

SISTEMA ECONÓMICO DE REFORÇO E DRENAGEM DE SOLOS PARA ESTABILIZAÇÃO DE TALUDES EM SOLOS SATURADOS

ECONOMIC SOIL REINFORCEMENT AND DRAINAGE SYSTEM TO STABILISE SLOPES IN LOOSE SATURATED SOIL

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RESUMO

Durante 2020, foi necessário conceber um método para fornecer estabilidade a uma escavação de solo solto e saturado inicialmente instável. A inclinação inicial atingia cerca de 45° em relação à horizontal, com o solo saturado causando deslizamentos localizados e queda de areia para dentro da escavação. A escavação estava cercada por uma infraestrutura sensível existente, não sendo viável construir uma escavação de pequena inclinação ou aplicar técnicas como estacas ou ancoragens. Também não era viável usar uma cortina de estacas-prancha ou uma parede de estacas secantes. Para permitir o reforço de solo e melhorar a estabilidade da encosta, foi projetado um sistema que incorporava a construção de um canal de pedra drenante angular altamente permeável ao longo da base da encosta, combinado com "cunhas" de pedra angular cimentadas e compactadas ortogonalmente à base da encosta. Este sistema combinado de drenagem e solo reforçado satisfaz o orçamento limitado do cliente. Para atender aos padrões de projeto do Eurocódigo 7, com fatores de segurança parciais apropriados, foi realizada uma análise PLAXIS 3D. Na análise foram consideradas as propriedades do solo não saturado. Os resultados da modelação foram traduzidos em especificações construtivas. Os desenhos foram usados para permitir a construção das encostas drenadas e reforçadas. Esta estabilização do solo foi bem sucedida, com a estabilidade mantida pelos dois anos necessários até que a infraestrutura fosse finalmente construída e aterrada.

ABSTRACT

During 2020, a method to provide stability to an initially unstable soil excavation was required in loose saturated soil. The initial slope angle was up to 45° to the horizontal, with saturated ground causing local slip failures and running sand into the excavation. The excavation was surrounded by existing sensitive infrastructure, so it was not feasible to construct shallow excavation slopes, nor feasible to use techniques such as soil nails or anchors. It was neither feasible to use a steel sheet pile wall nor a secant pile wall. To provide soil reinforcement to enhance slope stability, a system was designed that incorporated the construction of a highly permeable angular drainage stone channel along the toe of slope, combined with aggregated cemented angular stone "wedges" compacted orthogonally to the toe of the slope. This combined drainage and reinforced soil system met the economic budget for the client. To meet Eurocode 7 design standards, including appropriate partial safety factors, a PLAXIS 3D analysis was conducted. Furthermore, to reach a fully utilized safety case where the design stabilisation resistance exceeded the design destabilisation actions, unsaturated soil properties were required to be used as input to the model. The results of the modelling were translated into design specification sequence drawings. The drawings were used to enable the construction of the drainage and reinforced slopes. This ground stabilisation was successful, with stability maintained for the required two years until the infrastructure was ultimately constructed and backfilled.

1. INTRODUCTION

The upgrade of an energy infrastructure of national significance took place in the UK in 2020. Part of this upgrade programme required the construction of a temporary battered excavation to install buried pipework infrastructure. The excavation level varied between 76.46m STN10 and 76.66m STN10, where STN10 was the site-specific datum.

One side of the excavation had exposed vertical piles, with pile caps constructed above. On three sides of the excavation, the slopes were battered at slope angles varying between 30°- 45° (Figure 1) and displayed soil instability. During an initial site visit, water ingress was observed seeping through the sides of the excavation, together with progressive slope failure in the form of running sand (Figure 2).

As it was intended that infrastructure construction would only take 12 weeks during which undrained soil conditions were in place, the excavation had no slope reinforcement. However, due to a delay in construction programme, the excavation needed to remain open for up to two years longer than the anticipated 12 weeks, so resulting in drained (effective stress) conditions occurring.

To provide stabilisation of the ground for a period of up to two years, the challenge to the civil engineers was to design a robust system to mitigate soil instability, including erosion protection. The constraints on any slope stabilisation were substantial, with traditional techniques such as sheet pile walls, king post walls, secant pile walls or soil anchors, excluded due to the potential impact on the surrounding sensitive infrastructure.

2. SITE CONDITIONS

The pipeline excavation had previously been constructed using battered slopes by a previous contractor. To the west, south and northern sides of the excavation, there were slopes which were calculated to be in the range of 30° to 45°.

However, as a consequence of the high water-table, and steep slope angle, under drained soil conditions, the combination of the low drained shear strength and high groundwater seepage face resulted in local instability and running sand (Figure 2).

The geology on the site is characterised by predominantly superficial clay soils which overlay limestone and mudstone. The clay soils contain frequent lenses of sand and silt. The water level across the site is typically one metre below ground level. The geology with the construction detail overlaid is depicted in Figure 3. All elevations are presented relative to the site datum named STN10, where +7.47m OD = 80.0m STN10.

3. GEOTECHNICAL SOIL PARAMETERS

The geological sequence, as depicted in Figure. 3, is summarised as follows:

- **Unit 0** – MADE GROUND comprising gravel sized fragments of flint & sandstone.
- **Unit 1** – Soft to firm slightly gravelly sandy CLAY.
- **Unit 2** – Firm mottled very sandy to sandy silty CLAY.
- **Unit 3** – Firm to Stiff dark grey silty CLAY.
- **Unit 4** – Weak to strong dark grey LIMESTONE.
- **Unit 5** - Extremely weak to weak thickly laminated grey MUDSTONE.

Derivation of the geotechnical design parameters for each unit are presented in the following sections. This derivation has required the incorporation of limited available site investigation, historical research, industry best practice, and authors' own judgement.

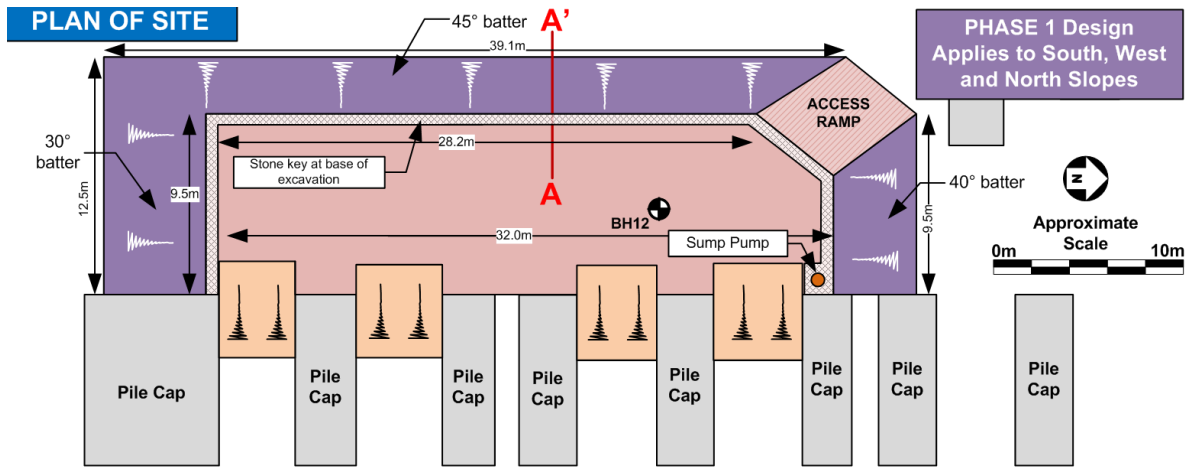


Figure 1. Site plan showing layout of the temporary excavation



Figure 2. Excavation with soil erosion and unstable sloping sides

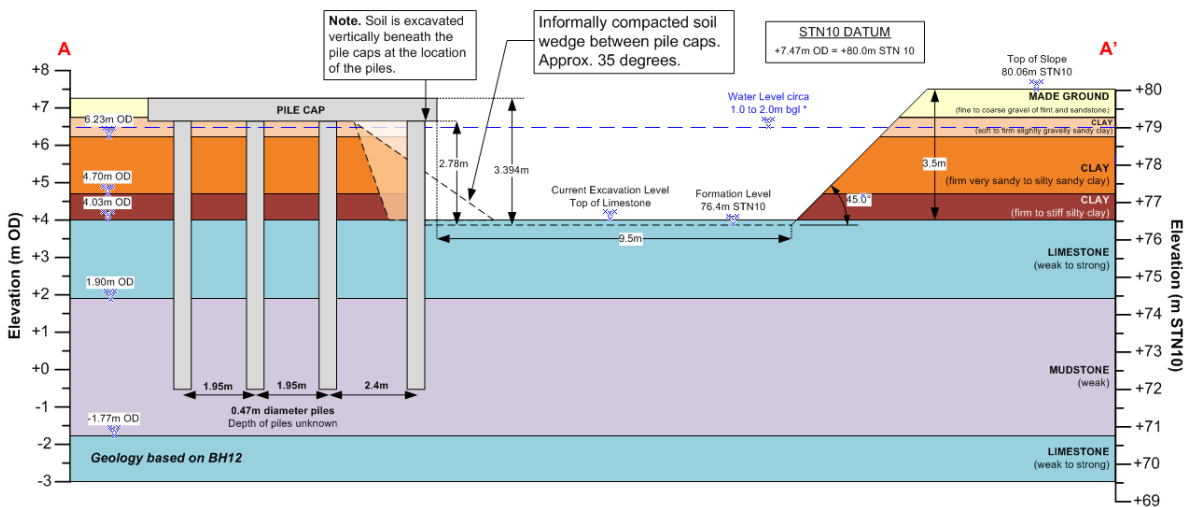


Figure 3. Conceptual model including the general excavation geometry.

3.1 Unit 0: MADE GROUND

Made Ground was described as fine to coarse gravel of flint and sandstone, from which it was considered appropriate to adopt drained parameters for this unit; with drained strength parameters presented in Table 1 used as inputs to the finite element analysis.

A conservative value of Young's Modulus has been selected (Table 1) based on a typical well-graded loose '*Sand\Gravel silty*' from Table 2 (Ref. 4. Obrzud & Truty, 2012). Poisson's ratio was selected from a table of typical values for soils (Ref. 7. Bowles, 1996).

Parameter	Characteristic Value
Weight density, γ	17 kN/m ³
Angle of internal friction, ϕ	25.0°
Effective cohesion, c'	0.0 kPa
Young's Modulus, E'	7.0 MPa
Poisson's ratio, ν'	0.2

Table 1: Geotechnical Characteristic Parameters – Unit 0 – Made Ground

USCS	Description	Loose	Medium	Dense
GW, SW	Gravels/Sand well-graded	30-80	80-160	160-320
GM, SM	Sand, uniform	10-30	30-50	50-80
GM, SM	Sand/Gravel silty	7-12	12-20	20-30

Table 2: Typical values of Young's Modulus for granular materials (MPa) (Ref. 4).

3.2 UNITS 1, 2 and 3 – CLAY Soils

The clay soils were broken down into three units based on the geological descriptions:

- UNIT 1: soft to firm dark brown slightly gravelly sandy Clay.
- UNIT 2: firm mottled orange and reddish brown very sandy to sandy silty Clay.
- UNIT 3: firm to stiff dark grey silty Clay

Weight density test data were not available for the natural clay soils, however a range between 17 kN/m³ and 21 kN/m³ was expected based on historical site investigation. Parameters in Table 2 have been selected based on this range and experience of working with similar soils.

Drained shear strength parameters for these units were selected on the basis that the excavation was expected to remain open for two further years. As shear strength data were not available to determine the drained strength parameters of the Clay, Plasticity Index (I_p) test data were used to determine undrained shear strength, together with equations outlined in British Standard: Code of Practice for Earth Retaining Systems (Ref. 3, Section 4.3.1.4).

Conservative values of Young's Modulus and Poisson's ratio were selected for each Clay unit using the typical values for soils (Ref. 7). Effective cohesion, c' is taken as 0.0kPa as suggested in British Standard: Code of Practice for Earth Retaining Systems (Ref. 3).

A summary of the characteristic parameters used for each Clay unit for the slope stability analysis are presented within Table 3.

Parameter	Unit 1	Unit 2	Unit 3
Weight density, γ (kN/m ³)	18	19	19
Angle of internal friction, ϕ'_{pk} (°)	21	21	21
Effective cohesion, c' (kPa)	0	0	0
Young's Modulus, E' (MPa)	2	5	7
Poisson's ratio, ν'	0.2	0.2	0.2

Table 3: Geotechnical Characteristic Parameters – Units 1, 2 and 3.

3.3 Units 4 and 5 – Limestone and Mudstone

Rock behaviour differs from soil as it is generally stiffer and stronger. A linear stress dependency, as obtained from the Mohr-Coulomb model is generally not sufficient. The Hoek-Brown failure criterion is a better non-linear approximation of the strength of rock, and accounts for several factors including strength, disturbance and weathered nature of the rock. Input parameters to PLAXIS 3D are as follows: Young's Modulus, E (kPa); Poisson's ratio, ν (-); Uniaxial compressive strength (UCS) of intact rock, σ_{ci} (kPa); Intact rock parameter, m_i (-); Geological strength index, GSI (-) and Disturbance factor, D (-).

Borehole descriptions suggest the Limestone is medium strong and thinly to medium bedded. The Mudstone is described as extremely weak to weak and thickly laminated to medium bedded. The selection of Young's Modulus and Poisson's ratio (Table 4) are based on typical values of Young's Modulus available in the literature (Ref 8).

Parameters	Limestone Unit 4	Mudstone Unit 5
Young's modulus, E (kPa)	1.5×10^7	1.0×10^6
Poisson's ratio, ν (-)	0.2	0.2
Uni-axial compressive strength of intact rock, σ_{ci} (kPa)	49,200	20,000
Intact rock parameter, m_i (-)	10	4
Geological strength index, GSI (-)	12	10
Disturbance factor, D (-)	0	0

Table 4: Summary of Design Rock Parameters – Unit 4 & Unit 5.

4. GROUNDWATER

Groundwater was encountered in boreholes outside the excavation at levels of c. 1m bgl, and was observed seeping into the excavation up to c. 2m above the toe of the slope. Approximately 30m away from the excavation was an area of a previous excavation which had been backfilled with sand and gravel. This area became waterlogged due to the surrounding clayey soil having a much lower permeability, and acted as a hydraulic boundary which defined the position of the water table boundary condition at a distance of 30m in the PLAXIS 3D finite element model (Ref. 6).

5. DESIGN METHOD FOR SOIL STABILISATION USING SLOPE REINFORCEMENT

The excavation was surrounded by existing sensitive infrastructure, so it was not feasible to construct shallow excavation slopes, nor was it feasible to use techniques such as soil nails or anchors. It was neither feasible to use a steel sheet pile wall nor a secant pile wall. To provide soil reinforcement to enhance slope stability, a system was designed that incorporated the construction of a highly permeable angular drainage stone channel along the toe of slope, combined with aggregated cemented angular stone "wedges" compacted orthogonally to the toe of the slope. The construction sequence is as follows:

- Install a stabilising angular stone key at the toe of the slope (Figures 4 & 9) so groundwater can seep out of the batter without ground loss due to loss of fines.
- Sump pump residual water away from sump in the corner of the excavation.
- Install cemented aggregate wedges in the slope at spacings of 2.5m. These wedges increase friction between the soil particles and prevent circular slip failure from occurring along the long west side of the excavation (Figures 4, 5 & 10).
- Place angular stone onto the remaining exposed slope face, then push into the soil and compact with the bucket of the excavator. This will mitigate the surface erosion due to rainfall. Also place the stone on the exposed surfaces of the aggregate wedges to give a uniform appearance across the slope.
- Visually monitor the embankments during and after stabilisation works to check for any ground movement.
- Regularly refer to the (ITM) Plan, and to the staged sequence of installation.

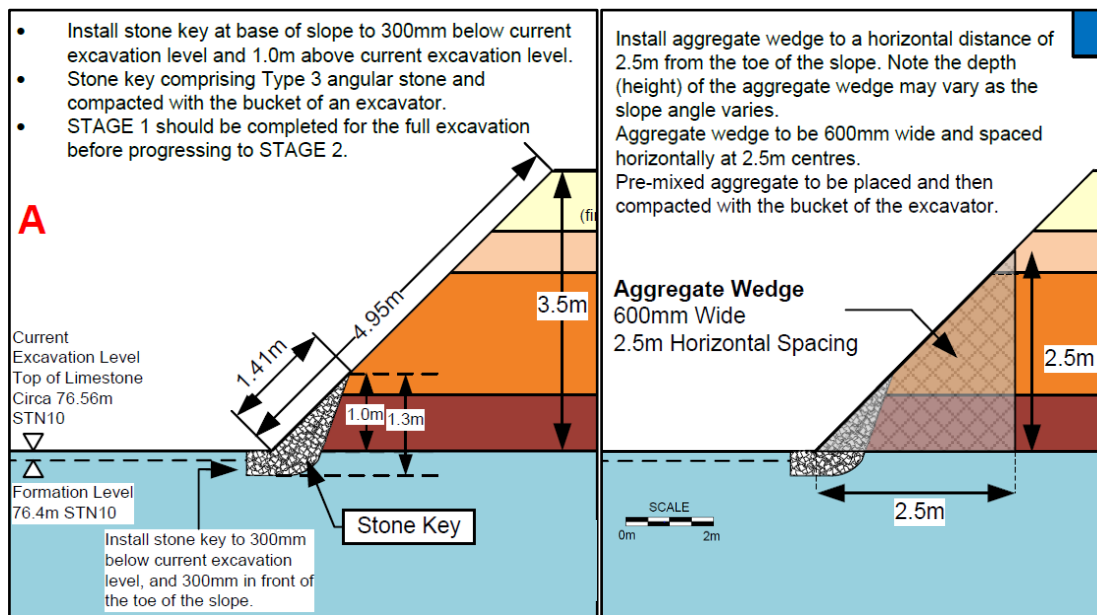


Figure 4. Cross section and inclined view of the stone key and aggregated wedges

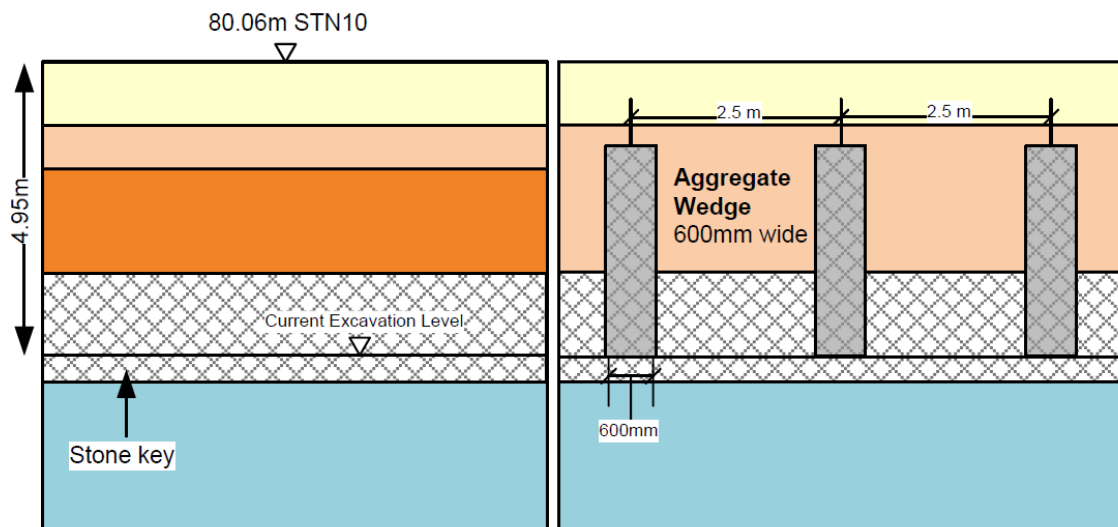


Figure 5. Cross section and inclined view of the stone key and aggregated wedges

4.1 Unit 6: Compacted Angular Stone Key used for Drained Reinforcement

The compacted angular stone key used MOT Type 3 (Figure 6) angular stone (Ref. 5), with the weight density (Table 5), selected using a table of typical values (Ref. 8).

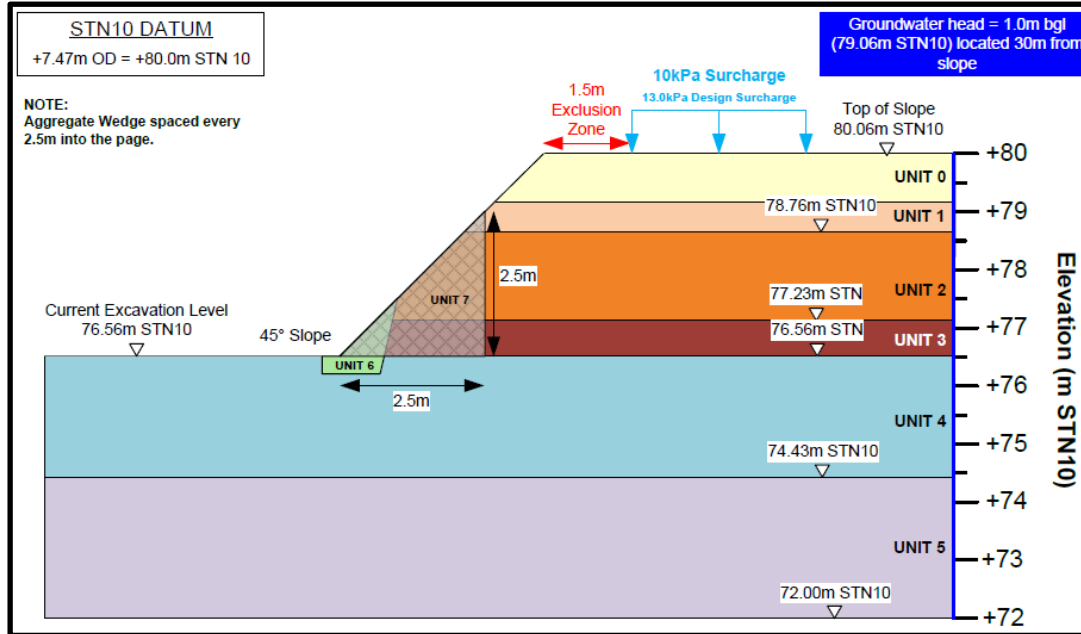


Figure 6. Conceptual model of the cemented angular stone soil reinforcement

Based on a gravel classification of 'GW - Medium Dense' Young's Modulus of compacted angular stone, was selected using values in Table 5. Shear strength was determined using the equations and tables presented in British Standard (Ref.3) Section 4.3.1.3:

Parameter	Unit 6
Weight density, γ (kN/m ³)	19
Young's Modulus, E (MPa)	100
Contribution from particle angularity ϕ'_{ang} (°)	4
Contribution from the PSD, ϕ'_{PSD} (°)	4
Effective angle of shearing resistance, $\phi'_{cv,k}$ (°)	38
Contribution from Dilatancy, ϕ'_{dil} (°)	6
Peak angle of internal friction, ϕ'_{pk} (°)	44
Effective cohesion, c' (kPa)	0
Poisson's ratio, ν'	0.2

Table 5: Geotechnical Characteristic Parameters – Unit 6 Angular Stone Key.

4.2 Unit 7 – Cemented Aggregate Wedge

Each aggregate wedge was constructed using angular stone mixed with cement. When cured, this resulted in a high strength similar to concrete. PLAXIS 3D Input parameters (Table 6) based on typical properties of concrete available in the literature (Ref. 8).

Parameter	Unit 7
Weight density, γ (kN/m ³)	21.0
Young's Modulus, E (kPa)	1.0×10^6
Poisson's ratio, ν'	0.2

Table 6: Geotechnical Characteristic Parameters – Unit 7 Cemented Aggregate Wedge

Slope stability validation was conducted in accordance with Eurocode 7 (Ref. 1 & Ref. 2), utilising the 'Geotechnical Design by Calculation' method, undertaken using PLAXIS 3D Version 2019. PLAXIS 3D (Ref. 6) is a finite element software package designed for three-dimensional analysis of deformation and stability in geotechnical engineering.

6.1 Partial factors of Safety

The following partial factors of safety were selected in accordance with relevant design standards (Ref. 3).

- Partial factor $\gamma_m = 1.25$ to reduce $\tan(\varphi')$ and c' for drained shear strength
- Partial factor $\gamma_m = 1.40$ to reduce σ_{ci} for rock
- Load factor, $\gamma_L = 1.30$ to increase destabilising actions of surcharge loading

The terminology defined in Eurocode 7, refers to the non-factored parameters as 'characteristic', with factored parameters referred to as 'design' and used in calculations.

6.2 Model geometry

The following geometry was adopted for input into the PLAXIS 3D Mathematical Model.

- Existing excavation level = 76.56m STN10
- Top of existing batter = 80.06m STN10
- Slope angle = 45°
- Cemented aggregate wedge – width = 0.6m
- Cemented aggregate wedge – height = 2.5m
- Cemented aggregate wedge – horizontal length = 2.5m
- Model width (i.e. wedge horizontal spacing) = 2.5m

6.3 Surcharge Loading and Exclusion Zone

A characteristic surcharge of 10 kPa has been applied to the top of the batter as required by Eurocode 7 (Ref. 1). The surcharge load has been specified 1.5m from the crest of the batter, meaning that there must be a 1.5m exclusion zone to plant and machinery.

6. RESULTS OF SLOPE STABILITY FINITE ELEMENT ANALYSIS VALIDATION

Numerical modelling of the slope stabilisation using the cemented aggregate wedge approach was carried out using PLAXIS 3D, based on the input values of Table 7.

Due to the symmetry between each of the aggregate wedges, the domain of the PLAXIS model needed to be only 2.5m wide. In the vertical dimension, the various geological units (Unit 0 to Unit 3) were input as horizontal layers. The slope angle was defined as 45°, with the 13kPa design loading applied downwards outside the 1.5m exclusion zone.

The compacted angular stone key (Unit 6) was simulated as an irregular shaped horizontal strip as outlined in Figure 4. The cemented aggregate stone wedge (Unit 7) was simulated as a 600mm wide triangular structure, 2.5m deep in the centre of the model (Figure 4 & Figure 6).

PHASE 1 - PLAXIS Model Input Parameters											
Material Properties											
CLAY Soils – Mohr-Coulomb Model											
UNIT	Material	γ (kN/m ³)	E' (kPa)	ν' (-)	ϕ'_c (°)	ϕ'_d (°)	c'_c (kPa)	c'_d (kPa)	k_x (m/s)	k_y (m/s)	
0	MADE GROUND (fine to coarse gravel of flint and sandstone)	17	7,000	0.2	25	20.46	0	0	1×10^{-4}	1×10^{-4}	
1	CLAY (soft to firm sandy clay)	18	2,000	0.2	21	17.07	0	0	1×10^{-6}	1×10^{-7}	
2	CLAY (firm very sandy clay)	19	5,000	0.2	21	17.07	0	0	1×10^{-6}	1×10^{-7}	
3	CLAY (firm silty clay)	19	7,000	0.2	21	17.07	0	0	1×10^{-6}	1×10^{-7}	
Partial Factors for (GEO) DA1 combination 2 failure, where $c'_d = c'_c \div 1.25$ and $\phi'_d = \tan^{-1}(\tan(\phi'_c) \div 1.25)$											
LIMESTONE AND MUDSTONE – Hoek & Brown Model											
UNIT	Material	γ (kN/m ³)	E' (kPa)	ν' (-)	σ_{elc} (kPa)	σ_{eld} (kPa)	m_i	GSI _c	GSI _d	k_x (m/s)	k_y (m/s)
4	LIMESTONE (weak to strong)	20	20,000	0.2	49,200	35,140	10	12	9.6	1×10^{-6}	1×10^{-7}
5	MUDSTONE (weak)	20	20,000	0.2	20,000	14,290	4	10	8	1×10^{-6}	1×10^{-7}
Partial Factors for (GEO) DA1 combination 2 failure, where $\sigma_{eld} = \sigma_{elc} \div 1.4$ and $GSI_d = GSI_c \div 1.25$											
COMPACTED STONE KEY											
UNIT	Material	γ (kN/m ³)	E' (kPa)	ν' (-)	ϕ'_c (°)	ϕ'_d (°)	c'_c (kPa)	c'_d (kPa)	k_x (m/s)	k_y (m/s)	
6	COMPACTED ANGULAR STONE KEY	20	100,000	0.2	44	37.69	0	0	1×10^{-3}	1×10^{-3}	
Partial Factors for (GEO) DA1 combination 2 failure, where $c'_d = c'_c \div 1.25$ and $\phi'_d = \tan^{-1}(\tan(\phi'_c) \div 1.25)$											
CEMENTED STONE COLUMN											
UNIT	Material	γ (kN/m ³)	E' (kPa)	ν' (-)	k_x (m/s)	k_y (m/s)					
7	CEMENTED AGGREGATE WEDGE	20	1.0E6	0.2	1×10^{-9}	1×10^{-9}					

Table 7: Geotechnical and Groundwater Parameters input to PLAXIS 3D (Ref. 6)

Figures 7 & Figure 8 depict the contours of simulated soil displacement (i) based on the working design loads, and (ii) when loads increased until simulated failure is reached.

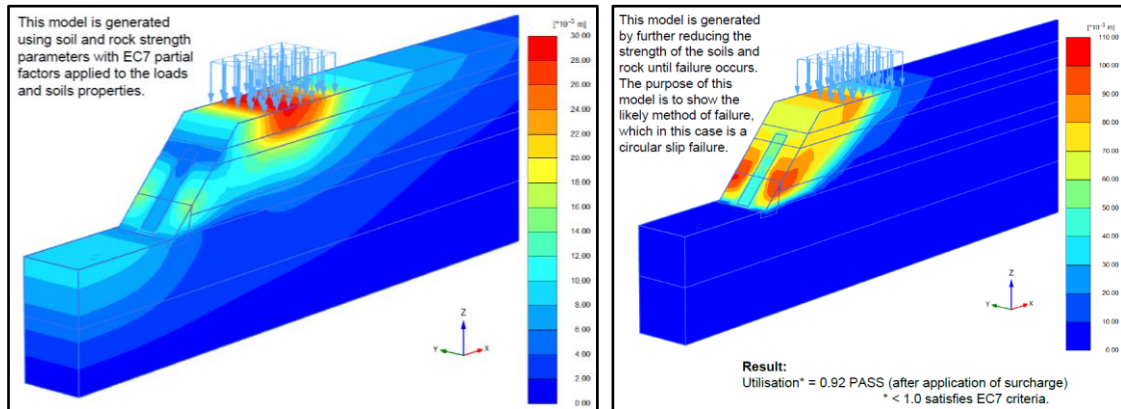


Figure 7. Displacement at working design load. Figure 8. Displacement at model failure

The Plaxis 3D model evaluates the stabilizing design parameters required to produce sufficient forces to resist the destabilizing design actions just prior to model failure. The model then calculates the utilisation factor **U**, from the equation E_d / R_d where:

E_d is the design value of the effect of destabilising actions i.e. loads & stresses.

R_d is the design value of the stabilising resistance to actions, i.e. the maximum load/stress that the structure can support before failure.

To demonstrate stability, Eurocode 7 requires the Utilisation, **U**, to be less than 1.0. For the parameters and partial factors described in Table 7, PLAXIS calculates **U = 0.94**

7. CONSTRUCTION METHODOLOGY

The design process resulted in the completion of a series of critical documents required to ensure (i) the construction fulfils the design objectives to stabilise the excavation slopes for a period of at least two years, (ii) there is no collateral damage to surrounding infrastructure, and (iii) the construction is conducted to defined safety standards.

These critical documents included:

- (a) Design Strategy document, including all relevant calculations
- (b) Clear construction sequence drawings
- (c) Design Risk Assessment (DRA)
- (d) Inspection, Testing & Monitoring (ITM) Plan
- (e) Signed design certificates
- (f) Insurance certificates

All site personnel were required to be suitably trained, qualified and certificated to work in this sensitive site. In summary, the construction proceeded safely, with the excavation producing safe and stable exposed slopes for more than the two-year duration required.



Figure 9. Construction of angular stone key. Figure. 10. Construction of aggregate wedge

8. REFERENCES

- Ref. 1 Eurocode 7 BS EN 1997-1:2004 'Geotechnical Design Part 1 General Rules Feb 2009'
- Ref. 2 Eurocode 7 NA to BS EN 1997-1:2004, 'UK National Annex to Eurocode 7: Geotechnical Design – Part 1: General Rules'.
- Ref. 3 British Standard 'Code of Practice for Earth Retaining Structures' Ref: BS 8002:2015.
- Ref. 4 Obrzud R. & Truty, A. The Hardening Soil Model – A Practical Guidebook Z soil. Ref: 100701 report, Date: 31.01.2012
- Ref. 5 SIMTEC Material Testing, 'Gradation Analysis Test Report Type 3 Subbase'
- Ref. 6 PLAXIS 3D (2019) Material Models Manual
- Ref. 7 Foundation Analysis and Design (5th Edition)', J.E. Bowles, 1996 McGraw-Hill.
- Ref. 8 Some Useful Numbers on the Engineering Properties of Materials' <https://www.jsq.utexas.edu/tyzhu/files/Some-Useful-Numbers.pdf>.