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GEOTECHNICAL
REPORT

job number	C3485/23/E/5292	date	14.07.23

site address	Former Quarry off Low Lane
	Draughton
	Skipton
	North Yorkshire BD23 6EA

written by	R.A. Palmer	checked by	I. Sakoor

issued by	R.A. Palmer

Please consider the environment before printing this report.

<ENVIRONMENTAL><GEOTECHNICAL>

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Appendices

Report on a Geotechnical Investigation

Location:	**Former Quarry off Low Lane** Draughton, Skipton, North Yorkshire BD23 6EA		
For:	Skyraikes Ltd		
Report No.	C3485/23/E/5292		
Version:	Final.v1	Report date:	14.07.23
Previous Versions	Draft.v1, dated 05.07.2023		

For and on behalf of **Rogers Geotechnical Services Ltd**

[redacted]

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1. Introduction

The land associated with the former limestone quarry off Low Lane, Draughton is to be developed by the construction of two residential properties. The quarrying operations have resulted in steep slopes adjacent to and below each plot. The site comprises two different plots; a low lying area at the base of the slopes associated with the quarry floor and a separate plot at the top of one of the slopes. It is understood that as part of the planning application, pre-commencement conditions have been raised by the local authority requesting that the stability of the slopes are inspected prior to construction.

Consequently, a site investigation has been undertaken in accordance with the instruction from the client in order to determine the nature of the underlying soils, to assess their engineering properties, to assist in the design of safe and economical foundations for the proposed development and to also assess the stability of the slopes. This report describes the work undertaken, presents the data obtained and discusses the ground conditions in relation to the proposed works.

2. Limitations

The recommendations made and opinions expressed in this report are based on the ground conditions revealed by the site works, together with an assessment of the site and of the laboratory test results. Whilst opinions may be expressed relating to sub-soil conditions in parts of the site not investigated, for example between investigatory locations, these are for guidance only and no liability can be accepted for their accuracy.

This report has been prepared in accordance with our understanding of current best practice. However, new information or legislation, or changes to best practice may necessitate revision of the report after the date of issue. It should be appreciated that this report only considers the geotechnical aspects of the site and not environmental or ground gas (carbon dioxide, methane or radon) issues.

3. Previous Investigation

An initial site investigation report was issued by RGS in August 2016, report ref J3571/16/E. In that investigation, a series of windowless sample boreholes, dynamic probes and machine excavated trial pits were undertaken. The report refers to two plots: Plot 1 is situated at a higher elevation i.e. top of the slope and Plot 2 is located at the toe of the slope.

The report states that approximately 2m of ground will be removed from the area of Plot 1 to create a level platform. As a result of this material being removed, it is specified that shallow foundations can be utilised, with suitable bearing strata present at 1m depth from the proposed formation level. With regards to Plot 2, the report states that the finalised formation level will be increase by up to 1m from the existing quarry floor level. Moreover, the conclusions are tentative in confirming whether the true presence of rockhead was identified within the investigation, or whether termination of the boreholes was actually due to boulder and cobbles associated with quarry fill. Therefore, drilled piles were recommended as an appropriate foundation solution for Plot 2.

Subsequent to the above report, a slope stability assessment was then issued by RGS, dated March 2017 with the same report reference as above – see Appendix 7. The report utilised estimated geotechnical parameters for the slope analyses. It was concluded that some slopes could be unstable in the long term and thus remedial measures were recommended.

4. Site Visit

A site visit was completed on the 4th of May 2023. The client provided a 14T excavator, which allowed a series of trial pits and trenches to be excavated. Trial pits were excavated in order to reveal the nature of the near surface soils using a tracked excavator. The soils were logged on site in general accordance with BS5930: 2015+A1: 2020. At regular intervals throughout the excavation of the pits, samples were taken for geotechnical testing. In two of the trial pits, soakaway infiltration tests were undertaken broadly in line with BRE365 to assess the potential drainage characteristics of the soils on site.

In addition to the above, a CBR test via plate load method was completed. Moreover, the numerous rock exposures on site were assessed by the on-site engineer which allowed rock descriptions and dip/strike data to be collected.

The data collected form the above activities is presented as follows:

Site Plans	- Appendix 1
Trial Pit Records	- Appendix 2
Soakaway Test Result Sheets	- Appendix 3
CBR Test Result Sheets	- Appendix 4
Site Photographs and Notes	- Appendix 5

5. Geology

The available published geological data for the site has been examined and the following table presents the anticipated geology:

Table 1: Geological Data for the Site

Strata Type	Strata Name[1]	Parent Group[2]	Description[2]
Superficial Geology	-	-	None indicated beneath the site surface.
Solid Geology	**Northern Section**		
	Hodder Mudstone Formation	Craven Group	Predominantly grey to dark grey mudstone, with subordinate and variable detrital limestone, siltstone and sandstone.
	Southern Section		
	Pendleside Limestone Formation	Craven Group	Grey, fine- to coarse-grained, bioclastic, commonly graded, erosive-based and bioturbated cherty packstones, interbedded with wackestone, sporadic intraformational and extraformational limestone conglomerate.

It should be appreciated that limited geological mapping has occurred within the Draughton area. Indeed, the solid geology beneath the site is not been presented on the New Series of BGS maps, whether this be 1:10000, 1:25000 and 1:50000. However, a review of the 1889 1:63360 map for Pateley Bridge reveals that the solid geology for Draughton and the surrounding areas has been subject to significant structural movement and folding. There are numerous dip indicators in the area suggesting that the solid geology can be dipping from near-horizontal to up to 82° in either a northern or southern direction over relatively short distances.

It would appear that the material which has been quarried on site is associated with the Pendleside Limestone Formation. It is apparent that there are numerous quarries associated with this limestone formation in the area.

[1] Sources: British Geological Survey (NERC) 1:63360 Sheet 61; Pateley Bridge Solid Edition, and GeoIndex Onshore [*online resource from www.bgs.ac.uk*]
[2] Sources: British Geological Survey (NERC) Lexicon of Named Rock Units [*online resource from www.bgs.ac.uk*]

6. Strata Conditions

In accordance with the geology of the area, the succession has been shown to include the following:

Table 2: Generalised Strata Profile

Depth m below ground level to underside of layer	**Strata Type**	**Positions Layer Revealed**	**Groundwater Strikes** m below ground level
0.1 – 0.5	TOPSOIL	All	-
0.9 – +1.85	MADE GROUND (Cohesive)	SA01, TP04A	-
1.1 – 2.1	MADE GROUND (Granular)	SA02, TP02A, TP03A, TP04A	-
1.0 – 1.3	Clayey silty sandy GRAVEL. Occasional cobbles and boulders.	TP01A-1, TP01A-2	-
+0.25 – +2.3	LIMESTONE (Pendleside Limestone Formation)	TP01A-1, TP01A-2, TP01A-3, TP02A, TP03A, TP04A	-

'+' denotes that the strata extended below the termination depth of the investigated positions, thus the extent of the deposit is only proven to the depths indicated.

6.1 General Strata

Trialpits TP03A and TP04A have confirmed that the refusals met in the previous investigation within the quarry floor area are indeed the limestone rockhead, as opposed to buried boulders and cobbles. The quarry floor comprises a capping of topsoil, beneath which both granular and cohesive quarry spoil is present.

The trialpits upon the slopes revealed a thin capping of topsoil, beneath which granular soils comprising clayey silty sandy gravel of limestone and mudstone was present. Limestone was then revealed beneath the granular soils. It should be noted that the limestone rockhead became shallower when progressing upslope. Indeed, limestone outcropped towards the top of the southern slope.

6.2 Groundwater

No distinct groundwater strikes were recorded or observed during the investigation.

7. Insitu Testing

7.1 Soakaway Tests

On reaching the elected soakaway test depth, the pit was trimmed and squared as much as practicable. Water was then introduced into the pit at a controlled rate to prevent collapse of the sides and the level monitored at time intervals relative to a reference bar at ground level. The results obtained from the soakaway tests are presented in Appendix 4 and are summarised below:

Table 3: Soakaway Test Results

Location	Soakage Area Dimensions (average) (m)	Depths of soaked strata (m)	Soil Description (of soaked strata)	Infiltration Rate (m/sec)	*Drainage Characteristics
SA1	0.7 x 1.7	1.445 to 1.85	Slightly sandy slightly gravelly CLAY	-	Practically Impermeable
SA2	0.7 x 1.8	0.94 to 1.33	Very clayey silty sandy GRAVEL	1.6×10^{-5}	Good
				2.2×10^{-5}	
				1.7×10^{-5}	

7.2 CBR Tests

As a consequence of the testing, CBR values have been derived in accordance with IAN 73/06 (HD25) Feb '06. The stated procedure utilises the constant penetration plate test method, in which, the force taken to mobilise the plate through 1.25mm of penetration is used to calculate the CBR. These results are presented in the following table:

Table 4: Summary of Results

Location	%CBR
CBR01	9%

8. Laboratory Testing - Geotechnical

The following programme of laboratory testing has been undertaken on samples obtained during this investigation:

Determination of water content	BS EN ISO 17892-1:2014
Determination of particle size distribution	BS EN ISO 17892-4:2016: 5.2
Sedimentation by pipette	BS EN ISO 17892-4:2016: 5.3 – 5.4
Determination of dry density/mc relationship	BS1377-1:1990: 4 - 3.4 & 3.6
Large shearbox	BS1377-7:1990: 5

The test results are presented in Appendix 6 and are summarised below:

Table 5: Summary of Geotechnical Test Results

Test Type	No.	Range of Results		Comments
Moisture content determinations	2	9.8% & 18%		
Particle size distribution (Wet sieve and sedimentation)	4	Gravel Sand Silt/Clay	11% & 49% 23% & 27% 24% & 66%	Samples from quarry base.
		Cobbles Gravel Sand Silt Clay	04% & 11% 53% & 69% 12% & 17% 09% & 11% 06% & 08%	Samples from quarry slopes.
Compaction (2.5kg rammer)	2	MC	8.1% & 11% 1.85 & 1.99 Mg/m^3	
Compaction (4.5kg rammer)	1	MC	8.5% 2.07 Mg/m^3	
Shearbox (Large)	1	Peak θ’	32.5°	Dry density – 1.86 to 1.95Mg/m^3 Testing carried out on material passing 20mm. PSD revealed this to be 45% of the soil sample – coarse particles have therefore not been included within the test. As such, value may be conservative.

8.1 Geotechnical Parameters

The idealised geotechnical properties to be employed in design are summarised below:

Table 6: Geotechnical Properties & Estimated Effective Stress Parameters

Description	PI (%)	Bulk unit weight, (kN/m^3)[4]	Angle of friction, ’ (°)	Effective cohesion, c’ (kN/m^2)
Plot 1 Material (Slightly gravelly silty CLAY)	29[3]	20	25 [4]	0
Plot 2 Material (Clayey silty sandy GRAVEL)	n/a	19	33 to 37 [5]	0
Compacted Granular Backfill (No clay or silt content)	n/a	20	45	0

9. Discussion of Ground Conditions - Geotechnical

9.1 Plot 1

It is understood that the development plans still include for the removal of material at this location, with up to 2.5m of material to be excavated to reduce site levels to the desired formation level. As such, it is considered that the foundation recommendations prepared within the RGS geotechnical investigation report from August 2016 remain valid and shallow foundations will be suitable once the site level is reduced.

[3] Utilising laboratory data from the 2016 investigation.
[4] Sources: Carter, M. and Bentley, S.P. (1991). Correlations of Soil Properties. London: Pentech Press.
[5] Based on shear box results.

It should be appreciated that soakaways are unlikely to be viable in and around this area due to the cohesive nature of the underlying soils. As such, a different form of surface water drainage should be sought for this plot.

9.2 Plot 2

Whilst site levels are to be reduced at the top of the slope, site levels shall instead be raised at the toe of the slopes (base of the quarry). Indeed, it is understood that the quarry floor shall be raised by 1.25m.

The detached property associated with Plot 2 is to incorporate a basement, therefore an allowance of 2.6m in height has been assumed for this structure. As such, in view of the uplift of material, the floor of the basement shall be 1.35m below the existing ground level. With reference to trial pit log TP04A which reveals that limestone is currently present at depths of up to 1.5m, this suggests that the floor of the basement shall be approximately 0.15m above the competent limestone or upon the rock itself even once site levels are uplifted. However, with reference to TP03A, it is evident that the level of the rock can be variable. For instance, in areas where the rock extends to greater depths, for instance at TP03A this was around 2.1m depth, the basement floor would then be 0.75m above the limestone.

In areas where the basement isn't present, the near surface floor could be some distance above the competent rockhead e.g. below the garage. For instance, where ground conditions similar to TP03A prevail, any structures at surface would be approximately 2.75m above the competent rock.

In view of the above, the following foundation solutions should be considered to support the basement and dwelling:

1) Install deep footings to the limestone rockhead from the underside of the basement floor and near surface structures.
2) Excavate the fill beneath the property, reinstate in an engineered manner and install a raft foundation and ground bearing floor slab.

9.2.1 Strip & Pier Footings

Whilst the made ground appears to be present in a relatively competent insitu condition, there remains a potential for excess differential settlement. Therefore, if the quarry fill is to be left insitu, then foundations will need to penetrate through this material to the competent limestone rockhead which has been observed at depths between 1.5m and 0.9m below the existing ground level at TP04A. As discussed above, when assuming rock head at 1.5m depth, the basement floor shall be present approximately 0.15m from above the limestone. Thus, strip footings would have to extend down 0.15m from the base of the basement floor – this could effectively create an edge-thickened raft. Clearly where the limestone is shallower, then a pragmatic approach can be undertaken and a traditional foundation may not be required as basement floor could be position on the rock itself. Conversely, should the rockhead level deepen in any areas, then footings will need to be depended accordingly.

It is understood that the majority of the property shall incorporate a basement, however it is understood that this shall not extend beneath the garage. Therefore, the garage will likely require deep strip footings. Alternatively, a pier foundation could be considered for the near surface structures. Using this approach, the ground would be excavated to rock and precast concrete rings would then be placed vertically on top of each other in order to form a circular column up to ground level. It would be necessary for the lowest ring to have a base or be filled with concrete in order to distribute foundation loads.

The limestone should possess a significant bearing capacity, probably being in excess of 250kN/m^2. Therefore, at a typical foundation load for a house the factor of safety against general shear failure will be high, probably exceeding 10. In addition, it is considered that nominal settlements will occur under the action of the proposed load.

It is considered that a ground bearing ground floor slab could be utilised for the basement. In this instance it would be necessary to compact the sub-grade using a vibrating roller to ensure the near surface soils are adequately compacted.

9.2.2 Raft Footings

It cannot be recommended that a raft foundation be constructed directly within the made ground without any prior treatment to the existing fill. These soils could be present in a weak and variable condition such that excessive total and or differential settlement could occur under moderately light surface loading. To incorporate a raft footing, which would ultimately form the basement floor, the made ground present beneath the proposed footprint could be excavated to expose the limestone rock head. The resulting excavation would then be filled with approved and compacted material such that a raft foundation could be constructed at the required depths. In principal this method of construction is considered viable, provided that:

- Testing of the source material (grading and dry density moisture content relationship) be undertaken to assess its suitability – this could possibly include the material which is excavated and has been tested during the current investigation.
- All of the weak ground is removed during excavation to reveal the underlying limestone.
- The excavation faces are battered to a slope not exceeding 45°.
- A suitable granular fill is brought up and compacted in layers not exceeding say 300mm.
- Each layer is subject to insitu density testing to ensure that it has been adequately compacted.

If the works are undertaken with due diligence then it is considered that settlement of the completed structure will be well within tolerable limits.

9.2.3 Retaining Walls & Basement Floor

It should be appreciated that in order to form the basement, the construction of retaining walls will be required. As the investigation was undertaken during a prolonged dry spell of weather it is possible that potential groundwater has not been observed. Indeed, during the wetter months, seepages of water could enter the excavation, particularly as granular soils will be present. In view of this, it should be assumed in any design that groundwater is actually present at ground level in the worst case. This is due to the potential for run-off entering any granular surround to the structure. Therefore, design should take into account the potential for up-thrust forces; 25kN/m^2 could be adopted when assuming a 2.6m excavation. It would also be prudent to account for the presence of groundwater for any retaining walls on site with a granular backfill.

In view of the granular nature of the near surface soils, it is not anticipated that the soils within the excavation will remain at a vertical angle in the short term. Therefore, it is reasoned that an angle of 45° or less should be adopted for a temporary excavation within the soil. As such, considering that the construction of this feature will reach a depth of 2.6m, it has been estimated that by battering the top of the excavation sides to an angle of 45° to 2.6m depth, then around 2.6m of landtake would be required around the basement area. The landtake would be reduced should construction of the basement walls precede the uplifting of the external areas.

In any event, it is strongly recommended that the construction of the basement walls is undertaken in short sections, in order to reduce the amount of time that an excavation is open. It should be appreciated that this method could require the removal of a large volume of material on the outer sides of the retaining walls, which should be reinstated or replaced ideally as an engineered fill in well-compacted layers. The materials which are placed should be accounted for in any retaining wall design. For reference, typical parameters have been estimated above in Table 6.

9.2.4 Excavations

If excavations are required to stand open for any period of time then a blinding layer of lean-mix concrete should be placed in the excavation bases. This expedient will reduce softening or loosening of the sub-grade due to the ingress of surface water.

If groundwater becomes prevalent, seepages could be controlled using a simple form of de-watering. Such a system could include the excavation of sumps from which the water could be pumped. However, it would be prudent to excavate some deep trialpits at the time the works are to start in order to establish if groundwater is likely to represent a significant issue during construction.

It is re-iterated that the stability of the excavation faces cannot be guaranteed thus temporary support to the excavation faces or battering the faces to a safe slope angle will be required. Under no circumstances should operatives be allowed to enter unsupported excavations.

9.2.5 Soakaways

Typically soakaways are not recommended in made ground due to the potential risks associated with mobilising contamination. However, the made ground at this site represents quarry spoil which mostly comprises reworked natural materials. The soakaway test undertaken within the vicinity of Plot 2 showed relatively good infiltration rates. As such, soakaways may possibly be adopted within the vicinity of Plot 2, albeit it is recommended that they are placed a sufficient distance away from the building. Indeed, the presence of soakaways close to the building could affect the engineering properties of the fill beneath the plot once placed. Placing soakaways too close to engineered fill can lead to collapse compression which can result in differential movement of the building.

9.3 Slope & Rock Face Stability

9.3.1 Rock Face Assessment

As discussed in Section 5 above, the published geological data for the site is limited, but it is evident that the quarrying will have targeted the Pendleside Limestone Formation. Numerous rock outcrops are present on the slopes surrounding Plot 2 and these have revealed that the solid geology has been subject to significant stresses which has altered the geological structure. Indeed, it appears that there is an anticline structure present on site, with the axial trend running roughly east to west. Consequently, the geology within the northern slope is dipping to the north, whereas the geology within the southern slope is dipping to the south. This has resulted in the rock dipping into the slope in these slopes, as opposed to out of the slopes. It is anticipated that these geological dips will aid in rock mass stability, as the rock is unlikely to slide along the bedding planes and out of the slope.

No distinct loose blocks were recognised during the investigation. However, it may be that some localised scaling is required to accommodate development and potential tree removal. It may be prudent to have an engineer supervise such works, if required.

9.3.2 Slope Stability

The previous slope stability report concluded that the slopes at the site were unstable as they were revealed to have factor of safety values of less than 1. The report concluded that slopes could be battered back to 30° or soil nails could be employed. Interestingly the estimated parameters utilised within the initial slope assessment correlate well with the testing results within this investigation. As such, credence can be given to the previous slope stability assessment as the parameters utilised are considered to be suitable.

The original slope assessment utilised a topographical profile which suggested that the slopes typically have a steep section towards the top of the slope which transitioned into a shallower slope towards the base. With reference to the appended photographs, it can be seen this is true. It should be appreciated that in some areas, the steeper sections of the slope comprise rock; this can be observed visually from the base of the slopes and are referenced on the site plans as 'rock outrops'. However, in some other areas, the steeper sections are covered by topsoil; reference should be made to the areas of 'terracing' highlighted on the site plans in Appendix 1.

Nonetheless, the series of trialpits referenced TP01A have revealed the make-up of the slope and confirm that the upper steeper section comprises 0.2m topsoil, beneath which competent limestone is present. This thin capping of topsoil has likely been subject to soil creep over the years and is an indicator of why the terracing structure has formed. It may be that topsoil was once present on the other rock outcrop faces, but said topsoil may have been subsequently been removed, either by soil creep processes or during quarrying operations. The rock faces typically have angles ranging between 46° and 50°, but can reach near vertical angles in some areas.

The original investigation indicated that the material comprised made ground comprising slightly sandy gravelly clay with cobbles, grading to cobbles and boulders of limestone and sandstone with much sandy gravelly clay. With reference to TP01A-1 and TP01A-2, which were completed within the vicinity of TP01 from the previous work, the make-up of the lower section of the slope is now clearly known.

It is evident that quarry spoil forms the lower sections of the slopes, and with reference to the geotechnical testing, such soils typically comprise clayey, silty, sandy gravels with occasional cobbles and boulders of limestone and mudstone. It is anticipated that these soils have not been subject to the slope failure in the past and the topography that is present now is likely to be how the spoil was placed immediately after quarrying.

In view of the above, it is anticipated that the upper steeper slopes are likely to remain stable, albeit spalling may occur through soil creep associated with the thin capping of topsoil on the rock faces. Given the consistent dip of the geology in to the rock faces, an engulfing failure within the rock mass is unlikely. Notwithstanding this, the lower slopes are considered to be unstable. Indeed, the observations and geotechnical testing results within the current investigation correlate with the previous slope assessment at the site. As such, remedial measure shall be required on the lower shallower slope areas.

9.3.3 Slope Remediation

It is understood that the client intends to retain the slopes at the current angles. Therefore, battering the slopes to 30° is not going to be considered as a remedial option.

In view of this, for the lower shallower sections of the slopes, it is anticipated that soil nails/ground anchors shall provide the most cost-effective method; a site plan is appended which indicates which areas require remediation. If utilised, care must be taken to ensure that soil nails are installed to beyond the potential failure slip circle. As such, the advice of specialist contractors will be required to assess the suitability of this method and, if deemed suitable, determine the length and spacing of the soil nails. The shallow presence of rockhead and the potential for cobbles and boulders within the fill must be considered. Indeed, this may preclude the use of certain types of anchors. Moreover, the presence of any trees to remain on site will need to be accounted for.

It may be prudent to utilise a geomat or geogrid textile system for the steeper sections of the site where soil creep may occur. Such a system may also be utilised in the shallower areas in combination with a ground anchor system, but the viability of this would need to be discussed with a specialist contractor.

10. Recommendations for Further Work

- This report should be forwarded to the relevant authorities as soon as practicable to ensure they have sufficient time to review and discuss any issues.
- Discussions with ground work contractors in relation to the possible construction issues and requirement for additional laboratory and insitu testing.
- Discussions with soil nail and ground anchor contractors with regards to the proposed remedial measures.
- Detailed design of the sub-structure.

Clearly Rogers Geotechnical Services Ltd would be happy to offer advice with respect to the above and assist where necessary.

11. References

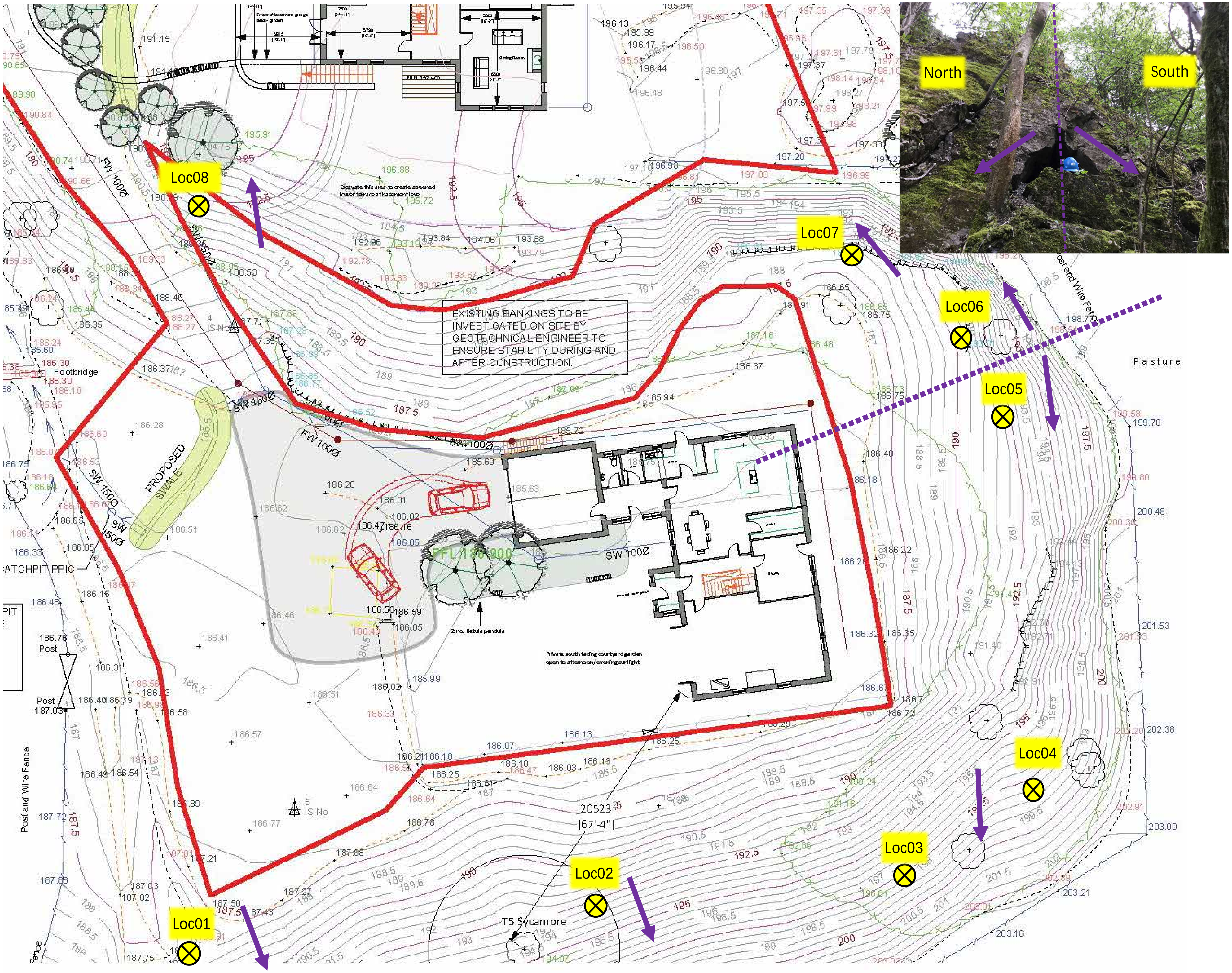
- British Standards Institution (1990) BS1377: *British standard methods of test for soils for civil engineering purposes,* B.S.I., London.

- British Standards Institution (2015) BS5930: *Code of practice for site investigations*, B.S.I., London.

- British Geological Survey (NERC) (2023), BGS, Keyworth.
  - Geology of Britain Viewer: (*http://maps.bgs.ac.uk/geologyviewer_google/googleviewer.html)*
  - Lexicon of Named Rock Units: (*http://www.bgs.ac.uk/lexicon/)*

Appendix 1

Site Plan



Notes:

Purple arrows indicate dip of solid geology. Dotted line is approx orientation of anticline axis.

Loc01 074/52°SSE
Loc02 083/44°S
Loc03 078/30°S
Loc04 078/30°S
Loc05 048/21°S
Loc06 096/78°NW
Loc07 068/52°NW
Loc08 074/34°N

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Client:

Richard Howson

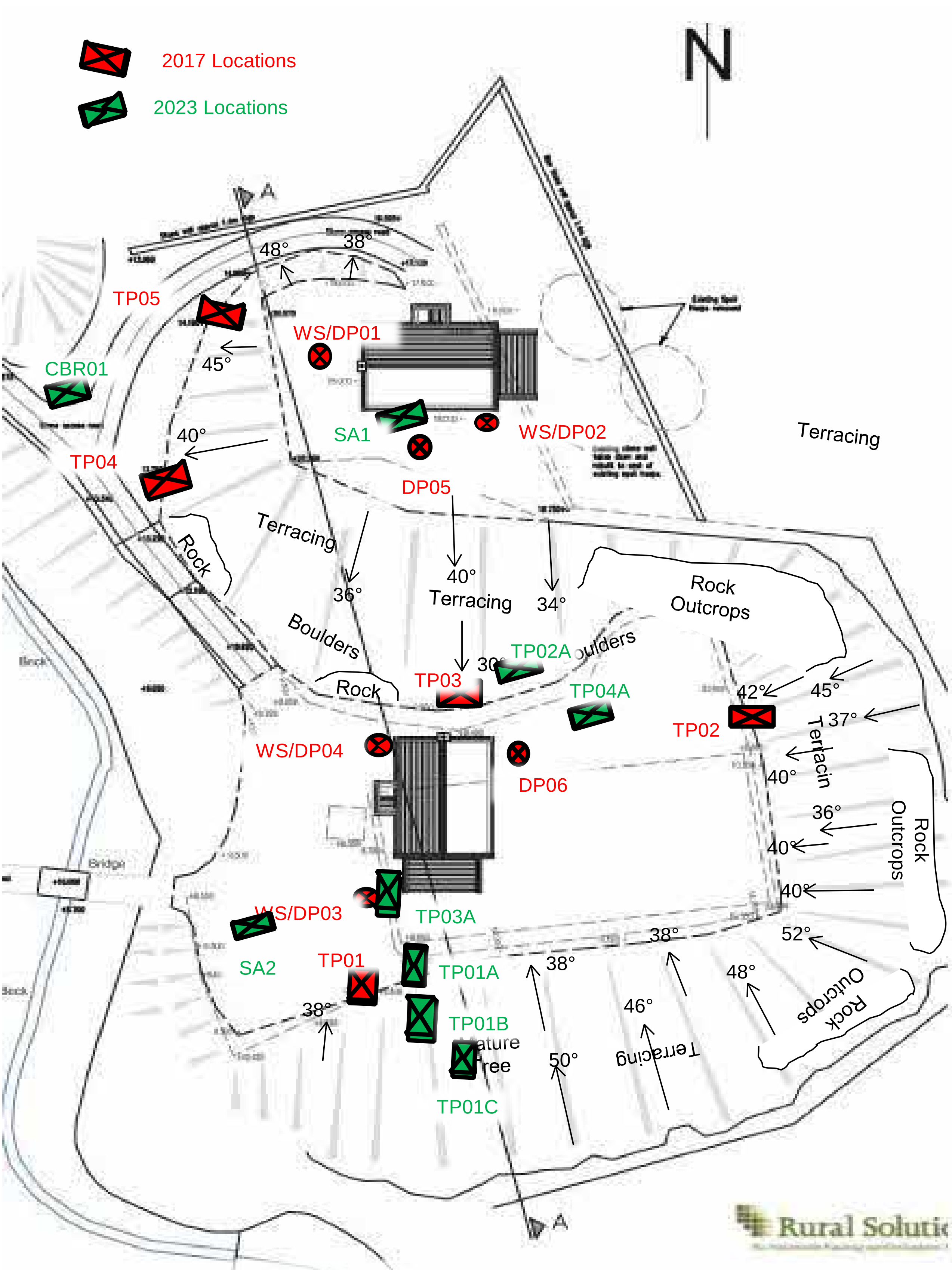
Job Number:

C3485/23/E

Project Details:

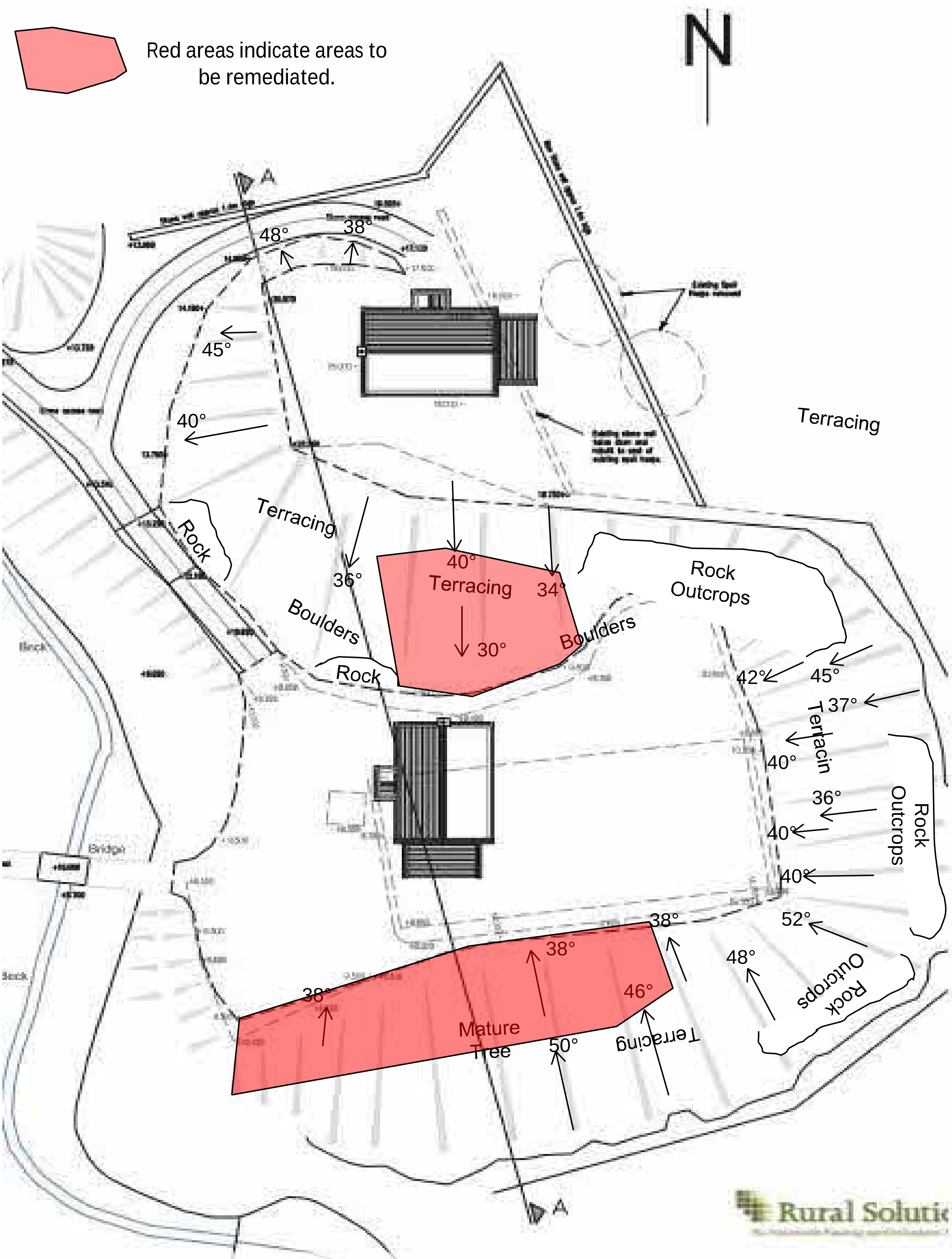
Draughton Quarry

Scale:	Not to scale - reference only



Plan not to scale and investigation positions approximated from site operative's notes.
It should be appreciated that the base map is from former plans.
However slope angles are current.

Title:	**Investigation Location Plan - Combined Locations (2017 & 2023)**	
Rogers Geotechnical Services Ltd	Site Name: **Draughton House, Draughton, Skipton BD23 6EA**	Job No: **C3485/23/E**



Plan not to scale and investigation positions approximated from site operative's notes.
It should be appreciated that the base map is from former plans.
However slope angles are current.

Title: **Slope Remedial Location Plan**		
Rogers Geotechnical Services Ltd	Site Name: **Draughton House, Draughton, Skipton BD23 6EA**	Job No: **C3485/23/E**

Appendix 2

Trial Pit Records

Trial Pit Log

Trialpit No	SA01	Sheet 1 of 1		
Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: -	Level:	Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.7 × 0.7	Depth 1.85	Scale 1:50
Client: Richard Howson				Logged RAP

Water Strike	Samples and In Situ Testing			Depth (m)	Level (m)	Legend
	Depth	Type	Results			
				0.20		
				1.85		

Remarks: .

Stability: Good

Trial Pit Log

Trialpit No	SA02	Sheet 1 of 1	
Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.8 × 0.7 Depth 1.33	Scale 1:50
Client: Richard Howson			Logged RAP

Water Strike	Samples and In Situ Testing: Depth	Type	Results	Depth (m)	Level (m)	Legend
				0.50		
				1.33		

Remarks: .

Stability: Good

Trial Pit Log

Trialpit No	TP01A-1	Sheet 1 of 1		
Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Date 04/05/2023	
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.2 x 0.7; Depth 1.10	Scale 1:50	
Client: Richard Howson			Logged RAP	

Water Strike	Samples and In Situ Testing			Depth (m)	Level (m)	Legend
	Depth	Type	Results			
				0.20		
				1.00		
				1.10		

Remarks: Refusal on bedrock; 14T excavator.

Stability:

Trial Pit Log

Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Trialpit No **TP01A-2** Sheet 1 of 1
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1 × 0.7 Depth 1.40	Date 04/05/2023
Client: Richard Howson			Scale 1:50 Logged RAP

Water Strike	Samples and In Situ Testing: Depth	Type	Results	Depth (m)	Level (m)	Legend
				0.20		
				1.30		
				1.40		

Remarks: Refusal on bedrock; 14T excavator.

Stability:

Trial Pit Log

Trialpit No	**TP01A-3**	Sheet 1 of 1	
Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 0.8 × 1.2 Depth 0.25	Scale 1:50
Client: Richard Howson			Logged RAP

Water Strike	Samples and In Situ Testing			Depth (m)	Level (m)	Legend
	Depth	Type	Results			
				0.20 0.25		

Remarks: Refusal on bedrock; 14T excavator.

Stability: Marginal

Trial Pit Log

Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Trialpit No **TP02A** Sheet 1 of 1
			Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.2 × 0.7 Depth 1.30	Scale 1:50
Client: Richard Howson			Logged RAP

Water Strike	Samples and In Situ Testing			Depth (m)	Level (m)	Legend
	Depth	Type	Results			
				0.30		
				1.10		
				1.30		

Remarks: Refusal on bedrock; 14T excavator.

Stability: Good

Trial Pit Log

Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: -	Level:	Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.8 × 0.7	Depth 2.30	Scale 1:50
Client: Richard Howson				Logged RAP

Trialpit No **TP03A**

Sheet 1 of 1

Water Strike	Samples and In Situ Testing: Depth	Type	Results	Depth (m)	Level (m)	Legend
				0.10		
				2.10		
				2.30		

Remarks: Refusal on bedrock; 14T excavator.

Stability: Good

Trial Pit Log

Trialpit No	**TP04A**	Sheet 1 of 1	
Project Name: Draughton Quarry	Project No. C3485/23/E/5292	Co-ords: - Level:	Date 04/05/2023
Location: Off Low Lane, Draughton, North Yorkshire BD23 6EA		Dimensions (m): 1.6 x 0.7, Depth 1.50	Scale 1:50
Client: Richard Howson			Logged RAP

Water Strike	Samples and In Situ Testing			Depth (m)	Level (m)	Legend
	Depth	Type	Results			
				0.10		
				0.90		
				1.40		
				1.50		

Remarks: Refusal on bedrock; 14T excavator.

Stability: Good

Appendix 3

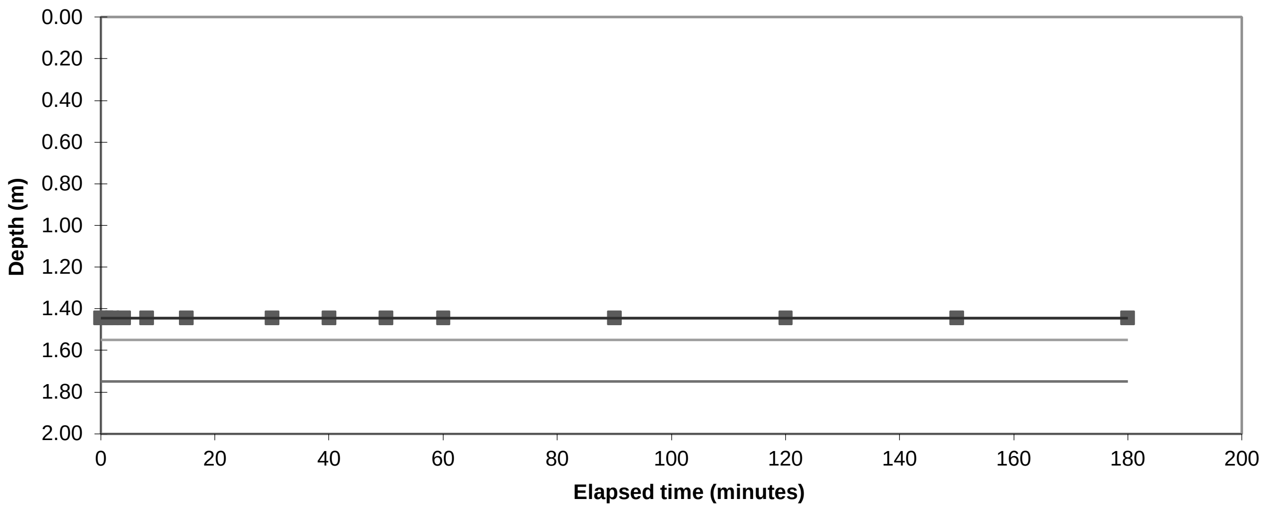
Soakaway Test Result Sheets

Rogers Geotechnical Services L

Soakaway Test

Trial Pit No:	SA1	Test No:	1	Date:	04/05/2023
Length (m):	1.700	Datum Height:		0.00	m agl
Width (m):	0.70	Granular infill:	None		
Depth (m):	1.85	Porosity of infill:		1	(assumed)

Elapsed time (minutes)	Water Depth (m below datum)	Elapsed time (minutes)	Water Depth (m below datum)
0	1.445		
1	1.445		
2	1.445		
4	1.445		
8	1.445		
15	1.445		
30	1.445		
40	1.445		
50	1.445		
60	1.445		
90	1.445		
120	1.445		
150	1.445		
180	1.445		



Start water depth for analysis (mbgl):	1.45		
75% effective depth (mbgl):	1.55	Elapsed time (mins):	#N/A
50% effective depth (mbgl):	1.65		
25% effective depth (mbgl):	1.75	Elapsed time (mins):	#N/A
Base of soakage zone (mbgl):	1.85		

Volume outflow between 75% and 25% effective depth (m³):

Mean surface area of outflow (m^2): 2.15
(side area at 50% effective depth + base area)

Time for outflow between 75% and 25% effective depth (mins):

Soil infiltration rate (m/s):	**Test incomplete as 25% effective depth not achieved. Unable to reliably determine soil infiltration rate.**

Remarks Results processed following BRE 365 (2007).

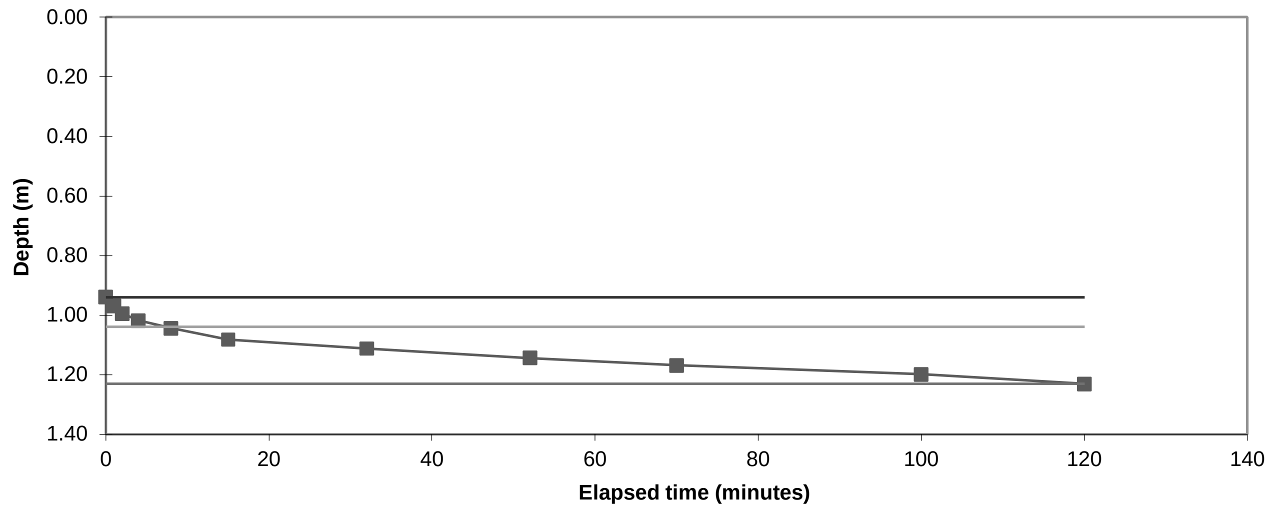
Client:	Richard Howson	**Job No:**
Site:	Draughton Quarry	C3485/23/E

Rogers Geotechnical Services L

Soakaway Test

Trial Pit No:	SA2	Test No:	1	Date:	04/05/2023
Length (m):	1.800	Datum Height:		0.00	m agl
Width (m):	0.70	Granular infill:	None		
Depth (m):	1.33	Porosity of infill:		1	(assumed)

Elapsed time (minutes)	Water Depth (m below datum)	Elapsed time (minutes)	Water Depth (m below datum)
0	0.940		
1	0.970		
2	0.995		
4	1.018		
8	1.044		
15	1.082		
32	1.112		
52	1.144		
70	1.169		
100	1.199		
120	1.231		



Start water depth for analysis (mbgl):	0.94		
75% effective depth (mbgl):	1.04	Elapsed time (mins):	7.4
50% effective depth (mbgl):	1.14		
25% effective depth (mbgl):	1.23	Elapsed time (mins):	119.4
Base of soakage zone (mbgl):	1.33		

Volume outflow between 75% and 25% effective depth (m³):	0.239
Mean surface area of outflow (m^2): (side area at 50% effective depth + base area)	2.21
Time for outflow between 75% and 25% effective depth (mins):	112.0

Soil infiltration rate (m/s):	**1.6E-5**

Remarks Results processed following BRE 365 (2007).

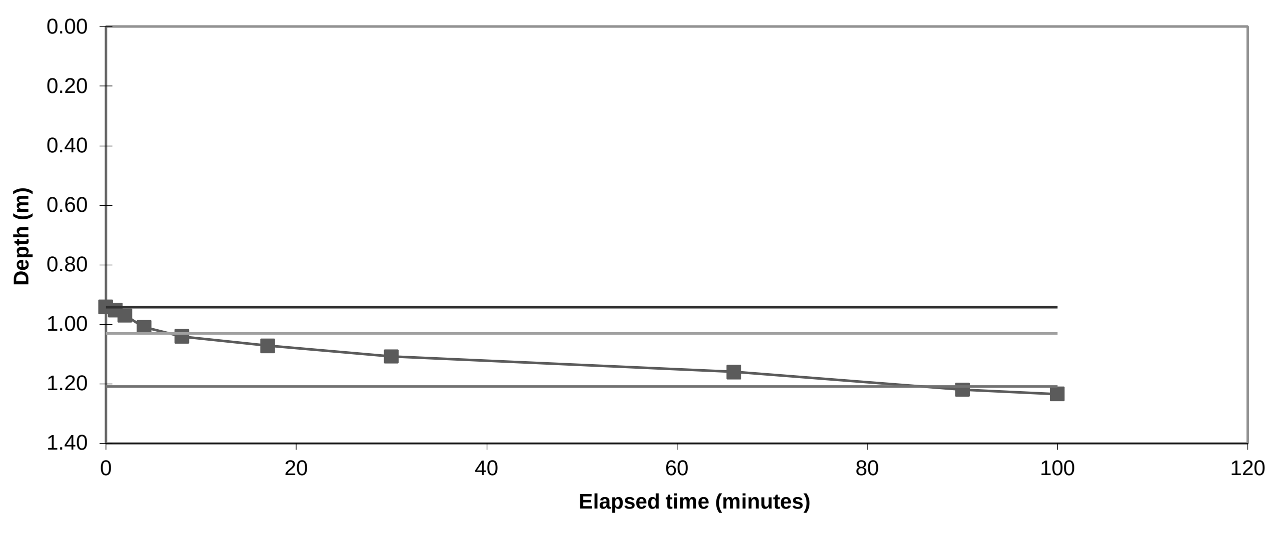
Client:	Richard Howson	**Job No:**
Site:	Draughton Quarry	C3485/23/E

Rogers Geotechnical Services L

Soakaway Test

Trial Pit No:	SA2	Test No:	2	Date: 04/05/2023
Length (m):	1.800	Datum Height:	0.00	m agl
Width (m):	0.70	Granular infill:	None	
Depth (m):	1.30	Porosity of infill:	1	(assumed)

Elapsed time (minutes)	Water Depth (m below datum)	Elapsed time (minutes)	Water Depth (m below datum)
0	0.942		
1	0.952		
2	0.971		
4	1.010		
8	1.041		
17	1.072		
30	1.108		
66	1.160		
90	1.220		
100	1.234		



Start water depth for analysis (mbgl):	0.94		
75% effective depth (mbgl):	1.03	Elapsed time (mins):	6.6
50% effective depth (mbgl):	1.12		
25% effective depth (mbgl):	1.21	Elapsed time (mins):	86.0
Base of soakage zone (mbgl):	1.30		

Volume outflow between 75% and 25% effective depth (m³):	0.227
Mean surface area of outflow (m^2): (side area at 50% effective depth + base area)	2.16
Time for outflow between 75% and 25% effective depth (mins):	79.4

Soil infiltration rate (m/s):	**2.2E-5**

Remarks Results processed following BRE 365 (2007).

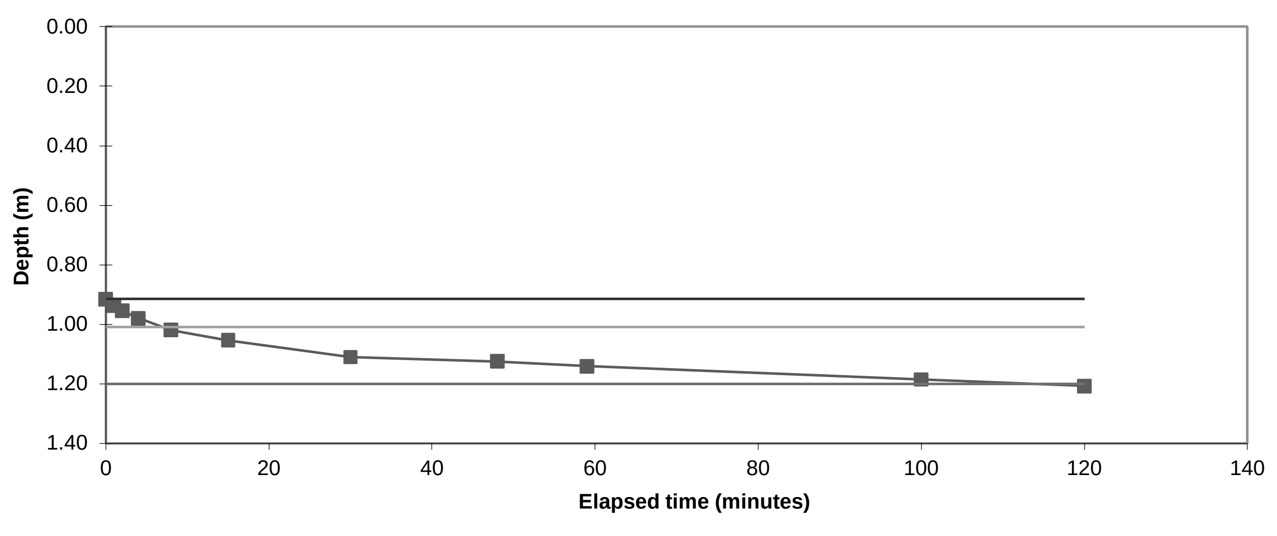
Client:	Richard Howson	**Job No:**
Site:	Draughton Quarry	C3485/23/E

Rogers Geotechnical Services L

Soakaway Test

Trial Pit No:	SA2	Test No:	3	Date:	04/05/2023
Length (m):	1.800		Datum Height:	0.00	m agl
Width (m):	0.70		Granular infill:	None	
Depth (m):	1.29		Porosity of infill:	1	(assumed)

Elapsed time (minutes)	Water Depth (m below datum)	Elapsed time (minutes)	Water Depth (m below datum)
0	0.915		
1	0.938		
2	0.955		
4	0.980		
8	1.020		
15	1.054		
30	1.110		
48	1.125		
59	1.141		
100	1.185		
120	1.208		



Start water depth for analysis (mbgl):	0.92		
75% effective depth (mbgl):	1.01	Elapsed time (mins):	7.0
50% effective depth (mbgl):	1.10		
25% effective depth (mbgl):	1.20	Elapsed time (mins):	113.0
Base of soakage zone (mbgl):	1.29		

Volume outflow between 75% and 25% effective depth (m³):	0.239
Mean surface area of outflow (m^2): (side area at 50% effective depth + base area)	2.21
Time for outflow between 75% and 25% effective depth (mins):	106.0

Soil infiltration rate (m/s):	**1.7E-5**

Remarks Results processed following BRE 365 (2007).

Client:	Richard Howson	**Job No:**
Site:	Draughton Quarry	C3485/23/E

Appendix 4

CBR Test Result Sheets

TEST REPORT
PLATE LOADING TEST

Project	Draughton Quarry	**Test No:**	1
Client	R Howson	**Lab Ref No:**	C3485
		Date Reported	07.06.23
		Weather Conditions	Dry
Technician	TM	**Air Temperature °C**	11

Date Tested	04.05.23	**Plate Dia (mm)**	450
Location	CBR01		
GPS Coord's	W 1° 56' 28.0", N 53° 58' 2.7"	**Depth (m)**	0
Material Type	Existing ground	**Reaction Type**	
No Cycles	1	**App Weight (kg)**	40

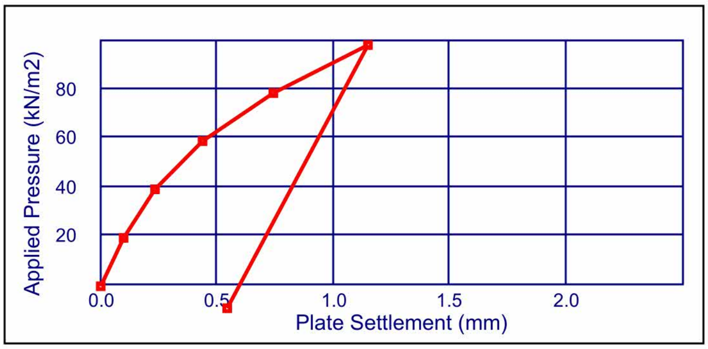


Plate Settlement (mm)	**Applied Pressure (kN/m2)**
0.00	0.0
0.10	19.8
0.23	39.8
0.44	59.5
0.74	79.2
1.15	98.9
0.54	-9.0

	Cycle 1
Maximum Applied Pressure (kPa):	**101**
Maximum deformation (mm):	**1.15**
Modulus of subgrade reaction K (MN/m3):	**83.1**
K762 (MN/m3):	**51.8**
Estimated CBR (%):	**9.0**

Comments:

[redacted]

Approved Signature
Roger Geotechnical Services
Tobias Merry
Laboratory Quality Manager

Plate Load - Tested in accordance with BS 1377 : Part 9 C
Moisture Content - Tested in accordance with BS 1377 : P
Opinions and interpolations expressed herein are outside the scope of UKAS acceditation