 5	10	-				
					(iii) Los Angeles coefficient	Clause 635	-	40				
					(iv) undrained shear parameters (c and ϕ)	Clause 633	Not required					
					(v) effective friction angle (ϕ') and effective cohesion (c')	Clause 636	c' = 0kPa ϕ' = 40°	-				
					(vi) permeability	Clause 640	Note 20	-				
					(vii) mc	BS 1377:Part 2 See Note 4	Note 15	Note 15				
					(viii) slope stability test (where required in Appendix 6/6)	Clause 610	App 6/6					

A minimum Ev2 of 60MPa was assumed for Class 6N6 for the settlement calculations.

Groundwater level used for the design

As per table 2-5, section 2.4.7 GDR (C02) and neraest borehole (ML061-WS018), the groundwater monitoring recorded depths between 1.0 mbgl and 2.5mbgl.
For culvert design, groundwater level conservatively considered at existing ground level on both sides.

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Ref	Durability Assessment	Output

OpenGround

Overview

Limited chemical testing data available for the specific location of the culvert and all the available chemical testing data (at the time of production of this calculation package, i.e., October 2023) has been considered (See tab 5a).

The chemical tests available were filtered for the top 1.5 m below ground level, to represent the superficial deposits.

Assumptions

Accounting for any existing/disused roads , **brownfield** locations were selected for the assessment.

Conservatively, **mobile** water was assumed.

BRE Special Digest 1

Table C2 Aggressive Chemical Environment for Concrete (ACEC) classification for brownfield locations[a]

Sulfate and magnesium						Groundwater		
Design Sulfate Class for location	2:1 water/soil extract[b]		Groundwater		Total potential sulfate[c]	Static water	Mobile water	ACEC Class for location
1	2 (SO_4 mg/l)	3 (Mg mg/l)	4 (SO_4 mg/l)	5 (Mg mg/l)	6 (SO_4 %)	7 (pH)[d]	8 (pH)[d]	9
DS-1	< 500		< 400		< 0.24	≥ 2.5	> 6.5[d]	AC-1s
							5.5–6.5	AC-1
							4.5–5.5	AC-2z
							2.5–4.5	AC-3z
DS-2	500–1500		400–1400		0.24–0.6	> 5.5		AC-4z
						2.5–5.5	> 6.5	AC-1s
								AC-2
								AC-2s
							5.5–6.5	AC-3z
							4.5–5.5	AC-4z
DS-3	1600–3000		1500–3000		0.7–1.2	> 5.5	2.5–4.5	AC-5z
						2.5–5.5	> 6.5	AC-2s
								AC-3
							5.5–6.5	AC-3s
							2.5–5.5	AC-4
DS-4	3100–6000	≤ 1200	3100–6000	≤ 1000	1.3–2.4	> 5.5		AC-5
						2.5–5.5	> 6.5	AC-3s
								AC-4
							2.5–6.5	AC-4s
DS-4m	3100–6000	> 1200[e]	3100–6000	> 1000[e]	1.3–2.4	> 5.5		AC-5
						2.5–5.5	> 6.5	AC-3s
								AC-4m
							2.5–6.5	AC-4ms
DS-5	> 6000	≤ 1200	> 6000	≤ 1000	> 2.4	> 5.5		AC-5m
						2.5–5.5	≥ 2.5	AC-4s
DS-5m	> 6000	> 1200[e]	> 6000	> 1000[e]	> 2.4	> 5.5		AC-5
						2.5–5.5	≥ 2.5	AC-4ms
								AC-5m

Notes
a Brownfield locations are those sites, or parts of sites, that might contain chemical residues produced by or associated with industrial production (Section C5.1.3).

Summary of data

PH	8.02		Max, considered for 4 samples within 1.5m bgl ALV
TPS (Total Potential Sulphate)	0.12	%	Max, considered for 3 samples within 1.5m bgl ALV
SO4 water soluble	13.71	mg/l	Max, cconsidered for 4 samples within 1.5m bgl ALV

Results based on limited local data for ALV:
Design Sulphate Classification: DS-1
ACEC Class: AC-1

The culvert is founded on Gault Clay formation at 78.941 mAOD. Due to unavailability of chemical testing data for Gault clay near the culvert , data were taken from the durability assessment for the Central Area C2, using route-wide test results based on the first geo-environmental report(Geo-environmental Assessment Report , 1MC06-CEK-EV-REP-CS03_CL17-000007), presented in Table 2-21 under section 2.4, GDR (C02).

Table 2-21 Summary of Soil Sulphate Chemistry

Geological Formation	Design Sulphate Class (DS)	Concrete Classification (ACEC)	Total Potential Sulphate (TPS) (%)	pH
Head Deposits	DS-1	AC-1s	0.20	7.7
Alluvium	No Testing Available			
Gault Clay Formation	DS-3	AC-1s	0.90	7.9
Whitchurch Sand Formation	DS-4	AC-3s	1.60	7.3
Purbeck Group	DS-4	AC-3s	1.5	7.4
Portland Sand/Stone Formation	DS-1	AC-1s	1.0	8.1
Kimmeridge Clay Formation	DS-5	AC-4s	12.2	7.6

Table 2-12 of Aylesbury Embankment GDR(C02) assigned Concrete Classifications of AC-1 and Design Sulphate of DS-3 for Gault Formation, which is an incorrect assessment. A conservative recommendation of Concrete Classifications of AC-3 and DS-3 has been made for Gault Clay.

Recommendations

Design Sulphate Classification: **DS-3**
ACEC Class: **AC-3**

Send via cover Email to Structural Engineer/ Ancillary Team

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Ref	Summary of Foundation Design Results	Output

Culvert

Settlement

	Min	Max	Tolerance	
Settlement of culvert	3.4	15.2	NA	mm
Rotation of segments	-0.1	0.1	NA	degrees
Relativel displacement	-4.3	2.1	10.0	mm

Recommendation: For relative displacemnt to be within tolerance, no E&R is required below the culvert segments.

Case 1 - Construction loading assuming β=0 with 20kPa surcharge applied to the headwall

Case 1a - Construction loading β=18 with 10kPa variable surcharge applied to the headwall

Case 2 - In service condition β=18 with 10kPa permanent surcharge applied to the headwall

Headwall

Sliding

	Foundation on Natural Ground (check along underside of shear key)			Foundation on 0.6m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	114%	116%	116%	95%	97%	97%
DA1,C2	63%	89%	89%	74%	74%	74%
SLS	72%	49%	49%	62%	62%	62%

Overturning

	Foundation on Natural Ground			Foundation on 0.6m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	78%	73%	73%	78%	73%	73%
DA1,C2	61%	56%	56%	61%	56%	56%
SLS	45%	41%	41%	45%	41%	41%

Bearing

	Foundation on Natural Ground			Foundation on 0.6m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	32%	26%	23%	6%	5%	5%
DA1,C2	33%	25%	25%	11%	9%	9%

Recommendation: Utilisation <100%, however, 0.6m of Class 6N6 fill (E&R) is required under headwall base slab to ensure stability.

Classifications for concrete (Gault Clay Formation)

Design Sulphate Classification: DS-3

ACEC Class: AC-3

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Appendix M

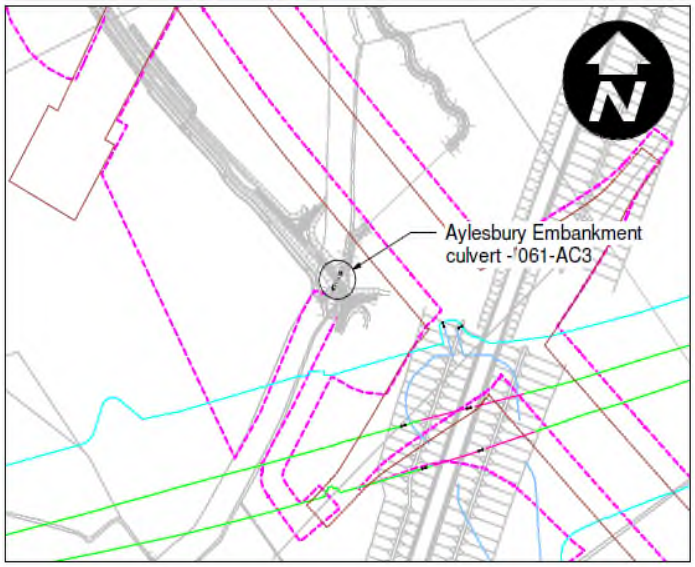
Summary of Design – Aylesbury Embankment 061-AC3 Ancillary Culvert

Number of Pages: 09

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A High-Speed Design Partnership
ARCADIS COWI

High Speed 2 - 1MC06 - Stage One C2 - MWCC – North Portal of Chiltern Tunnels to Brackley
Geotechnical Design
Box Culvert - 061-AC3 - IDR 207.2

Ref	Introduction to Geotechnical Design	Output
PowerBI & GIS Maps	**Introduction** The purpose of this spreadsheet is to undertake the the geotechnical design for the box Culvert 061-AC3 (asset ID: HS2-00002N7VR) as part of IDR 207.2. The asset is located offline at the approximate HS2 chainage of 61+300 within the Aylesbury Embankment Level 1 Asset Group. The asset sits beneath the road maintenance access road to pond (south arm off Calley Farm-south) (asset ID: HS2-00002BJSK, part of IDR 208.4) and allows Sedrup North Watercourse diversion (Asset ID: HS2-00002CT5D) to flow beneath the maintenance access road. The following geotechnical assessments are presented in this calculation pack for the asset: * Assessment of ground conditions and geotechnical risks; * Concrete Durability; * Settlement of culvert and headwalls along with Spring Stiffness values where necessary; * Bearing Capacity, Overturning and Sliding check of the culvert (only applicable to box culverts); * Bearing Capacity, Overturning and Sliding check of the headwalls.	Tabs 11 to 21
Drawings provided by Ancillary team	**Input Data** The culvert geometry and details are based on the following drawings and should be read in conjunction with this calculation pack: 1MC06-CEK-CV-DGA-CS04_CL18-000071.pdf C01 1MC06-CEK-CV-DGA-CS04_CL18-000073.pdf C01 1MC06-CEK-CV-DGA-CS04_CL18-000248.pdf C01 1MC06-CEK-CV-DSE-CS04_CL18-000050_edited.pdf C01	-
	Location Plan  Location plan of Aylesbury Embankment culvert - 061-AC3 (HS2-00002N7VR)	
GDR	**Submission** This calculation pack includes all details for the geotechnical design inputs applicable to the culvert, however, this is not a formally submitted deliverable. Instead a summary of this calculation pack is included within the Aylesbury Embankment GDR 1MC06-CEK-GT-REP-CS04_CL18-000037 C05	-

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A High-Speed Design Partnership
ARCADIS COWI

High Speed 2 - 1MC06 - Stage One C2 - MWCC – North Portal of Chiltern Tunnels to Brackley
Geotechnical Design
Box Culvert - 061-AC3 - IDR 207.2

Ref	Assumptions	Output
1	Ground conditions considered as part of the design will be validated on site prior to construction of the culvert and headwalls. The minimum requirements for site testing will be defined on the culvert drawings and in the IDR Specification.	
2	A reinforced concrete density of 25kN/m3 has been assumed for the culvert and headwalls where concrete self weight is unfavourable but an inferior concrete density of 24kN/m3 should be taken where the self weight is favourable eg sliding.	
3	Any changes to the culvert, headwall structure's geometry and/or loading will result in a further geotechnical verification check being required. Culvert and headwall dimensions and levels are provided by the Ancillary/Watercourse/Drainage teams and are assumed to be final.	
4	Due to limited GI data available at the location of the 061-AC3 ancillary culvert, the ground model and parameters in Section 4.5.5 of the GDR (C02), related to Sedrup North Culvert (asset ID: HS2-000001403), Table B-1 Justification of Characteristic Parameters – Mainline Earthworks Appendix B, have been considered most relevant to the 061-AC3 Ancillary culvert and were adopted for ground model and parameters respectively. It is assumed that the ground model and geotechnical parameters presented in the Aylesbury Embankment GDR (C02) have been through a full PCRA	
5	A groundwater level of exisisting ground level has been used in the design of Sedrup North culvert in the GDR (C02). Similarly, due to the presence of the land drainage and possibility of localised higher groundwater level in the area, flood groundwater level for this culvert has been assumed to be at ground level. The groundwater level is assumed to be the same on both sides of the culvert.	
6	For durability assessment and concrete classifications in Portand stone/sand formation, a mobile groundwater and brownfield site (accounting for any existing/disused roads) were conservatively assumed.	
7	The partial factors for actions for bridges were used for the foundation design of the ditch slab as per Table A.NA.3 of NA+A1:2014 to BS EN 1997-1:2004+A1:2013. Reference is made to Set B and Set C partial factors (Tables NA.A2.4(B) and NA.A2.4(C)) of NA to BS EN 1990:2002+A1:2005.	
8	As the Culvert is partially buried relative to existing ground level, the net pressure, self weight of culvert and the soil excavated for placing the box culvert below EGL has been considered in settlement analysis.	
9	In the sliding check calculations for headwall foundation, the friction between Class 6N6 and head wall foundation(cast-in-situ concrete) is taken as phi (delta=1.0* phi). Friction with the precast box culvert bottom are assumed to be 2/3 phi, because of pre-cast resulting in less friction between the foundations and the Class 6N6 material below (delta=2/3* Phi).	
10	Where founding stratum (ie base of Class 6N6) has an undrained shear strength <40kPa, material shall be excavated in depth increments of at least 200mm and the formation re-tested until an undrained shear strength >40kPa is achieved. The excavated soils are to be replaced as soon as practicable with Class 6N6 and the level brought up to the underside of the proposed foundation level	
11	it is considered that the foundation on the Portland sand formation meets the requirement for Cu>40kPa with the excavate and replace, 2m depth from ground level (0.8m from foundation level). However there might be a risk of thick layer of Alluviumin at particular location where there is an existing watercourse.	
12	Backfill properties of class 6N6 material has been assumed to have peak friction angle of 40deg and critical friction angle of 34deg with dry and saturated unit weight of 20 and 22kN/m3 respectively.	
13	Any benefits of positive arching has been ignored. Negative arching effects considered via supplementary model factor.	
14	Groundwater levels are assumed to be at existing ground level, whereas for uplift calculations for culvert water table is considered at the top of base slab as the fill used behind the headwall is 6N6 which is highly permeabile material and will be free-draining so no water pressure build-up in the 6N6 is assumed	
15	Culvert Structure has been considered as empty for sliding checks	
16	Interface friction between soil and wall has been ignored for all the cases for the design to be conservative.	

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Ref	Ground Model and Geotechnical Parameters	Output

Extract from GDR C02

Introduction

The ground conditions has been assesed considering the following sources:
* Aylesbury Embankment Supplementary GIR (1MC06-CEK-GT-REP-CS04_CL18-000011 C02)
* Aylesbury Embankment GDR (1MC06-CEK-GT-REP-CS04_CL18-000037 C02)
* Available GI from OpenGround
* GIS Viewer

Table 2-3: Summary of Ground Conditions at Different Chainages

Table 4-2 Ground Model-1 (GM-1) at Sedrup North Culvert

Geological Unit	Top of Stratum Range (m AOD)	Design Top Level (m AOD)	Range of Thickness (m)	Design Thickness (m)
Topsoil (TOP-CZ)	77.87-77.38	77.70	0-0.3	0.30
Alluvium (ALV-CZ)	77.57-75.48	77.40	0.3-2.2	1.00
Head Deposits (HEAD-S V SV)	77.18-75.68	76.40	0.5-2.0	1.00
Portland Stone Formation (POST-CZ)	75.48-69.86	75.40	2.2-7.82	3.70
Kimmeridge Clay Formation (KC-CZ)	71.35-57.67	71.70	6.52-20.2	21.7*

* The maximum depth of investigation data available is 20.2m. *Kimmeridge clay depth is assumed to be extended 50mAOD.

Table 5-1 Characteristic Geotechnical Parameters of In-situ Material

Geological Strata	γ_b (Mg/m³)	SPT (N)[1]	PI (%)	LL (%)	c_u[13] (kPa)	c_u[14] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	ν_u	ν'	E_u (MPa)	E' (MPa)	E_{oed} (MPa)	m_v (m²/MN)	c_v (m²/year)
Alluvium	1.8	0-22	48	36	$0 \rightarrow 2.0 = 20$	$0.0 \rightarrow 2.0 = 30$	0-2	21	23	0.5	0.2	6.0	4.8	5.28	0.190	11.05
Head	1.8	$7 + 0.3z$	43	50	$0 \rightarrow 2.0 = 30$ $2.0 += 40$		0-2	21	23	0.5	0.2	$0 \rightarrow 2.0 = 9.0$ $2.0 += 12.0$	$0 \rightarrow 2.0 = 7.2$ $2.0 += 9.6$	$0 \rightarrow 2.0 = 8.0$ $2.0 += 11.0$	$0 \rightarrow 2.0 = 0.125$ $2.0 += 0.090$	5.05
Gault Clay Formation (GLT)	1.9	$6 + 8.6z$	47	70	$50 + 10z \rightarrow 150$	$2.0 \rightarrow 4.5 = 40$	8	21	23	0.5	0.2	$15 + 3z \rightarrow 45$	$12 + 2.4z \rightarrow 36$	$13.3 + 2.64z \rightarrow 40$	$\frac{1}{(13.3 + 2.64z \rightarrow 40)}$	1.89
Soft Portland Stone/Sand Formation (POST/POSA)	2.0	$0 - 2$ m $5 + 11z$	35	34	20	$z > 4.5 = 40 + 8z \rightarrow 150$	0-2	23	25	0.5	0.2	6.0	4.8	5.28	0.189	12.62

Geological Strata	γ_b (Mg/m³)	SPT (N)[1]	PI (%)	LL (%)	c_u[13] (kPa)	c_u[14] (kPa)	c' (kPa)	ϕ'_{cv} (°)	ϕ'_{peak} (°)	ν_u	ν'	E_u (MPa)	E' (MPa)	E_{oed} (MPa)	m_v (m²/MN)	c_v (m²/year)
Portland Stone/Sand Formation (POST/POSA)	2.0	$2.0\text{ m} + 9 + 13.5z$	20	34	$50 + 10z \rightarrow 130$		8	26	28	0.5	0.2	$15 + 3z \rightarrow 39$	$12 + 2.4z \rightarrow 31$	$13.3 + 2.64z \rightarrow 34.1$	$\frac{1}{(13.3 + 2.64z \rightarrow 34.1)}$	12.62
Whitchurch Formation (WHS)	2.0	50[2]	20	37	50		10	22	24	0.5	0.2	15	12.0	13.20	0.075	10.10
Purbeck Formation (PB)	2.0	35 – 50	No data[3]	No data	80		10	22	24	0.5	0.2	24	19.2	21.12	0.047	0.01[4]
Kimmeridge Clay Formation (KC)	2.0	$8 + 3z$	25	44	$50 + 10z \rightarrow 150$		10	33	35	0.5	0.2	$15 + 3z \rightarrow 45$	$12 + 2.4z \rightarrow 36$	$13.3 + 2.64z \rightarrow 40$	$\frac{1}{(13.3 + 2.64z \rightarrow 40)}$	6.94

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Available GI

The figure and table below summarise the exploratory holes which have been considered when assessing the ground conditions local to the asset.

The data set corresponds ASC Detailed Design Data Drop 40 (Month 2023) from Open Ground and no further GI is planned around this asset to inform detail design.

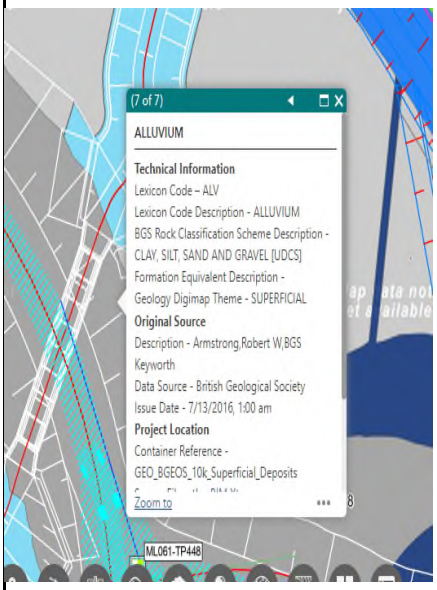
Local GI locations from GIS Maps

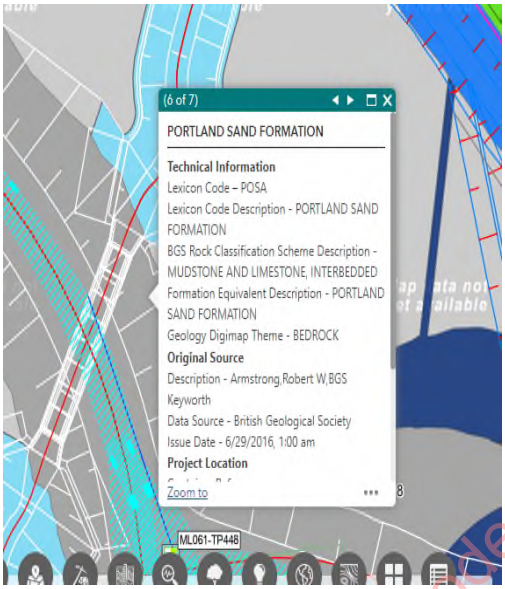
GIS Maps



Geology

The BGS maps accessed through the GIS mapping indicates the presence of Alluvium superficial geologies, and the presence of Portland sand formation as the bedrock geology.





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OpenGround Data

Localised data within approximately 72m radius of the Culvert were checked for this location to confirm the ground model from the GDR (C02).The exploratory holes mentioned below were checked. The ML061-TP448 is considered because it is located very close to the asset and it aslo compared with other one trial pit and one borehole while deciding the ground profile.

ML061-CR013.pdf
ML061-TP447.pdf
ML061-TP448.pdf

		Boreholes			Considered for culvert 061-AC3
Strata		ML061-CR013	ML061-TP448	ML061-TP447	
Top soil/Made Ground	Top	78.06	78.06	77.6	**77.7**
	Bottom	77.76	77.86	77.4	**77.4**
	Thickness	0.3	0.2	0.2	**0.3**
Head	Top	-	-	77.4	**Not considered**
	Bottom	-	-	77.2	
	Thickness	-	-	0.2	
Aluvium	Top	77.76	-	-	**Not considered**
	Bottom	76.06	-	-	
	Thicknees	1.7	-	-	
Purbeck Lime stone formation	Top	76.06	-	-	**Not considered**
	Bottom	73.56	-	-	
	Thickness	2.5	-	-	
Portland stone/sand formation	Top	73.56	77.86	77.2	**77.4**
	Bottom	59.46	76.56	76.1	**59.5**
	Thickness	14.1	1.3	1.1	**17.9**
Kimmeridge clay Formation	Top	59.46	-	-	**59.5**
	Bottom	58.06	-	-	**58.1**
	Thickness	1.4	-	-	**1.4**

Design Ground Model for Culvert foundation design

S.No	Geological Unit	Top of the Stratum (m AOD)	Bottom of the Stratum (m AOD)	Thickness, m
1	Top soil	77.70	77.40	0.30
2	Portland Sand/ Stone Formation	77.40	59.50	17.90
3	Kimmeridge clay Formation	59.50	58.10	1.4

Invert level of the culvert 76.823 m AOD

As the invert level of culvert is 0.9m below EGL (77.7m), where culvert rests on Portland Stone/sand formation.ThereforePortland Stone/sand formation was considered as the geology within the zone of influence of foundation design for bearing/sliding/punching and settlement checks.

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Parameters adjustment for Head Deposits

The values of soil parameters used for Sedrup North culvert are higher than mainline earthworks. Therefore parameters are considered from the mainline earthworks (Appendix B of the GDR C02). Portland Sand/stone formation's undrained shear strength is selected from the Figure B-15 of appendix B of GDR C02 (Culvert is located at the chainage 61+300). Since the bottom of the culvert foundation is 1.2m below ground level the Cu value is conservatively assumed as 30 kpa (please refer the notes below the graph -Figure B-15). In order to get Cu value of 40kpa, 0.8m depth excavation and replace required from the foundation bottom (From the note below the graph -Figure B-15)

since Cu of POSA 30 kPa, Excavate and replace is required

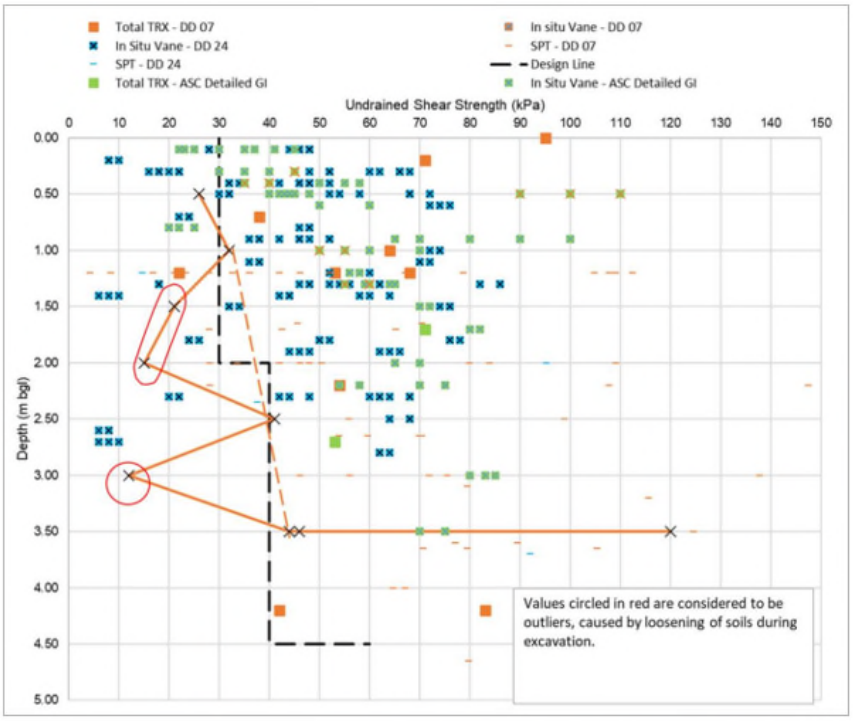


Figure B-15 - Undrained Shear Strength for Kimmeridge Clay, Portland Stone/Sand Formation from Ch.61+200 to Ch.61+763

Conservative strength parameters adopted within Alluvium Extent:

Depth from 0.0 to 2.0 mbgl: Cu = 30 kPa

Depth from 2.0 to 4.5 mbgl: Cu = 40 kPa

Depth > 4.5mbgl: Cu = 40+8z

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Table 6-1 of Contract-specific tables to 600 Series Appendices (1MC06-CEK-GT-REP-C002-000154)

Class 6N6 (granular fill) parameters

The Following parameters were adopted for the Class 6N6 in the design:

6	N	6	Selected well graded granular material	High Strength Fill to non-HSR structures	Natural gravel, natural sand, crushed gravel, crushed rock, crushed concrete, slag, well- burnt colliery spoil or any combination thereof. None of these constituents shall include any argillaceous rock. Recycled aggregate except recycled asphalt. Where material is imported onto site which is not 'as dug' it shall be aggregate conforming to BS EN 13242 from one or more of the following source codes, see Notes 8, 9 and 10: P (natural aggregates – except shale, siltstone or slate, see Note 7); A2 (crushed concrete); A3 (crushed bricks, masonry); G1 (red coal shale)	(i) grading	BS 1377: Part 2 (on-site)	Tab 6/2	Tab 6/2	End product 95% of maximum dry density of BS 1377: Part 4 (vibrating hammer method)	6	N	6
							BS EN 933-2 (Imported onto site)	Tab 6/5	Tab 6/5				
						(ii) uniformity coefficient	See Note 5	10	-				
						(iii) Los Angeles coefficient	Clause 635	-	40				
						(iv) undrained shear parameters (c and ϕ)	Clause 633	Not required					
						(v) effective friction angle (ϕ') and effective cohesion (c')	Clause 636	c' = 0kPa ϕ' = 40°	-				
						(vi) permeability	Clause 640	Note 20	-				
						(vii) mc	BS 1377:Part 2 See Note 4	Note 15	Note 15				
						(viii) slope stability test (where required in Appendix 6/6)	Clause 610	App 6/6					

A minimum Ev2 of 60MPa was assumed for Class 6N6 for the settlement calculations.

Groundwater level used for the design

A groundwater level is varied between 0.54mbgl -2.07m bgl at the related boreholes but has been considered at existing ground level for designing of Sedrup North Culvert in the GDR (C02). However, due to the presence of the land drainage and possibility of localised higher groundwater level in the area, the groundwater level for this culvert is assumed to be at the existing ground level but for the uplift calculation groundwater level is considered at top of the base slab as fill material behind the culvert which is 6N6 is highly permeabile and will be free-draining so no water pressure build-up in the 6N is assumed. The groundwater level is assumed to be the same on both sides of the culvert.

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Ref	Durability Assessment	Output

OpenGround

Overview

Due to limited GI and chemical testing data available for the specific location of the box culvert, all the latest chemical testing data, for the Maintenance Access Road to Pond (South Arm off Calley Farm - South) were considered for this assessment (**see Tab 5a**).
The chemical tests available were filtered for whole depths of POSA/POST and Kimmeridge Clay layers. It has been found that Kimmeridge clay is pyritic. Hence considered concrete classification for foundation as DS4-AC4 which is derived from the assesements of Kimmeridge clay.

Assumptions

Due to the construction activities of Maintenance Access Road to Pond (South Arm off Calley Farm - South),the brownfield locations were selected for the assessment.
Conservatively, mobile water was assumed.

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Table C2 Aggressive Chemical Environment for Concrete (ACEC) classification for brownfield locations[a]

Sulfate and magnesium						Groundwater		ACEC
Design Sulfate Class for location	2:1 water/soil extract[b]		Groundwater		Total potential sulfate[c]	Static water	Mobile water	Class for location
1	2 (SO_4 mg/l)	3 (Mg mg/l)	4 (SO_4 mg/l)	5 (Mg mg/l)	6 (SO_4 %)	7 (pH)[d]	8 (pH)[d]	9
DS-1	< 500		< 400		< 0.24	≥ 2.5		AC-1s
							> 6.5[d]	AC-1
							5.5–6.5	AC-2z
							4.5–5.5	AC-3z
							2.5–4.5	AC-4z
DS-2	500–1500		400–1400		0.24–0.6	> 5.5		AC-1s
							> 6.5	AC-2
						2.5–5.5		AC-2s
							5.5–6.5	AC-3z
							4.5–5.5	AC-4z
							2.5–5.5	AC-5z
DS-3	1600–3000		1500–3000		0.7–1.2	> 5.5		AC-2s
							> 6.5	AC-3
						2.5–5.5		AC-3s
							5.5–6.5	AC-4
							2.5–5.5	AC-5
DS-4	3100–6000	≥ 1200	3100–6000	≥ 1000	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4
						2.5–5.5		AC-4s
							2.5–6.5	AC-5
DS-4m	3100–6000	> 1200[e]	3100–6000	> 1000[e]	1.3–2.4	> 5.5		AC-3s
							> 6.5	AC-4m
						2.5–5.5		AC-4ms
							2.5–6.5	AC-5m
DS-5	> 6000	≥ 1200	> 6000	≥ 1000	> 2.4	> 5.5		AC-4s
						2.5–5.5	≥ 2.5	AC-5
DS-5m	> 6000	> 1200[e]	> 6000	> 1000[e]	> 2.4	> 5.5		AC-4ms
						2.5–5.5	≥ 2.5	AC-5m

Summary of data

PH	7.855	Mean of lowest 2 samples of 5 test results-considered Kimmeridge clay as it has pyritic characteristics
Total Potential Sulphate (TPS)	7.32 %	Mean of highes 2 samples of 5 test results-considered Kimmeridge clay as it has pyritic characteristics
SO4 water soluble	889.26 mg/l	Mean of highest 2 samples of 5results-considered Kimmeridge clay as it has pyritic characteristics

Results

Design Sulphate Classification: DS-4

ACEC Class: AC-4

Send via cover Email to Structural Engineer/ Ancillary team

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A High-Speed Design Partnership
ARCADIS COWI

High Speed 2 - 1MC06 - Stage One C2 - MWCC – North Portal of Chiltern Tunnels to Brackley
Geotechnical Design
Box Culvert - 061-AC3 - IDR 207.2

Ref | Summary of foundation design Results | Output

Culvert

Settlement

	Min	Max	Tolerance	
Settlement of culvert	4.53	22.83	NA	mm
Rotation of segments	-0.08	0.19	NA	degrees
Relativel displacement	-3.55	8.11	10.00	mm

Recommendation: For Relative displacemnt to be within tolerance, 0.8m E&R is required below the culvert with Class 6N6 material

Box Culvert - Stability checks for the finished permanent structure in-service

	Sliding		Overturning	Bearing	
Analyses	Sliding check at Culvert and Granular Fill or Subgrade Interface	Sliding check at Granular Fill and Subgrade Interface	Foundation on natural ground	Foundation on natural ground	Foundation on 0.8m of Class 6N6
DA1,C1	45%	38%	63%	76%	5%
DA1,C2	38%	33%	51%	78%	9%
SLS	94%	84%	93%	-	-

Recommendation - 0.8m class 6N6 fill required below the box culvert

Headwall

Case 1 - Construction loading assuming β=0 with 20kpa surcharge applied to the headwall

Case 1a - Construction loading β=18 with 10kpa variable surcharge applied to the headwall

Case 2 - In service condition β=18 with 10kpa permanent surcharge applied to the headwall

Headwall

Sliding

	Foundation on Natural Ground	Granular/ subgrade interface		
Analyses	Case 1	Case 1	Case 1a	Case 2
DA1,C1	145%	53%	56%	86%
DA1,C2	53%	41%	42%	64%
SLS	113%	35%	35%	57%

Overturning

	Foundation on natural ground/ granular fill		
Analyses	Case 1	Case 1a	Case 2
DA1,C1	39%	39%	39%
DA1,C2	30%	30%	29%
SLS	25%	25%	25%

Bearing

	Foundation on natural ground			Foundation on 0.8m of Class 6N6		
Analyses	Case 1	Case 1a	Case 2	Case 1	Case 1a	Case 2
DA1,C1	59%	64%	49%	6%	6%	6%
DA1,C2	80%	89%	69%	11%	13%	13%

Recommendation: Utilisation <100%, however, 0.8m of Class 6N6 fill required under head wall to ensure stability.

Classifications for concrete

Design Sulphate Classification: DS-4

ACEC Class: AC-4

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