BAM Construction

Manchester City Football Academy

Geotechnical Design Report

REP/226518/G001

Issue 4 | 2 May 2013

This report takes into account the particular instructions and requirements of our client.

It is not intended for and should not be relied upon by any third party and no responsibility is undertaken to any third party.

Job number 226518

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1 Introduction

The purpose of this Geotechnical Design Report is to inform building and infrastructure design for the site wide development of the Manchester City Academy development.

In preparing this report, the following information has been considered:

- The Employers Scheme Design.
- The Employers Performance Criteria with regards to Constraint Removal Works (CRW) and site preparation in order to meet the scheme design requirements.
- The CRW contractors approach to meeting the Employers Performance Criteria.
- The CRW works that have taken place and validation reports that demonstrate that the CRW works meet the requirements of the original design concept.

In addition, historical ground investigation data and interpretation has been reviewed in order to build up a ground model to combine with information provided from the subsequent constraint removal works.

The Geotechnical Design Report summarises the findings of the above summary and provides guidance on design assumptions that should be considered for key buildings and site wide infrastructure.

The report primarily considers the Phase 1 Central Training Facility (CTF), the Phase 2 Facilities Management Building, and the Phase 4 North West Arena Building. However an overview of the geotechnical issues in other areas of the development is provided where CRW validation reports are available.

If any further significant geotechnical issues are identified within the remaining phases as Constraints Removal Works (CRW) information becomes available, this will be provided separately, and issued as a report addendum.

2 Reference Documents

The following documents have been used in the preparation of this report and are available to the design team:

2.1.1 Client Design Performance Criteria

SS017978 Project Nectar Scheme Design Report Stage D

MCFA Performance Specification and Basis of Design_CSS Rev 2.pdf

2.1.2 Historical Ground Investigation data and Interpretation

Structural Soils Ground Investigation Factual Report Nov 2010 Ref 761679

Structural Soils Ground Investigation Factual Report OMG Site May 2011 Ref 761919

WYG Interpretative Report March 2011 report ref. A064184-SI-V2

OMG Boreholes and Tar Wells Site Plan DWG No. WP/0068/A

OMG Summary of Works Document (unreferenced author unknown)

2.1.2.1 Other sources

Dreyfus Village ERM May 2007 ref. 0061687 (sourced by Kate Canning)

2.1.3 Client Construction Specification (issued for Tender Feb 2012)

NBS D20 Specification Excavation and Filling

NBS D30 Specification Piling

NBS E05-60 Specification In-situ Concrete/Large Precast concrete

2.1.4 Constraints Removal Works (CRW) Employers Requirements, Specification and Construction Drawings

WYG Environmental Openshaw West – Constraints Removal Works, Employers Requirements (Works Information Contract Data Part 1) June 2001

Dwg NTH/1315/HD/172 P2 Attenuation/Irrigation Tank Excavation Works

2.1.5 BWB Consultancy Specifications

Earthworks Specification NTH1315/07/V2 Oct 2011

2.1.6 BWB Consultancy Validation Documents

The CRW works are being implemented by Buckingham Group and reports by their consultants, BWB Consultancy, are being produced and submitted following the completion of each area. Current report availability is as follows:

Site Wide Documents

Mine Stabilisation Works validation Report Revision 4 – NTH1315E/12/MWR4/V2 -Works complete as of 30 January 2013

Mine Stabilisation Works validation Report Revision 6 – NTH1315E/12/MWR6/V2 -Works complete as of 5 March 2013

Phases 1 to 3 Earthworks

Phases 1, 2 &3 Earthworks Validation Report – NTH1315E/13/EW/V2~Dated Aug 2012 - Final Issue V2

Enhanced Earthworks Validation Report – NTH1315E/11/V2~Dated Aug 2012 – Final Issue 11/V2

Phase 4 and 10 Earthworks

Phase 4Z-99 Earthworks Validation Report - NTH1315E/16/EW/V2 ~Dated Feb 2013 – Final Issue V2 – Note this appears to be superseded by the below report.

Phase 4Z and 10 Earthworks Validation Report - NTH1315E/18/EW/V2 ~Dated April 2013 – Final Issue V2

Phase 5 and 5A Earthworks

Geotechnical validation not available

Phase 6,7 & 9 Earthworks

Phases 6,7 and 9 Earthworks Validation Report – NTH1315E/15/EW/V2 – Dated Dec 2012 – Final Issue V2

Phase 8, 9, 10A, and 11 Earthworks

Geotechnical validation not available

Remediation Validation Reports

In addition to the Earthworks validation reports, remediation validation documents are available for several of the areas but these have not been reviewed as part of this process.

- Remediation validation Report **Phases 1, 2 & 3** – NTH1315/13/R/V2- Dated 23 August 2012 – Draft Issue 2
- Remediation Verification Report : **Phase 4/Z East & 10A-L** – NTH1315/17/R/V2 – Dated 20th March 2013 – Final Issue 2
- Remediation Verification Report : **Phase 4/Z & 10** – NTH1315/18/R/V2 – Dated 9th April 2013 –Issue 2
- Remediation Verification Report : **Phase 4/Z -99**– NTH1315/16/R/V2 – Dated 11th March 2013 – Final re-issue

- Remediation Verification Report : **Phase 5 & 5A** – NTH1315E/14/R/V4 – Dated 19th March 2013 – Final Issue 4

  Remediation Verification Report : **Phase 6,7 & 9** Vol 1 and 2 – NTH1315E/15/R/V3 – Dated 19th Dec 2012 – Final Issue 3

3 Proposed Development

The City Football Academy will include the following:

- A new Academy for up to 400 young players with classrooms for 200;
- On site accommodation for junior and senior players;
- 16.5 football pitches, 12 of them dedicated to players aged from 8 to 21;
- A state of the art first team building with changing rooms, gym, refectory and injury and rehab centre;
- A 7000 capacity stadium for youth matches;
- Staff offices and a dedicated media centre;
- A bridge linking the site to the Etihad Stadium and the rest of the Etihad Campus.

The overall vision and proposed layout of the final development is as illustrated in **Figure 1** below:



Figure 1 Proposed Development

The works are to be undertaken by BAM Construction, following a phased hand over from the Enabling Works/Constraints Removal Works contractor Buckingham Group Contracting Ltd (BGCL). The phased sequence of hand over is indicated in **Figure 2** below.

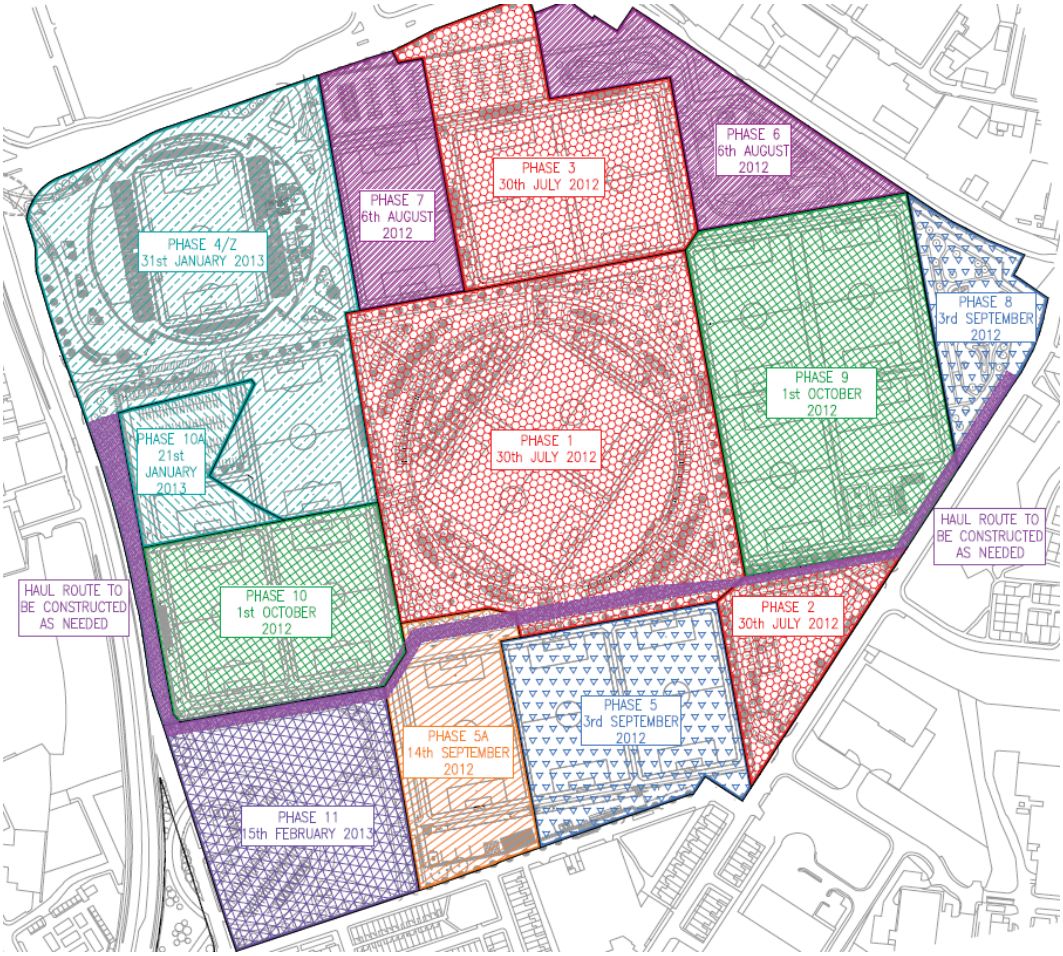


Figure 2 Original Development Phasing

Due to delays in the execution of the CRW, the work phasing has now been further split, to allow early hand over of intermediate parts of some of the phases. The revised phasing is illustrated below.

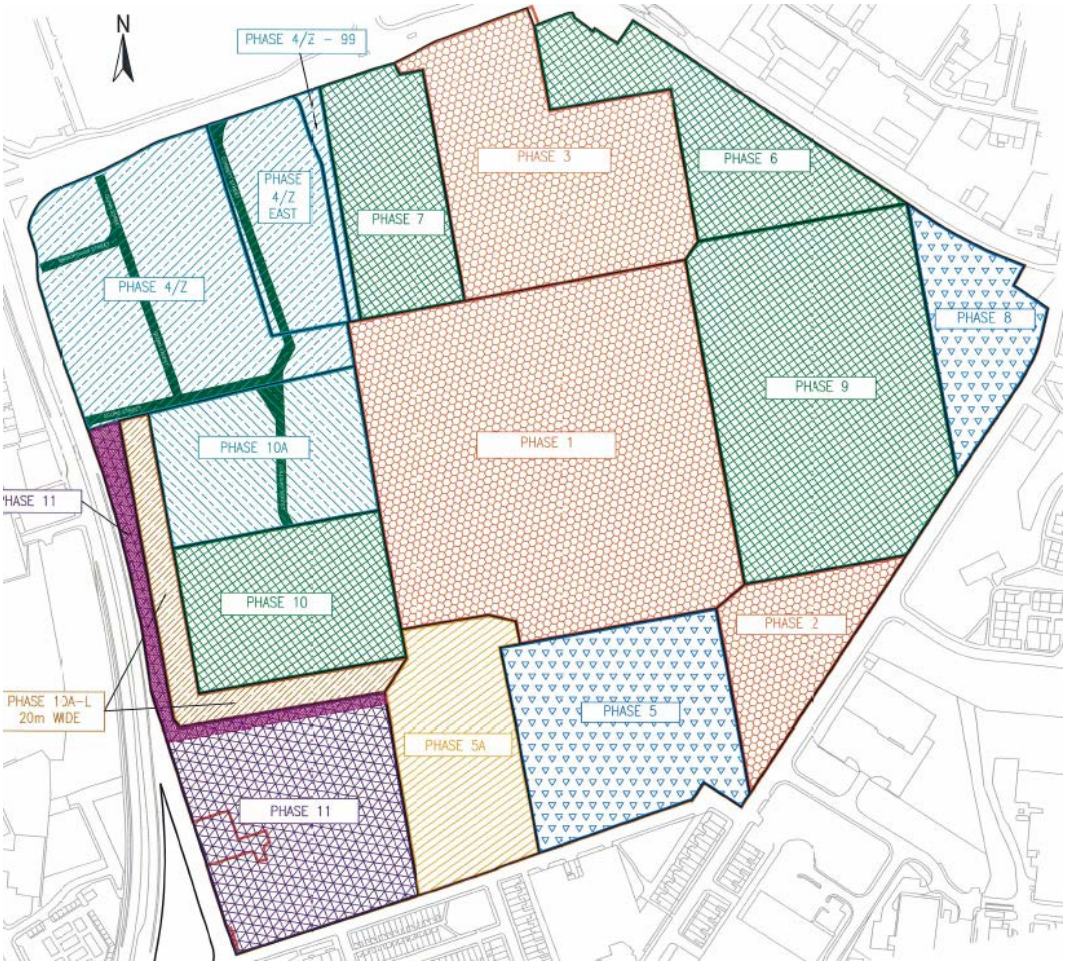


Figure 3 Revised Development Phasing

4 Geotechnical Assessment

The geotechnical assessment has considered information that can be interpreted from a review of historical data collected for the site in combination with consideration of the implementation of the constraints removal works (CRW) undertaken between 2011 and 2013. .

4.1 Constraints Removal Works

Prior to commencement of the construction, an advance phase of constraints removal work (CRW) was developed by White Young Green and implemented by contractor Buckingham Group Contracting Ltd (BGCL). In terms of the works that may impact on the structural and infrastructure proposals, the works included:

- Remediation of contaminated ground;
- Further investigation primarily relating to contamination and mine workings rather than to inform Engineering design;
- Removal of ground obstructions to varying depths in different site areas;
- General earthworks using primarily site won/reprocessed fill materials;
- 'Enhanced' earthworks in the locations of the proposed Phase 1 Elite and Academy buildings, to allow shallow foundations to be adopted;
- Locating and treating mineshafts and grouting up shallow mine workings;
- Installation of ground gas monitoring points to inform future design.

The CRW works are on-going but close to completion, at the time of writing this report. Some validation records (see section 2.1.6) have been provided by BGCL although some data remains outstanding at present.

4.2 Historical Ground Investigation Data

The ground investigation undertaken at the scoping stage appears to have been undertaken to inform the reclamation strategy rather than meet requirements to support detailed engineering design. However, the following information on stratigraphy and ground water is presented in the WYG Report and a review of the factual information available appears to confirm the interpretation by WYG. Although from the CRW understanding there are clear changes to the shallow stratigraphy the review of historical interpretation goes some way as to inform understanding and behaviour of underlying stratigraphy.

Overview

Previous reports indicate up to 5m of Made Ground in some parts of the site. This is underlain by up to 20m of Glacial Till (clay). There are two faults crossing the site, as a result of which the eastern third is underlain by Sherwood Sandstone (a Principal Aquifer) whilst the remainder of the site is underlain by Coal Measures (a Secondary Aquifer).

Superficial Deposits

The site is underlain by superficial deposits of Glacial Till which extend to depths of approximately 20mbgl. The Till comprises a stiff slightly sandy slightly gravelly clay with occasional thin sand lenses and rare bands (2-3m thickness) of silty sand. The rarity of permeable layers within the Till and stiff nature of the clay will afford a significant degree of protection to the underlying aquifer units.

Solid Geology

The underlying solid geology comprises Coal Measures strata on the western half of the site and the Triassic Sherwood Sandstone/Mercia Mudstone Group on the eastern half of the site. These areas are separated by a north-south trending fault, which passes through the centre west of the site. The downthrown side of the fault is to the east. Based upon the site investigation works this fault is shown to be located significantly further to the west than indicated on the geological maps for the area. This has important implications for considerations of potential mining hazards and possible requirements for grouting or treatment of shallow mine workings.

The Coal Measures are generally very weak to medium strong mudstone and/or siltstone with bands of coal. The Sherwood Sandstone typically comprises a red/brown sandstone that is weak or highly weathered in the upper portion.

Groundwater

Separate groundwater bodies have been identified within the Made Ground soils (circa 61-62mAOD), permeable bands within the Glacial Till (circa 58-59mAOD) and in the sandstone aquifer (circa 42-43mAOD). A fourth groundwater body may be present within the Coal Measures but there is limited information to support this.

4.2.1 Additional Information

In addition to ground investigation gathered from the tender documentation other ground investigation data has been identified from Arup archives to supplement understanding of ground conditions across the site.

4.3 Anticipated Stratigraphy of the Development Site

4.3.1 Engineered Fill

Information on the expected stratigraphy under the site has been determined from a combination of historical reports combined with an understanding of the Constraints Removal Works.

Given the scope of the CRW works, it is evident that the near surface ground conditions within the site are significantly different from those which might be inferred from the historical ground investigation.

In general terms, the CRW has included the removal and reprocessing of all of the Made Ground deposits down to the underlying Glacial Till deposit, and in places deeper.

BGCL are to survey the base of the excavation as each area is complete. Part of this survey, covering the Phase 1 area has been completed.

The excavated Made Ground has been processed to remove oversized components of 'hard dig' and the remainder appears to have been processed as a Class 1 General granular or Class 2 General cohesive fill.

Following validation of the natural formation and survey of the base, refilling of the site has then taken place, with materials placed and compacted in accordance with the CRW Earthworks Specification prepared by BWB Consultants on behalf of BGCL to meet the performance criteria of the Employer's Requirements (WYG, June 2011). Validation testing using Plate Load Testing and Mexe Probe was performed within the fill layers to confirm the CBR values.

In addition, enhanced earthworks were undertaken in localised areas with specific placement requirements and validation testing as discussed later in this report.

BGCL have undertaken to survey the completed site level which will form the commencing surface for subsequent works by BAM.

The validation report for Phases 1, 2 and 3 indicates that no original Made Ground has been left in place. This has been excavated and replaced with engineered fill during the CRW. It is understood that this philosophy to fully excavate the Made Ground was adopted throughout the other phases of development.

By comparing the original topographical levels to the levels at the base of the BGCL excavation, it is evident that between about 2 and 11m of Made Ground has been removed, but typically between 2 and 6m from below the Central Training Facility (CTF).

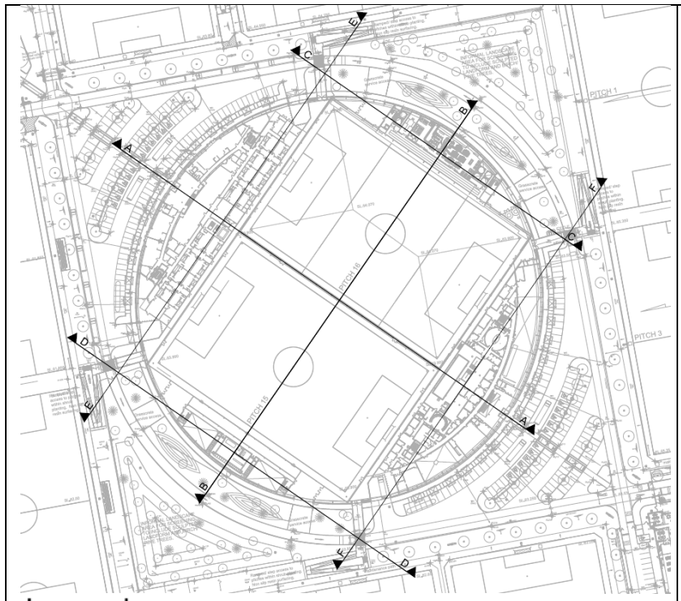
4.3.1.1 Indicative CRW Surfaces at CTF (Phase 1)

As part of the design validation works, Arup have produced a series of 3D surface models using a combination of proposed surfaces and where known, as built surfaces. The main surfaces modelled are as follows:

- Surface A: The finished development levels – As provided by Architect: Planit
- Surface F: The Pre- CRW Topographical Survey level- Provided by Survey Operations
- Surface B : The designed formation level – As provided by BGCL
- Surface C : The 'As built' formation level – As provided by BGCL
- Surface E: The base of CRW excavations – As provided by BGCL

The level of Surface C is not available for the site and the information for Surface E is available for only part of the site. This is due to the on-going nature of the CRW. It is envisaged that this information will be made available for the full site as the CRW works progress.

The base of excavation (Surface E) is available for the Phase 1 area and has been used to produce the following cross sections for the CTF area (**Figure 4**):



Legend

A - Proposed Final Level

C - Proposed Formation Level

E - Base Of Dig

F - Orginal Topo

Key Plan and Legend for Sections AA to FF

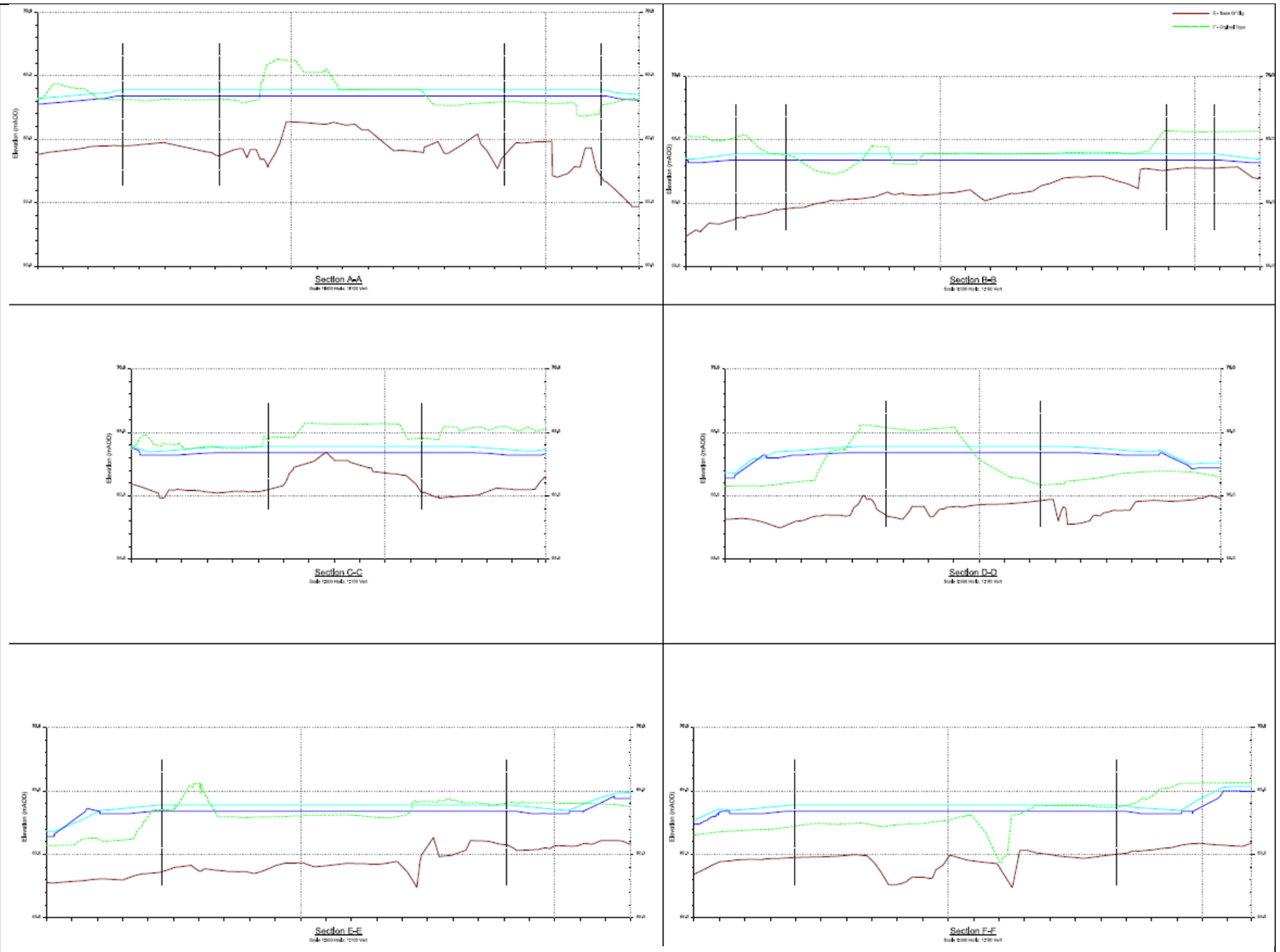


Figure 4 Earthworks Cross Sections for CTF Area (Sections AA to FF)

The data appears to suggest a thickness of engineered fill of varying thickness (typically 0.5m to 5m) and a thickness of 'enhanced' engineered fill typically between 3m and 5m.

It is also noted that the development platform is above the original pre CRW levels in some locations and below it in others, resulting in differences in the likely responses of the underlying soils, due to either reduction in stress, where levels have been reduced, or increase in surcharge stresses where levels increased.

4.3.2 Overview of Ground Conditions below CRW

4.3.2.1 Glacial Till

The Glacial Till below the CRW is anticipated to be between 15m and 20m in thickness below the site.

The Glacial Till is expected to have an undrained shear strength (Cu) of 100kPa at the top of stratum. This is based on the CRW Earthworks Specification prepared by BWB Consultants on behalf of BGCL to meet the performance criteria of the Employer's Requirements (WYG, June 2011). It is also possible to determine the site wide increase in strength of the Till with depth from available historical ground investigation data.

4.3.2.2 Solid Geology

The underlying solid geology comprises Coal Measures strata on the western half of the site and the Triassic Sherwood Sandstone/Mercia Mudstone Group on the eastern half of the site. The geology is split by the Bradford Fault which passes through the centre west of the site crossing both the Academy building and the southern CTF area (formerly Laundry building).

A review of the geological map for the area shows the fault to be about 45 degrees to the horizontal as shown below (**Figure 5**). The fault down throw is to the east. The shallow named seams are likely to outcrop to the west of the fault and the Parker Mine seam appears to be deep (>500m below surface).

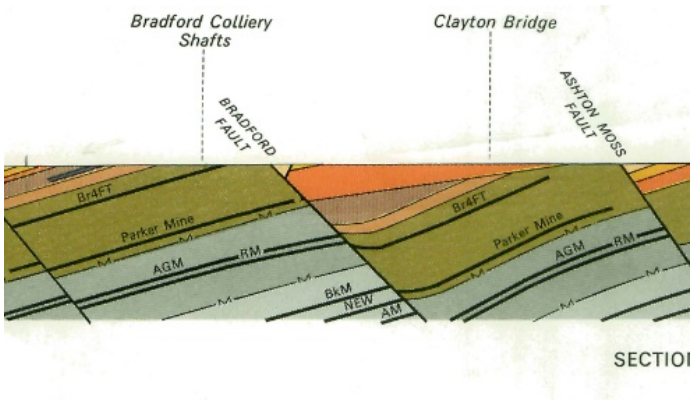


Figure 5 Geological Map extract showing Bradford Fault (section of west to east orientation)

The performance criteria of the Employer's Requirements (WYG, June 2011) required the CRW contractor to both investigate the extent of worked coal seams across the site and also treat any worked seams up to a depth that is potential influential on the proposed development including shafts.

As a result substantial potential mining hazards are considered to have been removed from the site. Further comment is made in Section 5.

4.4 Expected Ground Conditions in Phase 1 CTF Area

Phase 1 is considered as three areas underlying three structures in this phase; energy centre, elite building and academy building. A fourth area is the southern CTF area which was originally the intended site of the Laundry area, which has now been relocated. The proposed usage of this area is not known but is believed to be non-structural. A fifth area is the proposed covered pitch structure which occupies the north eastern portion of the pitches, adjacent to the energy centre.

Following the excavation of Made Ground, the energy centre and southern CTF sites are underlain by around 2 to 6m of engineered fill, 18m of Glacial Till over bedrock. The fill beneath the academy and elite building areas has been placed to a higher standard as 'enhanced' earthworks and as a result the stratigraphy typically comprises 3 to 5m of 'enhanced' engineered fill over 15-18m of Glacial Till overlying bedrock. The Bradford Fault bisects the academy building with Coal Measures sub-cropping to the west of the fault and Sherwood Sandstone to the east.

4.4.1 Indicative Cross Sections from Historical Ground Investigation

Cross-sections under each building area are shown in **Figure 6**-**Figure 9** below, based on existing borehole data from within around 20m of the line bisecting the building. It should be noted that the proposed Laundry Centre in the south of the CTF (Figure 9) has now been relocated away from the CTF.

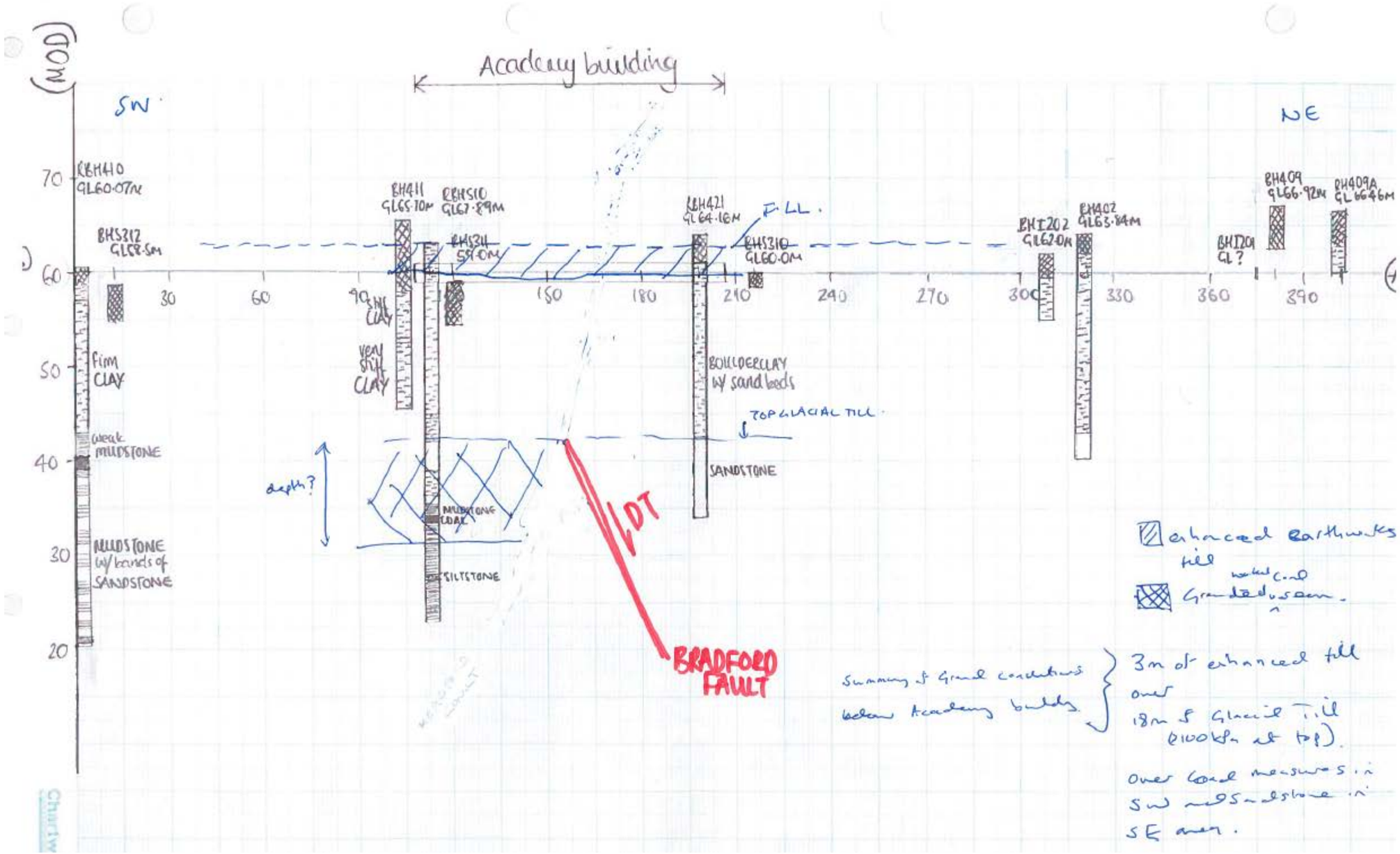


Figure 6 Indicative Geological Cross Section through Academy Building (Section looking to the North West i.e. Section line SW to NE)

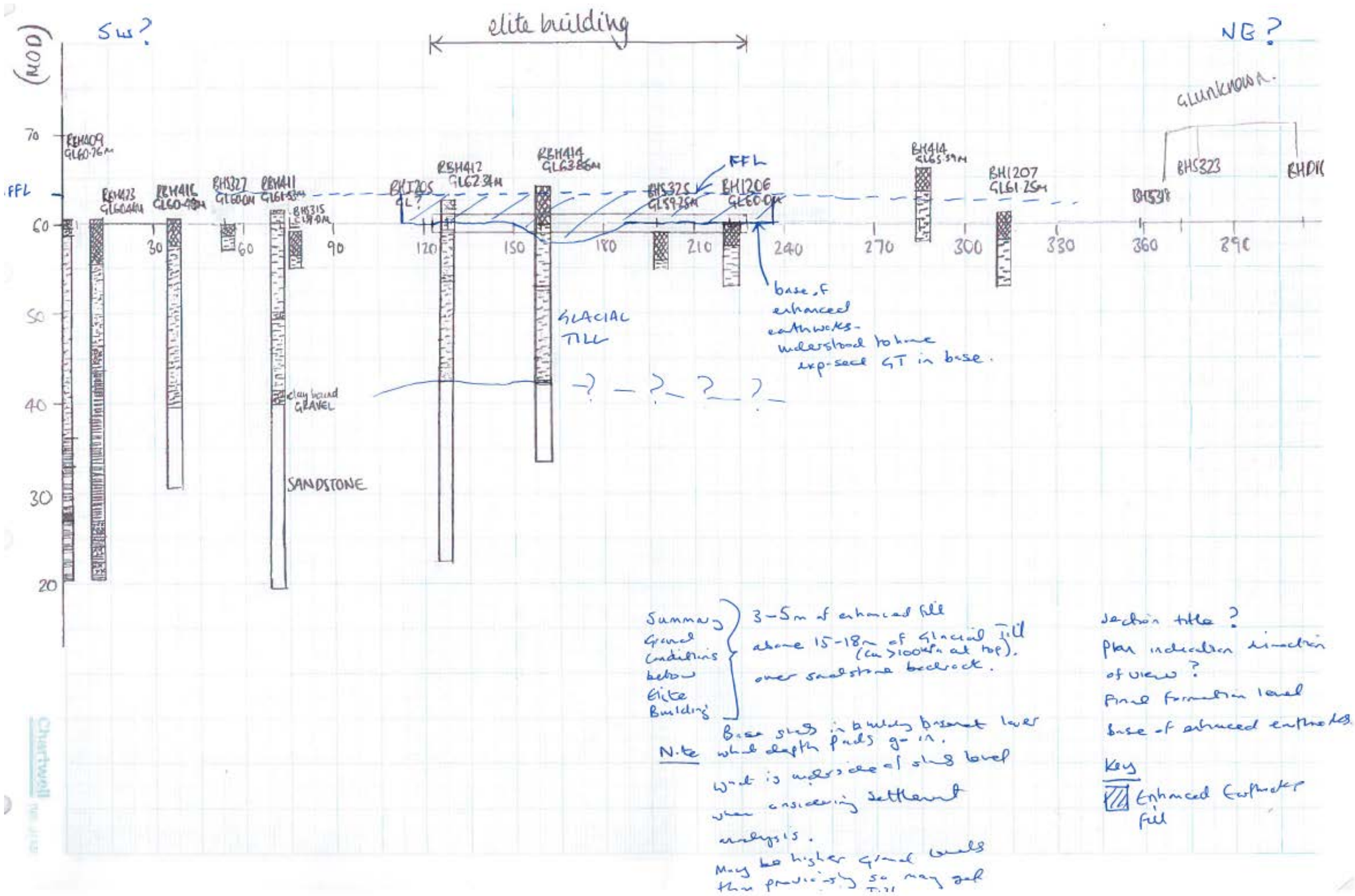


Figure 7 Indicative Geological Cross Section through Elite Building (Section looking to North West i.e. Section line SW to NE)

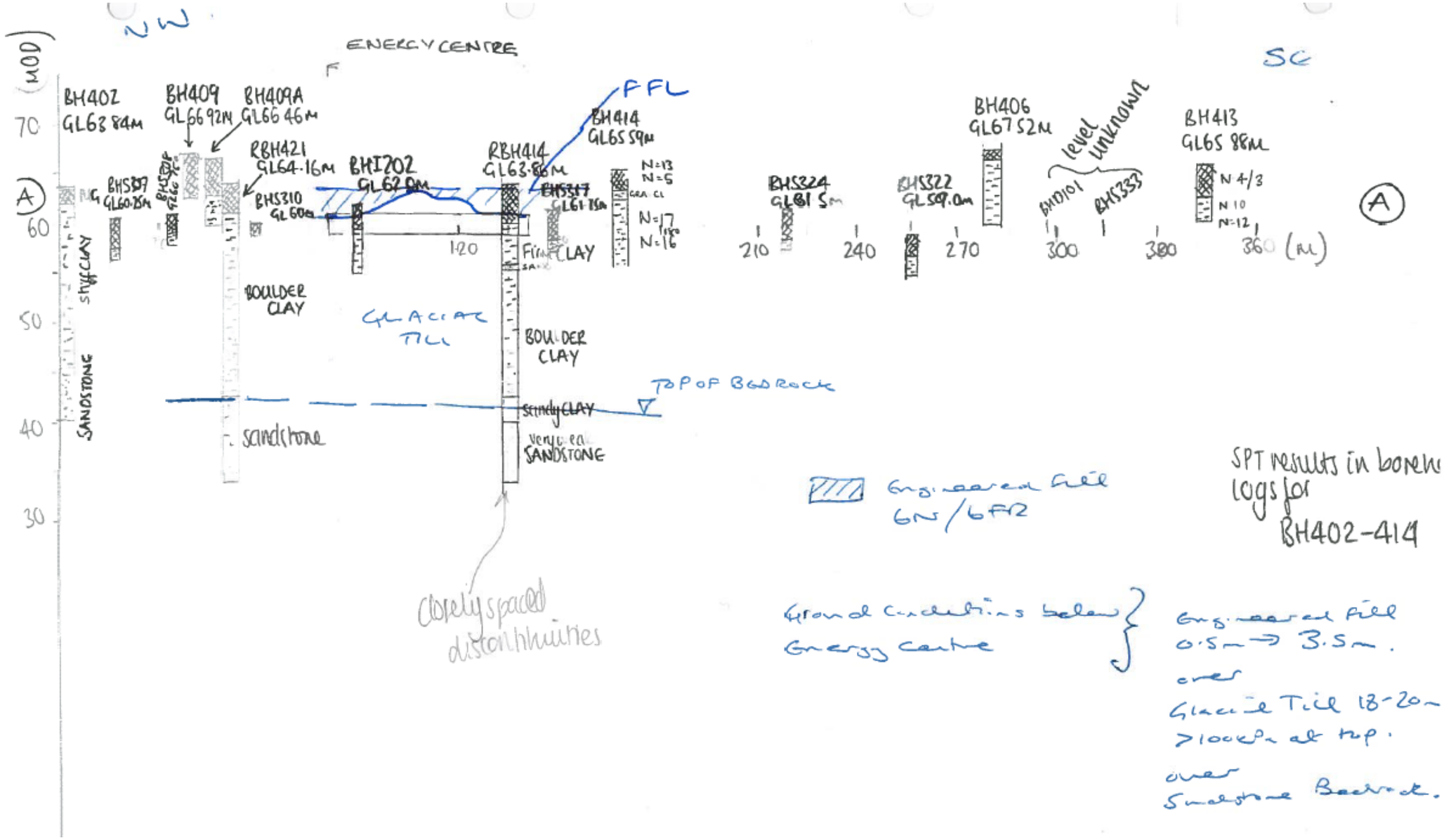


Figure 8 Indicative Geological Cross Section through Energy Centre (Section looking to North East i.e. Section line NW to SE)

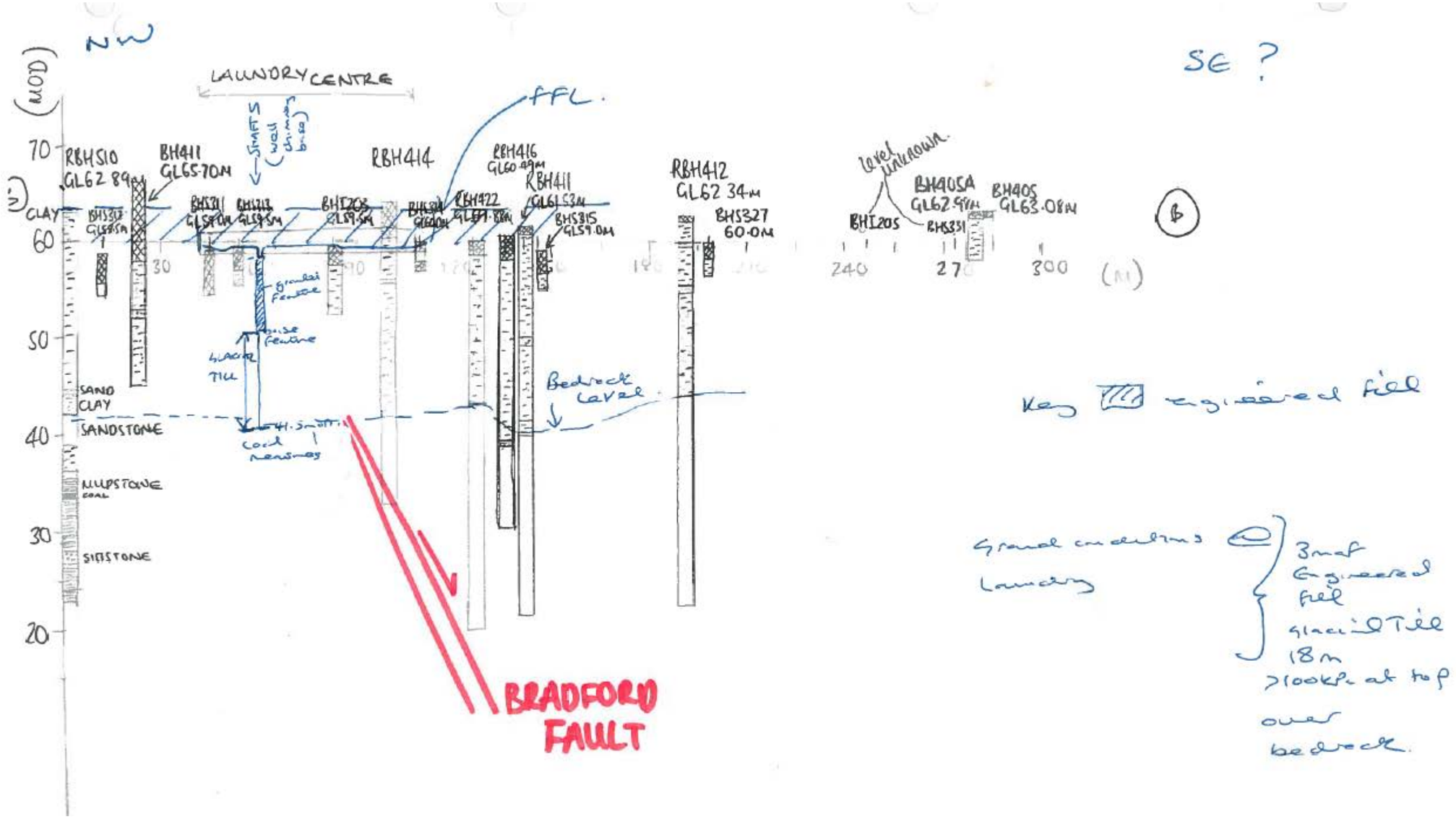


Figure 9 Indicative Geological Cross Section through southern CTF i.e. former Laundry Centre (Section looking to North East i.e. Section line NW to SE)

4.5 Properties of the Engineered Fill placed as part of Phase 1 CRW

There is no ground investigation data available to confirm parameters of the engineered fill. Properties of the engineered fill have been determined based on analysis of the following available data from the CRW validation reports:-

- Bench Trial Data
- In situ testing (CBR from Mexe probe and plate bearing test data)

4.5.1 Bench Trials

During the CRW, an earthworks bench trial study was undertaken (around November 2011) to assess the performance of the engineered fill and validate proposed compaction against Employers requirements for fill performance.

The bench trial was developed to confirm that the method specification for the bulk earthworks would meet the Employers Requirements for a CBR value of 5% at formation level and a maximum allowable settlement of 25mm. The actual meaning of the later requirement on settlement is not clear and does not appear to have been proven as part of either the benching study or later assessment of the as constructed works. The BWB approach to meeting this settlement criterion is to limit settlement through the achievement of 95% compaction of available site won material using the method specification.

The benching exercise correlated a requirement for 95% compaction against an equivalent lab based CBR value determined by Mexe probe. This value was used to evaluate the compaction method through comparison of laboratory results with in-situ CBR, Dynamic Cone CBR and the in-situ density measured by sand replacement tests.

Two trial areas were adopted, one using reworked Glacial Till and one with granular Made Ground.

The work reported the following:

- For cohesive fill, the method specification will provide the required maximum dry density for the fill. A close relationship was reported between the MEXE CBR, DCP CBR and plunger CBR values. The proposal was to validate fill placed in situ using MEXE Probe to obtain an average CBR of 4% with no single result falling below 3%.
- For granular fill, a series of 6 No. MEXE Probe tests, 3 No. DCP CBR tests and a single Plate Load CBR test were conducted. The optimum moisture content was not determined due to the quantity of particle sizes being in excess of 20mm. The CBR values were greater than 5% which BWB concluded would indicate very little anticipated settlement.

The trial philosophy looks at limiting settlement by achievement of 95% compaction and assessing equivalent CBR for this compaction to validate earthworks on site. The approach looks at short term loading and does not confirm the anticipated range of settlement as a result of variation in thickness of fill, the influence of slightly lower compaction on variation in settlement, or any

direct indication of stiffness of the material. The testing undertaken during the benching study and subsequent earthworks does not appear to be particularly well suited to informing assumptions with regards to detailed design of bearing foundations/slabs without further interpretation and possibly further investigation to confirm assumptions.

In addition, no hand vane shear tests appear to have been undertaken as part of the trial to confirm soil strength, even though the requirement is detailed in the BWB specification for earthworks. Therefore no strength testing has been provided for cohesive fill as part of the CRW works which could be used for interpretation of strength and stiffness parameters for design purposes.

4.5.2 Validation during earthworks

As part of the CRW, BGCL were reported to have excavated to the base of Made Ground and screened and processed the materials for use as general fill (Class 1 or 2), with hard components being crushed to form an 'enhanced' engineered fill (Class 6).

The areas were refilled from the level of the natural Glacial Till, with the majority of the fill being classified as General Cohesive Fill (HA Class 2A and 2C).

Validation testing using Plate Load Testing and Mexe Probe was performed within the fill layers to confirm the CBR values in accordance with the BWB Earthworks Specification.

The results from the field tests have been used to assess values for Stiffness Modulus (ratio of applied stress to induced strain) based on correlations between E and CBR published in IAN 73/06, Rev 1 (Section 5.11 of Interim Advice Note 73/06, DfT Highways Agency, 2009) i.e. $E = 17.6 (CBR)^{0.64}$ MPa. E is considered to be short term based on the tests used in its determination and the general cohesive nature of material encountered. Results are presented for the areas of general engineered fill tested beneath the Energy Centre and former Laundry Building locations/vicinity (based on panel ID locations) in **Figure 10** and **Figure 11**.

The results show results typically in a range for Eu of 40-60MPa for both locations.

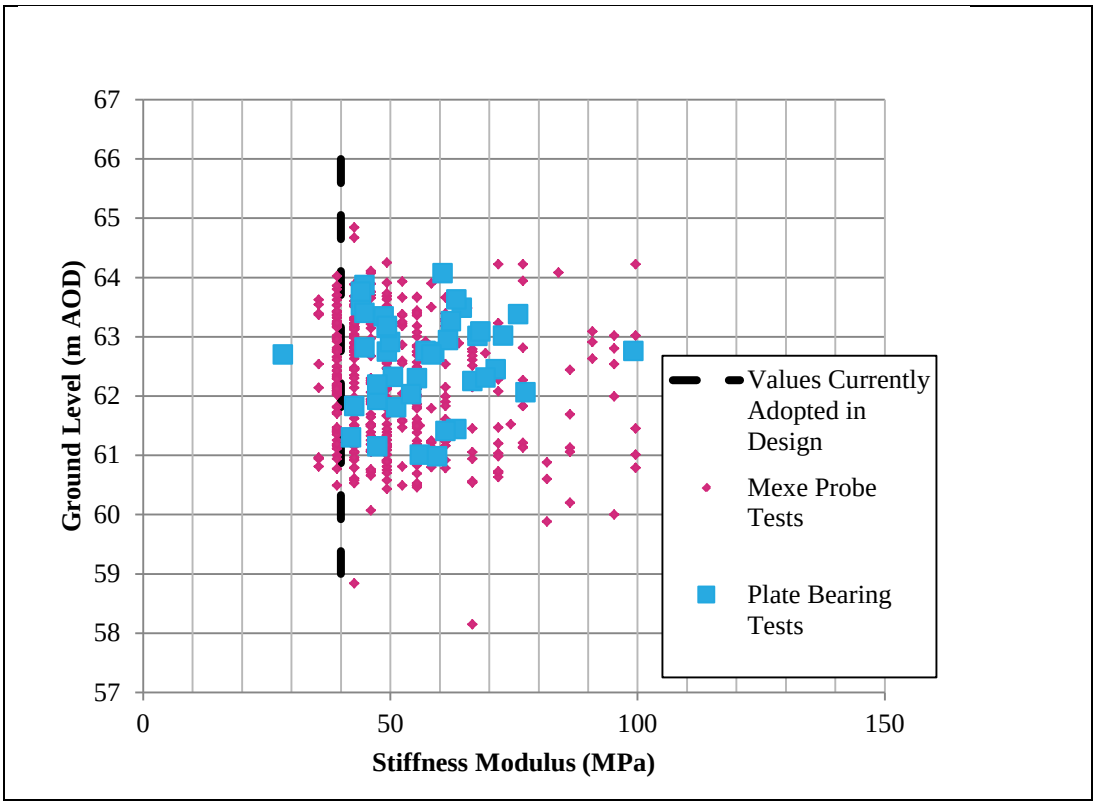


Figure 10 Assessment of Stiffness Modulus Eu for General Engineered Fill below Energy Centre based on field tests.

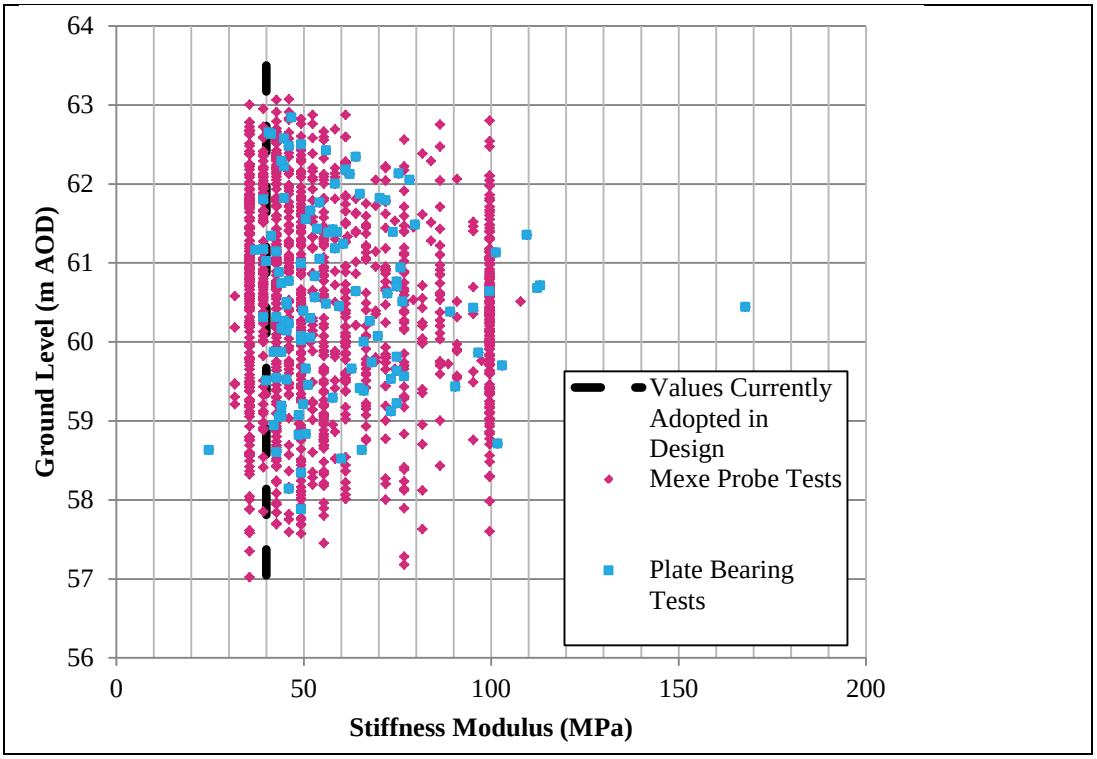


Figure 11 Assessment of Stiffness Modulus Eu for General Engineered Fill below southern CTF (former Laundry Building) based on field tests

4.6 Properties of the 'Enhanced' Engineering Fill placed as part of Phase 1 CRW

Within the areas of enhanced earthworks beneath the Elite Training and Academy buildings in the Phase 1 area, enhanced earthworks have been completed using Class 6N and 6F2 granular fill.

The BWB specification for the 'Enhanced' engineered fill states that the material should achieve a Modulus of Subgrade Reaction (K) = 25,000 kN/m^2/m. Equivalent requirements for E' (drained stiffness modulus) would therefore vary depending on the thickness of fill but for say for 3m of fill this would equate to approximately E' of 75MPa. This requirement was to be verified using 600mm diameter plate bearing tests with up to 150% of anticipated building load. Testing was undertaken using an offset 50m grid every other layer.

The stiffness of the enhanced engineered fill, underlying the Elite and Academy buildings, was also assessed through the End Product Verification plate bearing tests (600mm dia.) carried out by BWB. The K762 value established through this test was converted to a Stiffness Modulus using correlations published in IAN 73/06 Rev 1 (DTp Highways Agency, 2009).

In addition, a series of 6 No. area load tests were performed within the 'enhanced' engineered fill, with 3 tests on both the Elite and Academy areas. The objective of this was to demonstrate that the settlement criteria was met. Testing took place at 62.9mOD and comprised either a 1.65m x 1.65m reinforced concrete pad or 1.5 x 1.5m steel plate loaded to 350kN.

Only 3 No results have been provided, all within the Elite building area. From these, the settlement range at 150 kN/m^2 stress was 3.5 to 5.4mm. Assuming the strain occurs over a depth of 1.5 x pad size, results in a stiffness value of 65 to 102 MN/m^2 and therefore meeting the specification requirements for K.

The results for interpretation of in situ testing are presented in **Figure 12** and **Figure 13**

Results show E' typically in a range of 50-100MPa for the 'enhanced' engineered fill with more reliable load testing at the Elite building showing E'>75MPa. The lower values for E' may be influenced by the depth of testing as results at depths closer to top of base of 'enhanced' engineered fill appear to show lower stiffness than those closer to the final formation level. This should be considered further when considering formation level of foundations and slabs in design.

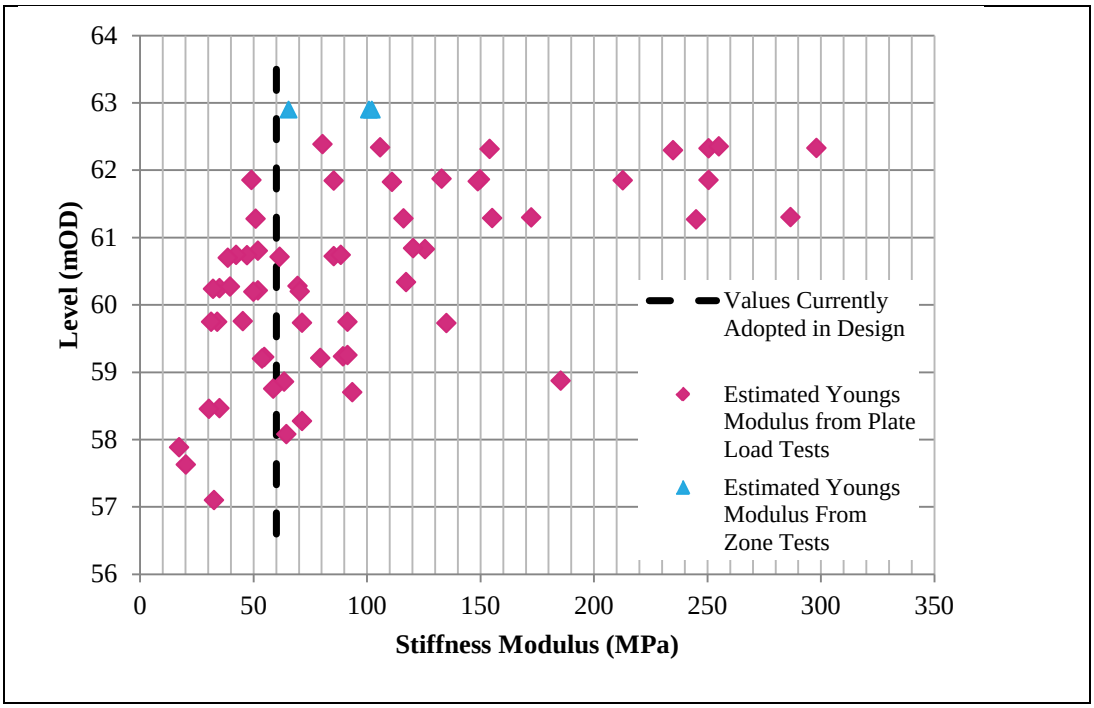


Figure 12 Assessment of Stiffness Modulus E' for Enhanced Engineered Fill below Elite Building from field tests

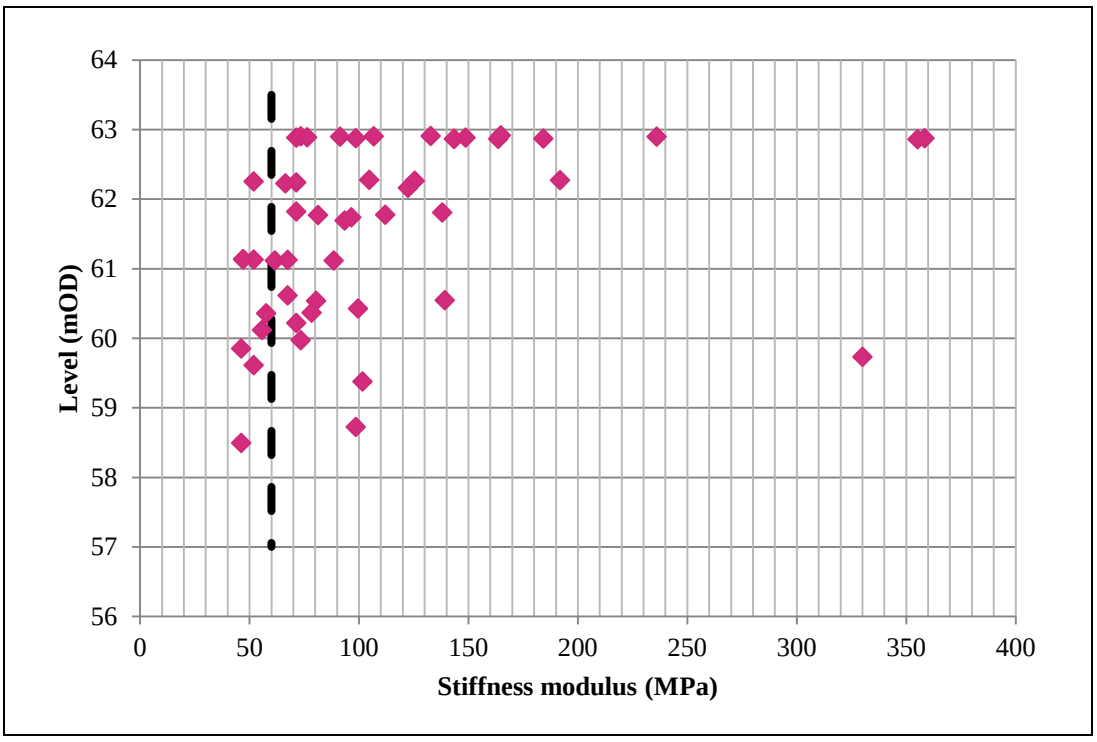


Figure 13 Assessment of Stiffness Modulus E' for Enhanced Engineered Fill below Academy Building from field tests

4.7 Properties of the Glacial Till

The Stage D design assumes the Glacial Till has a minimum strength of 100kPa at the top of stratum. It is not clear how this was determined; however it does not seem to be unreasonable from knowledge of the local region.

Interpretation of available historical records within the Glacial Till indicates that the material primarily comprises a stiff clay.

Validation reports for the enhanced earthworks and a benching trial undertaken to inform the earthworks approach included strength testing in the top of formation of Till. The results of hand vane tests show an undrained shear strength in excess of 100kPa and frequently in excess of 150kPa (assumed to be the test limit of the apparatus) as shown in **Figure 14**.

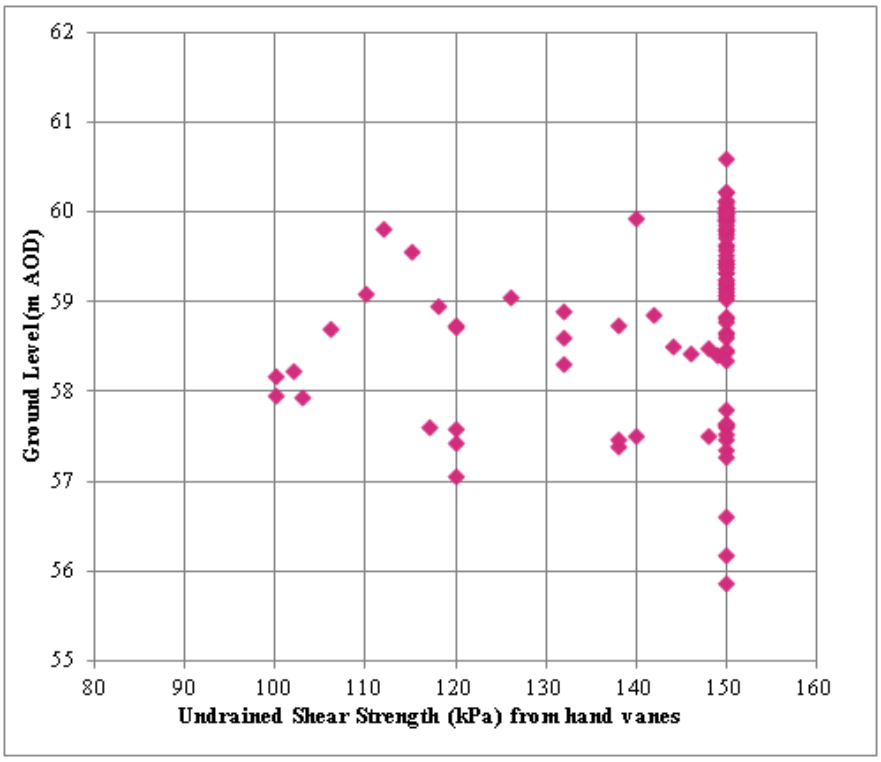


Figure 14 Undrained Shear Strength (Cu in kPa) at top of Glacial Till (rolled base of excavation in enhanced earthworks area)

In addition, an assessment has been made of the strength of the Glacial Till using the results of site wide Standard Penetration Tests (SPTs) from the WYG investigation and other historical ground investigations as shown in **Figure 15**.

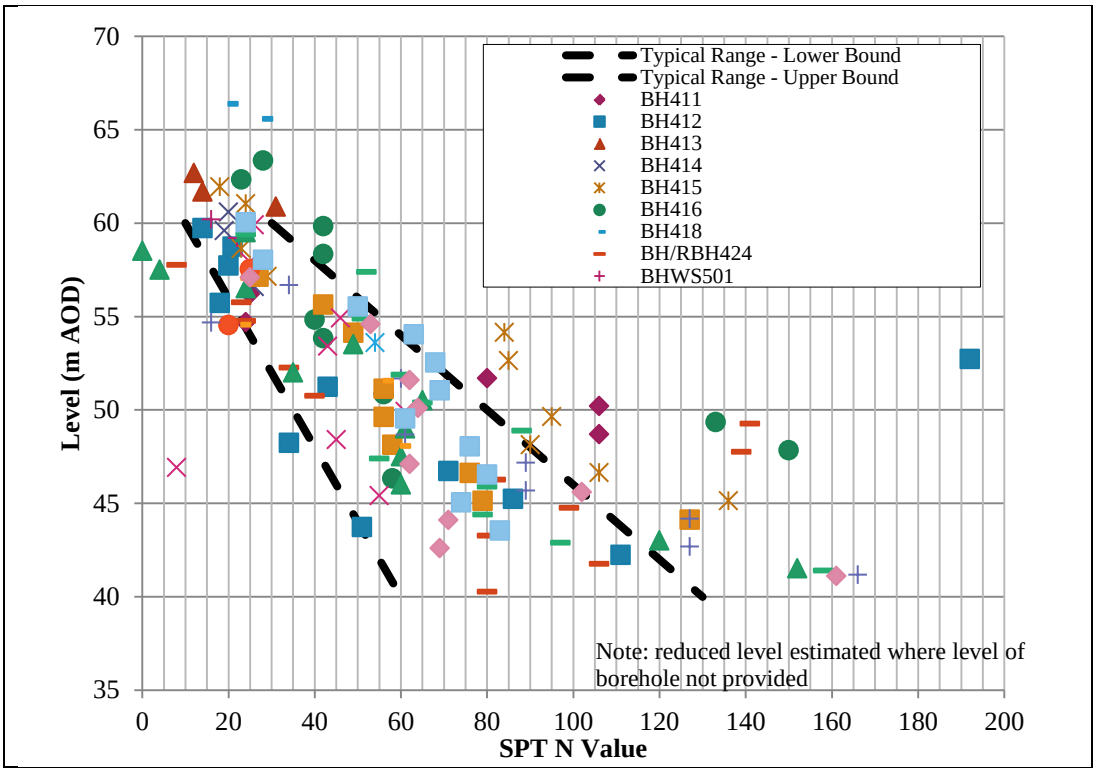


Figure 15 Site wide SPT vs. Reduced Level Plot for Glacial Till

This has been compared to shear strength measured in undrained compressive triaxial tests, also undertaken as part of historical investigations. For UK over consolidated clays, the SPT shows a good correlation to undrained shear strength (c_u), as shown in Figure 31 of CIRIA 143, 1985.

A plot of the triaxial test results of c_u, along with c_u values derived from the relationship c_u = 5 x SPT 'N' are shown in **Figure 16**.

The shear strength derived from SPT tests appears to be comparable with the triaxial results.

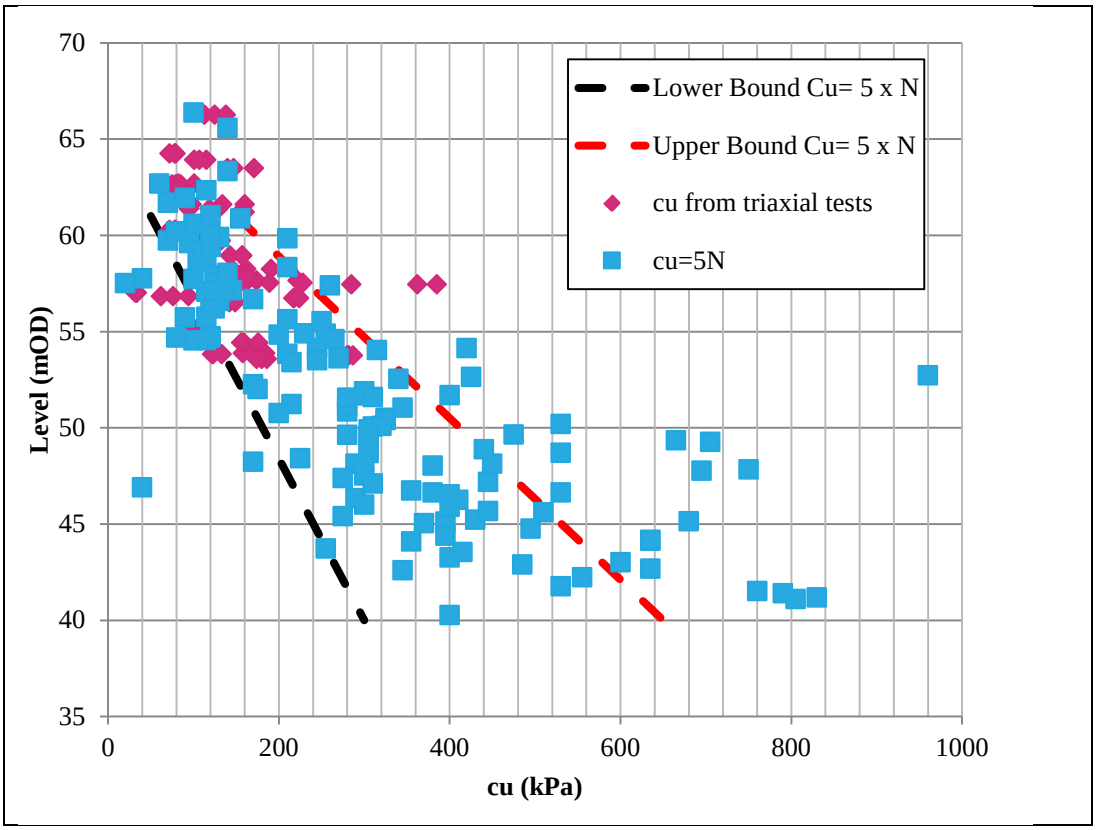


Figure 16 Undrained shear strength, c_u vs. Reduced Level from site wide results

The shear strength of the Glacial Till increases significantly with depth. For the settlement prediction model a generally lower bound range of shear strengths has been considered, with strength of 100kPa at the base of the placed fill, increasing linearly with depth to 300kPa at rockhead.

The strength of the Glacial Till was also verified against the investigation of the adjacent Sportcity East/West and Etihad Stadium projects. In both cases, the profile of strength against reduced level was comparable, with typical strengths of 75 to 100 kPa to about 55mOD followed by a generally more rapid increase in strength below this.

The stiffness of the Glacial Till for the purposes of the settlement assessment is taken as Undrained Modulus Eu (MN/m^2) = 500xCu at the surface based on the plasticity index and a degree of over consolidation of a Glacial Till (where E' can be taken as 0.7Eu). However taking into consideration the relationship of stiffness with strain (Stroud 1988) the ratio of E' to Cu may be higher than this where the degree of net load/ultimate load and equivalent strain is lower i.e. where the glacial till is at depth below the foundation and/or at depth where the load above is less influential.

4.8 Summary of Design Parameters

The assumed stiffness and strength parameters for the CTF area are as indicated in **Table 1** below.

Table 1: Summary of Design Parameters Interpreted from Available Data for the Phase 1 Area

Material Type	Design Shear strength	Design Undrained Young's Modulus Eu	Design Drained Young's Modulus E'	Source of testing data
General Engineered Fill	N/A	40MPa (typical range 35-60MPa)	25MPa (based on 0.7Eu, typical range 25-42MPa)	Insitu CBR/plate bearing tests carried out by CRW contractor
'Enhanced' Engineered Granular Fill	N/A		18-360MPa (typically 40-120 MPa) Assumed 60MPa in settlement assessment.	Plate Load Tests and zone tests by BWB
Glacial Till	Top of stratum 100kPa Base of stratum 300kPa		Assumed 35MPa at top of stratum Currently assumed min 100MPa at base of stratum - increases with depth possibly up to 300MPa at 20mbgl to be assessed during design analysis depending on assessment of strain	In-situ hand vanes in top of formation by CRW contractor Historical Site Investigation Data (SPT and Triaxial testing)

4.9 Bedrock

As discussed earlier interpretation of available historical records above suggests Coal Measures strata on the western half of the site and the Triassic Sherwood Sandstone/Mercia Mudstone Group on the eastern half of the site. The site is split by the Bradford Fault. Based upon the site investigation works this fault is shown to be located significantly further to the west than indicated on the geological maps for the area.

Shallow worked coal seams within the site area are reported to have been treated with grout to x10 seam thickness below top of bedrock up to the western edge of the fault outcrop line. It is therefore assumed that bedrock is reasonably competent across the site. However in order to avoid overstressing of the underlying coal and seat earth deposits, it would be recommended that piling is terminated at least 3 x pile diameter above rockhead.

4.10 Groundwater

Prior to the excavation of Made Ground undertaken as part of the Constraints Removals Works, site investigation records indicate that there was localised perched groundwater within the Made Ground. WYG indicated that separate groundwater bodies were present in the Made Ground (c.61-62mOD), in permeable bands within the Glacial Till (c.58-59mOD) and in the underlying Sandstone aquifer (c. 42-43mOD). It is likely that the groundwater in the Made Ground comprised localised areas of perched water, confined by elements of remnant substructure and the underlying low permeability Glacial Till.

WYG indicated that the Ashton Canal may have been leaking into the east of the site. This has since been discounted by investigations conducted by BGCL under the CRW.

The extent of groundwater encountered during the CRW is unclear. Some validation photographs indicate pumping was required to pools of water within excavations of old structures and rainfall present within the earthworks. Aerial progress photographs indicate large pools of water. However, the validation reports provide insufficient data to conclude whether perched water tables lie within the newly placed fill materials.

It is very possible that most of the perched ground water will have been dealt with during the works and either pumped off site or otherwise incorporated in the generated fill.

As the majority of the general engineered fill is cohesive, this will be of low permeability and therefore if recharged by rain water infiltration or ground water from the edges of the site it would not be expected to transmit large volumes of water in to localised excavations. It is envisaged that if water is present, is should be temporarily controllable by sump and pump methods.

However, the Class 6 granular materials used in the Enhanced Elite/Academy areas will be more permeable than the surrounding general cohesive fill and these pockets of fill could attract surface runoff or ground water and behave like sumps to the phase 1 area. Therefore there is a higher probability of water ingress in to excavations in these areas.

4.11 Obstructions

See Section 5.1.5.

5 Phase 1 Buildings – Summary of interpretation of site conditions informing design

5.1 General Design Considerations

5.1.1 Influence of increase in site levels on ground movements

5.1.1.1 Loading of Glacial Till

As discussed in Section 4.3 it has been established that the CRW formation level in the Central Training Facility in some instances is up to 3m higher than original site levels pre CRW works. As a result, a degree of settlement would be anticipated in the underlying Glacial Till due to:-

- The unloading and reloading of Glacial Till due to the CRW works and
- The increase in site development levels.

Detailed assessment of settlement has not been undertaken due to the varying nature of the thickness of engineered fill/ 'enhanced' engineered fill and varying degree of increase in site levels. However a simple calculation of total long term settlement has been undertaken based on:

Settlement = 1/E'x increase in stress at centre of Glacial Till layer x thickness of Glacial Till

Settlement = 1/35MPa x (3x20) x 17m = 29mm

Clearly, a lower bound stiffness for the Till has been adopted in the calculation. In addition some settlement would be anticipated to have taken place during earthworks filling. Anticipated further settlement as a result of reloading/additional loading of the Glacial Till would therefore be expected to be insignificant.

5.1.1.2 Consolidation of Engineered Fill under Self Weight

The Engineered Fill appears to have been placed to achieve maximum density for the material won from the remediation of the site in order to limit settlement of formation to within the criteria set in the specification. The specification for the CRW aims at achieving 95% compaction which should limit the future settlements of the general engineered fill due to self-weight. However, given the placed fill thicknesses are of up to 6m (but typically 3m), there is the potential for some time related self-weight consolidation of the fill. The magnitude of this settlement is typically in the range of 0.5 to 1% for well compacted fills (BS6031-2009 7.6.3.2) which would equate to between 15mm and 60mm. It is possible that much of this settlement will have taken place during/since construction. However a programme of surface monitoring has been recommended in order to verify absence of significant on-going settlement of formation (Refer to Section 9 for further details of the monitoring).

As the 'Enhanced' engineered Fill is granular in nature, future settlement of the fill under self-weight is not anticipated to be as significant as for the general Engineered Fill due to point to point contact of the gravel graded material. Settlement under self-weight load would be expected to occur during placement. However, surface monitoring has been proposed to confirm the formation stability.

The Employers Requirements for the CRW requested a maximum permissible formation settlement of 25mm was to be achieved during the earthworks. It is not clear from the ER how this was to be demonstrated and over what timescale and so it is difficult to identify if there is a requirement to allow for up to 25mm of formation settlement in the detailed design solution. Although it is considered on-going settlement due to self-weight compaction is likely to be negligible the above proposed surface monitoring should demonstrate whether any significant post construction movements are still taking place which need to be considered further for detailed design.

5.1.2 Groundwater

The actual shallow ground water conditions at the end of the Phase 1 CRW are not clear at this stage. It is unlikely that there is sufficient time available within the programme to install monitoring wells and allow these to stabilise to determine the long term levels. Trial pitting in the more permeable enhanced earthworks area may show water recharge has started but even if water is found, it might not be at its maximum level.

In the absence of any further information it is suggested that the design incorporates an assumption of a high static ground water level behind retaining structures (which would also take consideration of a burst water main event) and a requirement for waterproof tanking to basement structures.

The design to resist water uplift pressures may be thought of as onerous in the expected post CRW ground conditions, particularly if the basement is within the lower permeability Glacial Till rather than the granular fill, as pore pressures below the slab would be very slow to develop. However there appears insufficient information at present to fully justify this assumption.

For structural design, it would seem pragmatic to adopt an assumed ground water level of 1m below final ground level.

5.1.3 Gas Protection

The ground gas regime is currently unclear given the extent of earthworks that is being undertaken by BGCL. Further gas monitoring is being undertaken within instruments installed within the Phase 1 area. Gas protection requirements are to be assessed following monitoring. At this stage it should be assumed that some measures are likely to be required.

5.1.4 Concrete Classification

WYG indicated AC-04 classification for concrete in their interpretative report. The Capita Symonds concrete specification indicates design concrete class: To suit AC-4 and DS-4 classifications.

Specific sulphate testing to inform concrete classification or to demonstrate requirements for remediation strategy was not undertaken during the CRW. However, during the CRW, sulphate testing undertaken by May Gurney (ref SI1565) to look at ground improvement using lime mixing concluded that sulphate levels were sufficiently low that use of lime as a modification would not pose a risk for heave of materials.

In lieu of further information, it is not possible to reassess the concrete classification from that reported from the original WYG investigation i.e. AC-4 and DS-4 classifications. In addition, as the site won material has now been placed as Engineered Fill it is likely that the concrete classification has altered due to remediation and removal of contaminated material and use of lime.

However, if additional testing of the Made Ground following completion of the CRW is undertaken, it may be possible to relax concrete classification from that presented as specification requirements but in the absence of additional data on the Engineered/Enhanced engineered Fill it is not possible to reduce classification requirements from that specified by Capita Symonds in the Employers Requirements for Concrete Specification

5.1.5 Obstructions

Following clarification during a meeting with BGCL/BWB on 28th September 2012, it has been confirmed that obstructions have then been chased out to a depth of 2m below the top of Glacial Till. Where obstructions extend beyond 2m, they have been left in situ. It is known that there are areas of piles where obstructions remain at depth as shown in **Figure 17**. No additional dig was undertaken to account for the deeper basement areas.

Comment on design implications/requirements regarding building specific obstructions are discussed further in the building specific section of this report.

5.1.6 Shaft Treatment

BGCL were required under the Employer Requirements to identify and treat mine shafts. In order to meet this requirement, BGCL have confirmed that their CRW scope included the excavation to confirm the base of Made Ground i.e. the top of the underlying Glacial Till. By undertaking this work and inspecting the formation, they believe they have mitigated the risk of encountering unforeseen shafts which could otherwise impact on the development.

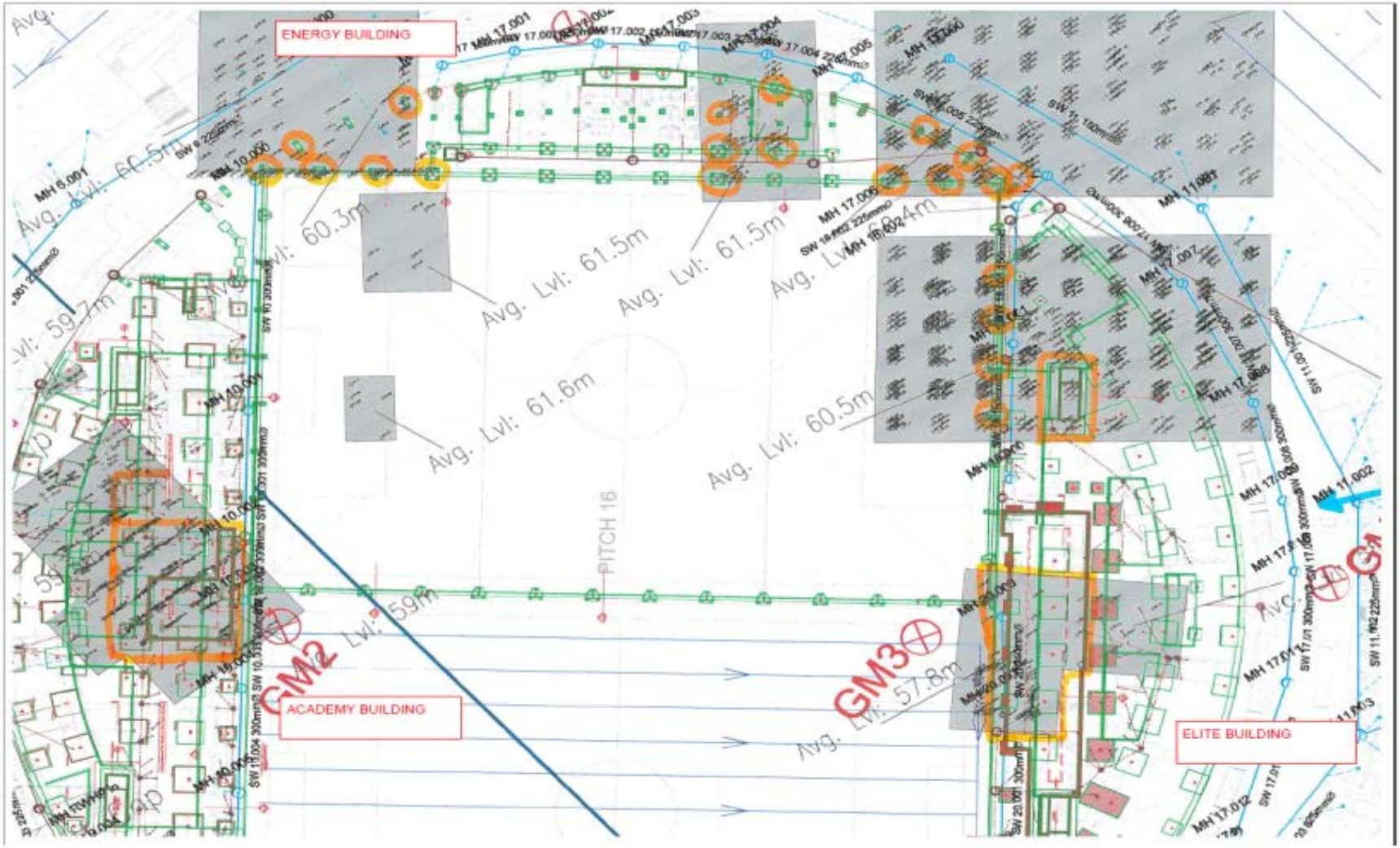


Figure 17: Location of Historical Piles broken down but still in situ

5.2 Considerations specific to the Elite and Academy Buildings

5.2.1 Basis of design

The following design basis is assumed for the Elite and Academy Buildings:-

Concept: Pad and ground bearing foundations placed on 'enhanced' engineered fill above Glacial Till

Finished Slab Levels: Typically 63.9mOD but localised lift pits and basement structures, with structural slab levels down to 59.95mOD.

Assumed Loads on ground below slabs: 14.25 kN/m^2

Foundation Loads: Pads assumed to be sized for 150kPa bearing pressure

The following drawings were used to as reference for the proposed pad layout and required level:-

- S112.F2.E1 and 2 rev C2
- S112.F2.E3 and 4 rev C3

The drawings show typical pad formation levels between 58.95mOD and 62.7mOD for the general building support.

The finished ground floor slab is understood to be at 63.9mOD and of 175mm thickness. The deeper pads are generally associated with the building core/lift shafts. The drawings highlight that for both buildings in the area of the basement incorporating the hydropool, the pad foundation for surrounding basement building columns are significant in size and have been positioned at basement slab level in order to avoid loading the basement walls.

The basement slab formation level is assumed to be 59.5mOD (450mm in thickness) and the pads are typically 900mm thick so founded at 59.05mOD.

It is understood that in the Arup design, the pads will be integral with the basement slab. This will avoid potential issues with bearing capacity failure and no checks have been undertaken on bearing capacity as a result.

In order to consider differential settlement and reinforcement requirements between the heavily loaded integral pad and the adjacent thinner basement slab to control cracking, the pads have been modelled separately to the basement slab. For this model the basement slab is founded at a proposed level of 59.5mOD with a founding level of 59.05mOD for the pads.

5.2.2 Ground Conditions below building footprints

5.2.2.1 Enhanced Engineered Fill

Material is believed to comprise between 3m and 8m of compacted 6N/6F2 granular fill. The base of the Enhanced Engineered Fill below the Elite Building varies between 57mOD and 60.5mOD. The base of the Enhanced Engineered Fill below the Academy Building varies between 57.5mOD and 61.5mOD.

Based on the typical level of pad foundations the thickness of Enhanced Engineered Fill below the pad foundations varies between about 1m and 5.5m.

Below the building core formation the Enhanced Engineered Fill is:-

- Typically 1 to 2m for the Academy Building
- Typically 1 to 3.5m for the Elite Building

In the area of the basement formation the Enhanced Engineered Fill is:-

- Typically 0m to ~2m for the Academy Building (partly founded in Till)
- Typically 0m to ~3m for the Elite Building (partly founded in Till)

Remediation validation report appears to show some treated Galligu material has been placed under the southern corner of the Academy Building. BWB initially indicated in a meeting of 28th September 2012 that Galligu placed below building footprint was subsequently removed and replaced. Further discussion revealed that the stabilised material was not removed and potentially remains below building footprints, albeit primarily below the southern CTF area (the former Laundry building).

The base of enhanced earthworks is understood to have exposed the Glacial Till.

The derivation of the stiffness of Enhanced Engineered Fill is defined in Section 4 and design parameters are presented in **Table 1**.

5.2.2.2 Glacial Till

The top of Glacial Till is defined by the base of Enhanced Engineered Fill above.

Glacial Till is anticipated to extend to a level of about 42 and 42.5mOD for these areas as shown in the cross-sections in **Figure 6** and **Figure 7**

Glacial Till with Cu>100kPa was confirmed at the upper surface based on hand vane tests on proof rolled Till and has been adopted in design.

The Till is assumed to increase in strength to around 300kPa at rockhead and has been adopted in design. The derivation of the stiffness of Glacial Till appropriate to the proposed design solution is defined in Section 4 and design parameters are presented in **Table 1**.

5.2.2.3 Bedrock and Bradford Fault

It appears that the Bradford Fault passes through the site of the Academy building. BWB's review of potential workings identified a requirement for grouting of

workings to the west of the fault. Drilling and grouting was on a 12m grid (with additional 6m grid drilling and grouting where notable grout takes were observed). A point of note is that the dip of the fault is at 45degrees to the horizontal and assuming the outcrop of the fault is shown on the drawing the grouting at depth is unlikely to be up to the line of the fault as shown in **Figure 6**. As a zone of faulting is anticipated either side of the fault line it is considered fairly unlikely that mining would have taken place in the location close to the fault line at shallow depth.

The Elite building footprint is expected to sit above the Sherwood Sandstone/Mercia Mudstone Group on the eastern half of the site. In places the Sandstone is very weak with very closely spaced discontinuities.

The area of east Manchester was affected by a swarm of minor earthquakes between October 2002 to January 2003, with a maximum recorded magnitude of 3.9. It is believed these originated at between 3 and 6km below ground.

There is no widespread evidence of significant movements occurring on the Bradford Fault but as it is a plane of geological weakness, future fault reactivation cannot be fully ruled out.

Historically it has been good practice to avoid building directly over faults in mining areas because of issues with movement of deep mine works close to faults. The level of movement that might be expected is mm's and as a result unlikely to affect the integrity of the structure but superficial cracking may be possible requiring redecoration and the architectural design should consider the location of the fault with regards to jointing of brickwork etc.

5.2.3 Ground Constraints

5.2.3.1 Historical Piles

There are areas of proposed basements in the Academy and Elite buildings where pile obstructions have not been removed to 2m below basement formation/level of foundations below the basement as was required in the CRW (see **Figure 17**). Consequently further constraints removal will be required in these areas. It is recommended that obstruction removal to a depth of 1m below underside of the basement structure would be sufficient to remove the hard spot risk.

Non-compliances with the original CRW requirements on obstruction removal are likely to result in requirements for additional design and influence design programme. In addition if additional CRW are required on site this is likely to influence construction risk, temporary works design and programme.

5.2.3.2 Carrier Drain Treatment

The constraints drawing shows a service easement crossing Phase 1 that crosses the Academy Building.

A grouted sewer pipe is shown at 5m below GL passing below the Academy building. This is potentially an obstruction to the basement slab.

BWB confirmed in a meeting on 28th Sept 2012 that the carrier drain which historically ran below the Academy Building and former Laundry building has been excavated and removed. Elsewhere in the Phase 1 area the drain has been grouted.

5.2.3.3 Treatment of Shafts

In the original constraints plan two shafts were shown potentially in the location of the Academy building or close proximity, Shaft H (not located) and Shaft Q (4m diameter). The mine workings validation report by BWB has identified that Shaft H was located just to the west of the record position outside the footprint of the building. Shaft H was recorded as 2.6m diameter, backfilled with concrete but without a 'cap'. The shaft was grouted and a concrete cap installed 6m by 6m. Shaft Q (also shown outside the footprint of the building to the NE of Shaft H) was not shown as a recorded shaft location but during site works a <2.5m diameter shaft was encountered. Treatment of Shaft Q comprised temporary backfilling of the top 4m of the shaft with 6F2 followed by drilling and grouting and installation of a cap 6m x 6m.

5.2.4 Settlement Analysis

The OASYS software PDISP was used in the settlement analysis of the proposed Elite and Academy Building Designs.

A simplified model was constructed for each area using the slab and foundation loading indicated above and design stiffness profiles presented in **Table 2** below.

Table 2: Design Stratigraphy and Parameters Specific for Elite and Academy Building

Strata	Assumed Vertical Extent (mOD)	Comments/ Assumptions	Parameters used in the assessment
Enhanced Engineered Fill (MG)	63.9 to 59.5mOD	Uniform thickness assumed but fluctuates over the site areas. There is an assumption that the site testing is representative of the drained stiffness of the fill. The stiffness is also weighted towards the results of CRW Area Load Tests.	Drained Young's Modulus E' = 60,000 kN/m^2, Poisson Ratio = 0.2
In-Situ Glacial Till	59.5 to 42.9mOD	Linear increase in strength. Assumed E = 500Cu. (conservative as strain related to net load/ult. load likely to be significantly lower at depth)	Drained Young's Modulus E' = 35,000 kN/m^2 at base of Fill increasing to 100,000 kN/m^2 at rockhead. Poisson Ratio = 0.2
Bedrock	All material below 42.9mOD	Little information on rock strength, however very little influence on settlements.	Assume rigid boundary to model

The limitations and assumptions of the settlement modelling undertaken are as follows:

- The software does not take in to consideration any self-weight settlement of the fill or settlements due to increases in site levels, as described previously. The issue is likely to be time dependent and have greater potential to impact on overlying construction for development taking place within a short time frame of completion of filling and become less significant with time. Monitoring of formation will confirm the required degree of consolidation has taken place prior to construction so further modelling and assessment is not required.
- No allowance has been made for variation in level of top of Glacial Till.
- The foundation sizing has been based on the limiting peak stress of 150kN/m^2 (as per the specification requirements) which is fairly uniformly distributed under the main pad foundation areas. Slab loads have been provided as appropriate.
- The software assumes that the loading is flexible and the rigidity of the foundation is not taken in to account. The inherent stiffness of the foundation itself would tend to equalise movements between the centre and edge of the loaded area.
- Only the pads in one half of each building have been modelled although the ground floor slab for the whole building area was modelled and also, the core slabs plus the full basement slab and associated pads. Output is only representative on the fully modelled portion.
- The loaded areas have been set at the base of the pad and slab formations as currently proposed.
- The proposed loadings for basement slabs have been adjusted to reflect the net stress increase at foundation depth. For load related to the hydropool the extreme load conditions have been considered as part of a sensitivity analysis.
- The load is expected to fluctuate under the ‘core’ areas where a significant load component is attributable to transient wind loading rather than permanent static loading. This transient load would be short term and so behaviour in the soils would be slightly different than assumed in modelling (which looks at long term stiffness). The model is simplified so that an average increase in live load is applied uniformly across the core slab on top of dead load. The dead load weight of the core foundation alone was also considered as part of a sensitivity analysis to give a more accurate view on the anticipated core behaviour.
- Settlement is reported at underside of ground floor slab level, taken as being 63.65mOD for both structures. Settlement is reported for basement slab level at 60.9mOD for the Academy Building and 59.5mOD for the Elite Building.

The model plan layouts are shown in **Figure18** and **Figure 24** for information and orientation for the results contour plans shown in **Figure 19** to **Figure 23** and **Figure 25** to **Figure 29**.

5.2.4.1 Results

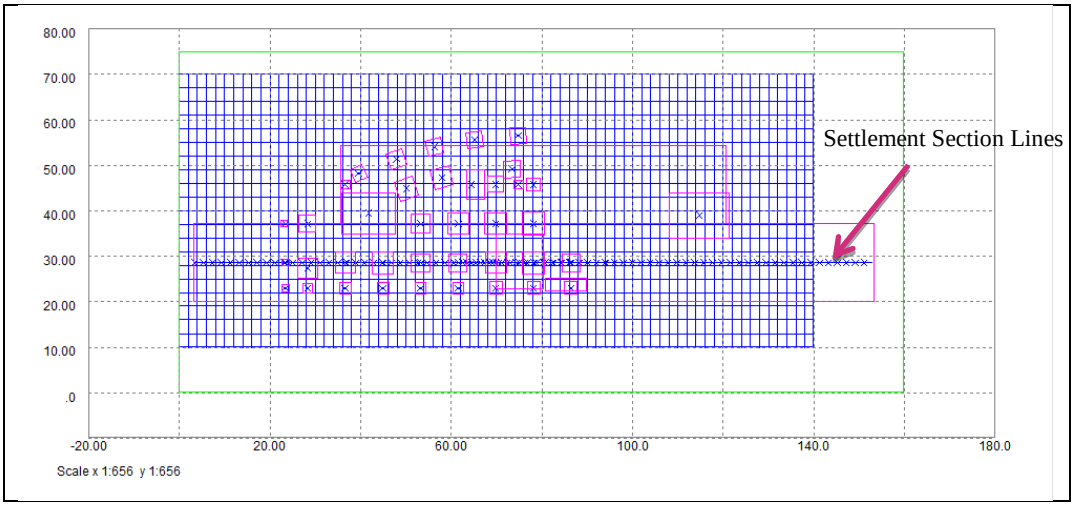


Figure 18: Academy Building PDisp Model Layout (southern half pads considered only)

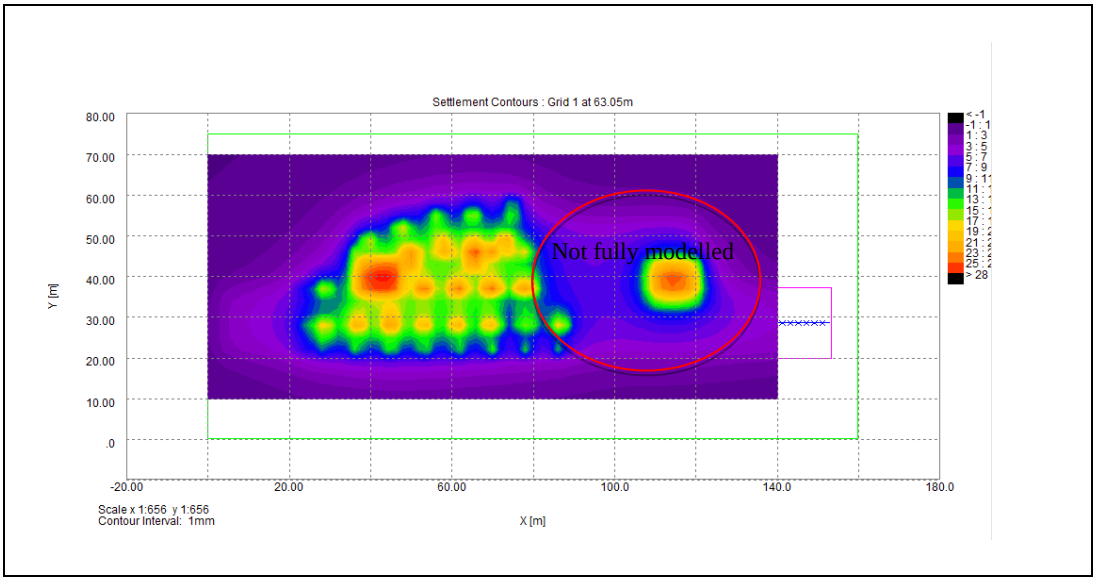
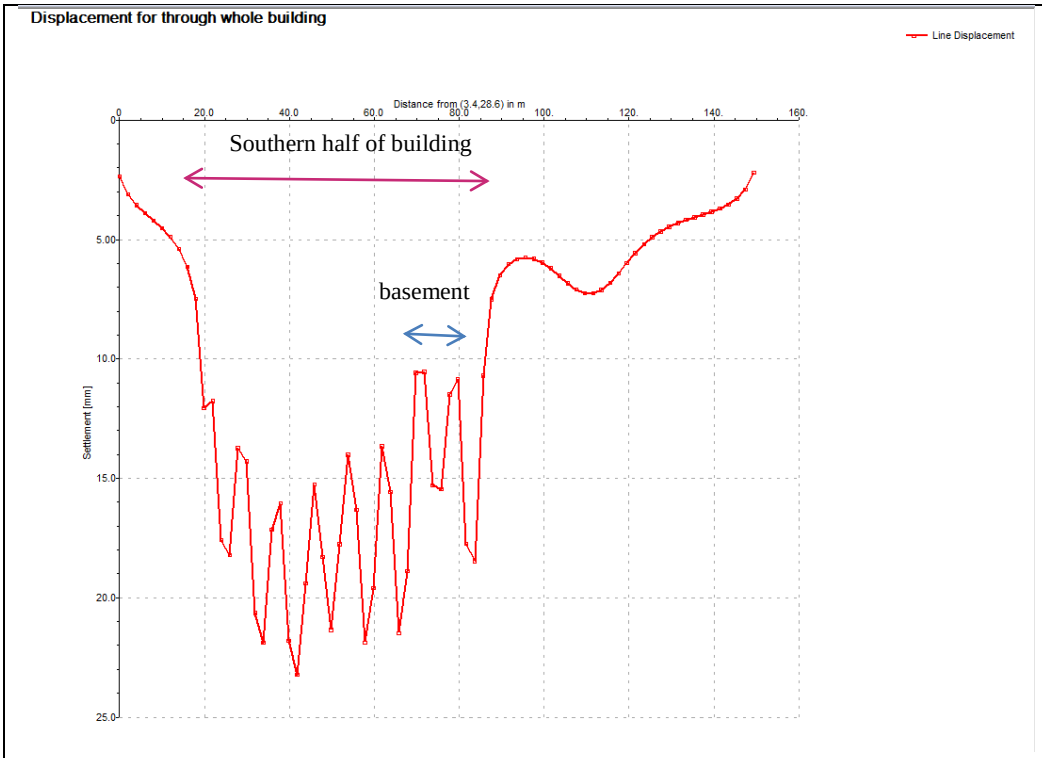
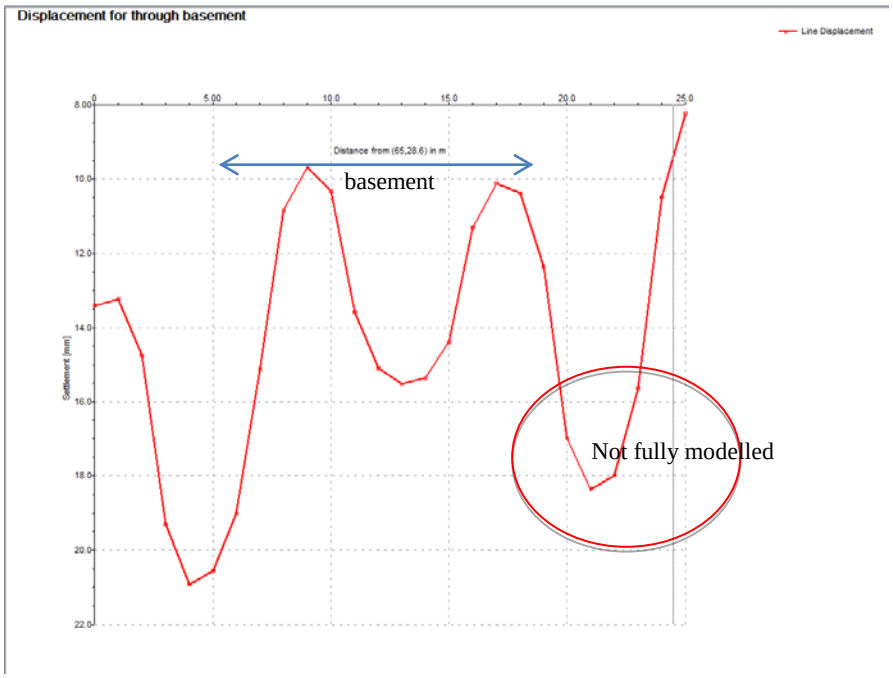


Figure 19 Run 1 Academy Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool empty, DL + ave. LL for core pads

Note: Southern half representative ONLY, based on lower bound E' values



Settlement at 63.05mOD Ground Floor Slab Formation



Localised Settlement at Basement Floor Slab Formation

Figure 20 Run 1 Academy Building Settlement –case: pool empty, DL + ave. LL for core pads

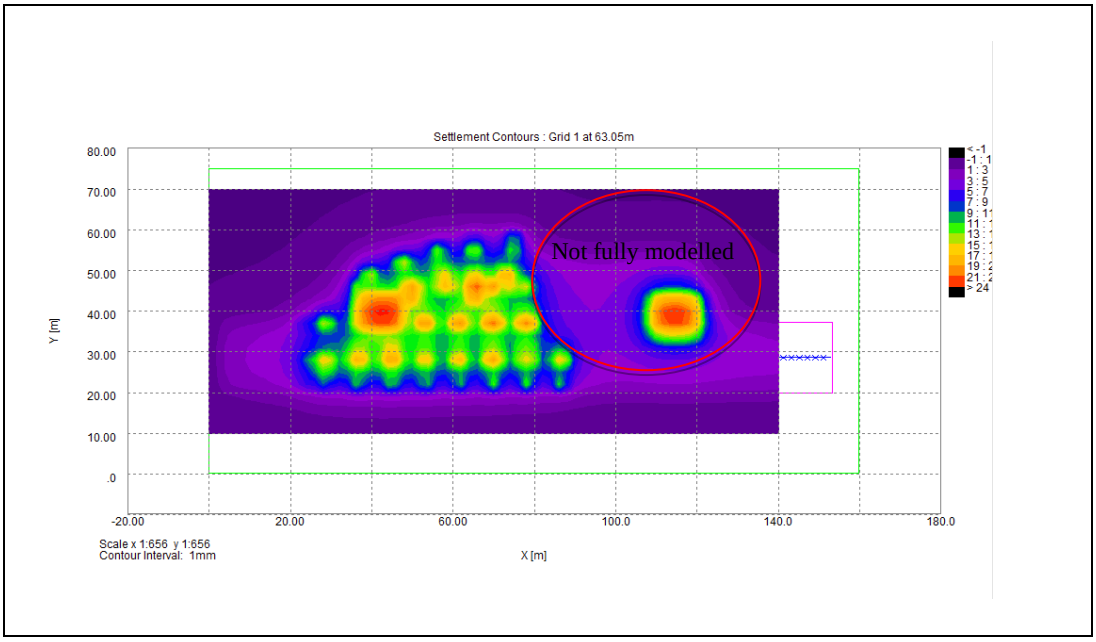
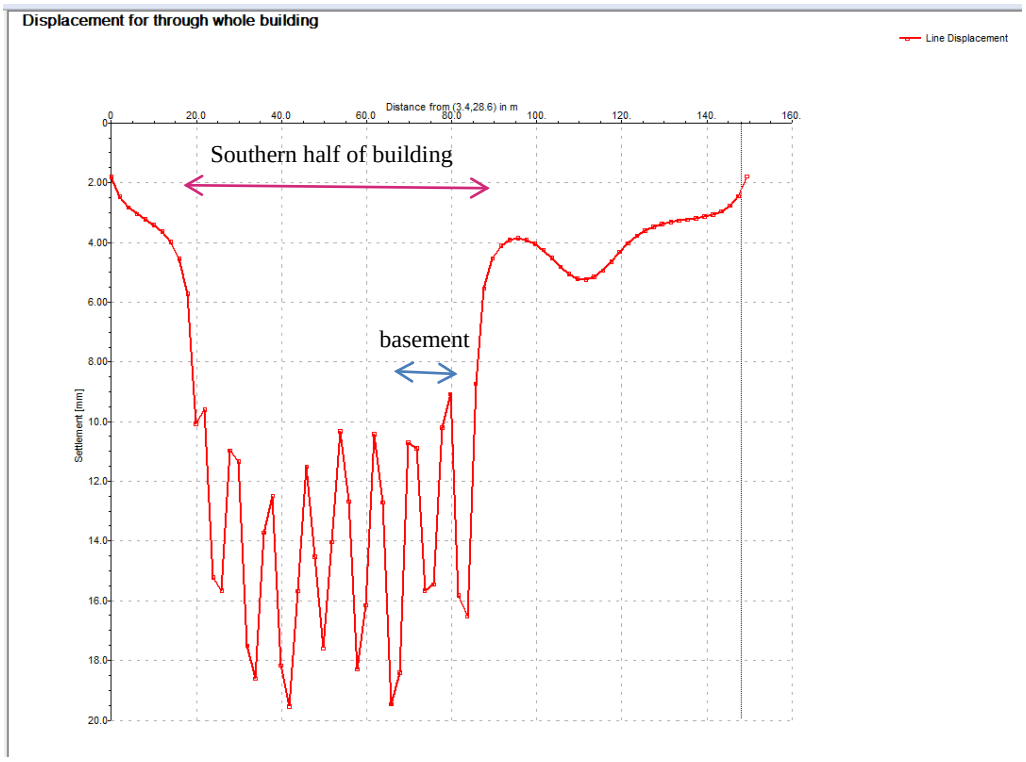
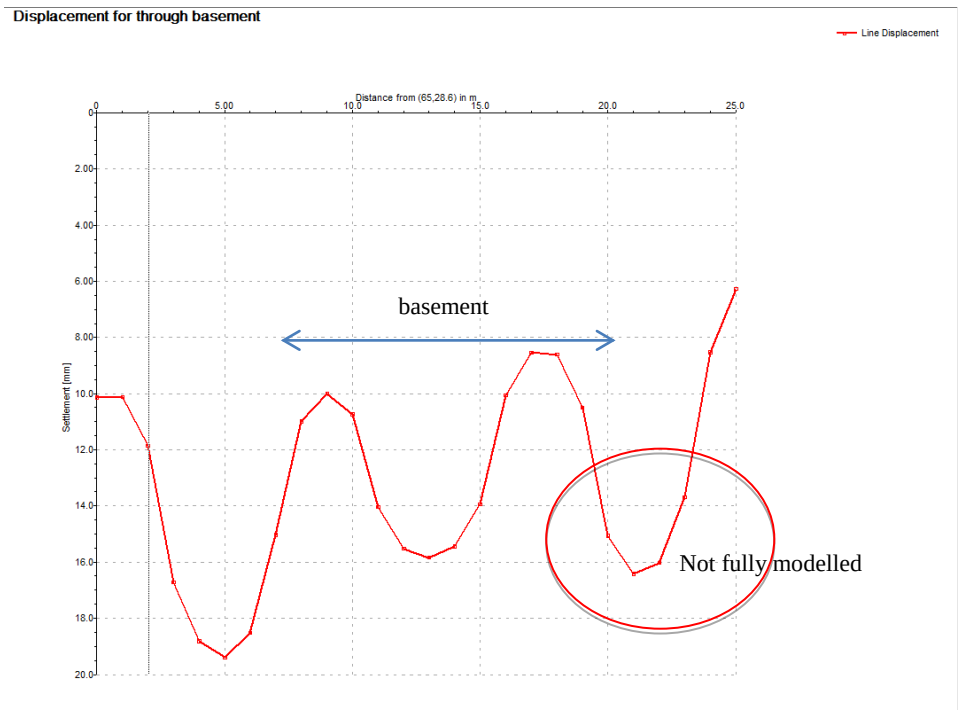


Figure 21 Run 2 Academy Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool full, DL + ave. LL for core pads

Note: Southern half representative ONLY, based on lower bound E' values



Settlement at 63.05mOD Ground Floor Slab Formation



Localised Settlement at Basement Floor Slab Formation

Figure 22 Run 2 Academy Building Settlement –case: pool full, DL + ave. LL for core pads

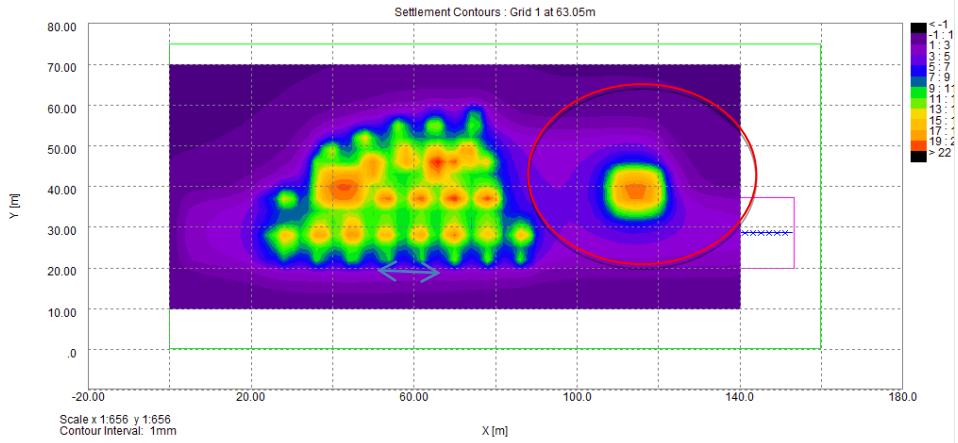


Figure 23 Run 3 Academy Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool full, DL only for core pads

Note: Southern half representative ONLY, based on lower bound E' values

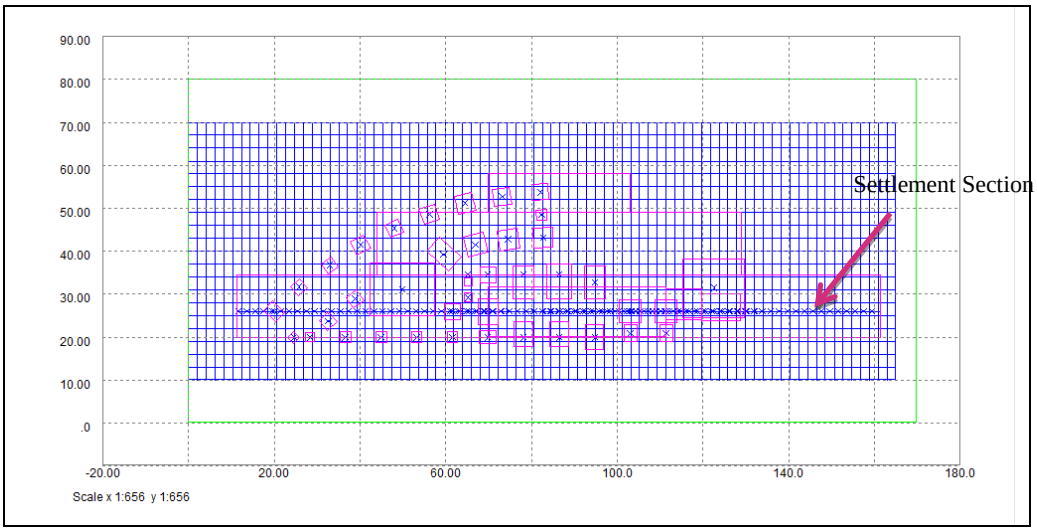


Figure 24 Elite Building Model PDisp Layouts (Northern Half pads considered only)

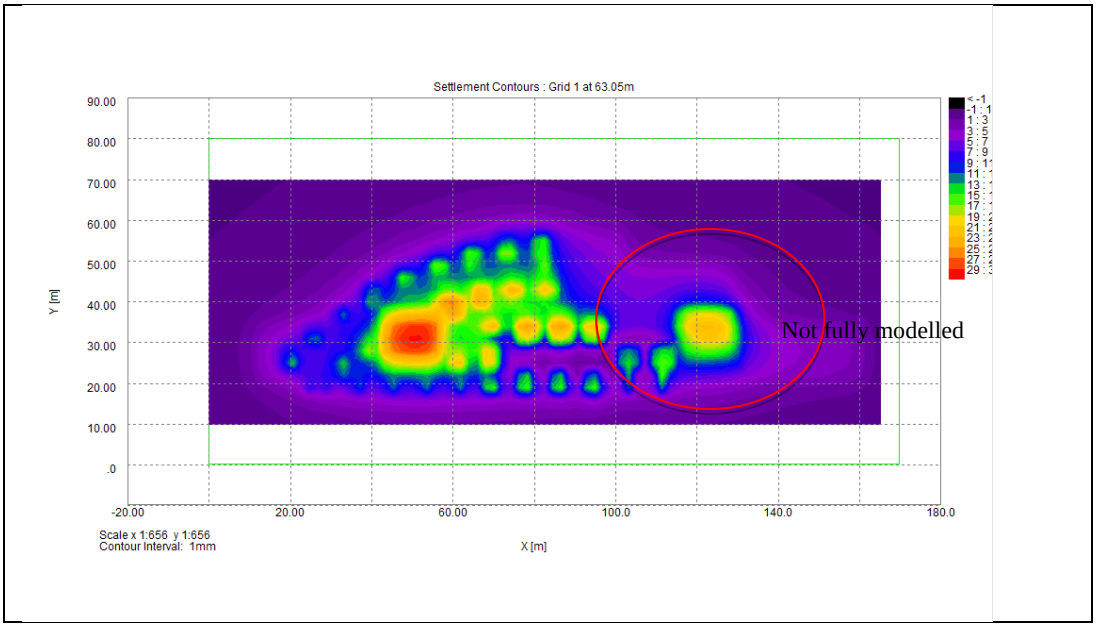
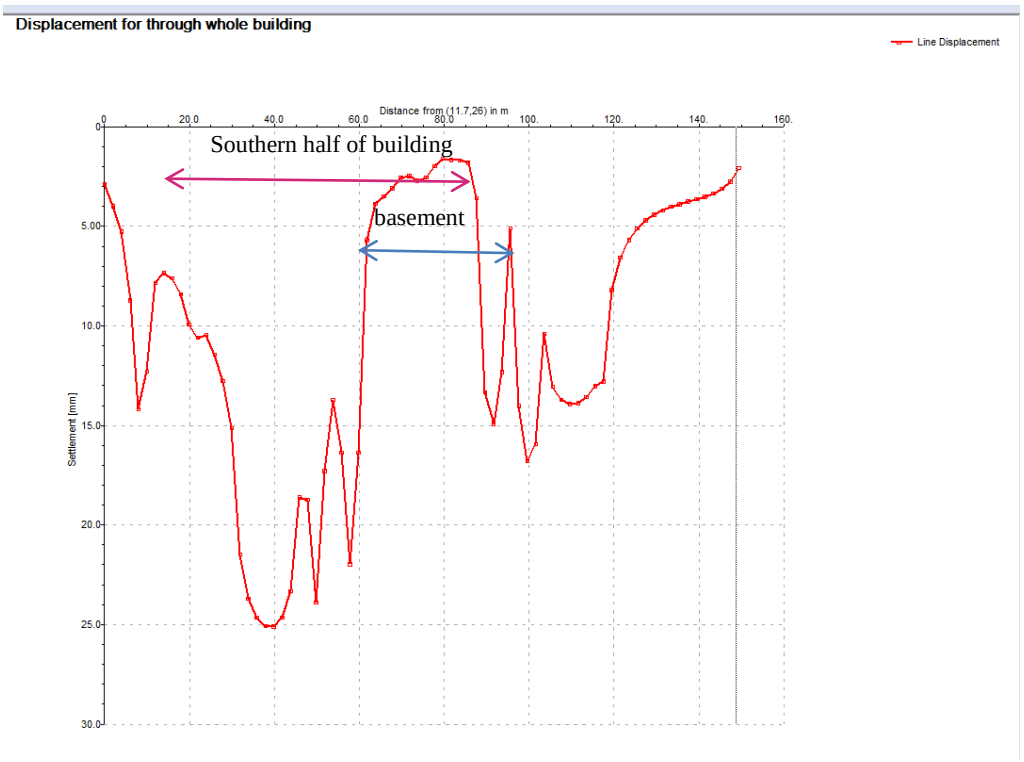
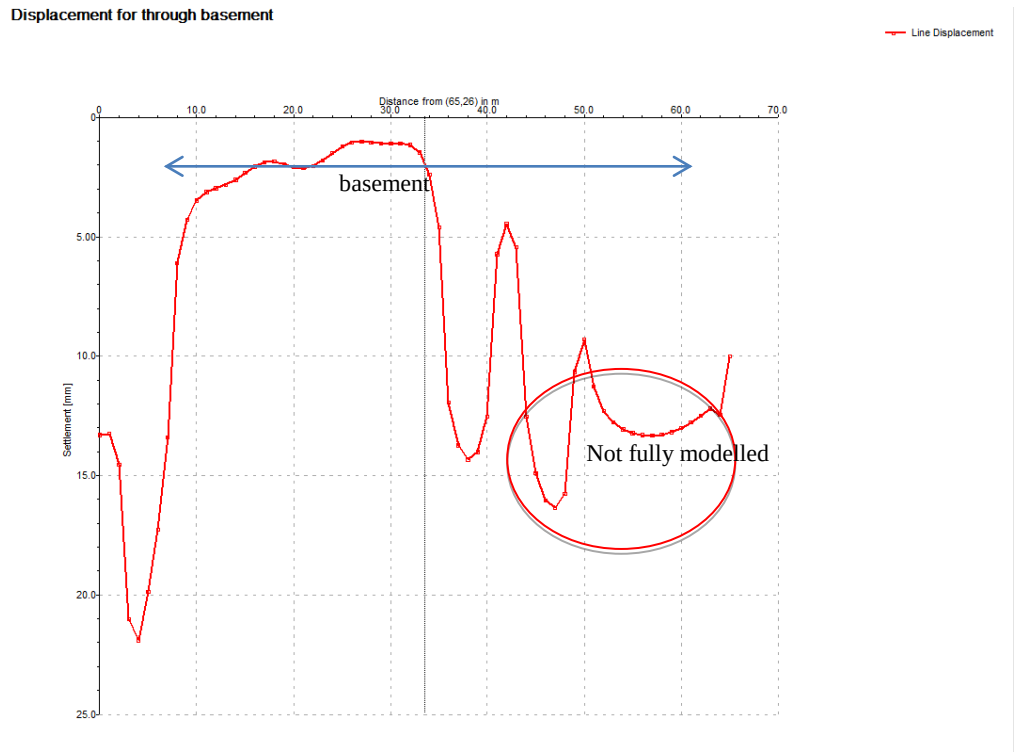


Figure 25 Run 1 Elite Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool empty, DL + ave. LL for core pads

Note: Southern half representative ONLY, based on lower bound E' values



Settlement at 63.05mOD Ground Floor Slab Formation



Localised Settlement at Basement Floor Slab Formation

Figure 26 Run 1 Elite Building Settlement –case: pool empty, DL + ave. LL for core pads

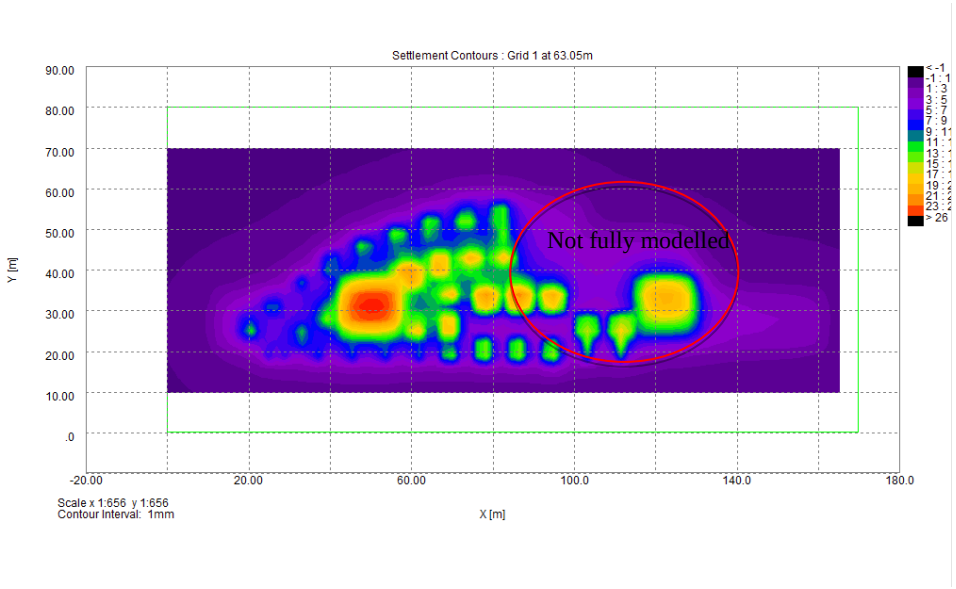
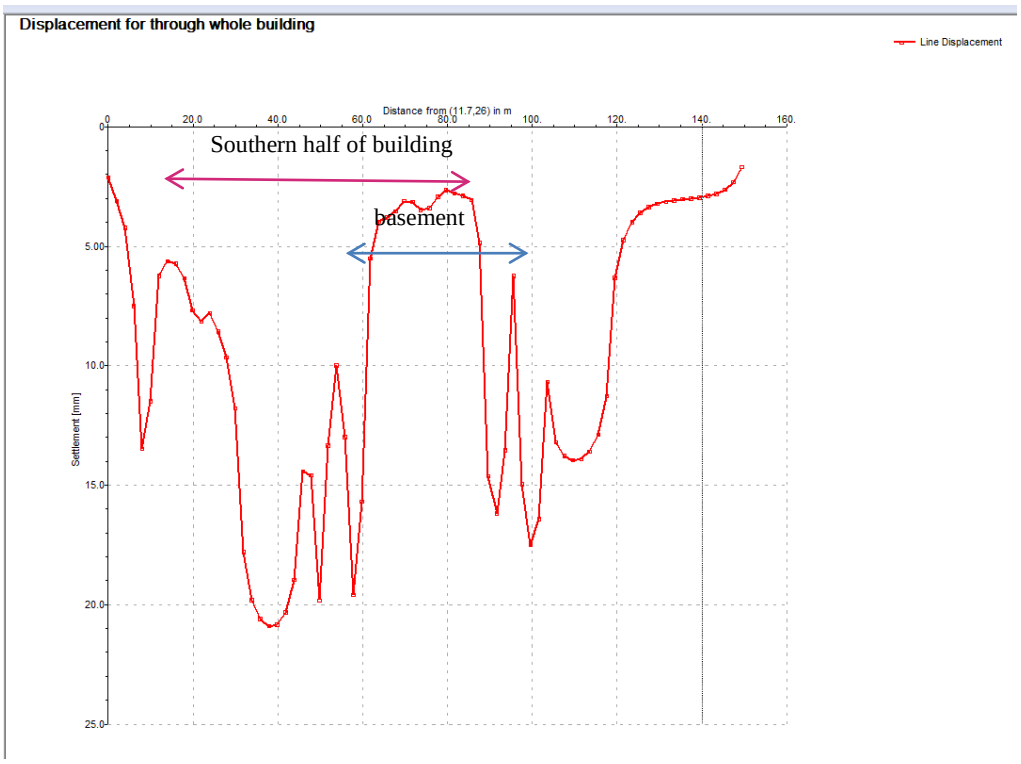


Figure 27 Run 2 Elite Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool full, DL + ave. LL for core pads

Note: Southern half representative ONLY, based on lower bound E' values



Settlement at 63.05mOD Ground Floor Slab Formation



Localised Settlement at Basement Floor Slab Formation

Figure 28 Run 2 Elite Building Settlement –case: pool full, DL + ave. LL for core pads

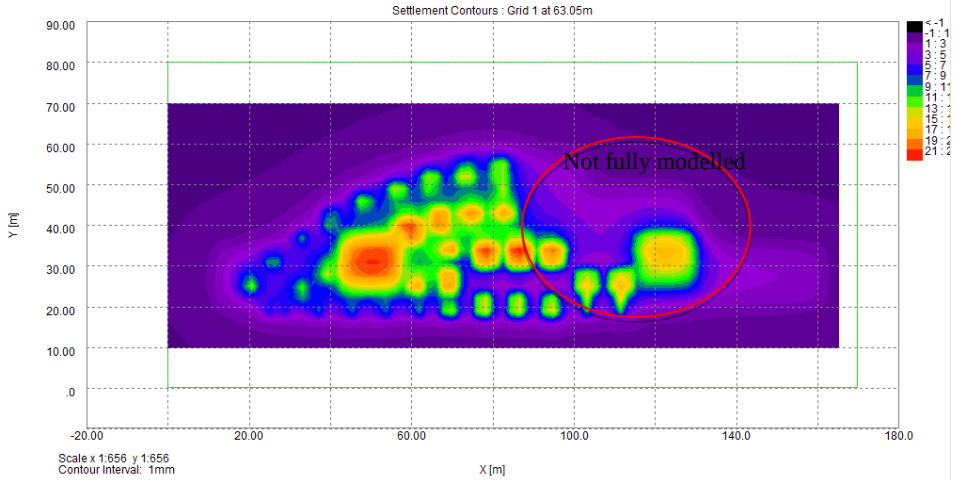


Figure 29 Run 3 Elite Building Settlement Contours at 63.05mOD Ground Floor Slab Formation –case: pool full, DL only for core pads

Note: Southern half representative ONLY, based on lower bound E' values

Notwithstanding the above assumptions and recommendations for monitoring, the estimated settlements due to the foundation loading alone considering sensitivity analysis to address variation in load conditions with time can be summarised in the following **Table 3** and **Table 4**.

Table 3: Summary of PDisp Runs for Academy Building

Academy Building			
Model Run	Description	Purpose	PDisp output
Run 1	DL + ave. LL cores, net load basement (hydropool empty)	Allowance for live load on core (conservative as E' considered) Worst heave condition for basement slab	Range of settlement = 5-25mm Differential settlement between basement slab and basement pad negligible as pads large in relation to basement area Differential Settlement at junction of ground slab and basement wall = 12mm
Run 2	DL + ave. LL cores, net load basement (hydropool full)	Allowance for live load on core (conservative as E' considered) Worst load condition for basement slab	Range of settlement = 5-21mm Differential settlement between basement slab and basement pad negligible as pads large in relation to basement area Differential Settlement at junction of ground slab and basement wall = 9.5mm
Run 3	DL only cores, net load basement (hydropool full)	Sensitivity analysis for core for range of anticipated load	Anticipated range of settlement for core slabs = 15-25mm with 10mm differential settlement at edge of core

Table 4: Summary of PDisp Runs for Elite Building

Elite Building			
Model Run	Description	Purpose	PDisp output
Run 1	DL + ave. LL cores, net load basement (hydropool empty)	Allowance for live load on core (conservative as E' considered) Worst heave condition for basement slab	Range of settlement = 5-30mm Differential settlement between basement slab and basement pad = 13mm Differential Settlement at junction of ground slab and basement wall = 20mm
Run 2	DL + ave. LL cores, net load basement (hydropool full)	Allowance for live load on core (conservative as E' considered) Worst load condition for basement slab	Range of settlement = 5-25mm Differential settlement between basement slab and basement pad = 13mm Differential Settlement at junction of ground slab and basement wall = 17mm
Run 3	DL only cores, net load basement (hydropool full)	Sensitivity analysis for core for range of anticipated load	Anticipated range of settlement for core slabs = 15mm to 30mm with 14mm differential settlement at edge of core

In summary, the current PDISP settlement assessment is believed to be an upper bound representation of displacements due to foundation loading. However no allowance has been made for variation in top of Till level which may cause some additional differential settlement across the buildings.

The assessment excludes any settlement as a result of consolidation of fill material and build-up of ground from historical levels discussed elsewhere.

5.2.4.2 Further Considerations

Due to the required depth and close proximity of the foundations there is a possibility that formation materials could be loosened during foundation construction. The construction sequence and temporary works requirements will need significant consideration to mitigate the risks of the formations being weaker than required for the design. In addition, the backfilling to the caps will need to ensure that the replaced materials are at least the same stiffness as the material installed during the CRW.

Settlement may also be influenced by ground water and rainwater ingress where founding material is granular although this is unlikely to be a significant issue. Where founding material is cohesive formation should be protected in short term by leaving excavation 200mm higher. Any material softened through water saturation should be removed prior to pad/slab construction and may result in lower pad levels than proposed in design. Requirements should be addressed in the specification for construction.

5.2.5 Further Monitoring

The CRW included the installation of 8 No settlement monitoring stations in the CTF enhanced engineered fill areas. It is recommended that these are monitored to establish whether any settlements are occurring. Furthermore it has been recommended that additional surface settlement stations are installed above deeper areas of fill, particularly where surface levels have been increased, and monitored until the effects of this filling are established.

5.3 Considerations specific to the Energy Building and Southern CTF area (formerly the Laundry building)

5.3.1 Basis of design – Energy Building

The foundation solution to the Energy Centre and southern CTF (formerly the Laundry Building) were originally intended to be piled as presented in the Stage D Capita design report.

However, as part of a value engineering exercise, a shallow foundation solution has been assessed for the Energy Building. The main reasons for considering this solution are:

- Discussions with the CRW contractor, Buckingham, and assessment of their validation data suggests that the quality control procedures and strength of the engineered fill adopted at the location of the energy building was high.
- It was established that the CRW had not fully removed the existing piled foundations from this area and therefore the obstructions would place significant constraints on the design and installation of further piles and substructure.

The following design basis is assumed for the Energy Centre.

Concept: Pad and ground bearing foundations placed on 'general' engineered fill above Glacial Till. Due to the varying depth of the Till some of the pads may lie directly on the Glacial Till.

Finished Slab Levels: Typically 63.9mOD but localised lift pits and basement structures are deeper.

Assumed Loads on Slabs:

Pads have been sized for a maximum of 150kPa bearing pressure; the actual loads on each pad were provided as part of the structural assessment.

The following drawings were used as a reference for the proposed pad and slab layout and required level:

- General Arrangement to Foundations - S112.F2.E6 revision C2
- General Arrangement to Ground Floor Slab - S122.00.E6 revision C2

The drawings show typical pad formation levels between 60.95mOD and 62.7mOD.

The finished ground floor slab level is understood to be at 63.9mOD; the slab has a 150mm thickness. The pads are generally 600mm thick with a top level of 63.3mOD.

The drawings highlight the requirement for large mass concrete pads underlying the service areas, containing water buffer vessels which when full would induce a large load. These pads are currently 1.2m thick with a top level of 63.9mOD.

The service basement has a top level of 61.8m; the slab has a 250mm thickness. The pads below this basement have a top level of 61.55m; the pads are 600mm thick.

5.3.2 Southern area of CTF (formerly Laundry area)

In March 2013, a decision was made to relocate the Laundry Building from the CTF area to the Facilities Management building in the Phase 2 site area.

This decision was made partly on cost grounds and partly due to uncertainties regarding the presence of stabilised Galligu below the proposed building footprint.

5.3.3 Ground Conditions Below Building Footprints

5.3.3.1 General Earthworks Fill

Records suggest that the existing Made Ground was excavated to depths of between 2 to 6m (base of excavation at 59-60mOD). The fill was processed and stockpiled as discussed in section 4.

Acceptable general fill, which was typically a Class 2A or 2C cohesive material, was then recompacted to formation level in line with the Earthworks specification and performance criteria. i.e. CBR of >5% at formation level and a target 95% compaction from equivalent CBR tests as per benching trials.

A final capping layer was placed on completion of sub formation which comprised 100mm to 150mm of 6F1. The base of the Engineered Fill below the southern CTF (formerly the Laundry Building) varies between 58.5mOD and 62.5mOD. The base of the Engineered Fill below the Energy Building varies between 60.5mOD and 63.0mOD.

5.3.3.2 Removal of Existing Made Ground

The CRW contract did not specify a requirement for Made Ground be removed and recompacted to natural Glacial Till level. It is possible that some existing Made Ground may have been left in place and just proof rolled at excavation base. However, according to the validation report for Phases 1, 2 and 3 there is no original Made Ground left in place, with all material having been excavated and replaced with engineered fill during the CRW. The design assessment therefore assumes this to be the case.

The Made Ground that was removed below the southern CTF (former Laundry Building) largely consisted of gravelly sand and sandy gravel with coal, clinker, brick and concrete fractions up to cobble sized; over gravelly clay.

The Made Ground in the Energy Building area consists of concrete, brick clay ash fill, a multi-coloured silt with occasional gravel and black tarry silt at 2.5m BGL with a strong hydrocarbon odour.

5.3.3.3 Glacial Till

The top of Glacial Till is defined by the base of Engineered Fill above.

The Till is anticipated to extend below the Made Ground and fill deposits to a level of around 42m and 43mOD within these areas.

Glacial Till with Cu>100kPa was confirmed at upper surface based on hand vane tests on proof rolled Till.

The Till is assumed to increase in strength to around 300kPa at rockhead as suggested in Section 4.

5.3.3.4 Bedrock and Bradford Fault

It appears that the Bradford Fault passes through the site of the southern CTF area (formerly Laundry Building). BWB's review of potential workings did not appear to show workings to the west of the fault in the area of the former Laundry Building. As a result no grouting of worked seams was undertaken within the footprint of the former Laundry Building although drilling and grouting of shallow worked seams was undertaken just to the south of the former Laundry Building. Drilling and grouting was undertaken on a 12m grid (with additional 6m grid drilling and grouting where notable grout takes were observed).

The Energy Building footprint is expected to sit above the Sherwood Sandstone/Mercia Mudstone Group on the eastern half of the site. In places the Sandstone is very weak with very closely spaced discontinuities.

East Manchester was affected by a swarm of minor earthquakes between October 2002 and January 2003, with a maximum recorded magnitude of 3.9. It is believed these originated at between 3 and 6km below ground.

There is no widespread evidence of significant movements occurring on the Bradford Fault but as it is a plane of geological weakness, future fault reactivation cannot be fully ruled out.

Historically it has been good practice to avoid building directly over faults in mining areas because of issues with movement of deep mine workings close to faults. The level of movement that might be expected is in the order of millimetres and as a result unlikely to affect the integrity of the structure. Superficial cracking may be possible which would require redecoration and the architectural design should consider the location of the fault with regards to jointing of brickwork etc.

5.3.4 Ground Constraints

5.3.4.1 Services

The constraints drawing show a services easement crossing Phase 1 that crosses the southern CTF area (formerly the Laundry Building) and Energy Building. However it does not appear that services have been left in situ in these locations.

A grouted sewer pipe is shown at 5m below GL passing below the site of the former Laundry Building. This would be a potential pile obstruction if it was not removed; BWB indicated in their meeting, on 28th September, that the carrier drain was grubbed out below the building footprint. If a shallow foundation solution is adopted for any future build in this area, this should not cause any obstruction.

5.3.4.2 Unidentified Structure

The proposed foundation solution includes foundation groups close to a 6m diameter shaft, “feature S”, in the southern CTF area (formerly the Laundry Building). The feature and potential impact on design are discussed in more detail below.

As part of these works a shaft like feature ‘Shaft S’ was identified below the former site of the proposed Laundry Centre, in the south west of the CTF. The records of this are presented in Appendix B of the BWB validation report for mine workings. The feature is identified on one of the photographs within the report and located as indicated in Figure 29.



Figure 30: Shaft Feature S

The following details have been established from the validation report:

- It appears that it was approx. 6m diameter circular brick lined feature centred on coordinates (387484.4, 398091.2).
- The feature was not a shaft recognised on the Coal Authority plans.
- The base of the feature is only 10m deep and terminates within the Glacial Till. Therefore it is not likely to be coal mining related as the bedrock is reported to be 21m below here. BWB believe it is a chimney base or tank.
- The feature was found to be filled with "soft and firm fill".
- The feature was grouted up by BGCL using 9 injection points at 1.5m centres.
- No cap has been constructed over this feature.
- BWB deem that no further works were necessary and the feature is stable.

The main potential impacts on the proposed development are as follows:

- It would represent a physical obstruction to piling or foundation construction within this area. The surveying accuracy, wall thickness and verticality of the feature are not known and would need to be considered if any future foundation solution is required in this area.
- The feature has been partially grouted but is clay filled. Consideration should be given to avoid transferring future foundation stresses on to it and minimising interaction in the event that it further deteriorates in the future.
- As the feature does not appear to be a coal mining shaft, it is possible that there may be an associated base to it that projects outside the plan surface footprint.
- If piled foundations are adopted in future, it may be possible to transfer the pile base to below the base of the feature (51.7mOD) in order to achieve base capacity. However, this would not be possible for a shallow foundation solution and more reliance would need to be placed on the integrity of the shaft treatment. Localised removal of part of the shaft and filling would be necessary to mitigate against any related hard or soft spots below pad formation.

5.3.4.3 Historical Piles

As shown in the Figure 16, there were a number of proposed foundations, in particular within the Energy Centre site, where the potential clashes between the new foundations and historical piles exists. Without knowing the accuracy of the 'as found' pile survey and diameter, depth and inclination tolerance of the historical piles it was considered impossible to know with any certainty which foundations would be affected. Based on a preliminary pile design it was estimated that about 21 Energy Centre pile groups would have been potentially affected. As a result of this, the option of using shallow foundations was developed for this area.

5.3.4.4 Stabilised Galligu

It is understood that significant quantities of Galligu impacted materials were encountered as part of the CRW. Galligu is an industrial waste produce generated from the Leblanc process. In addition to significant levels of contamination, the untreated material possesses very poor geotechnical properties. It is understood that where this was encountered, the material was excavated and stabilised before being reused in the works.

The validation report ('Phases 1,2 and 3 Earthworks Validation Report', ref NTH1315E/13/EW/V2 Final dated August 2012.) suggests that Galligu material was cement stabilised and placed predominantly in areas beneath proposed car parking and football pitches, and covered with reworked Glacial Till material.

The report includes an Appendix 4 entitled ' Treatment of Galligu Material Report' but this appears to refer to additional testing information which isn't in the version of the report referred to above.

It is understood from discussions at a site meeting attended by Arup, BGCL, BWB and BAM dated 26th Sept 12, that Buckingham confirmed that no Galligu material had been placed below proposed building footprints. However, BWB indicated at the meeting that there was some Galligu originally placed at the Phase 1 Academy (Panel 179 and 87) which was subsequently replaced with 6N/6F2 material.

However Figure 31 of the Phase 1, 2 and 3 Remediation Verification report (NTH/1315/13/R/V2) suggests that there is also a significant extent of treated Galligu materials under the southern CTF area (formerly the site of the proposed Laundry building).

Following discussions it is now understood that this material may be present under the south of the Phase 1 CTF. The primary risks of this material are as follows:

- It is understood that the stabilisation was only ever intended for use outside the building footprints and therefore isn't necessarily appropriate or suitable for use below structures. It has also not been possible to obtain any technical papers documenting the use of this below structures.

- Potential expansive reaction between the high sulphate and lime resulting in heave/softening of the stabilised material resulting in differential movements of the building foundations/slabs.
- Risk of the generation of Hydrogen Sulphide gas with distinctive 'rotten egg' odour as this material weathers.
- Very high sulphate content which may exceed the design values and impact on foundation durability.
- Additional release of chemical contaminants due to the stabilisation.

Predicting a design life for cement-stabilised material is difficult, particularly for such a heterogeneous and chemically complex material as Galligu.

Understanding of the long-term properties of stabilised Galligu is limited in scientific literature. There was a recent project undertaken at the University of Greenwich (PASSIFY) which assessed cement-stabilised Galligu from a site in Widnes three years after treatment. X-ray diffraction and electron microscopy identified significant quantities of ettringite and gypsum minerals within the treated material, both of which can lead to expansion of the matrix (and failure), although geotechnical properties were not specifically examined as part of the study as it was focused on contamination risk and not used under buildings.

Standard industry guidance (including BRE Special Digest 1, HA 74/00, Britpave BP/16, BS EN14227-10) is focused on the assessment of untreated soils, not materials that have already been stabilised.

Given the heterogeneity of Galligu, any investigation and assessment would need to be comprehensive, and specialised microstructural analysis and assessment (using electron microscopy techniques) may be needed. However, it is still not certain that such an exercise would provide the necessary level of confidence that the material was durable or would enable an estimated design life to be provided, especially given the findings of the PASSIFY study.

The potential impact of the instability of this material is unknown in both timescale and effect. It is believed to be a medium to long term issue with the effect on stability being a function of both the thickness of deposit and the depth of conventional soil above the deposit. It would also be prudent to ensure any overlying construction is made to be as resilient to ground movements as possible to minimise any future effects. This should include an assessment of risks to site utilities, highways and pitch tolerances.

The MCFA Client have requested that Buckingham assemble case studies and research on the use of stabilised Galligu below buildings and BAM have also engaged an expert consultant from URS to review the potential impact of the stabilised Galligu on the development. The client has also elected to relocate the proposed Laundry building from the Phase 1 CTF to the Phase 2 FM building.

5.3.5 Shallow Foundation Solution Design

A preliminary design of the shallow foundation solution has been undertaken to evaluate the capacity of pad sizes shown in drawings S112.F2.E6 and S122.00.E6.

The pads were sized based on a maximum bearing pressure of 150kPa.

The top of the glacial till was assumed to have an undrained shear strength, $c_{u,}$ of 100kPa and increasing to 300kPa with depth up until the rockhead level (42.9mOD) which is assumed to be a rigid boundary.

5.3.6 Energy Building Settlement Analysis Methodology

An analysis of the Energy Centre, with a slab and pad foundation solution, was carried out using Oasys PDISP, in a similar manner to the assessment of the Elite/Academy buildings.

A simplified model was constructed for the building area using the slab and foundation loading indicated above and design stiffness profiles presented in **Table 5** below.

Table 5: Design Stratigraphy and Parameters Specific for Elite and Academy Building

Strata	Assumed Vertical Extent (mOD)	Comments/ Assumptions	Parameters used in the assessment
Engineered Fill (MG)	63.9 to 63.0 (High Till) or 60.5mOD (Low Till)	There is an assumption that the site testing is representative of the drained stiffness of the fill. The depth of fill was considered under two scenarios: • High till (base at 63.0m) • Low till (base at 60.5m)	Drained Young's Modulus E' = 25,000 kN/m^2 Poisson Ratio = 0.2
In-Situ Glacial Till	to 42.9mOD	Linear increase in strength. Assumed E = 500Cu. (conservative as strain related to net load/ult. load likely to be significantly lower at depth)	Drained Young's Modulus E' = 35,000 kN/m^2 at base of Fill increasing to 100,000 kN/m^2 at rockhead. Poisson Ratio = 0.2
Bedrock	All material below	Little information on rock strength, however very little	Assume rigid boundary of model

	42.9mOD	influence on settlements.	

The limitations and assumptions of the settlement modelling undertaken are as follows:

- The software does not take into consideration any self-weight settlement of the fill or settlements due to increases in site levels, as described previously. The issue is likely to be time dependent and have greater potential to impact on overlying construction for development taking place within a short time frame of completion of filling and become less significant with time. Monitoring of formation will confirm that the required degree of consolidation has taken place prior to construction so further modelling and assessment is not required.
- The foundation sizing has been based on the limiting peak stress of 150kN/m^2 (as per the specification requirements) which is fairly uniformly distributed under the main pad foundation areas. Slab loads have been provided as appropriate.
- The software assumes that the loading is flexible and the rigidity of the foundation is not taken in to account. The inherent stiffness of the foundation itself would tend to equalise movements between the centre and edge of the loaded area.
- The loaded areas have been set at the base of the pad and slab formations as currently proposed.
- The loads applied to the pads include dead load, live load self-weight of the pads and an allowance for pad thickening. A sensitivity analysis has been run in the model which includes the effects of wind loading on the pads.
- In the lift pit area an additional load of 12.4kPa has been applied along with the dead and live load on the slab.
- The load due to the 700mm high bund wall has been modelled as an additional load of 4.45kPa across the area of the wall.
- The areas containing the water buffer vessels have a 1.2m thick base with full water tanks and so have been assumed to carry a stress of between 60-80kPa.
- The service pits carry a load at their base which is supported by two pads which may act independently or be integrated in to the base slab. Both scenarios have been modelled to assess their effect on settlement.
- The chimney is potentially subject to large wind loads and so a maximum pad load of 90kPa has been applied; a sensitivity has also been modelled of the chimney with no wind and so a load of 7.45kPa.
- Settlement is reported at underside of ground floor slab level, taken as being 63.75mOD. Settlement is also reported at the base of fill level.

5.3.7 Energy Building Settlement Results

The run log and a summary of the results are provided in the table and figures below. The model layout is shown in **Figure 30**.

Table 6: Energy Centre PDISP Run Log

Energy Centre				
Model Run	Description	Purpose	PDisp output	Location of maximum displacement
Run 1	Pads subject to dead+live+self-weight+pad thickening Slab subject to DL+LL Low Glacial Till level at 60.5mOD Chimney subject to maximum load (90 kPa).	Expected most common scenario. Low glacial till level is a conservative scenario as the assumed stiffness of till is greater than the engineered fill.	Range of settlement = 5-16mm Maximum differential settlement between service basement and slab approx 8mm Contour plan and displacement plots are shown in **Figure 31**.	Water tanks Chimney Pad 1450
Run 2	Pads subject to dead+live+self-weight+pad thickening Slab subject to DL+LL High Glacial Till level at 63.0mOD Chimney subject to maximum load.	Sensitivity analysis for range of till levels. High till level assumed.	Range of settlement = 4-13mm Maximum differential settlement between service basement and slab = approx 7mm Contour plan and displacement plots are shown in **Figure 32**.	Water tanks Chimney Pad 1450
Run 3	Pads subject to dead+live+self-weight+pad thickening+wind Slab subject to DL+LL Low Till level at 60.5mOD	Sensitivity analysis for possible pad loads (now including wind load)	Range of settlement = 5-16mm Maximum differential settlement between service basement and slab = approx 8mm	Water tanks Chimney Pad 1450

	Chimney subject to maximum load.		Contour plan and displacement plots are shown in **Figure 33**.	
Run 4	Pads subject to dead+live+self-weight+pad thickening Slab subject to DL+LL Low Till level at 60.5mOD Chimney subject to DL+LL	Sensitivity analysis for chimney load, not including wind load	Range of settlement =5-16mm Maximum differential settlement between service basement and slab =approx 8mm Contour plan and displacement plots are shown in **Figure 34**.	Water tanks Pad 1450
Run 5	Pads subject to the maximum load of 150kPa Slab subject to DL+LL Low Till level at 60.5mOD Chimney subject to maximum load	Sensitivity analysis for pads under worst case loading	Range of settlement = 5-21mm Maximum differential settlement between service basement and slab = approx 10mm Contour plan and displacement plots are shown in **Figure 35**.	Pad 2020 Pad 2010
Run 6	Pads subject to dead+live+self-weight+pad thickening Slab subject to DL+LL Low Till level at 60.5mOD Chimney subject to maximum load. Basement modelled as composite foundation carrying slab and pad loads over entire area	Sensitivity analysis for basement considering it's slab and pads loads acting as one foundation load	Range of settlement = 5-16mm Maximum differential settlement between service basement and slab = approx 4mm Contour plan and displacement plots are shown in **Figure 36**.	Water tanks Chimney Pad 1450
Run 7	REVISED STRUCTURAL LOADS Slab subject to DL+LL Low Till level at 60.5mOD Chimney subject to maximum load	Checking effects of change in structural loads	Range of settlement =5-16mm Maximum differential settlement between service basement and slab = approx 7mm Contour plan and displacement plots are shown in **Figure 37**.	
Run 8	REVISED STRUCTURAL LOADS (including wind)	Checking effects of change in structural loads	Range of settlement =5-17mm Maximum differential settlement	

	Slab subject to DL+LL Low Till level at 60.5mOD Chimney subject to maximum load.		between service basement and slab = approx 7mm Contour plan and displacement plots are shown in **Figure 38**.	

The results are shown in **Figure 32-Figure 39** to accompany the run log in **Table 6**

The input model of the Energy Centre is shown in **Figure 31**; it shows the displacement grid and lines used to monitor the displacement in under the various loading scenarios.

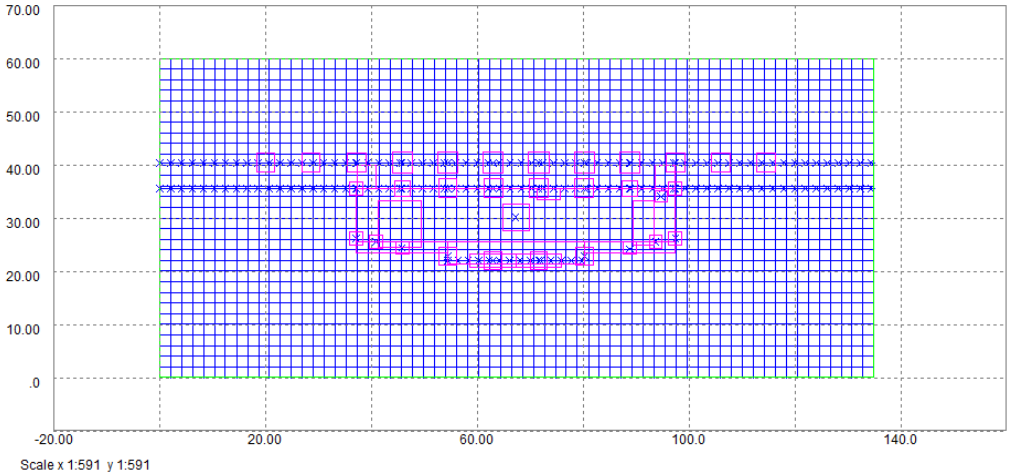


Figure 31: Energy Centre PDISP model layout

Run1: Till at 60.5m

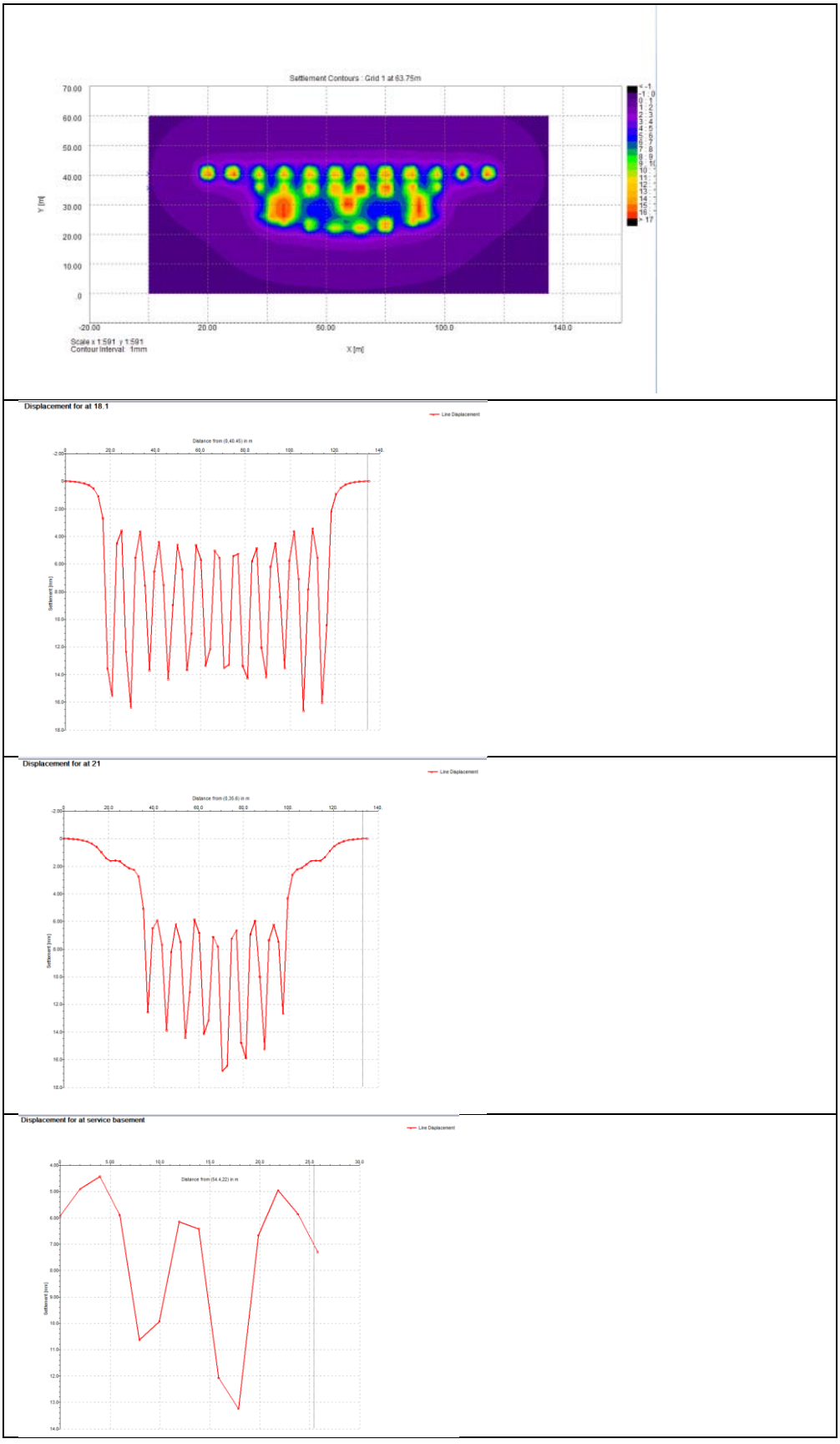


Figure 32: PDISP Run 1 outputs

Run 2: till at 63.0m

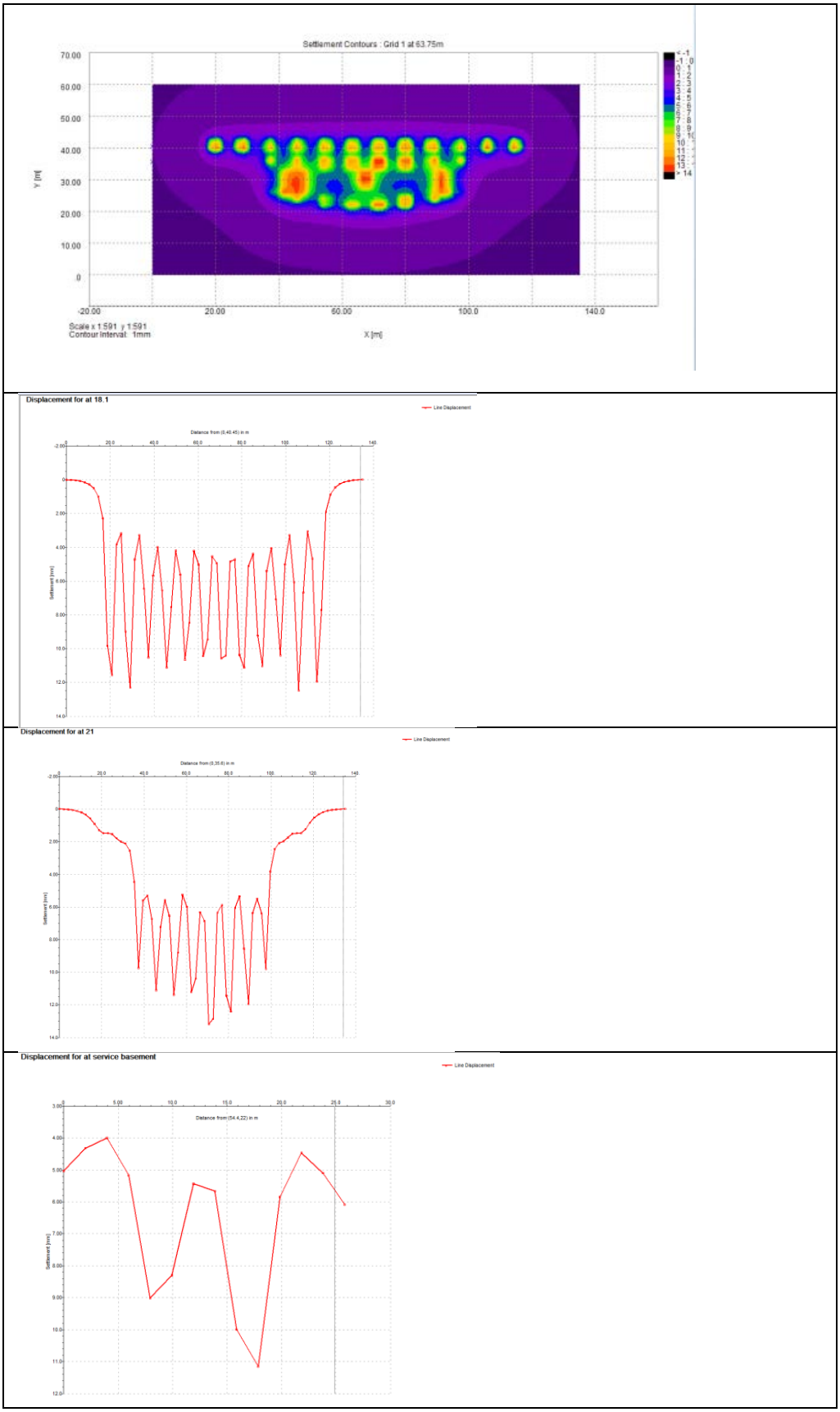


Figure 33: PDISP Run 2 outputs

Run 3: including wind loads

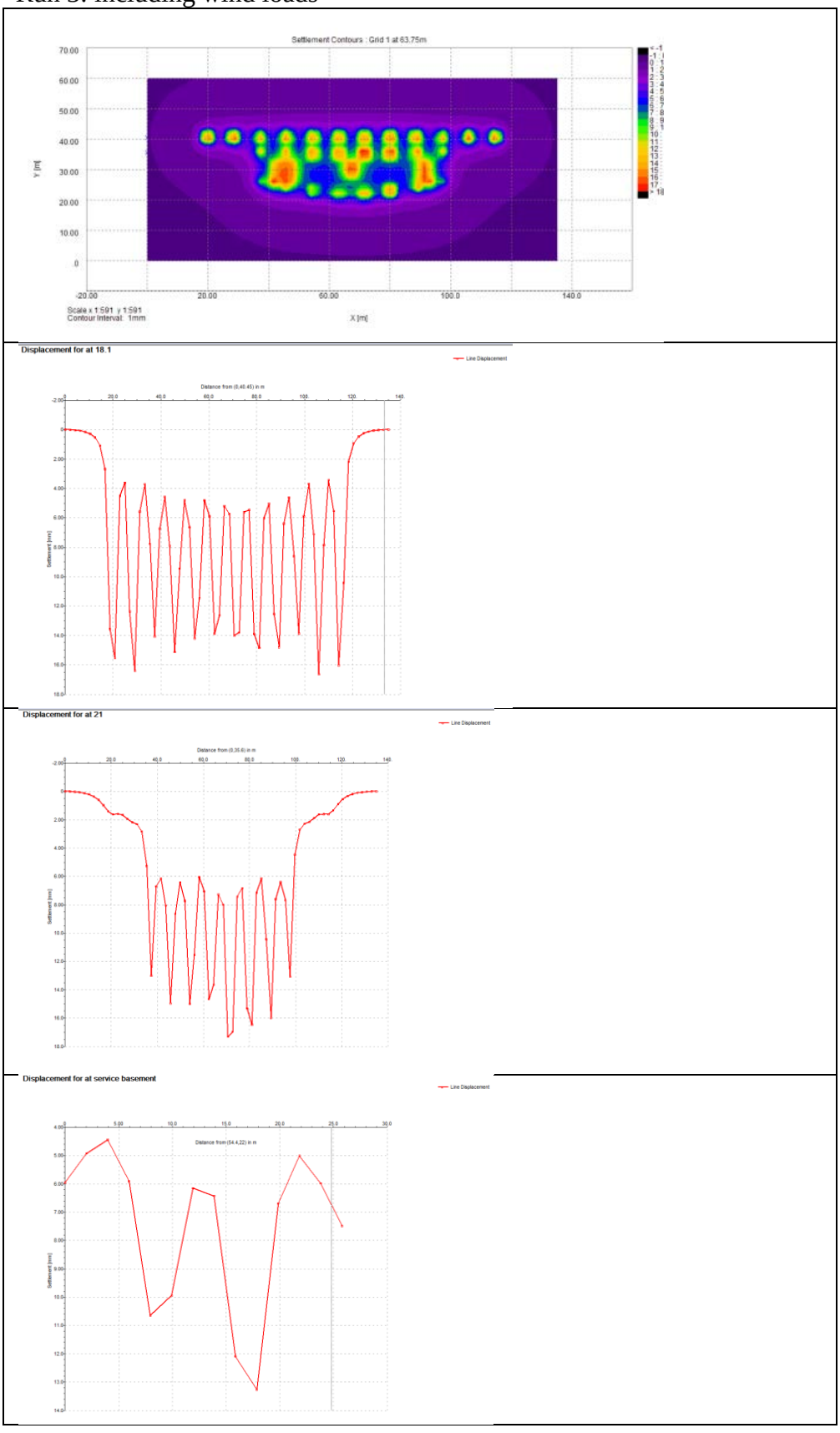


Figure 34: PDISP Run 3 outputs

Run 4 minimum chimney load of 7.45 kPa

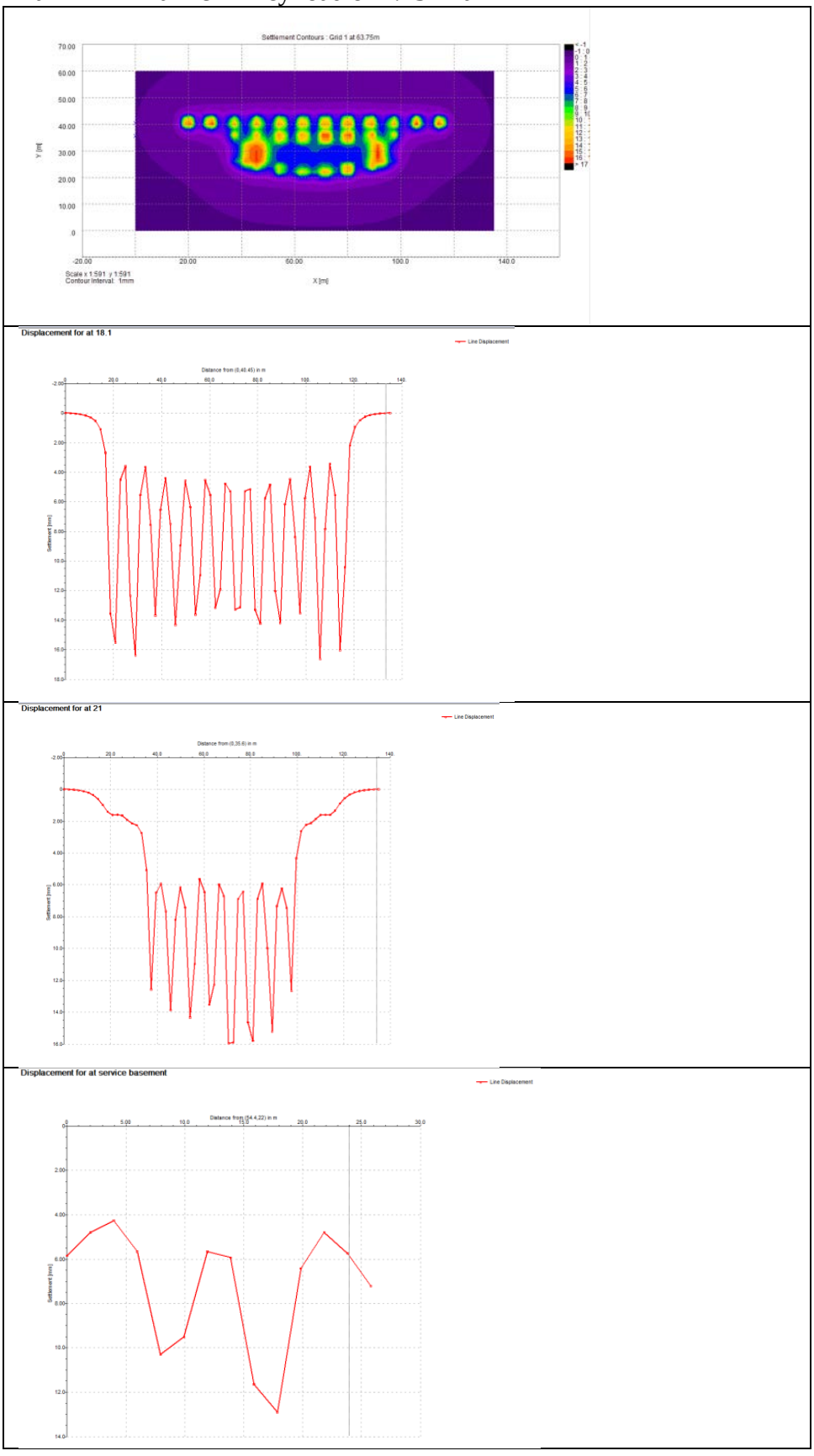


Figure 35: PDISP Run 4 outputs

Run 5 (all pads carrying maximum load of 150kPa)

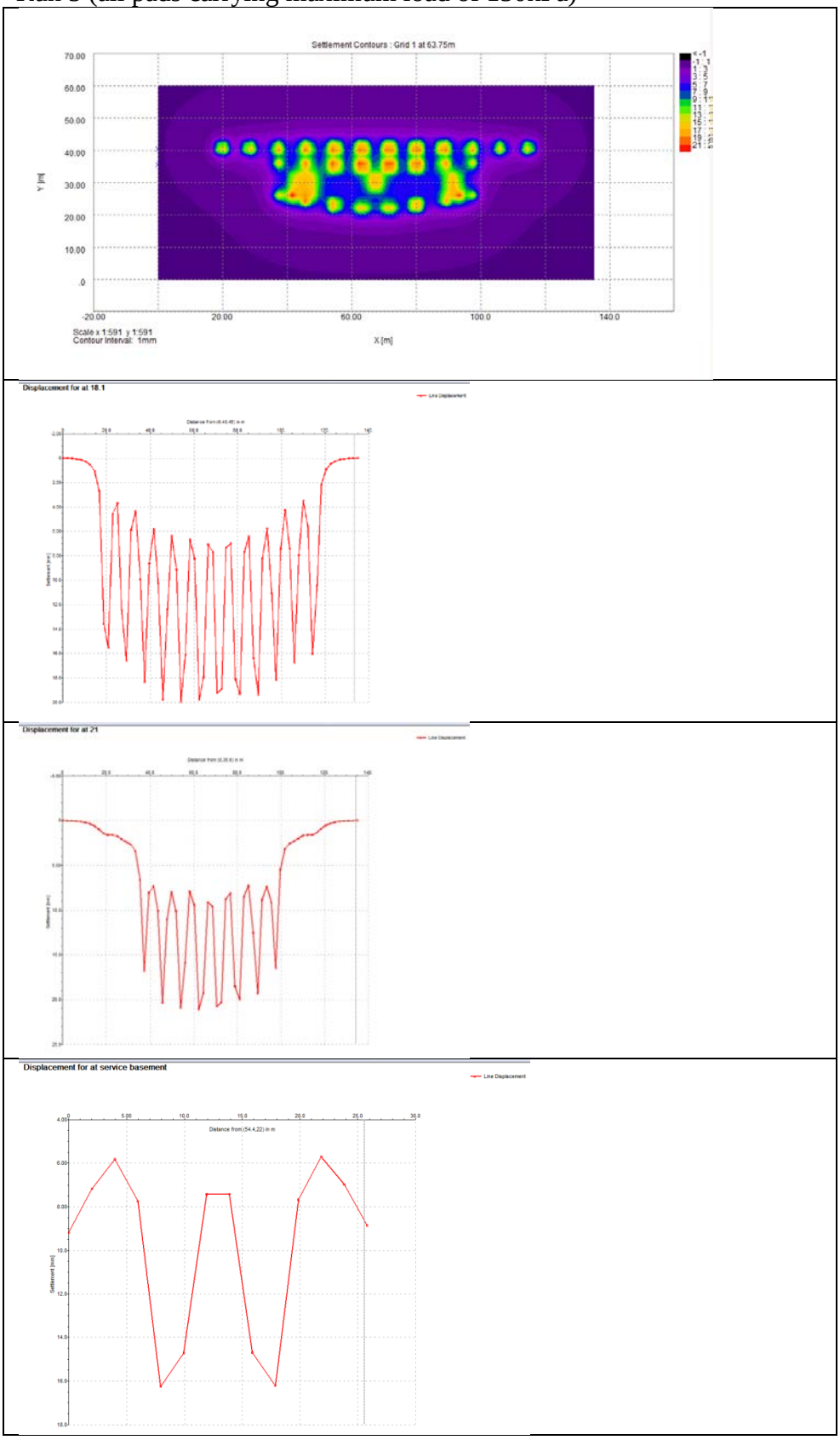


Figure 36: PDISP Run 5 outputs

Run 6 composite basement

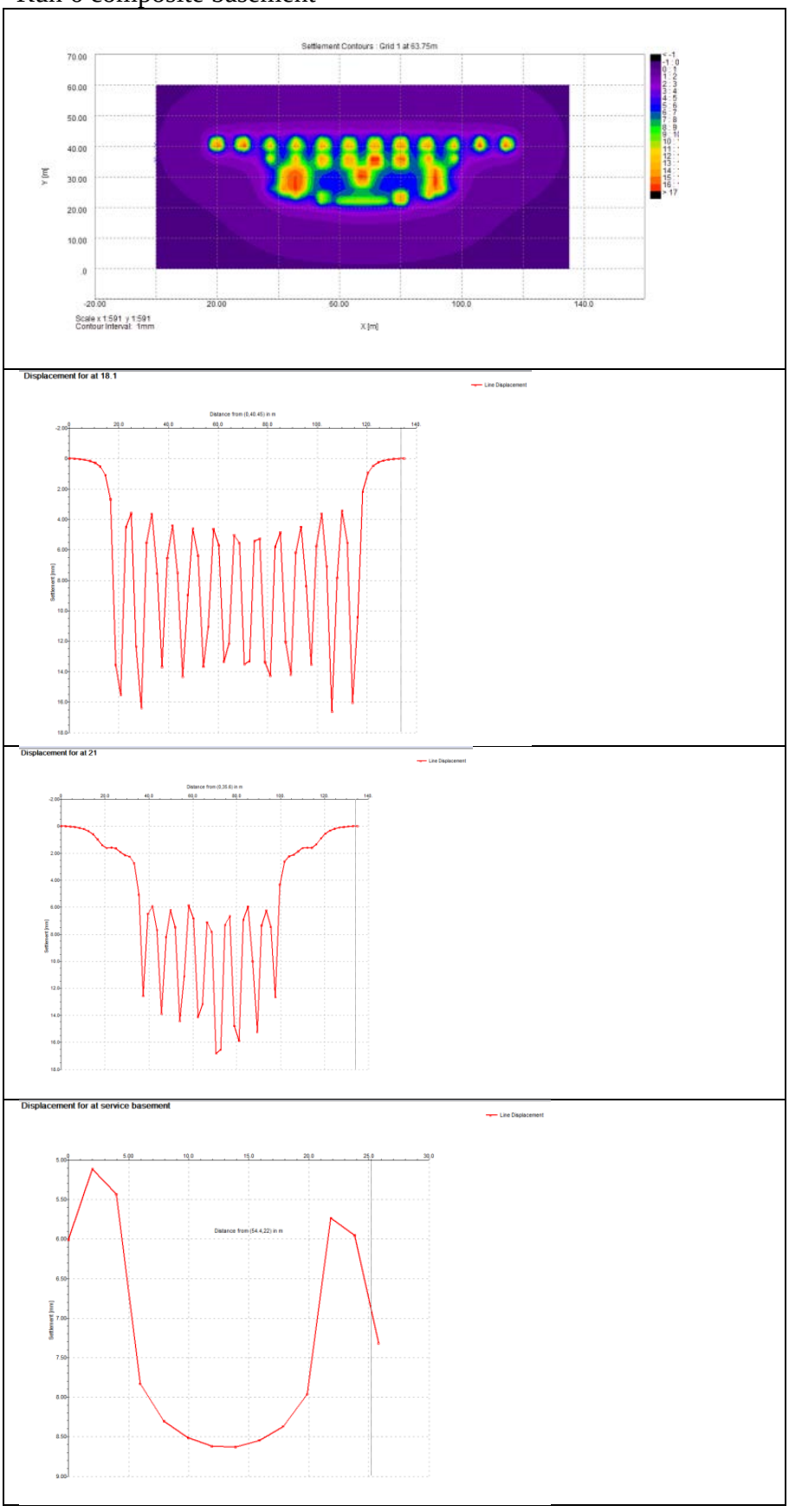


Figure 37: PDISP Run 6 outputs

Run 7: Energy Centre PDISP results with revised structural loads : pad loads excluding wind; till at 60.5mOD; maximum chimney load;

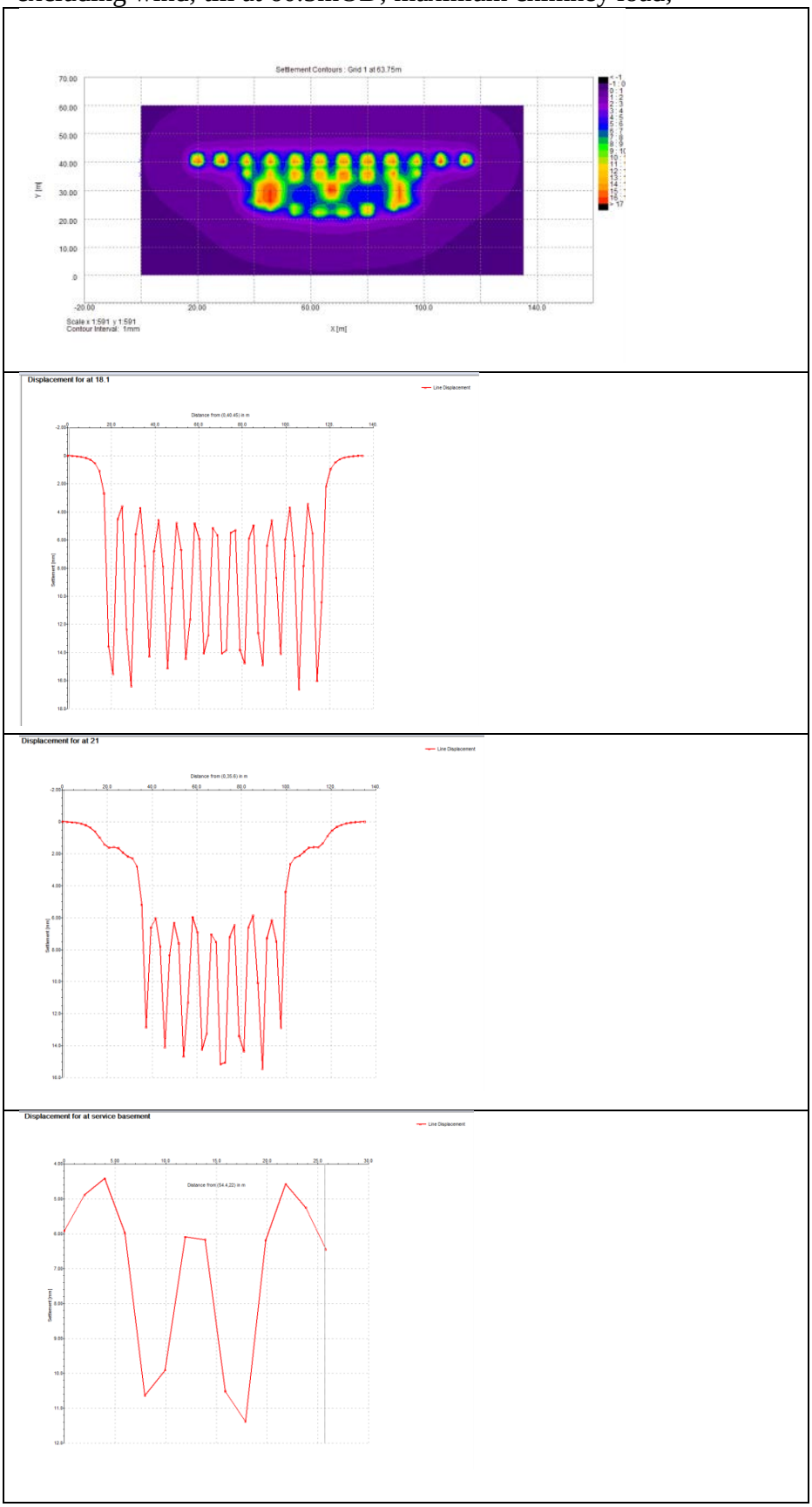


Figure 38: PDISP Run7 outputs

Run 8: Energy Centre PDISP results with revised structural loads: pad loads including wind; till at 60.5mOD; maximum chimney load;

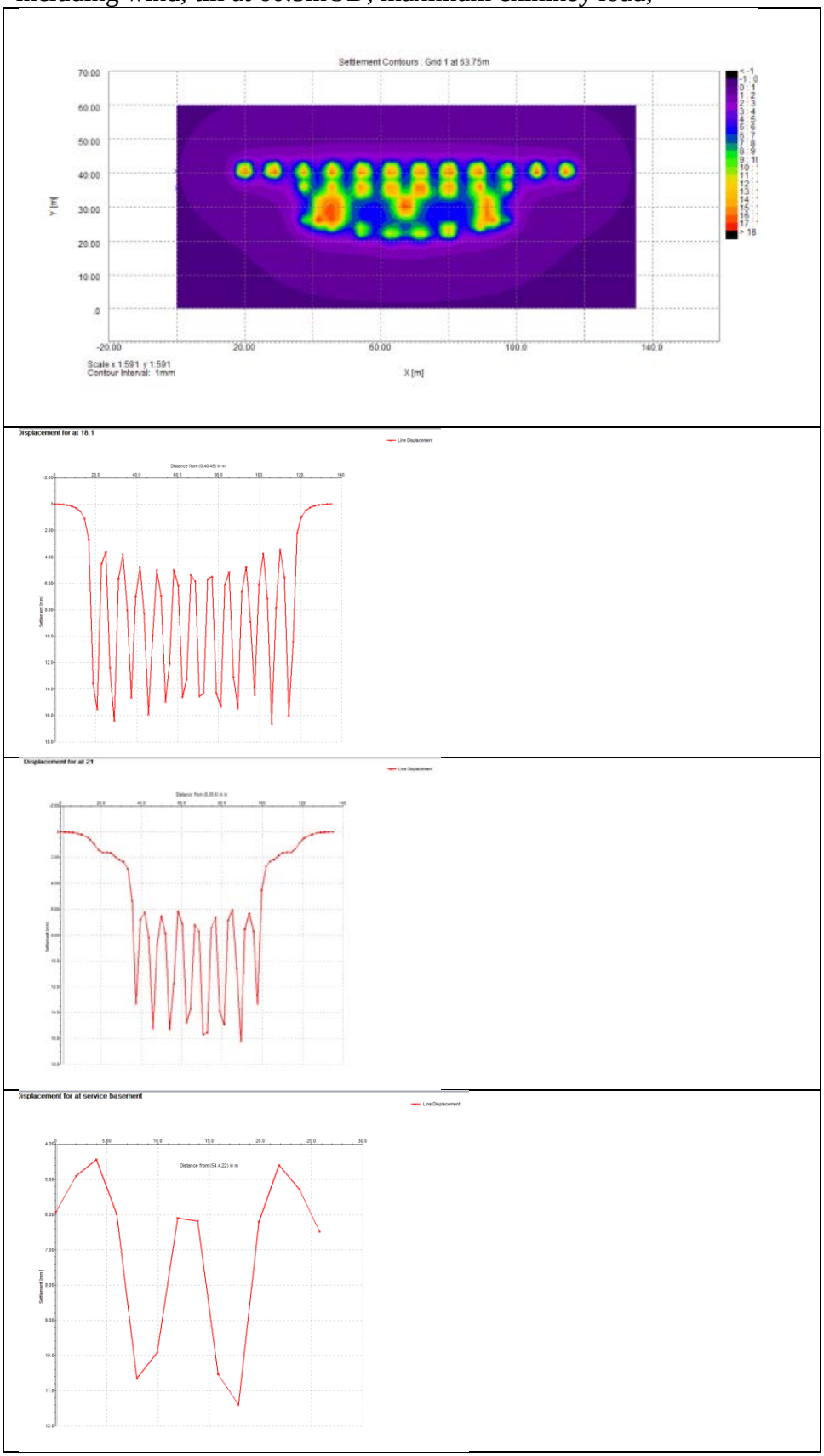


Figure 39: PDISP Run 8 outputs

In summary, the results show that:

- Raising the level of the Glacial Till reduces the range of settlements and the maximum settlement, as it is a stiffer deposit compared to the engineered fill;
- Including the wind load slightly increases the differential settlement between the service basement and the slab;
- Excluding wind load from the chimney pad slightly reduced the differential settlement between the service basement and the slab
- Applying the maximum 150 kPa load to all pads across the building area (a worst case scenario) increased the settlement of the pads by around 5mm. However they are still within the acceptable level (<25mm).
- Treating the service basement as a composite slab as opposed to separate pads and slab results in a decrease in the differential settlement between the service basement and the main slab.
- The revisions in structural loads provided on 15th November caused some pad stresses to increase whilst other were decreased. The result is that the range of settlements is largely unaffected whilst the differential settlement between the service basement and the main slab has decreased.

5.4 Perimeter Screening Wall to CTF

The foundation solution for the Perimeter Screening Wall consists of shallow pad footings, of various dimensions, some of which are connected with a ground beam whilst others work independently of each other. The foundation layouts and dimensions are detailed on the construction drawings listed below:

Title	Drawing No.	Revision
Academy Building Zone E1 General Arrangement to Foundations	S112.F2.E1	C6
Academy Building Zone E2 General Arrangement to Foundations	S112.F2.E2	C6
Academy Building Zone E3 General Arrangement to Foundations	S112.F2.E3	C8
Academy Building Zone E4 General Arrangement to Foundations	S112.F2.E4	C9
Academy Building Zone E5 General Arrangement to Foundations	S112.F2.E5	C2
Academy Building Zone E6 General Arrangement to Foundations	S112.F2.E6	C6

An allowable bearing capacity of 150kPa has been determined for the pad foundations, which is consistent with the CTF general engineered fill in the energy building and former laundry building. However an allowance for a temporary overstress of 25% has been applied in accordance with BS8004 Clause 2.3.2.4.3, giving a maximum allowable pressure of 187.5kPa.

The foundations are to be founded in the general engineered fill at levels between 61.3mOD and 62.5mOD. Some pad foundations will be placed on a mass concrete fill to allow interaction between adjacent structures and service trenches. The mass fill will mostly be founded at levels deeper than the other pad foundations, to depths up to 63.0mOD.

The ground conditions below this structure are expected to vary between the Enhanced Engineered Fill used in the Elite and Academy areas, and the general engineered fill used below the Energy Centre and CTF south (former laundry area).

Based on the ground parameters being similar to the general engineered fill, it is expected that the settlements should be similar to those calculated for the foundations in the Energy Centre i.e. typically less than 25mm.

It is noted that in the southern area, the perimeter wall is founded within an area of stabilised Galligu. As discussed in section 5.3.4.4 the long term performance of this material is currently unknown due to a number of uncertainties. BAM have engaged an expert in stabilised Galligu from consultant URS to examine the potential risks of this material on the development. A separate report has been prepared by URS.

The uncertainty on the performance was a factor in deciding to relocate the proposed laundry building away from the CTF area.

The screening wall is a less critical structure than the former laundry building. However in the absence of any definitive guidance on the risks posed by this material, the wall cladding is selected to accommodate vertical movement which might occur if the material either degrades or swells in future.

5.5 Phase 1 Pitch Enclosure

The foundation solution for the Pitch Enclosure consists of shallow pad footings, of various dimensions, connected with a ground beam located around the perimeter of the structure. In the corners of the structure the columns are supported by strip footings. The foundation layouts and dimensions are detailed on the construction drawings listed below:

Title	Drawing No.	Revision
Central Training Facility Zone E Substructure Sections and Details	S324.09.E	C4
Covered Pitch Zone E8 General Arrangement to Foundations	S112.F2.E8	C5
Central Training Facility 3D View Covered Pitch	S000.33.E	C3

An allowable bearing capacity of 150kPa has been determined for the pad foundations. The foundations are to be founded in general engineered fill at levels from 59.05mOD to 62.7mOD. Pad foundations with a formation level above 61.1mOD are placed on mass concrete fill.

Based on the ground parameters being similar to the general engineered fill, it is expected that the settlements should be similar to those calculated for the foundations in the Energy Centre i.e. typically less than 25mm.

6 Phase 2 Facilities Management Building

6.1 Proposed development

The main development in this area is a two storey steel framed structure to be used as the main facilities management for the Etihad Campus development. In March 2013, a decision was also made to relocate the Energy Centre from the CTF area in to the Facilities Management building.

6.2 Available Information

Information on the CRW earthworks validation for the Phase 2 area is included in the following report, produced by BWB Consultancy:

Phases 1, 2 &3 Earthworks Validation Report – NTH1315E/13/EW/V2~Dated Aug 2012 - Final Issue V2

The Phase 2 area has been briefly reviewed in terms of the potential impacts on the proposed Facilities Management building.

6.3 General Design Considerations

6.3.1 Expected Ground Conditions

The proposed facilities management building is situated adjacent to the south eastern site boundary and comprises a single storey steel framed structure on shallow foundations. It is understood that this building is also to house the Laundry facilities which were previously planned to be constructed in the south of the CTF area.

The ground conditions in this area are expected to comprise an engineered fill placed under the Buckingham CRW Works, overlying Glacial Till.

The CRW Contractor Buckingham elected to excavate and re-engineer all the overlying existing Made Ground deposits to mitigate risks of mineshafts, remove obstructions and deleterious materials and produce a material with more uniform properties.

The reduced levels of the base of this excavation from the Buckingham CRW works from the validation report (presumably corresponding to the top of the Glacial Till) is between c. 58 and 61.5mOD under the proposed building (see Figure 39 below).

The validation report suggests that about 65% of the material was a Class 2A (wet cohesive fill), 31% Class 2C (stony cohesive fill) and 4% Class 1B (Uniformly graded granular fill). CRW records suggest the moisture content was high and above optimum, requiring extensive use of lime to reduce moisture and achieve compaction. It is understood that the target CBR values were the same as per the CTF general engineered fill i.e. 4% average value but with no value less than CBR 3%.

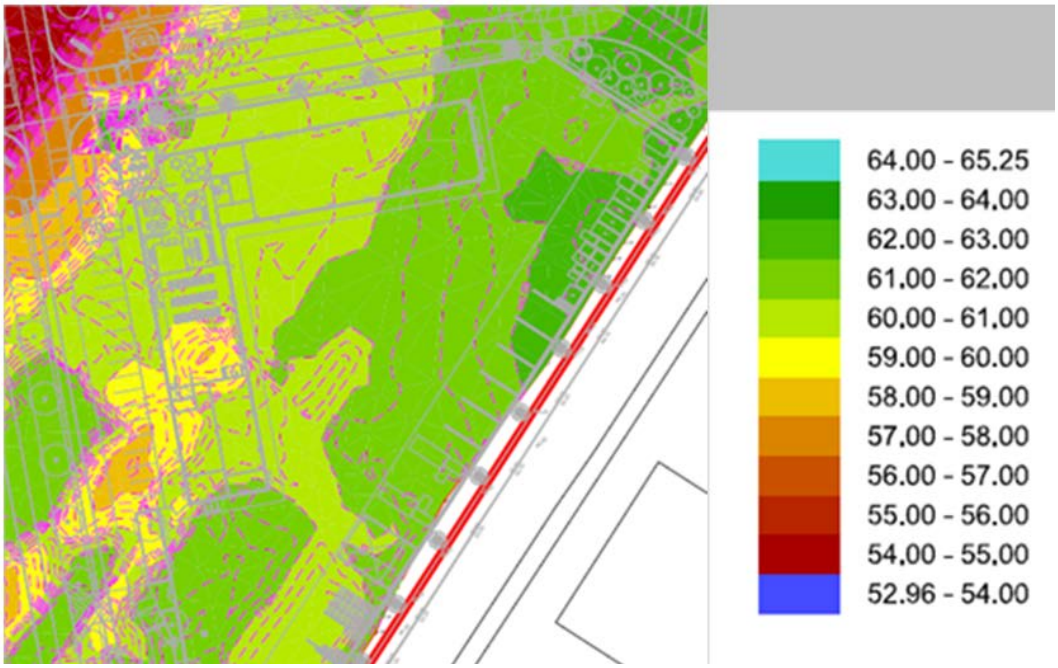


Figure 40 Base of CRW excavation (mOD)

The ERs for the CRW works specified a maximum earthworks formation settlement of 25mm, but without being clear over what timescale and loading.

The filling has been completed for several months therefore any residual settlements that might occur as a result of self-weight settlement of the new fill or any increase in site levels could be reasonably expected to have largely taken place.

The fill materials in this area are being reported to be of similar nature and performance to materials used in the general engineered fill to the CTF (Energy centre and southern CTF area i.e. former Laundry building) and have been well compacted similarly to the CTF fills.

The site is located to the east of the Bradford Fault, and therefore underlain by Triassic Sherwood Sandstone therefore shallow coal mining and shafts should not be present.

6.3.2 Proposed Building Foundations

The reference foundation design undertaken by Capita was to comprise shallow foundations on to shallow natural glacial till, with an assumed allowable bearing capacity of 100 kPa.

The proposed foundations have subsequently been sized based on a peak bearing stress of 150 kPa. However excluding the transient wind loading, the typical dead plus live loads are less than 100kPa.

The revised structural design assumes that a bearing capacity of 150kPa is achievable on the general engineered fill, which is assumed to be similar in nature to the fill used in the CTF (former) laundry and energy areas.

The structural formation levels of the proposed pads is c 62.8mOD suggesting that there is between 1 and 4m of new general engineered fill between the pad formation and the natural till.

The proposed structure is relatively insensitive to settlement. The nature of the earthworks materials together with the method of placement indicates that the filled ground has been placed in a controlled manner in accordance with the earthworks specification, and to an appropriate performance criteria. It is not anticipated that there would be significant residual settlement occurring within this material and any additional settlement due to loading from the new foundations will be relatively small and uniform across the structure. On this basis, it is considered that the engineered fill provides an appropriate foundation stratum for the FM structure.

6.3.3 Proposed Slab Formation

The current proposal assumes a ground bearing slab on the engineered fill material. CBR testing of the formation surface was to be validated during the CRW using 300mm diameter plate bearing tests to provide a 5% CBR value but the achieved final CBR results are missing from the Table 5 in the CRW Validation for the Phase 2 area.

Based on the CBR values of the general engineered fill it would be expected that below this surface, the fill materials would provide a CBR of a minimum of 3% with average of 4%+, as is the case for similar general engineered fill in the CTF Energy and former Laundry areas (these had average CBRs of about 6%).

In terms of the slab performance, the slab is relatively lightly loaded compared to the CTF building slabs and a target CBR of 4% should be provided at the underside of the subbase, which should have obtained by default given the target CRW values.

Additional plate load testing has been conducted by BAM, which indicated CBR values of 2.0, 3.2, 10.3, 0.5, 0.7 and 4.8% which are generally lower than would be expected.

The general cohesive based fills on the site are particularly weather susceptible and assuming the CRW material was put in correctly, the surface is very likely to have been degraded during prolonged wet weather over the winter 2012/13.

It is recommend that areas of CRW formation which are visibly soft or wet and areas where the CBR is less than 4%, are excavated as soft spots and replaced with capping back to underside of the proposed subbase. A Mexe cone device could be used as a rapid test to quickly obtain CBR results in finer CRW material,

and therefore identify areas needing capping, however it would not be suitable for testing the capping itself.

It is recommended that a trial area is also prepared with Type 1 sub base and tested at underside of slab level to validate the CBR achieved at the surface.

7 Phase 3 Football Pitches and Car Park

7.1 Proposed development

The Phase 3 area is to be developed as two outdoor sports pitches and the eastern half of the northern car park.

7.2 Available Information

Information on the CRW earthworks validation for the Phase 3 area is included in the following report, produced by BWB Consultancy:

Phases 1, 2 &3 Earthworks Validation Report – NTH1315E/13/EW/V2~Dated Aug 2012 - Final Issue V2

The Phase 3 area has been briefly reviewed in terms of the potential impacts on the proposed Facilities Management building.

7.3 Overview of CRW Validation

7.3.1 Earthworks Validation

Historical ground investigation suggests the site was underlain by variable deposits of both granular and cohesive Made Ground overlying firm to very stiff Glacial Till deposits.

The BWB Remediation report also indicates that significant thicknesses of Galligu were located in the Phase 3 area.

Buckingham Group has conducted a Constraints Removal Works (CRW) to excavate out all Made Ground, remove obstructions and deal with mine shafts and coal workings. The area has been filled with compacted engineered general engineered fill.

Isopach information indicating the range of fill depths across Phase 3 is currently missing from Phase 1, 2 &3 Earthworks Validation Report-NTH1315E/13/EW/V1 dated August 2012.

A base of excavation survey received from Buckingham suggests a typical excavation level of 61 to 64mOD in this area. However, Mexeprobe CBR data for each fill layer indicates level during placement ranging between 58.47mOD and 64.75mOD, suggesting a maximum depth of 6.3m.

Excavated Made Ground was processed as follows:

- A significant volume of Made Ground excavated was screened to remove material size over 100mm. Foreign material such as steel, plastic, tyres and wood noted during screening were removed prior to placement and compaction.

- Bricks and concrete removed from Made ground was crushed to a 6F2 materially initially and stockpiled.

Upon completion of earthworks over this phase, there was a requirement to leave on site stockpiles of 16,600m^3 of Type 3 material and 32,796m^{3}in accordance of SHW Series 800 as well as 8,557m^3 of 6F2 in accordance with SHW Series 600. It is assumed that this was for incorporation in to the later works.

The report states that obstructions encountered within Phase 3, such as the remnants if the historical canal arm, footings, drainage features and basement structures were broken out with a hydraulic breaker and removed. Any piles encountered beneath building footprints were cut down as a minimum to the base of Made Ground in accordance with WYG requirements. However, the contactor has extended this to at least 3m below the formation level. The remaining site piles outside building footprints were cut down to a level of 1m below the formation level.

A drawing presented as Figure 6 in the BWB Earthworks validation report indicates a number of piles (approx. 30 to 35) which have been left in situ on the southern edge of the Phase 3 area. However no significant structures are planned in this area therefore they may not impact on the proposed development.

A number of services have been removed as part of the works, with a number of services diverted along site boundaries.

Of the materials sampled for Geotechnical Analysis, approximately 63% of samples were Class 2C, 34% were Class 2A and 2.5% of samples were Class 1B. The Glacial Till material was predominantly classified as Class 2A.

The Employer has not requested any performance criteria for the bulk fill but has specified a 5% CBR at formation level and maximum settlement of 25mm. However, it is understood from their specification that Buckingham have proposed to produce a general engineered fill with an average CBR performance of 4% CBR, with no values of less than 3%.

All of the natural moisture contents were found to be higher than optimum and lime modification was used to condition fill which is wet of optimum. Table 7 shows the range of CBR values determined as part of the CRW works.

Table 7: Summary of CBR Data

Level where CBR was tested	**CBR target for specified level**	**Minimum CBR Value**	**Average CBR Value**	**Maximum CBR Value**
Formation Level	5%	* TBC	* TBC	* TBC
Fill Material	4%	1.5	6.6	15

*TBC – No results included within the validation report.

Where CBR results were recorded below the minimum requirement of 4%, BWB confirm that additional works were undertaken and the area retested for compliance. The final formation level was to demonstrate CBR values of 5% but the results are not included in the report. The formation was capped with 100mm of 6F2.

It is understood that the Galligu materials were excavated and stabilised. It is not clear from the validation report where this material was permanently deposited but one of the drawings shows this to be within the Phase 3 area. The earthworks validation states that Galligu material was placed predominantly beneath the proposed car parking and football pitches, with the placement of Galligu beneath buildings and roadways avoided where possible. Galligu material was placed in 250mm layers and cement stabilised with 2% OPC.

7.3.2 Mine treatment Validation

7.3.2.1 Mine Shafts

No mine shafts were expected within the Phase 3 area and none have been identified and treated as part of the earthworks.

7.3.2.2 Mine seam treatment

Treatment Grid 1 partially encroaches on to the northwest boundary of Phase 3. The purpose of the treatment grid was to understand the presence of fireclay workings and confirm the location of the sub crop of the Bradford Four Foot coal seam.

BWB have concluded the drilling and grouting works within Treatment Grid 1 have been completed satisfactorily. Low grout takes with in holes within Treatment Grid 1 indicate either no workings have been undertaken in this part of the site; or that historic workings have since collapsed. Collapsed historic workings are anticipated by BWB to be fully choked or at such a depth that they will not affect the surface.

7.4 General Design Considerations

The proposed function of this area is car parking and external pitches. Refer to Section 13.2 and 13.3 for general geotechnical considerations with regards to these uses.

8 Phase 4 North West Arena

8.1 Proposed development

The main development in this area is the MCFC Northwest Arena stadium. The stadium comprises two large steel framed stands (East and West) founded on concrete pads which generally increase in size with distance from the pitch. The North and South stands are smaller in size and comprise a steel and concrete construction.

Phase 4 will also be occupied by hard standing and landscaping associated with the stadium as well as the foundations and access ramps for a proposed footbridge connecting Manchester City Stadium with Phase 4.

8.2 Available Information

Information on the CRW earthworks validation for the Phase 4 area is included in the following report, produced by BWB Consultancy:

Phase 4Z and 10 Earthworks Validation Report – NTH1315E/18/EW/V2~Dated April 2013 - Final Issues V2

This report supersedes an earlier report which contained very limited validation for a small portion of the east of the area.

Phase 4Z-99 Earthworks Validation Report - NTH1315E/16/EW/V2 ~Dated Feb 2013 – Final Issue V2

8.3 Overview of CRW Validation

8.3.1 Earthworks Validation

Historical ground investigation at the site of the North West Arena suggests the site was underlain by variable deposits of both granular and cohesive Made Ground overlying firm to very stiff Glacial Till deposits.

Significant areas of heavy contamination were reported within this area including tar well structures. In addition, pockets of unstable ‘self combustible’ materials were encountered towards the south of the area.

Buckingham Group has conducted a Constraints Removal Works (CRW) to excavate out all Made Ground, remove obstructions and deal with mine shafts and coal workings. The area has been filled with compacted general engineered fill.

Excavated Made Ground was processed as follows:

- Made Ground over 100m in size was screened. Foreign material such as steel, plastic, tyres and wood noted during screening were removed prior to placement and compaction.
- Bricks and concrete removed from Made ground was crushed to a 6F2 materially initially and stockpiled.

Upon completion of earthworks over the whole site area, there was a requirement to leave on site 18,086m^3 of Type 3 material in accordance of SHW Series 800 and 5,000m^3 of 'sub soil'.

A significant quantity of crushed and screened material (Type 6F1, 6F2, 6N and Type 3) was generated from this area, totalling 140,000 m^3.

It was reported that obstructions such as historic foundations, footings, drainage features and basement structures were broken out with a hydraulic breaker and removed. Where identified, piles have been cut down to the base to the made ground. Where Made Ground was less than 1m in depth, the pile were cut down to a minimum of 1m below formation level.

Figure 6 of the validation report indicates a number of piles (approx. 30 to 40 No.) that have been left in situ which might impact on below ground works in these areas. In addition 4 No mine shaft caps are indicated, one of which is below the east stand of the proposed structure.

A number of services have been removed as part of the works and services were diverted along the site boundaries.

The depth of CRW excavation of Made Ground located in phase 4Z range between approximately 53.00 mOD to 61.00 mOD.

The Employer has not requested any performance criteria for the bulk fill but has specified a 5% CBR at formation level and maximum settlement of 25mm. However, it is understood from their specification that Buckingham have proposed to produce a general engineered fill with an average CBR performance of 4% CBR, with no values of less than 3%. It is understood that lime modification is being used to condition fill which is wet of optimum.

Table 8 shows the range of CBR values determined as part of the CRW works.

Table 8: Summary of CBR Data

Level where CBR was tested	**CBR target for specified level**	**Minimum CBR Value**	**Average CBR Value**	**Maximum CBR Value**
Formation Level	5%	2%*	25%	66%
Fill Material	4%	1%	7%	16%

Where CBR results in the formation and general engineered fill were recorded below the minimum requirement of 5% and 4% respectively, additional works were undertaken and the area retested for compliance.

Fill thickness across the site have been established from IsoPach's on plans submitted by BWB. Table 9 summaries the fill thickness for areas within Phase 4Z.

Table 9: Summary of Fill Thicknesses

Area	Minimum Fill Thickness (m)	Typical site wide fill thickness (m)	Maximum Fill Thickness (m)
Phase 4Z	1.00-2.00	3.00-4.00	7.00-8.00
Phase 4Z East	1.00-2.00	2.00-3.00	7.00- 8.00
Phase 4Z-99	1.00-2.00	2.00-3.00	6.00-7.00

The fill material breakdown is as follows:

- 6% Class 2A Material (Selected well graded granular Material);
- 1% Class 1A (well graded granular material);
- 78% Class 2C (Stony Cohesive Material);
- 14% Class 1B (Uniformly graded granular material);
- 1% of material could not be classified in accordance with SHW Series 600.
- Natural Glacial Till was generally classified as a Class 2A material.

BWB report that the materials were very wet in comparison with the optimum moisture content required for adequate compaction. The average natural moisture content was 29% compared to the average optimum moisture content of 19%.

8.3.2 Mine treatment Validation

8.3.2.1 Mine Shafts

Four mine shafts were expected within the Phase 4Z area and all four appear to have been identified and treated as part of the CRW works. Table 10 provides a summary of the mine shaft investigation and treatment works undertaken in Phase 4Z.

Table 10: Summary of Phase 4Z Mine Shafts

Shaft Ref.	Shaft centre Coordinates	Shaft Diameter (m)	Depth at base of shaft	Current works completed
C	387241.8, 398358.0	2.0	27m	Shaft identified during earthworks at the base of the excavation. Shaft drilled, grouted with a 10:1 ratio of PFA to OPC grout and capped with a 4m by 4m concrete cap in accordance with CIRIA Report SP32.
D	387271.8, 398336.3	3.3	18.4m	Shaft identified during earthworks at the base of the excavation. Shaft drilled, grouted with a 10:1 ratio of PFA to OPC grout and capped with two 10m by 10m Tensar Geo-textiles.
E	387292.2, 398334.8	2.0	16m	Shaft identified during earthworks. Shaft drilled, grouted with a 10:1 ratio of PFA to OPC grout and capped with a 4m by 4m concrete cap in accordance with CIRIA Report SP32.
F	387341.5, 398343.5	1.9	27m	Shaft identified during earthworks. Shaft drilled, grouted with a 10:1 ratio of PFA to OPC grout and capped with 4m by 4m concrete cap in accordance with CIRIA Report SP32.

No evidence of additional unrecorded shafts has been reported.

It is noted that Shafts C and F appear to be located below the structural footprint of the north and east stands respectively. This will require coordination with the proposed foundation solution.

8.3.2.2 Mine seam treatment

Eight treatment grids were established on Phase 4Z as part of the mine workings stabilisation, which are summarised in Table 11.

Table 11: Summary of Treatment Grids in Phase 4Z

Seam Treated	Grid Number	Grid Spacing	Scope of works
Bradford 4 Foot and Fireclay	2	6m/12m	Drill to investigate and treat the possible shallow mien working from sub-crop to anticipated depth of 10x seam thickness
Bradford 3/4	3	6m	Drill to investigate and treat the possible shallow mine workings from sub-crop to anticipated depth of 10x seam thickness. 6m, grid beneath lightly loaded future parking and 12m grid beneath future football pitches.
Fireclay	4	12m	Drill to investigate and treat the possibility of fireclay working based on the works undertaken at the ASDA site adjacent.
Bradford 3/4	5	6m	Drill to investigate and treat possible shallow mine workings from sub crop to anticipate the depth of 10x seam thickness.

Seam Treated	Grid Number	Grid Spacing	Scope of works
Bradford 3/4	6	12m	Drill to investigate and treat possible shallow mine workings from sub-crop to anticipate depth of 10x seam thickness.
Charlotte	7	6m	Drill to investigate and treat the possible shallow mine workings from sub-crop to anticipate depth of 10x seam thickness. 6m grid beneath future football pitches.
Charlotte	8	12m	Drill to investigate and treat the possible shallow mine workings from sub-crop to anticipate depth of 10x seam thickness. 12m grid beneath future football pitches.

BWB conclude that for all of the treatment grids, the work has been completed satisfactorily and : '*The low grout takes indicate that either no workings have been undertaken in that area of the site, or that any historic workings which have been undertaken have since collapsed and fully choked or are at such a depth not to affect the surface.*'

8.4 Geotechnical Design Considerations

8.4.1 Proposed Building Foundations

The original reference foundation design by Capita was to adopt vibro ground improvement to support the main structure foundations to achieve an allowable bearing capacity of 150 kN/m^2.

Based on information known or assumed to date, in addition to technical, commercial, CDM and programme constraints, a number of foundation options were reviewed and briefly appraised as follows:

Table 12- Review of Foundation Options

Option	Option Description	Potential Benefits	Potential Risks
1	Ground Bearing directly on the CRW engineered fill i.e. Test the body of fill following handover using intrusive and geophysical techniques and review of validation data with a potential to use shallow foundations directly on the fill.	Least cost / Value Engineering Option. Programme beneficial for site operations.	Fill is not a structural fill and may be inherently unsuitable for direct loading i.e. review/testing may indicate that this option cannot be used. Fill is newer than CTF therefore risk of residual settlements if foundation construction commences early. Also the earthworks have been conducted during a period of high seasonal rainfall. No opportunity to specify appropriate enhancements to the CRW works undertaken Programme prohibitive to await validation

Option	Option Description	Potential Benefits	Potential Risks
			reports/additional GI and consider further interpretation (similar to the CTF areas). Implications of excessive settlements are significant in respect to the building to be constructed.
2	Mass Fill Concrete to underlying Glacial Till levels	Based on SI data to be obtained and validated by Arup. Offers a consistent / reliable ground conditions for the pad foundations to achieve the bearing capacity needed and could provide an increase in the assumed capacity. Offers opportunity to reduce ground beams needed to mobilise adjacent pad foundations if Vibro solution used.	Deeper excavation operations on site – may need temporary works CDM consideration. Usually these foundations only limited to c. 3m depth. But might need to be deeper on this site to reach the till. Relatively large volumes of concrete to be used, and thus associated costs of mass concrete understood to be greater than Vibro solution. Requires SI to be undertaken prior to confirmation of solution – risk will remain in the project until that time. Restrictions to gaining access to the site for SI.
3	Ground Improvement – Vibro Stone Columns	Offers a warranted design from the associated sub-contractor. Based on a performance specification, with the contractor undertaking any further SI and validation needed. Based on original Employers Requirements. Mobilisation understood to be achievable within BAM's programme requirements. Likely to be lower cost than mass concrete fill option, based on current assumptions.	Possibly not improving the ground bearing capacity over that which already exists. Variability of materials in CRW, unknown but possibly high Ground Water could cause bore collapse. Requires appropriate method statement for Vibro. Significant extent and depth of drainage excavations is required in close proximity to the proposed foundations which will loosen the stabilised fills and invalidate the Vibro warranty in those areas / levels. Requires coordination of setting out and hybrid mass fill solution in affected areas following discussion with Vibro specialist. Areas of Vibro Piled ground will be excavated following the operations, such to achieve the foundation levels. The depth of the remaining Vibro piles in the west stand may be relatively shallow, subject to CRW validation information. Not usually guaranteed in the upper 350-

Option	Option Description	Potential Benefits	Potential Risks
			500mm due to lack of confinement – Top 500mm would need to be re-engineered / compacted for slab areas. Vibro contractor may require validation reports from the CRW and trial pitting in order to inform their design prior to commencement. Stone columns could increase rate of settlement in cohesive fills as they will act as drains. Also may alter ground gas pathways, which will require consideration in the protection specification and solution. Potential water pathways in to main body of CRW fill - Risk of causing inundation settlement of the fill if it's been placed dry of optimum [Possibly a low risk owing to wet conditions during placement] Will only influence settlement of top 4m or so of site so there would still be an impact on self weight settlements of deeper fills. The use of stone columns could potentially impact on the ground gas regime as these are pathways for ground gas to migrate to the surface. If CRW fill is similar in stiffness for NWD as for CTF the settlement will be similar with stone columns also, as we would not expect much enhancement in already competent material.
4	Vibro Concrete Columns This is similar to the Vibro stone column solution but includes the incorporation of cement in to the material which is reported to solidify after placement	May improve the strength of column and therefore reduce impact of adjacent excavations.	Similar to stone columns but it is not a widely used a technique therefore may be more uncertainty on performance and validation.
5	Piled Foundations (probably Driven but could be CFA)	Based on Arup Specification to specialist design – offers greatest certainly in performance and less reliance on the nature of the general engineered fill placed in the CRW.	Structural redesign of all pad foundations required as they behave differently from pads. However caps could be significantly smaller in size. Programme prohibitive

Option	Option Description	Potential Benefits	Potential Risks
		Force paths better defined and transferred below CRW fill therefore the risk of undermining pile caps with drainage trenches is much reduced. Minimal requirement for mass concrete below pile caps. The risk of obstructions, (which is usually the problem for piling) should have been removed by the CRW. Driven pile solution has been adopted on adjacent residential developments therefore has been proven to be feasible in the local conditions.	Design Coordination with drainage and services delayed Cost unverified compared to other options but likely to be higher than a vibro option.
6	Composite Vibro Stone Columns/Mass Concrete Fill This solution would involve the installation of vibro stone columns as per Option 3 for all foundations, but provide additional mass concrete fill where excavation is required within 45 degrees of the top of stone column, to transfer loads below drainage.	Requirement for mass concrete fill is lower than option 2 as still supported by stone columns below.	Costs likely to be in between options 2 and 3, Stone columns above the base of mass concrete are abortive works but might be reusable as a reasonable backfill to foundations.

It was initially concluded that the preferred technical solution was either the piled option (option 5) or a composite vibro/mass concrete solution (option 6).

A shallow foundation solution founding on the Engineered fill was briefly considered but was discounted due to uncertainties associated with the thicknesses, quality and long term performance of the fill. The risks of this option were deemed to be greater than the foundations on fill at the CTF area.

The option of supporting the 150kPa foundation loads on ground improvement by vibro stone columns was selected as the most appropriate method of producing a support of uniform stiffness within the site constraints.

A ground improvement performance specification was developed which is based on the requirements of the “Specification for Ground Treatment” published by Thomas Telford Services 1987, with additional references to more recent

standards including BRE publication “Specifying vibro stone columns”, BR391 (published 2000).

These works are planned to commence in early May 2013.

8.4.2 Proposed Building Slab Foundation

It is proposed to adopt a ground bearing slab founded on vibro stone columns which are designed to support a bearing pressure of 50 kN/m^2. The assumed extent of concurrent fully loaded slab will need to be communicated to the vibro contractor to ensure the settlements can be designed to be within tolerable limits.

8.4.3 General design considerations

In addition to the foundation and slab requirements associated with the main structure, there are also areas of proposed external pavements, pitch construction and external soft landscaping. Further considerations with regards to these elements are included in Section 13.

9 Phase 5, 5A and 8 Geotechnical issues.

9.1 Proposed development

The Phase 5 and 5A areas are situated in the south of the development site. The area is to be developed in to three external training pitches and a pitch maintenance storage area known as the 'BoneYard', which contains several small silos and structures.

The Phase 8 area is located in the east of the site and is to comprise an external training area including a running track, running hill, sand pit and external exercise equipment set in an area of grassland/woodland.

9.2 Available Information

It is understood that the Geotechnical validation of these three areas is to be included within a single report, to be produced by BWB Consultancy. This report was not available at the time of writing.

9.3 General Design Considerations

Design considerations are expected to be covered by the general considerations within Section 13.

10 Phase 6,7 and 9 Geotechnical Issues

10.1 Proposed development

All of these areas are to be developed with a primarily external usage, with no significant structures being proposed.

The Phase 6 area is to be developed as an external training area, Phase 7 as an external training pitch and the western half of the northern car park. Phase 9 is an area which is to include three training pitches, a half sized pitch area and an area of smaller sports facilities.

10.2 Available Information

Information on the CRW earthworks validation for these three Phases is included in the following reports, produced by BWB Consultancy:

- Composite Master Validation Document for Phase 6,7 and 9, NTH1315SE/23/CMV/V3Rev 2, Final : Nov 2012
- Phases 6,7 and 9 Earthworks Validation Report – NTH1315E/15/EW/V2 – Dated Dec 2012 – Final Issue V2, received 25th Jan 13.

Mining related work for these areas was described in:

- Mine Stabilisation Works Validation Report Revision 3 – Works Complete as of 31 Oct 2012NTH1315E/12/MWR3/V2, Final : Nov 2012

These phases are situated to the north eastern, northern and eastern parts of the site respectively. The proposed development in these areas is understood to include external football pitches, a car park and an external fitness training area.

A high level review of the conclusions and key drawings for this work has been undertaken to determine the main considerations that might impact on the development.

10.3 Overview of CRW Validation

10.3.1 Earthworks Validation

In relation to Earthworks, BWB confirm in their composite master document the following:

- **Phase 6** – BWB state "...As this area of the site was designated as a borrow pit the excavation was then continued into the Glacial Till to create the pre-prescribed shape of the borrow pit …No backfilling of the created void was undertaken as part of this contract "
- **Phase 7** – BWB have excavated to natural ground, removed contamination and obstructions then refilled to provide a final formation CBR of 5%.
- **Phase 9** – BWB state that the objectives were similar to Phase 7. However an attenuation tank was also installed with excavation to a depth of approximately 3m from the top of Glacial Till.

- BWB state that they have used lime modification as part of the earthworks.

BWB present further detail of their earthworks validation works in the validation document (NTH1315E/15/EW/V2). This report is a PDF of some 8097 pages long, divided in to 9 main sections, 6 tables, 30 figures and 15 Appendices. The conclusions are as follows:

9.0 CONCLUSIONS

9.1 Based on the information contained within this report it is concluded that all made ground material within Phases 6, 7 and 9 have been excavated to the depth of Glacial Till with all obstructions having been removed to this level, with the exception of those structures, such as mineshafts caps and services, which have previously been agreed to remain in place.

9.2 The base of excavation has been inspected by the BWB resident engineer for remaining obstructions, none were observed in this area of site. The base of excavation was surveyed upon completion of this inspection.

9.3 The areas have been suitably backfilled with compliant material in 200mm to 275mm layers and compacted with a vibratory roller to formation. Each layer was tested for compliance. All layers demonstrated an average CBR value of or above 4%. Where this was not achieved, modification or removal of this material was undertaken and replaced with compliant material.

9.4 The final formation level demonstrated CBR values of 5%. The formation was capped with 100mm to 150mm of 6F2/6F1.

In addition,

- Figure 6 of the report indicates that a number of residual obstructions in the form of piles have been left in situ within Phases 7 and 9.
- In Area 7, a 3m diameter mine shaft has been found and plugged.
- No Galligu was reported to have been found in Phase 7 or 9 areas but was present in the Phase 6 area in Panel 204 (north of area).
- Phase 6 is shown to be clear of residual obstructions.
- An assessment may be required to understand if there are any clashes between these and the proposed construction.
- Figure 15 indicates a very deep circular feature was excavated in Phase 7 to around 56mOD. Figure 18 suggests that almost 7m of fill was required to fill this feature and this is clearly would be a risk area for settlement.
- Figure 24 suggests a series of deep excavations were undertaken in Phase 9, down to 53mOD. Figure 27 in the report suggests that this required up to 11m of fill placement which could result in overlying settlement.
- The report contains extensive sets of test results for these areas which have not been reviewed.

10.3.2 Mine Treatment Validation

In relation to mining, BWB confirm in their composite master document the following:

- **Phase 6** - "This area of the site is located to the east of the Bradford Fault and therefore not within the Coal Measures Strata; therefore no works to treat shallow mineworkings were required."

- **Phase 7** - "One Mineshaft, Shaft G, was indicated on the Coal Authority records to be located within Phase 7 of the site which had previously been identified on site and treated. This shaft required validation works to confirm the competency of the historic treatment works, involving drilling the shaft and re-grouting to assess competence of the backfill. These works confirm the historical works were adequate. Three areas of drilling and grouting of shallow mineworkings were required within Phase 7 of the site. Grid 1 to treat the fireclay workings and workings within the Bradford Four Foot Seam, Grid 2 to treat workings within the Bradford Four Foot Seam and Grid 5 to treat workings within the Bradford Three Quarter Seam".

- **Phase 9** - "This area of the site is located to the east of the Bradford Fault and therefore not within the Coal Measures Strata; therefore no works to treat shallow mineworkings were required."

BWB present further detail of their mining works in the validation document (NTH1315E/12/MWR3/V2). This is a 846 page PDF report comprising 7 sections, 5 tables, 8 figures and 31 Appendices. The document appears to cover site wide treatment rather than specifically the Phase 6,7 and 9 areas and is stated to include work up to 31 Oct 2012. The overall conclusions are as follows:

7.0 CONCLUSIONS

7.1 Based on the information contained within this report it is concluded that the issues associated with the past historical mining activity within handover Phases 6, 7 and 9 of the site as well as within Phases 1, 2, 3, 5, 5A and 8 of the site have been treated in accordance with the specification and the employers requirement.

7.2 Further treatment of shafts and mineworkings in the remaining handover phases will be reported in subsequent revision of this report as works are completed.

A previously treated mine shaft, Shaft G, is reported to be located in Phase 7. BWB report that earlier work in 1976 proved the shaft to be 3m diameter and 108m depth. The shaft had historically been grouted and plugged to form a plug of 6m diameter at the surface.

The CRW works drilled the shaft to 50m depth then grouted to validate the competency of the previous work. The BWB report concludes that for shaft G:

5.57	Due to the low grout take identified in the validation borehole, combined with the presence of the substantial concrete plug it is considered that the historical works have been suitably validated in accordance with the contract requirements and no further works are required.

Figure 5 Sheet 1 of 4 indicates the confirmed location of shaft G in Phase 7 and the treatment.

No shafts are indicated in the Phase 6 and 9 areas.

In addition to the treatment of shaft G, the report also includes details of mine stabilisation works undertaken in the Phase 7 area.

Figure 8 Sheet 2 of 4 indicates the extent of mine working treatment in Phase 7 area. No treatment is indicated in the Phase 6 and 9 areas, which is consistent with these areas being in non mined parts of the site.

10.4 General design considerations.

Based on an overview of the conclusions of the earthwork and mining elements of the documents received, BWB are suggesting the works were undertaken in accordance with the Employer Requirements. The utility and remediation works have not been assessed.

However there are issues that potentially impact on the geotechnical performance of the development. These are primarily the residual below ground pile obstructions and the wide variation in fill thicknesses.

10.5 Phase 9 – External fitness area

The proposals for this area are not fully developed. However it is currently understood that the area will comprise primarily soft landscaping with footways and training equipment. There are a number of potential geotechnical considerations associated with this area, in addition to geo-environmental issues:

- Earthworks requirements – It is understood that this area is currently receiving general fill materials excavated from the site which, due to subsequent exposure, are deemed unsuitable for use in the works on account of very high moisture content. Consideration will need to be given to the treatment and reuse of this material and appropriate specification.
- The area is adjacent to a canal and a slope assessment would be required to establish whether the proposed slope geometry might pose a risk to the stability.
- Increasing the levels of site above the general existing levels might result in ground settlements which although might not be problematic to the proposed surfacing, may impact on any proposed drainage and utilities in the area.
- Geo-environmental – Given the proposed more sensitive ‘park’ end use compared to the other Phases, and the close proximity to the canal, it is possible that additional chemical targets and testing might be required.

11 Phase 10 and 10A Geotechnical Issues

11.1 Proposed development

These areas are to be developed with a primarily external usage, with no significant structures being proposed.

The Phase 10 area is to be developed as two external training pitches. The Phase 10A is located to the north of Phase 10 and includes a goal keeping training area and an external car park.

11.2 Available Information

Information on the CRW earthworks validation for Phase 10 is included in the following report, produced by BWB Consultancy:

- Phase 4Z and 10 Earthworks Validation Report – NTH1315E/18/EW/V2~Dated April 2013 - Final Issues V2

Geotechnical validation of the Phase 10A are is not available at present.

11.3 Overview of Phase 10 CRW Validation

11.3.1 Earthworks Validation

Buckingham Group has conducted a Constraints Removal Works (CRW) to excavate out all Made Ground, remove obstructions and deal with mine shafts and coal workings. The area has been filled with compacted general engineered fill.

Isopach information indicating the range of fill depths across Phase 10 (Figure 21 of the validation report) suggests fill thicknesses range between 1 and 8m, with typical thicknesses being 2 to 5m.

Excavated Made Ground was processed as follows:

- A significant volume of Made Ground excavated was screened to remove material size over 100mm. Foreign material such as steel, plastic, tyres and wood noted during screening were removed prior to placement and compaction.
- Bricks and concrete removed from Made ground was crushed to a 6F2 materially initially and stockpiled.

Upon completion of earthworks over this phase, there was a requirement to leave on site stockpiles of 18,086m^3 of Type 3 material and 32,796m^{3}in accordance of SHW Series 800 as well as 5,000m^3 of ‘subsoil’. It is assumed that this was for incorporation in to the later works.

Figure 6 of the report confirms that the only residual obstruction left in place was mine shaft P, located near the southern boundary of the area.

Of the materials sampled for Geotechnical Analysis, approximately 79% of samples were Class 2C, 18% were Class 2A and 3% of samples were Class 1A. The Glacial Till material was predominantly classified as Class 2A.

The Employer has not requested any performance criteria for the bulk fill but has specified a 5% CBR at formation level and maximum settlement of 25mm. However, it is understood from their specification that Buckingham have proposed to produce a general engineered fill with an average CBR performance of 4% CBR, with no values of less than 3%.

Table 13 shows the range of CBR values determined as part of the CRW works.

Table 13: Summary of CBR Data

Level where CBR was tested	CBR target for specified level	Minimum CBR Value	Average CBR Value	Maximum CBR Value
Formation Level	5%	8.8%	93.5%	211%
Fill Material	4%	1.5%	6.1%	15%

Where CBR results were recorded below the minimum requirement of 5% at formation and 4% in general, BWB confirm that additional works were undertaken and the area retested for compliance.

All of the natural moisture contents were found to be higher than optimum and lime modification was used to condition fill which is wet of optimum. The average natural moisture content was 29% whilst the average optimum moisture content was 22%.

It is reported that small pockets of Galligu were encountered in the Phase 10 area in Panels 120-122 and this was moved to a designated area and stabilised.

11.3.2 Mine treatment Validation

11.3.2.1 Mine Shafts

One mineshaft was expected to be situated within the Phase 10 area, shaft. A further unrecorded feature, Shaft P was found during the excavation works.

Shaft P was found to be about 2m in diameter and the feature was capped with two 10m x 10m layers of geotextile placed at the level of competent glacial till.

BWB concluded that shaft K was located below a service easement for Mill Street Sewer, or is off site to the west.

11.3.2.2 Mine seam treatment

Treatment Grid 9 encroaches on to the east and north east of the Phase 10 area. The purpose of the treatment grid was to investigate and treat the Charlotte coal seam.

BWB have concluded the drilling and grouting works have been completed satisfactorily. Low grout takes indicate either no workings have been undertaken in this part of the site; or that historic workings have since collapsed. Collapsed historic workings are anticipated by BWB to be fully choked or at such a depth that they will not affect the surface.

11.4 General Design Considerations

The proposed function of this area is car parking and external pitches. Refer to Section 13.2 and 13.3 for general geotechnical considerations with regards to these uses.

12 Phase 11 Geotechnical Issues

12.1 Proposed development

This are is located in the south western corner of the site. It is proposed to develop this area for two educational buildings, with associated external car parking and landscaping.

12.2 Available Information

It is understood that the Geotechnical validation of this area is to be included within a CRW validation report, to be produced by BWB Consultancy. However, this report has not yet been made available.

This area is currently outside the area to be considered under the BAM contract.

12.3 General Design Considerations

This area is outside the current BAM/Arup scope. Further advice on the ground conditions and geotechnical considerations within this area should be confirmed once the proposals have been developed.

13 Other site wide considerations

13.1 External retaining walls and free standing walls

There are a number of external low height retaining walls and free standing separation walls required as part of the development.

In terms of low height retaining walls, a generic design was undertaken for walls with retained heights in bands of 0 to 0.5m, 0.5 to 1.0m and 1.0 to 1.5m.

These have been designed to the principles of BS8002, using the following assumptions:

- Assumed uniform surcharge at back of wall : 10 kN/m^2
- Moist/Saturated density of retained material : 18 to 21 kN/m^3
- Peak Phi of soil at back of wall : 34 degrees
- Design Phi of soil at back of wall : 24 degrees
- Design base friction angle : 18.6 degrees
- Unplanned overdig on passive side of wall : 200mm
- Allowable bearing pressure : 75 kN/m^2

Similarly, a 4m high RC wall (non-retaining) was required for the goalkeeper training area. The structural design based on 2m wide foundation resulted in a maximum static dead load of approx. 25 kN/m^2 and peak load (in wind conditions) of approx. 70 kN/m^2.

On this basis it was recommended that the shallow foundation was suitable providing the base of wall is positioned sufficiently deep to avoid the effects of frost, and allow for future shallow pavement constructions and services.

The above approaches are considered conservative providing that the following recommendations and verifications are carried out during the works:

- The Contractor is to initially excavate to the minimum proposed wall base formation level (i.e. underside of the 75mm blinding level) and inspect.
- In addition to the visual validation, Mexe probe testing is to be conducted on an approximately 5m grid to help identify soft or hard spots and verify that the stiffness/degree of compaction of the Made Ground material at formation level will provide an allowable bearing capacity in excess of 75 kN/m2
- Where existing hard spots are encountered at this level, these are to be removed to a depth of 0.5m below this level.
- Should soft, loose or organic deposits be encountered, these unsuitable founding materials should be removed.
- The base of excavation shall be prepared by proof rolling to seal the surface and compact any loose materials.

- The base of excavation level shall be kept free of water and vehicular trafficking minimised.
- Photographs shall be taken of each formation following preparation and prior to mass concrete/blinding.
- The overdug areas where soft or hard spots have been removed shall be backfilled with mass concrete.
- The foundation shall be blinded within 6 hours of exposure, or sooner if inclement weather is expected or otherwise agreed with the CA.
- The design assumes that no permanent ground water level exists above the base of the wall. If ground water ingress is noted during the excavation this should be brought to the attention of BAM.
- The backfill to the walls should be compacted class 6F1, 6F2, 6N or 6P granular material with Type A filter drain material being placed above the heel of the wall up to weephole level (approx 75mm above the lower retained level).
- Fill is not to be placed behind retaining walls or above sub structure until the concrete has attained full design strength and the structural configuration complies with the design intent.

13.2 Proposed external playing fields

The development incorporates a series of fifteen full sized sports pitches and two half pitches. It is understood that the detailed design of these is to be undertaken by specialist sports field contractors J. Mallinson Ltd. It is recommended that Mallinson review the validation reports in order to confirm that the as built conditions are in line with their design requirements for the pitch build up and any associated drainage and utilities.

13.3 Proposed Highways and car parks

There are a number of proposed external car parks, service yards and roadway areas with both asphalt and concrete finishes. The plan extent and type of pavement is indicated on the following drawing:

- Site wide Pavements and Roads – Pavement General Arrangement, Construction Types, Drawing C060.00.

The proposed construction thicknesses for these surfaces are provided on drawing:

- Site wide Pavements and Roads – Pavement Details, Drawing C764.55

The sub-base and capping thicknesses will be a function of the subgrade CBR value and a table of options are to be provided ranging from CBR of 2.5% to CBR 15%. The expected CBR values from the CRW suggest that CBRs of 4 to 5% would be obtained. However given the potential degradation of the surface materials since placement, it is possible that the actual values could be significantly lower (as found in the subsequent testing for the FM building see section 6.3.3). The contractor will need to undertake CBR/Plate load testing and determine the actual CBR design values prior to construction.

13.4 Proposed Covered seating/dug out shelter structures.

The development includes a series of six exterior covered seating (dug out) structures located adjacent to the playing pitches in the Phases 3,4,10 and 10A. The structures are lightweight polycarbonate shelters located on concrete base.

The applied ground bearing pressures are expected to be low (less than 20 kN/m^2) and consequently shallow ground bearing sub-structure is considered appropriate. The general requirements for validating the foundation quality will be similar to the points noted in Section 13.1 above. Any build up required to the base of foundation should be either in mass concrete or compacted Type 1 subbase.

13.5 Stabilised Tar materials

It is understood from discussions with Buckingham that significant quantities of tar impacted materials were encountered as part of the site wide CRW. It is understood that the CRW contractor Buckingham Group undertook stabilisation of this material and it was reused within the works.

It is believed that the tar impacted material was stabilised with ash and lime/cement and the material compacted in to pre-dug repositories within Phase 10A, to the south of the North West Arena.

Information indicating the precise locations where this material was deposited has not been provided. Details are also required of the intended and achieved geotechnical characteristics and durability of the material.

This should be reviewed in consideration of the proposed overlying development, although it is understood that this area 10A had been selected for the repository as the development is car parking and playing fields, with no proposed structures. However depending on the level of materials placed, there could be impacts on the proposed deep drainage within this area.

13.6 Stabilised Galligu materials

As discussed in Section 5.3.4.4 significant Galligu deposits have been encountered on site and this material was stabilised and reused on site. The original intention of the CRW was to use this outside the building footprints. However it’s been confirmed that material has been deposited below parts of the CTF area. It is possible that there are further areas where Galligu impacted materials have been deposited outside of the agreed areas.

A third party expert has been appointed from URS to review the potential risks of Galligu stabilised materials on the development.

13.7 Site wide clean cover requirements

It should be noted that there is a requirement to ensure that all materials placed within 450mm depth below surface of pitches and 300mm below surface of

landscaping are chemically tested for and meet a series of development criteria which have derived by White Young Green.

For other primarily hard end uses such as building slabs and bitmac roadway pavements and car parks, the chemical composition of imported materials should be tested against and not exceed the White Young Green remediation criteria. Site won materials should have already been remediated against these criteria under the CRW by Buckingham Group.

13.8 Earthworks – reuse of general engineered fill.

It is evident that significant quantities of lime modification were used in the CRW earthworks, primarily to reduce the moisture content of wet materials. It should be noted that the 'lime modification' process is different from the process of 'lime stabilisation', which generally requires higher quantities of lime/cement and requires extensive site trials. Buckingham reported that between 1 and 2% of lime was typically added for the modification purpose.

As the purpose of the lime addition was primarily to dry the material rather than to create any long term strength improvement through pozzolanic action (which would have required a stabilisation process). Providing that the moisture content is consistent with that when the modified material was first placed, and the material has not otherwise been altered, it may be possible to re-compact it as a general engineered fill.

If the material:

- has been altered E.g. due to rain/ground water,
- does not comply with the current earthworks specification,
- or does not compact to a well closed stable surface (showing no movement under compaction plant)

then it should not be incorporated in to the further earthworks without further modification.

It should be pointed out that much of the general engineered fill is a Class 2C (general stony cohesive fill) and will be susceptible to degradation in the presence of excess moisture.

13.9 Mine stabilisation works

It should be noted that the mine treatment works are presented in the BWB report:

- Mine Stabilisation Works validation Report Revision 6 – NTH1315E/12/MWR6/V2 -Works complete as of 5 March 2013

With the exception of the items already highlighted, no detailed assessment of the quality or completeness of the mine stabilisation works have been undertaken. It is assumed that BWB have designed the treatment works in cognisance of the proposed end use of the site, including overlying structures, and in accordance with the Employers Requirements for the Constraints removal works produced by White Young Green.

In Section 2.8.2 of the Employer Requirements it is stated that formal sign off from the Coal Authority, to confirm that they consider the works to be complete to their satisfaction shall be submitted. It is assumed that this has been obtained. However it was not included in the report.

14 Surface Level Monitoring

14.1 Scope of monitoring works

As discussed in earlier sections, given the significant thicknesses of engineered fill placed as part of the CRW works, there is a possibility of residual ground movements due to either self weight consolidation of the placed fill or, where ground levels have been increased, due to the surcharge loading on the underlying clay.

A series of 8 No. settlement monitoring points were installed by Buckingham as part of the CRW earthworks in the Phase 1 CTF area, to allow post CRW level monitoring. These comprised 4 monitoring points in both of the Elite and Academy areas.

In order to better understand the ground movements and provide early identification of any significant movements, Arup recommended that the monitoring of these 8 No. monitoring stations plus installation and monitoring of additional monitoring points should be incorporated in to the earthworks for the remainder of the site.

The particular areas where this monitoring was to be implemented, in addition to the 8 No stations installed in the Elite/Academy are as follows:

- Southern CTF area (formerly the Laundry area) – 4 no. points
- Proposed Energy building – 4 no. points
- Centre of pitch – 1 no. point
- Proposed North West Development – Upon completion of filling in each area – 4 no. points per stand plus centre point of pitch (total 17 points)

In addition, Arup recommended that similar stations were to be installed in other external areas of deeper general engineered fill which might be subjected to ground movements which could impact on proposed construction.

Arup recommended that levelling of these pins was to be conducted immediately after construction of the survey station, which should be as soon as possible after placement of the fill, then on a frequency of twice per week initially for a period of 1 month. Following this, Arup recommend that the frequency can be adjusted or points omitted depending on whether a measurable movement is still occurring.

The monitoring points which are currently being recorded are indicated on Figure 41 below.

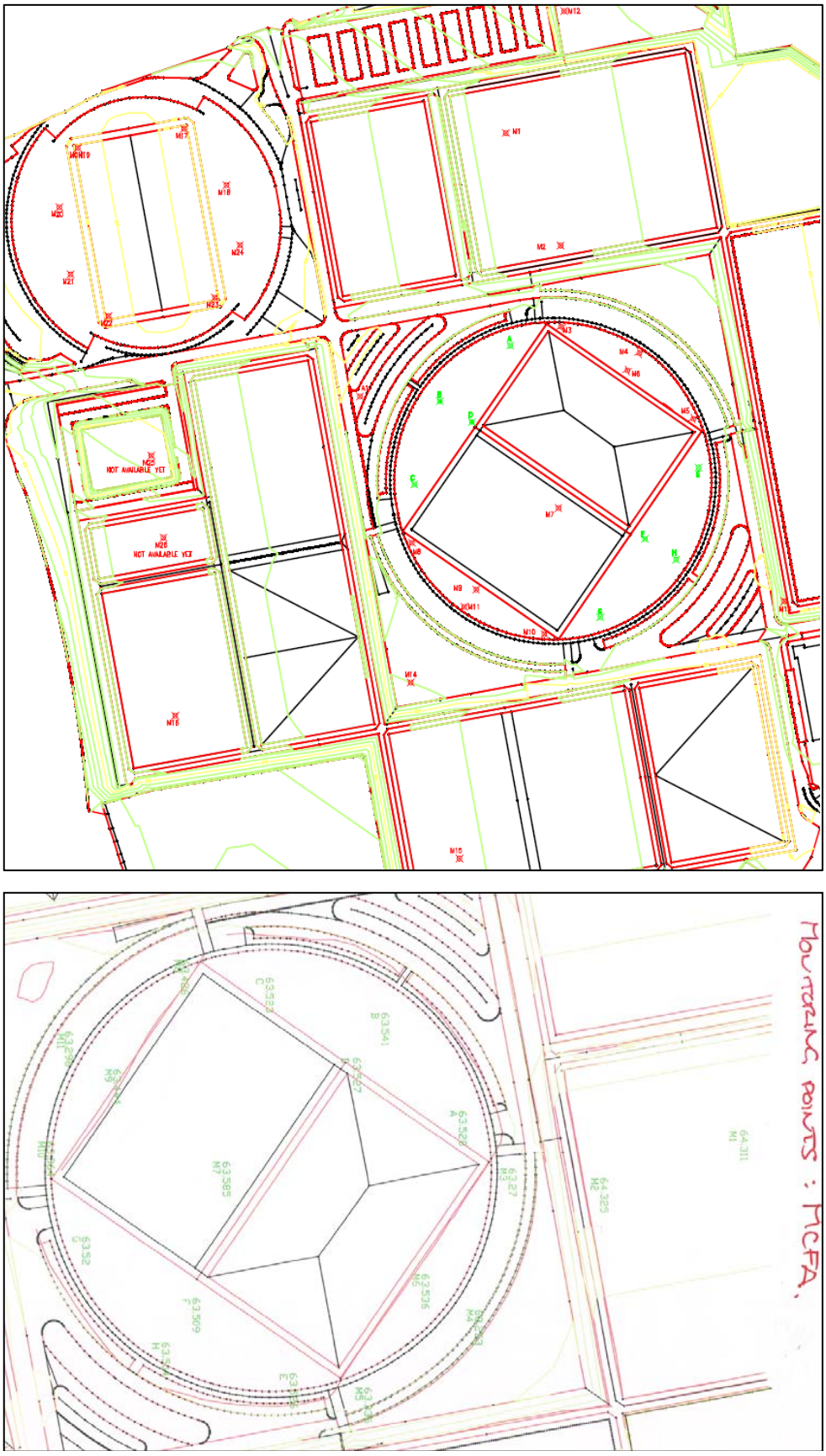


Figure 41 Sitewide and CTF Settlement Monitoring Points

14.2 Results of surface level monitoring undertaken up to 8th April 2013

14.2.1 Monitoring in the CTF area

A total of 8 surface monitoring stations were installed on the enhanced CRW earthworks areas by Buckingham, with 4 being on the Academy and 4 in the Elite areas. These are indicated as A1 to D1 in the Academy area and E1 to H1 in the Elite area.

Level monitoring is understood from the provided records to have started on 30th October and 9 rounds of monitoring are indicated up to end of Nov 12.

Up until the last 2 readings (26th and 29th Nov), all readings are within a range of +/- 3mm which suggests, that no significant residual settlements are occurring and the measurements may actually be due to instrument limitation/surveying tolerances. No further readings have been provided since 29th Nov 12.

The substructure works were reported to have commenced on 24th Nov and the last two rounds indicated a maximum 39mm upwards displacement (position E1) and 11mm downwards displacement (position G1). However BAM report that these stations had been visibly disturbed by the groundworks, and therefore are probably not representative of underlying settlements.

It is understood that further unavoidable disturbance of these has occurred as the works progressed and consequently these stations have now been lost.

14.2.2 Site Wide Monitoring Points

14.2.2.1 Positions M1 to M11

A total of 11 surface monitoring stations have been installed primarily in the Phase 1 CTF area, situated on general engineered fill deposits. These are indicated as M1 to M11 on the BAM drawing and were installed 12th Nov and monitored on five subsequent periods.

Positions M1 and M2 are situated north of the CTF area. The displacements indicated so far are from +2mm to -9mm, with a larger increase in settlement between 22nd Nov and 26th Nov possibly due to weather conditions/low instrument visibility.

Positions M3 to M6 are situated in the proposed Energy building. Displacements are generally low, between +5mm and -3mm.

Position M7 is in the centre of the pitches and has displaced between +3 and -3mm, which again is not considered significant.

Position M8 to M11 situated in the formerly proposed Laundry building have displaced between +6 and -4mm, which again is not considered significant.

It is recommended that the stations continue to be monitored on a reduced frequency of every 3 weeks rather than weekly.

No further readings have been provided since 29th Nov 12.

14.2.2.2 Positions M12 to M26

A further set of 15 monitoring stations have been installed within the development. Monitoring data has been provided for up to 4 rounds of monitoring between 4th March 13 and 8th April 13.

Position M12 to M16 have been monitored on three occasions after the base line reading and indicate vertical displacements of up to 5mm, with some points increasing in apparent level.

Positions M17 to M24 are in the North West Arena (Phase 4) and have been monitored once after initial baseline reading and indicate displacements between 1mm settlement and 3mm of heave.

Positions M25 and M26 have not yet been monitored but are in the Phase 10A area in the west of the site.

No further readings have been provided for these positions since 8th Apr 13.

14.3 Summary

The monitoring reported so far is indicating that there are no gross settlements occurring in these areas over the timescale of measurements. BAM have confirmed that the reference datums are off site and therefore assumed stable. BAM also confirmed that the accuracy of readings is believed to be in the range of +/- 2mm.

Providing the rates of displacement remain stable or are decreasing, this provides confidence that residual ground movements should be low.

When new stations are installed in site wide locations of deep fill, they should similarly be monitored weekly for a period of 4 weeks then providing stability is proven, the frequency can be reduced.

From earlier discussions with MCC Building Control, it is understood that they are very interested in the implementation of this monitoring and it is recommended that this information is also forwarded to them.

15 Design Risk Register

Initiating a geotechnical risk management process is the most effective method of reducing ground related risks, potentially reducing costs and construction time. This enables mitigation measures to be incorporated within the design, reducing the vulnerability of the project to factors such as unforeseen ground conditions.

A preliminary risk register of identified geotechnical risks is presented below, **Table 14**, which will be further developed as more information becomes available in subsequent stages of project development.

Table 14: Geotechnical Risk Register

		Register reference	4-08-04
Project	MCFA – Site wide	Job number	226518
Package/Topic	Geotechnical – Design Issues and Commercial Risk Register	Design stage	CONSTRUCTION – C1 Costing Drawings Issue

Remember: Avoid – Reduce – Control and communicate relevant information to others (CDM Regulation 11)

Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
25/10/12 TWES/LDJ	SW Drainage	Ground settlement leading to physical damage to drainage infrastructure/ loss of functionality. Settlements are expected to be time related and will have less impact if the time interval between CRW earthworks placement and drainage construction is extended.	Use of Rocker pipes to allow a degree of articulation in the event of settlement, check minimum gradients and avoid shallow gradient runs. No specific shallow hard spots are identified however there may be some differential movement due to the varying thickness of placed CRW fill. Surface level monitoring has been		R		Review settlement amount being monitored at CTF and adjust any assumptions Ensure that fill levels have stabilised in areas of deep fill prior to drainage installation.	Arup/ BAM	Active

		Register reference	4-08-04
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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
			recommended in areas of deeper fill to verify the movements have stabilised.						
8/11/12 LDJ	CTF Shallow Foundations	Design assumes foundation conditions are as shown in the CRW validation: Variation of pad formation levels and proximity to basement will require tight control of temporary works to ensure stability and avoid weakening adjacent formations.	Construction sequencing and temporary works control.		R	C	Construction sequence and temporary support measures to be developed and implemented.	BAM	Active
8/11/12 LDJ	CTF Shallow Foundations	Design assumes foundation conditions are as shown in the CRW validation: Protection of excavations from weather/water ingress from ground seepages or runoff during the	Ground and surface water control. Leaving protective layer above pad formation until immediately prior to			C	Ground and surface water control measures to be developed and implemented. Leave formations high in inclement weather.	BAM	Active

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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
		excavations, (particularly if conducted in Winter) - would result in softening of founding materials which could be weaker than assumed.	pad construction.				Contractors QA procedures to include inspection of formations and in-situ testing.		
8/11/12 LDJ	CTF Mine Treatment works	Extent of treatment of shallow workings close to the Bradford Fault are assumed to be limited – If workings are more extensive than assumed, this would result in additional ground settlements which could potentially impact on the CTF structures and require additional treatment.	No further mitigation at this stage		R	C	No further action at this stage.	BAM	Active

		Register reference	4-08-04
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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
8/11/12 LDJ	Location of Bradford Fault	A known fault (Bradford Fault) passes below the southern CTF area (former Laundry) and Academy buildings. It is usually good practice to avoid building over faults in mining areas. There is a residual risk of future fault reactivation resulting in ground displacements and structural damage.	The potential to relocate the CTF facilities to an off fault location is no longer practical. Avoidance of deep rock socketted piles will help isolate the structure from movements at rockhead. Architectual layout to be checked and allowances made in detailing E.g. brickwork jointing.		R		Jointing in brickwork. Client to be made aware of the residual risk of superficial cracking in internal decorating.	BAM/ MCFC	Active
8/11/12 LDJ	Removal and reprocessing of Made Ground	It is assumed that the CRW contractor has excavated all Made Ground to confirm natural Glacial Till deposits	Potentially review the reported base of excavation compared to the historical base		R	C	Contractor to be alert for excavated materials which include clearly unprocessed and variable	BAM	Active

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		in all areas of the site even though this was not required by the ERs. If not undertaken or if there's a change in this strategy for parts of the site, Made Ground may be left in site below any engineered fill. The unprocessed Made Ground is likely to be of a more variable stiffness compared to the engineered fill assumed in the foundation design and result in additional ground movements or higher differential settlements. In addition, there would be no certainty that	of Made Ground from the ground investigation. Observations during future excavations.				Made Ground.		

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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
		unrecorded mineshafts were not present in these areas.							
9/11/12 LDJ	Clashes between existing substructure and new foundations/below ground drainage.	Existing piled foundations were encountered during the CRW excavations in the CTF area, in particular the Energy, Elite and Academy areas. These features are shown in Figure 6 of the BWB Phases 1, 2 and 3 Validation report (NTH1315E/13/EW/V2). Clashes with new shallow foundations will need to be eliminated by further pile removal. Additional work costs and	Adoption of a shallow foundation solution in these areas will reduce the potential impacts of new pile installations. Review of residual risks for each area in advance of construction. Additional breaking out work to ensure existing pile head 1m below new foundation base.		R	C	Identify risk areas for foundation clashes to allow better planning of works. Further breaking out of foundations to remove risk of ‘hard spots’	BAM	Active

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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
		potential delays in the construction of new foundations.							
12/11/12 LDJ	CTF Shallow foundations and slabs	Variation in founding material across building is beyond that allowed for in analysis.	Sensitivity checks in analysis. In-situ testing at formation		R	C	Develop testing strategy and incorporate in to specification.	Arup/ BAM	Active
12/11/12 LDJ	CTF Shallow foundations and slabs	On-going settlement of formation as a result of self weight compression and surcharging due to newly placed fill.	A settlement monitoring strategy has been outlined and recommended to BAM to identify any on-going settlement in areas of deeper fill.		R	C	Implement surface level monitoring as recommended. Review of results prior to construction on these areas.	BAM	Active
12/11/12 LDJ	Site wide – Below ground structures	Sulphate levels in backfill are unmeasured. Potential that following the CRW they are higher than suggested by the GI.	Further testing		R	C	Further testing	BAM	Active

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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
12/11/12 LDJ	CTF Areas – Enhanced Engineered Fill	Enhanced engineered fill is of a higher permeability to the surrounding general engineered fill and may act as a water sump where runoff can collect. Presence of water may result in softening of the underlying clay during foundation construction.	Review formations and water ingress during construction. Water control measures.		R	C	To be included in contractor methodology.	BAM	Active
13/11/12 LDJ	Site Wide	Lime stabilisation has been used as part of the CRW to improve the soil strength. This may have impacted on the soil chemistry potentially resulting in the release of additional contaminants.	Buckingham to be asked to provide evidence to demonstrate this issue does not impact on the residual ground risks.		R	C	BAM to request assurances from Buckingham. Further testing in areas of lime stabilisation may be required if not already conducted.	BAM	Active

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			Further leachability testing would be required to address this issue.						
13/11/12 LDJ	CTF Shallow foundations and slabs	Backfilling to shallow foundations and basement is critical as this provides support to overlying ground bearing slabs and foundations at higher levels. Wet weather may render the excavated materials unsuitable for replacement.	Tight controls on fill types and validation testing to be included in Earthworks specification. Use of imported structural fill where materials unsuitable for reuse.		R	C	Control and validation of filling works.	Arup/ BAM	Active
23/11/12 LDJ	CTF Shallow foundations and slabs – Southern CTF area (formerly Laundry Building)	An unrecorded shaft feature Shaft S was encountered under the former laundry area. In addition to the physical	Shaft treatment works have been undertaken under CRW. If future piled foundations adopted,		R	C	It is assumed that the CRW has fully stabilised the feature. Foundation design to consider the potential	Arup/ BAM	Active

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Date (+ initials)	Area/Location of Risk Exposure	Description of Issue and Risk Exposure	Mitigation of Risk (Potential or Achieved)	A	R	C	Further Action	by	Status Active/closed
		presence, there is a risk of instability to overlying foundations.	new piles to be 1.5 to 2m from edge of shaft and extending to a depth below shaft. If shallow foundations used, formation levels and plan positions to be considered relative to shaft and substructure modified or further treatment proposed to minimise residual risk.				impact on the shaft in both plan and elevation.		
23/11/12 LDJ	CTF Shallow foundations and slabs – Impact of drainage works	Drainage installation sequence and protection. Risk of undermining shallow foundations if installed before drainage and drainage is deeper. Potential vertical and	Localised deepening of pads using mass concrete to transfer load to below proposed drainage excavations.		R	C	Implementation of mitigation measures. Construction sequence to be carefully considered.	Arup/ BAM	Active

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		lateral clashes with existing ground obstructions.	Further breaking out of existing piles where clashes with proposed drainage and shallow foundation construction.						
23/1/12 LDJ	North West Arena - Foundations	No validation data available as works are currently incomplete. Ground conditions and residual mining features may significantly impact on the proposals.	Clarify conditions prior and validate design assumptions prior to installation of new foundations.	A	R		Vibro contractors may require review of validation testing and further trial pitting after completion of CRW.	BAM	Active
23/1/12 LDJ	North West Arena - Foundations	Installation of drainage is required below proposed pad foundation levels. Mass concrete fill	Use of mass concrete fill below pads to a depth below proposed drainage to avoid subsequently		R	C	Design of suitable temporary works to provide safe support.	BAM	Active

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		requirement.	undermining the pads. Temporary works are required to ensure support of adjacent operations and staff safety.						
17/4/13 LDJ	Site Wide – Areas of stabilised Galligu.	The location and long term Performance of the stabilised Galligu material is uncertain. Materials have been deposited below the CTF building.	Consultant URS are conducting an appraisal of the likely impact of this treatment on the development. The proposed CTF Laundry building has been relocated to the FM building.	A	R	C	Review findings of URS report.	BAM	Active

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Date (+ initials)	**Area/Location of Risk Exposure**	**Description of Issue and Risk Exposure**	**Mitigation of Risk (Potential or Achieved)**	**A**	**R**	**C**	**Further Action**	**by**	**Status Active/closed**
17/4/13 LDJ	Site Wide - Areas of stabilised Tar.	Details of the tar treatment areas and the types of treatment adopted have not been reported. Uncertainties with regards to impacts on the development.	Performance of the material to be clarified.		R	C	Obtain and review Buckingham report on this treatment.	BAM	Active