

Appendix C

Coal Authority Mining Report



The Coal
Authority

Resolving the **impacts** of mining

CON29M Non-Residential Mining Report

70A MIDDLECLIFFE LANE
LITTLE HOUGHTON
BARNSELEY
S72 0HN

Date of enquiry: 16 May 2017
Date enquiry received: 16 May 2017
Issue date: 16 May 2017

Our reference: 51001457843001
Your reference:



CON29M Non-Residential Mining Report

This report is based on, and limited to, the records held by the Coal Authority and the Cheshire Brine Subsidence Compensation Board's records, at the time we answer the search.

Client name

John Pearcey

Enquiry address

70A MIDDLECLIFFE LANE, LITTLE HOUGHTON,
BARNLEY, S72 0HN

How to contact us

0345 762 6848 (UK)
+44 (0)1623 637 000 (International)

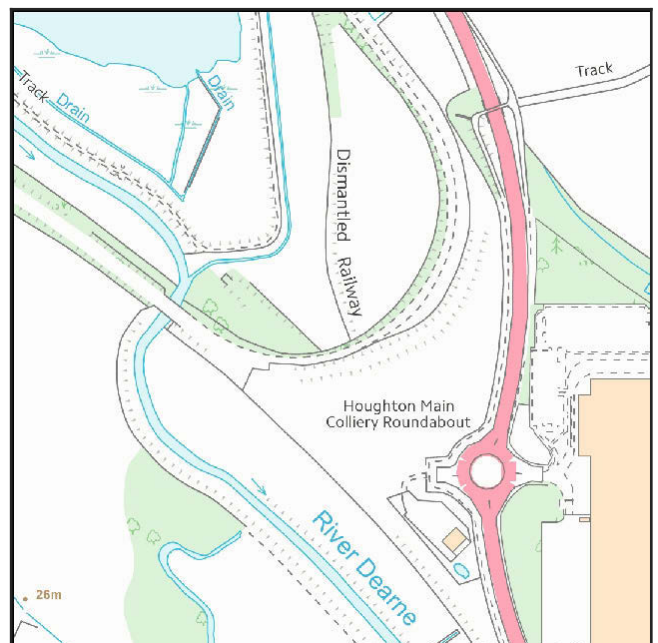
200 Lichfield Lane
Mansfield
Nottinghamshire
NG18 4RG

www.groundstability.com

 /company/the-coal-authority

 /thecoalauthority

 /coalauthority



Approximate position of property



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Summary

Has the search report highlighted evidence or potential of		
1	Past underground coal mining	Yes
2	Present underground coal mining	No
3	Future underground coal mining	Yes
4	Mine entries	Yes
5	Coal mining geology	No
6	Past opencast coal mining	Yes
7	Present opencast coal mining	No
8	Future opencast coal mining	No
9	Coal mining subsidence	No
10	Mine gas	No
11	Hazards related to coal mining	No
12	Withdrawal of support	Yes
13	Working facilities order	No
14	Payments to owners of former copyhold land	No
15	Information from the Cheshire Brine Subsidence Compensation Board	No

For detailed findings, please go to page 4.

Detailed findings

1. Past underground coal mining

The property is in a surface area that could be affected by underground mining in 10 seams of coal at 320m to 840m depth, and last worked in 1991.

Any movement in the ground due to coal mining activity should have stopped.

In addition the property is in an area where the Coal Authority believe there is coal at or close to the surface. This coal may have been worked at some time in the past. The potential presence of coal workings at or close to the surface should be considered prior to any site works or future development activity. Your attention is drawn to the Comments on the Coal Authority information section of the report.

2. Present underground coal mining

The property is not within a surface area that could be affected by present underground mining.

3. Future underground coal mining

The property is not in an area where the Coal Authority has plans to grant a licence to remove coal using underground methods.

The property is not in an area where a licence has been granted to remove or otherwise work coal using underground methods.

The property is not in an area likely to be affected from any planned future underground coal mining.

However, reserves of coal exist in the local area which could be worked at some time in the future.

No notices have been given, under section 46 of the Coal Mining Subsidence Act 1991, stating that the land is at risk of subsidence.

4. Mine entries

There are no known coal mine entries within, or within 20 metres of, the boundary of the property.

There may however be mine entries/additional mine entries in the local area which the Coal Authority has no knowledge of.

5. Coal mining geology

The Coal Authority is not aware of any damage due to geological faults or other lines of weakness that have been affected by coal mining.

6. Past opencast coal mining

The property is within the boundary of an opencast site from which coal has been removed by opencast methods.

7. Present opencast coal mining

The property does not lie within 200 metres of the boundary of an opencast site from which coal is being removed by opencast methods.

8. Future opencast coal mining

There are no licence requests outstanding to remove coal by opencast methods within 800 metres of the boundary.

The property is not within 800 metres of the boundary of an opencast site for which a licence to remove coal by opencast methods has been granted.

9. Coal mining subsidence

The Coal Authority has not received a damage notice or claim for the subject property, or any property within 50 metres, since 31 October 1994.

There is no current Stop Notice delaying the start of remedial works or repairs to the property.

The Coal Authority is not aware of any request having been made to carry out preventive works before coal is worked under section 33 of the Coal Mining Subsidence Act 1991.

10. Mine gas

The Coal Authority has no record of a mine gas emission requiring action.

11. Hazards related to coal mining

The property has not been subject to remedial works, by or on behalf of the Authority, under its Emergency Surface Hazard Call Out procedures.

12. Withdrawal of support

The property is in an area where a notice to withdraw support was given in 1982.

The property is not in an area where a notice has been given under section 41 of the Coal Industry Act 1994, cancelling the entitlement to withdraw support.

13. Working facilities order

The property is not in an area where an order has been made, under the provisions of the Mines (Working Facilities and Support) Acts 1923 and 1966 or any statutory modification or amendment thereof.

14. Payments to owners of former copyhold land

The property is not in an area where a relevant notice has been published under the Coal Industry Act 1975/Coal Industry Act 1994.

15. Information from the Cheshire Brine Subsidence Compensation Board

The property lies outside the Cheshire Brine Compensation District.

Comments on the Coal Authority information

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In view of the mining circumstances a prudent developer would seek appropriate technical advice before any works are undertaken.

Therefore if development proposals are being considered, technical advice relating to both the investigation of coal and former coal mines and their treatment should be obtained before beginning work on site. All proposals should apply good engineering practice developed for mining areas. No development should be undertaken that intersects, disturbs or interferes with any coal or mines of coal without the permission of the Coal Authority. Developers should be aware that the investigation of coal seams/former mines of coal may have the potential to generate and/or displace underground gases and these risks both under and adjacent to the development should be fully considered in developing any proposals. The need for effective measures to prevent gases entering into public properties either during investigation or after development also needs to be assessed and properly addressed. This is necessary due to the public safety implications of any development in these circumstances.

Additional remarks

Information provided by the Coal Authority in this report is compiled in response to the Law Society's Con29M Coal Mining and Brine Subsidence Claim enquiries. The said enquiries are protected by copyright owned by the Law Society of 113 Chancery Lane, London WC2A 1PL. Please note that Brine Subsidence Claim enquiries are only relevant for England and Wales. This report is prepared in accordance with the Law Society's Guidance Notes 2006, the User Guide 2006 and the Coal Authority and Cheshire Brine Board's Terms and Conditions applicable at the time the report was produced.

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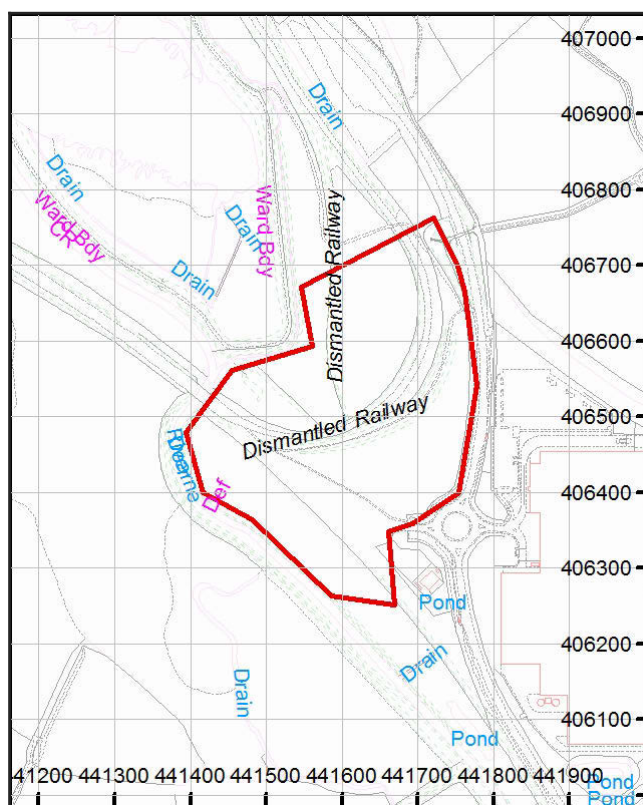
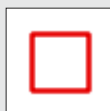
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Enquiry boundary

Key

Approximate position of enquiry boundary shown



How to contact us

0345 762 6848 (UK)
+44 (0)1623 637 000 (International)

200 Lichfield Lane
Mansfield
Nottinghamshire
NG18 4RG

www.groundstability.com

 /company/the-coal-authority

 /thecoalauthority

 /coalauthority



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VAT receipt

Issued by	The Coal Authority 200 Lichfield Lane Mansfield Nottinghamshire NG18 4RG
Tax point date	16 May 2017
Issued to	JOHN PEARCEY 9 HORNBEAM SQUARE SOUTH HARROGATE NORTH YORKSHIRE HG2 8NB
Property search for	70A MIDDLECLIFFE LANE LITTLE HOUGHTON BARNSELY S72 0HN
Reference number	51001457843001
Date of issue	16 May 2017
Cost	£78.30
VAT @ 20%	£15.66
Total received	£93.96
VAT registration	598 5850 68

Appendix D

01975AB_R_001A_DAS - Foundation Assessment for Office Extension

**Buckingham Group
Foundation Assessment for Office Extension
ASOS, Houghton Main**

14 February 2017



Results emerge
when local knowledge
intersects with
global expertise

Document Verification

Page 1 of 1

Job title		Houghton Main Office Extension		Job number	
				01975AB	
Document title		Foundation Assessment for Office Extension ASOS, Houghton Main			File reference
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		Name	David Stone	Nabeel Bux	Andrew Smith
		Signature			
		Filename			
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		Name			
		Signature			
		Filename			
		Description			
			Prepared by	Checked by	Approved by
		Name			
		Signature			

Coffey Geotechnics Limited

 9/11 Hornbeam Square South, Hornbeam Park, Harrogate, North Yorkshire HG2 8NB United Kingdom
 T (+44) (0) 1423 878 130 F (+44) (0) 1423 878 131 coffey.com

 Registered Office: 1 Northfield Road Reading Berkshire RG1 8AH United Kingdom
 Registered in England No. 06328315

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1 INTRODUCTION

Coffey Geotechnics Ltd (Coffey) has been commissioned by Buckingham Group as geotechnical designer to design the founding solution for a proposed extension at the existing ASOS warehouse at Park Spring Road, Grimethorpe, Barnsley. The site location is shown on 01975AB_D_001.

The lead designer for the Scheme is RPS Ltd, the Architect is AJA Architects LLP and the project manager for the scheme is Knight Webb.

2 PROPOSED DEVELOPMENT

The Proposed development comprises the construction of an office extension to the east of the existing warehouse. The proposed extension location is shown on AJA Architects drawing 6010 - 025.

At the time of writing, limited information was available to design the foundations for the scheme. Therefore, the foundation design has been based on preliminary un-factored column loads provided by RPS and enclosed as Appendix A.

3 REFERENCES

In preparation of this document we have reviewed the following information;

- “RSA Geotechnics Ltd – *Ground Investigation Report*” (Ref – 9735), RSA, Dated August 2005
- “RSA Geotechnics Ltd – *Interpretive Geotechnical Report*” (Ref – 13378GI), RSA, Dated December 2012
- “RPS Civil and Structural Requirements *ASOS Warehouse Extension, Barnsley*” (Ref – NK014833) November 2012.
- “RJB Mining (UK) Ltd Correspondence – *Houghton Main Site – Treatment to shafts*” (Ref – JNA/HS), Dated 18th September 2000.
- “The Coal Authority – Coal Mining Report - *Houghton Main Colliery Shafts, Darfield, South Yorkshire*” (Ref – 00007712-06) Dated 21st March 2006.
- “The Coal Authority – Non Residential Coal Mining Report - *Site at Houghton Main Barnsley*” (Ref – 710000290861002) Dated 30th October 2012.
- Babtie UK Coal Mining Limited *Houghton Main Opencast Site Draft Completion Report* (Ref – 24130/R22) Dated 16th September 2004.

4 SITE DEPOSITIONAL HISTORY

The existing warehouse is constructed on a former UK Coal owned site which had been subject to opencast mining.

The records reviewed indicate that the opencast site was excavated and backfilled between 1997 and backfilled by 2001. The approximate thickness of fill generally varies from 10m at the south west side of the site to 26m at the north east side of the site.

The specification for reinstatement of the former opencast site required the backfill to achieve a safe bearing capacity of 110kPa or better for a 2m by 2m foundation, without producing more than 20mm of settlement.

The existing distribution warehouse was constructed in 2006. Dynamic compaction was carried out prior to construction within the proposed building footprint, which extended to 5m beyond the proposed build line. The ground improvement was designed to compact the top 6m of fill in order to reduce settlements from the building load.

In 2013 two extensions, approximately 0.6 Ha each in area were constructed to adjoin to the existing warehouse. One extension was constructed at the west side of the warehouse and one was constructed at the south of the warehouse. Dynamic compaction could not be undertaken in close proximity to the existing building without causing damage to the structure. Therefore, High Energy Impact Compaction (HEIC) was used to densify the upper 3m of the fill reducing the magnitude of total and differential settlement between the original warehouse and the extensions.

For details refer to 01975AA_L_001A_DS and 01975AA_R_001A_DS enclosed as Appendix B.

5 INVESTIGATION

Investigations were undertaken on site by RSA Geotechnics Ltd in 2005 for the original construction which comprised

- Eight cable percussive boreholes to a maximum depth of 15.0 m
- Seven dynamic probes to a maximum depth of 12.1 m
- Six trial pits to a maximum depth of 4.8 m

Limited geotechnical laboratory testing was carried out during the investigation. However, in-situ SPTs were carried out in all the exploratory holes.

For details of all testing undertaken on site refer to RSA Geotechnics Ltd – Ground Investigation Report” (Ref – 9735).

5.1 Ground Conditions

The ground Conditions encountered on site comprised made ground to the base of all of the exploratory holes at 15 m depth. The made ground comprised firm to stiff friable very gravelly clay with gravel of weak to moderately strong mudstone, siltstone and sandstone or weak gravel and boulders of mudstone, siltstone and sandstone within a clay matrix. Historical contour plans of the base of the opencast mine indicate that the thickness of the made ground on site is up to 26 m.

The base of the made ground was encountered in two holes BH2 and BH7 at 12 m and 10 m depth respectively. The exploratory hole records show that the made ground is underlain by moderately weak mudstone. However, based on the historical extent of the mining it is likely that the mudstone encountered in BH7 is not the base of the made ground and could be a boulder in the backfill. These boreholes are, however, some distance from the location of the proposed extension.

5.2 Groundwater

Groundwater was encountered during the investigation between 9.5 m and 12 m in the boreholes. Groundwater was not encountered in the trial pits undertaken on site.

5.3 Soil Properties

The results of the SPT undertaken on site are shown on Figure 1 below. The SPT's indicate that in the upper 4 m of the backfilled material is very dense / stiff with SPT's in the range from approximately 30 to 50. Below 4 m the density of the fill becomes more variable with SPT N values ranging from 12 to 80.

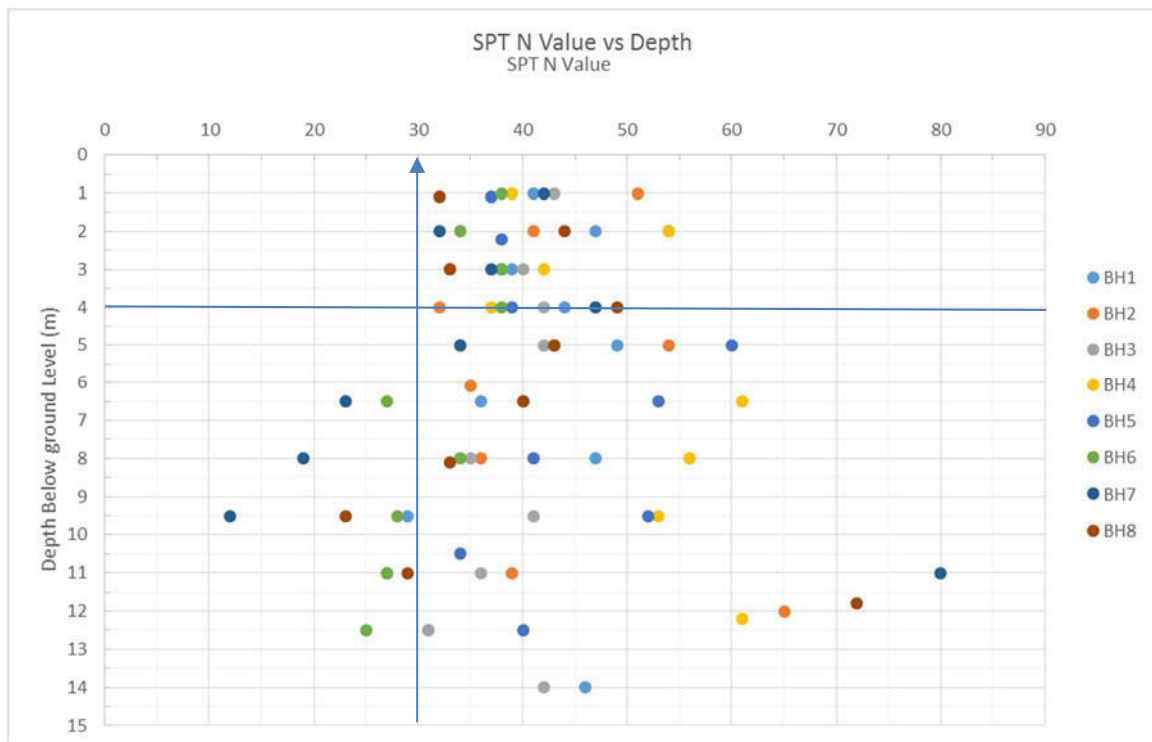


Figure 1 – Graph showing SPT N with depth

The investigation undertaken by RSA Geotechnics was undertaken prior to the dynamic compaction works on site. It is therefore, likely that the variation in density is due to the method of fill placement outlined in the Babbie Compaction Certification Report, which indicates that the upper fill was subject to a more rigorous compaction specification than the deeper fills.

Because of the variability of the lower fills a moderately conservative characteristic design SPT N value of 30 has been selected to determine the strength of the made ground.

Multiplying the SPT N value of 30 by 4.5 after Stroud (Stroud, 1975), an undrained shear strength of 135 kPa can be assumed for the fill material.

6 ANTICIPATED DESIGN ISSUES

At the location of the proposed extension, the site is underlain by re-compacted / engineered opencast backfill which generally varies in thickness between 22 m and 26 m.

As identified in the previous extension phase and reported in 01975AA_L_001A_DS enclosed as Appendix B, the fill on site is likely to be undergoing settlement due to creep because of its composition and compacted state. Following guidance set out in “Specifications for and performance of compacted opencast backfills” (Hodgetts et al), a prediction of the likely creep settlement of the fill since its placement can be estimated and a rate of future settlement can be assessed. At the time of writing this report 15 years have passed since the placement of the opencast backfill.

The guidance indicates that the creep settlement of a backfilled opencast site can vary from 0.1% to 0.2% of the fill thickness per log cycle of time for an engineered fill, to 0.65% of the fill thickness per log cycle of time for an uncompact fill. Assuming an optimistic value of 0.2% creep settlement for an engineered fill, between 28 mm (where the fill is 22 m thick) and 35mm (where the fill is 26 m thick) of creep settlement has been assessed at the location of the extension over a typical design life of 60 years irrespective of imposed load. This creep settlement will also be occurring below the existing connecting structures on site.

The proposed office extends 12 m beyond the eastern edge of the original warehouse. As dynamic compaction was used for the original warehouse with the dynamic compaction extending 5 m beyond the footprint of the original warehouse, the proposed office extension is situated on both un-compacted fill and fill that has been compacted to 6 m depth. Therefore, there is a risk of differential settlement occurring between the foundations between the two areas of the office extension.

7 FOUNDATION SELECTION

On the previous extensions to the existing warehouse High Energy Impact Compaction (HEIC) was used to densify material in the extension footprint, reducing differential settlement. However, for HEIC to be effective the area to be compacted needs to have a minimum area to allow the equipment to impart sufficient energy to the ground. In the case of this extension the area to be compacted is only 7 m wide by 32 m long. Therefore, HEIC would not be effective at this location and an alternative foundation solution is required.

7.1 Ground Improvement and Shallow Foundations

Ground improvement could be adopted on site. However, because works would be undertaken within 5 m of existing structures the vibrations from the ground improvement works could be potentially damaging to the existing building.

As discussed above, HEIC is not feasible on such a small area and it is inadvisable to carry out dynamic compaction within 30 m of an existing building.

A possible option for improvement would be to carry out Rapid Impact Compaction (RIC) of the site surface. RIC densifies shallow soils, using a hydraulic hammer typically of 7 to 10 tonnes which repeatedly strikes an impact plate. The energy is transferred to the underlying soils and rearranges the particles into a denser configuration.

This compaction method can be used in close proximity to buildings (literature suggests as close as 6 m). However, this still present a vibration risk to the existing structures and monitoring / vibration cut-off trenches would be required during the works to monitor and reduce vibrations.

The RIC is likely to only be effective in improving the upper 3m of the fill. Therefore, while improvement reduces the risk of differential movements, there is still likely to be some differential movement across the building footprint between the HEIC and RIC areas.

From conversations with the Knight Webb vibrations would be potentially disruptive to the operation of the warehouse and have therefore been discounted.

7.2 Piled Foundations

As an alternative to ground improvement, a floating pile solution could be adopted. The floating pile option would comprise the installation of piles to support the building which are founded wholly in the opencast backfill.

The piles would be designed to extend below the improved zone and found on the soils below which have not been improved. By bypassing this improved zone the risk of differential movement due to the variation in ground improvement method is negligible.

Because it is a piled solution the settlement of the foundations due to imposed load is typically less than 10 mm. This would be relatively low in comparison with the settlement caused by creep in the fill. Creep will affect both the piled foundations and the existing structures on site causing settlements of similar magnitude for both. Therefore, the differential settlement between the existing and proposed structures is expected to be from the imposed loading and would be approximately 10 mm

There is, however, a residual risk that differential settlements may occur between piles due to the variability of the density of the fill in this lower un-improved zone.

To ensure that the correct toe depth has been achieved and to minimise vibration to the adjacent buildings the piles would either need to be bored piles or CFA. Bored piles are likely to require casing to prevent collapse of the open bore during installation. Therefore we expect the preferred option to save programme would be to install piles using CFA.

8 PILE DESIGN

8.1 Negative Skin Friction and Pile Movement

As creep occurs in the fill on site, an additional load will be imposed onto the pile (negative skin friction) from the surrounding soil as it settles around the pile dragging it down. In conventional pile design, the piles would be designed to resist the additional load from negative skin friction due to the creeping fill which would require the piles to be sufficiently long to provide a suitable founding base. Additionally in conventional pile design for negative skin friction, as the pile is founded on a stable base, the soil settles around the pile giving large relative movements between the pile and the ground.

As the proposed extension is to be connected to an existing building which is already subject to creep, if the negative skin friction were resisted by the piles designed using conventional methods, there would be large differential settlements between the existing building (which is allowed to settle) and the proposed extension (which would not settle). In order to prevent the differential settlement, the founding

piles have been designed as “floating piles” so that they settle with the fill whilst providing adequate bearing capacity for the proposed structure.

The creep settlement will occur relatively slowly over the lifetime of the building and will continue for the whole design life. Therefore, the design of “floating piles” has to consider the long term load and settlement characteristics of the piles as follows.

In the initial condition when the piles are first loaded they will settle under imposed load (Refer to Figure 2 (a)). The initial loading of the pile has been taken as a datum for consideration of settlement of the fill. Any fill settlement that has occurred before this time is irrelevant. Consequently, there will be no negative skin friction on the pile and the pile will be subject to full positive skin friction. In this case the floating pile needs to be designed for ultimate limit state conditions with appropriate factors applied to the design (Eurocode 7).

Over time, following installation and loading, the fill will settle around the pile imposing negative skin friction forces.

The magnitude of this negative skin friction is dependent the location of the neutral point. The neutral point is defined as the point down the pile at which the pile settlement is the same as the settlement of the surrounding soil. Above the neutral point the pile is subjected to negative skin friction and below this point the pile is subject to positive skin friction.

As the fill begins to settle around the pile, the neutral point moves down the pile as more load is applied, increasing negative skin friction and decreasing positive skin friction. This decreases the factor of safety of the pile until the working load plus negative skin friction equals the positive skin friction below the neutral point and the end bearing capacity of the pile (giving a factor of safety of unity). At this point the pile movement for serviceability limit state conditions can be calculated. This can be seen in Figures 2 (b) and 2 (C). At this point the ultimate limit state condition will still be satisfied as any additional movement of the pile will reverse the negative skin friction, temporarily increasing the factor of safety as the pile settles relative to the soil.

To design the “floating pile”, a design depth for the neutral point of the pile was fixed at the base of the improved soils. By calculating and fixing the neutral point of the pile at a defined depth we can calculate the length of pile required for a safety factor of 1 for serviceability limit state conditions at full design life. By plotting the settlement profile of the fill and the pile on a chart assuming a design life and the neutral point fixed we can calculate the maximum differential settlement between the pile and the fill for the building design life (Refer to Figure 2 (c)). This is the serviceability limit state condition.

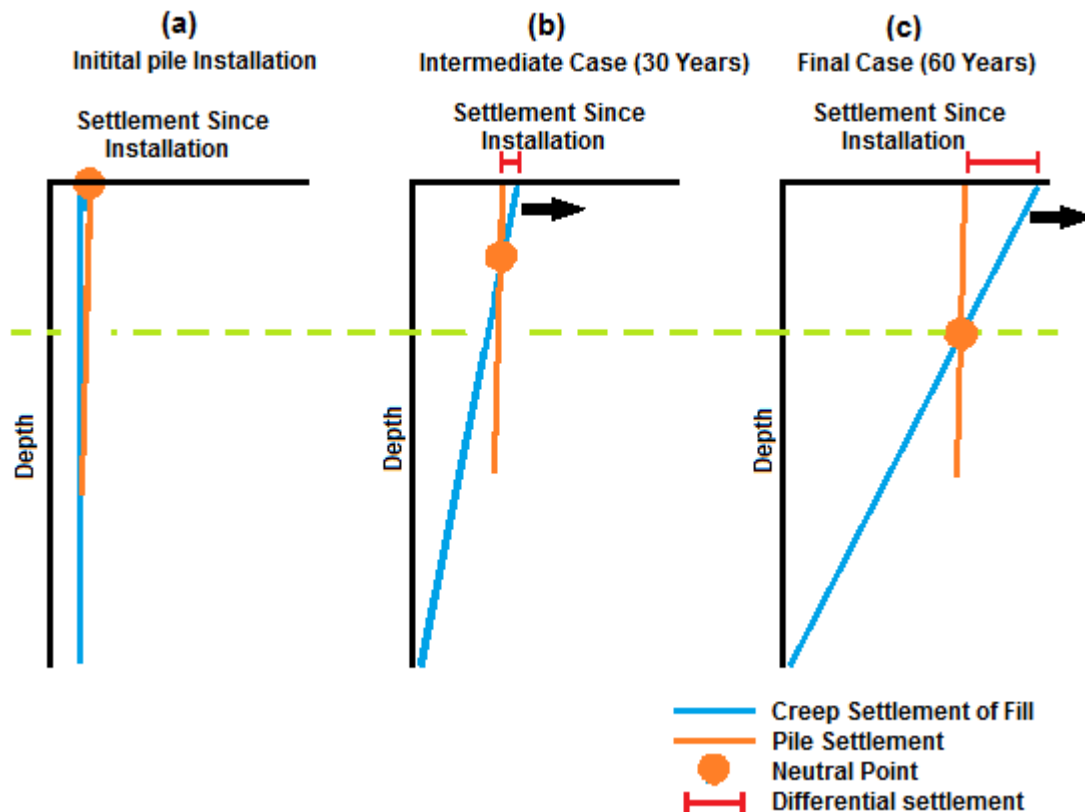


Figure 2 – Conceptual Design

8.2 Calculation of capacity

In order to determine the magnitude of negative skin friction on the pile a load compatibility assessment has been carried out using unfactored loads and assuming a neutral point at 6m which coincides with the depth of ground improvement undertaken on site.

The neutral point is the point on the pile at which the working load and down drag forces on the pile are equal to the pile resistance. Below this point the pile settles relative to the soil and positive skin friction is generated on the pile. The length of the pile was then calculated assuming unfactored loads and resistances with negative skin friction applied.

The pile length from the above calculation is then checked for ultimate limit state conditions (ULS). For the ULS case it is assumed that the “floating pile” will have full positive skin friction as the for the pile to fail under ULS conditions the applied load must be greater than resistance of the pile using full positive skin friction and end bearing.

The ULS condition was checked using a partial factored approach in accordance with the requirements of BS EN1997-1:2004 Eurocode 7 (British Standards Institute, 2010).

The Partial factors adopted for design are summarised in Table 1

Table 1 – Partial Factors for design of piles.

	Permanent Actions γ_G	Variable actions γ_Q	Base Resistance γ_b	Shaft Resistance γ_s
Combination 1	1.35	1.5	1	1
Combination 2	1	1.3	1.5	1.3
1. A model factor of 1.4 should be applied to both base resistance and skin friction in addition to the factors above. 2. The partial factor on material properties is 1. 3. Combination 1 partial factors selected assuming bored / CFA piles with at least 1% of piles tested to 1.5 x characteristic load.				

An initial assessment was carried out using a variety of pile diameters and grouping to optimise the pile size for the load compatibility assessment. Based on the results of this preliminary assessment, a representative design pile diameter of 450 mm has been chosen with an associated unfactored working load of 430 kN per pile based on a given number of piles supporting each column.

To minimise differential settlement between the dynamically compacted area and the non-compacted area of the proposed office extension, a neutral point of the pile at 6 m depth has been selected. This has been chosen so that settlement of the piled foundation will be the same for both the compacted area and non-compacted area at this depth. The pile length was designed such that the neutral point in terms of load compatibility was achieved at 6 m depth as shown in Figure 3 below. Figure 3 presents the applied working load plus negative skin friction at a given depth below the top of pile against the available pile capacity at a given depth for a specified pile length. In this case, to have a neutral point at 6 m depth, the overall pile length needs to be 11.5 m.

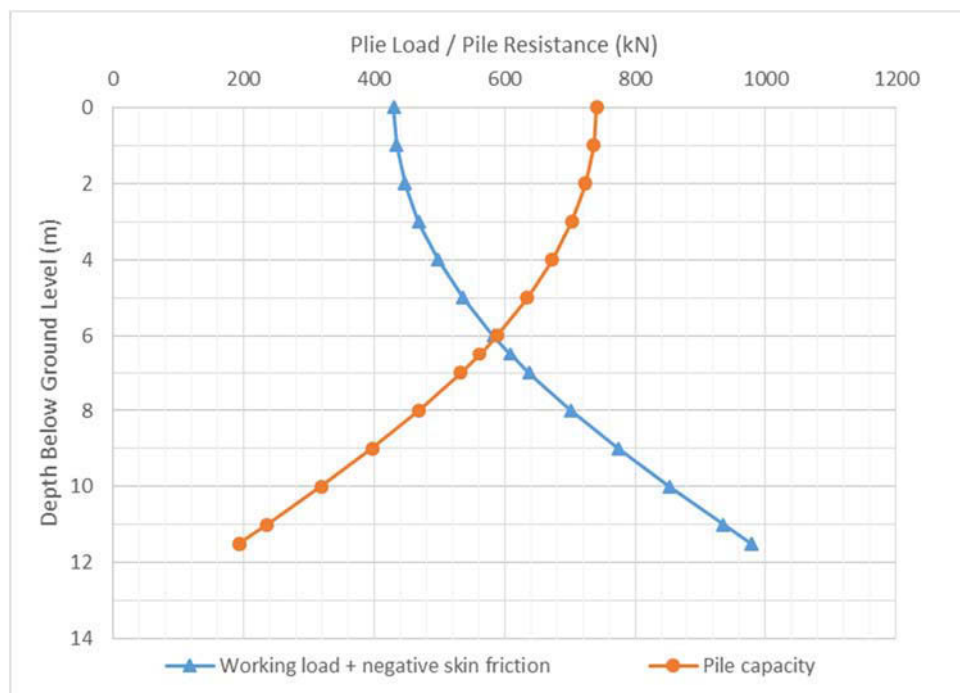


Figure 3 – Load compatibility assessment for pile

The overall pile length has then been checked against ULS conditions assuming full positive skin friction along the full length of the pile. The results of this check indicate that the pile has an adequate factor of safety for ULS conditions.

8.3 Settlement Assessment

The settlements of the piles were calculated using a strain compatibility assessment comparing creep settlement of the site to settlement of the pile due to imposed load and creep at the pile toe.

In order to carry out the assessment, the creep settlement of the fill was calculated at different depths in the fill and plotted onto a graph.

As a neutral point of the point of 6 m depth has been designed using load compatibility of the pile, at this point the soil would be settling at the same rate as the pile. Therefore, the calculated creep settlement of the fill below 6m was added to the pile settlement to determine the total settlement of the pile.

This process was followed for each of the pile groups based on the column loads for the proposed building and plotted against creep settlement on Figure 4.

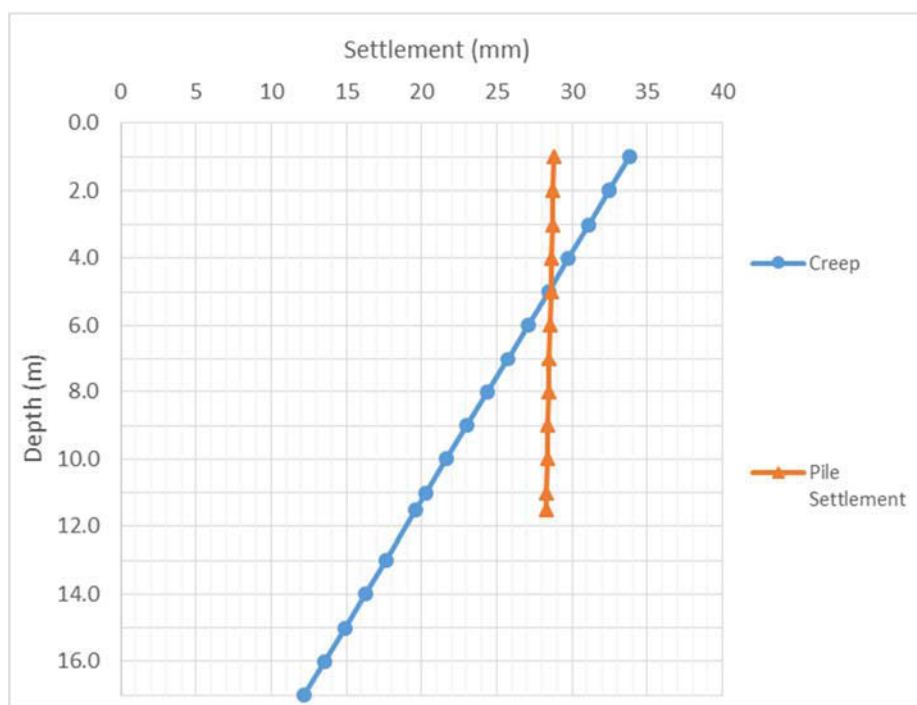


Figure 4 – Strain compatibility assessment results

9 RESULTS

The results of the assessment indicate that if a pile design is adopted for the site, then 450 mm bored or CFA piles will be required to a depth of 11.5 m. The pile settlement following construction will be less than 3mm under imposed loading. However, taking into account the creep settlement of the fill, the pile settlement over the assumed 60-year design life would be approximately 30 mm.

It is anticipated that as the original warehouse has been in place for ten years, the settlement from imposed load would be complete. Assuming that the warehouse is subject to a similar creep settlement as the office extension the anticipated creep settlement of the warehouse over its remaining design life will be approximately 35 mm. Therefore, the maximum calculated differential settlement between the existing building and the proposed extension would be 5 to 10 mm over the 60 year design life.

10 CONCLUSION

The proposed foundation design has been undertaken to avoid variation in density in the opencast backfilled material near the site surface to reduce differential settlement.

The piled foundation has been designed using “floating piles” within the fill to supporting the proposed extension while allowing the extension to settle at a similar rate to the existing building minimising differential settlement.

The pile requirements for each of the column loads supplied by RPS are summarised in Table 2.

Table 2 – Summary of Pile requirements.

Pile Group	Dead Load (kN)	Live load (kN)	Diameter (m)	Length (m)	Number of Piles in group	Working Load Per Pile. (kN)
B-12/13	535	650	0.45	11.5	3.0	395
A - 12/13/14	350	325	0.45	11.5	2.0	338
C-12	350	325	0.45	11.5	2.0	338
A-15	200	150	0.45	11.5	1.0	350
B-15	325	350	0.45	11.5	2.0	338
C-13A/14A	350	450	0.45	11.5	2.0	400
A-11A	220	210	0.45	11.5	1.0	430
C-11A	290	275	0.45	11.5	2.0	283
B-11A	370	450	0.45	11.5	2.0	410

Adopting these piled foundations the maximum anticipated settlement from imposed load is 3 mm and the differential settlement between the existing warehouse and proposed extension has been calculated to be 6 mm over the 60 year building design life.

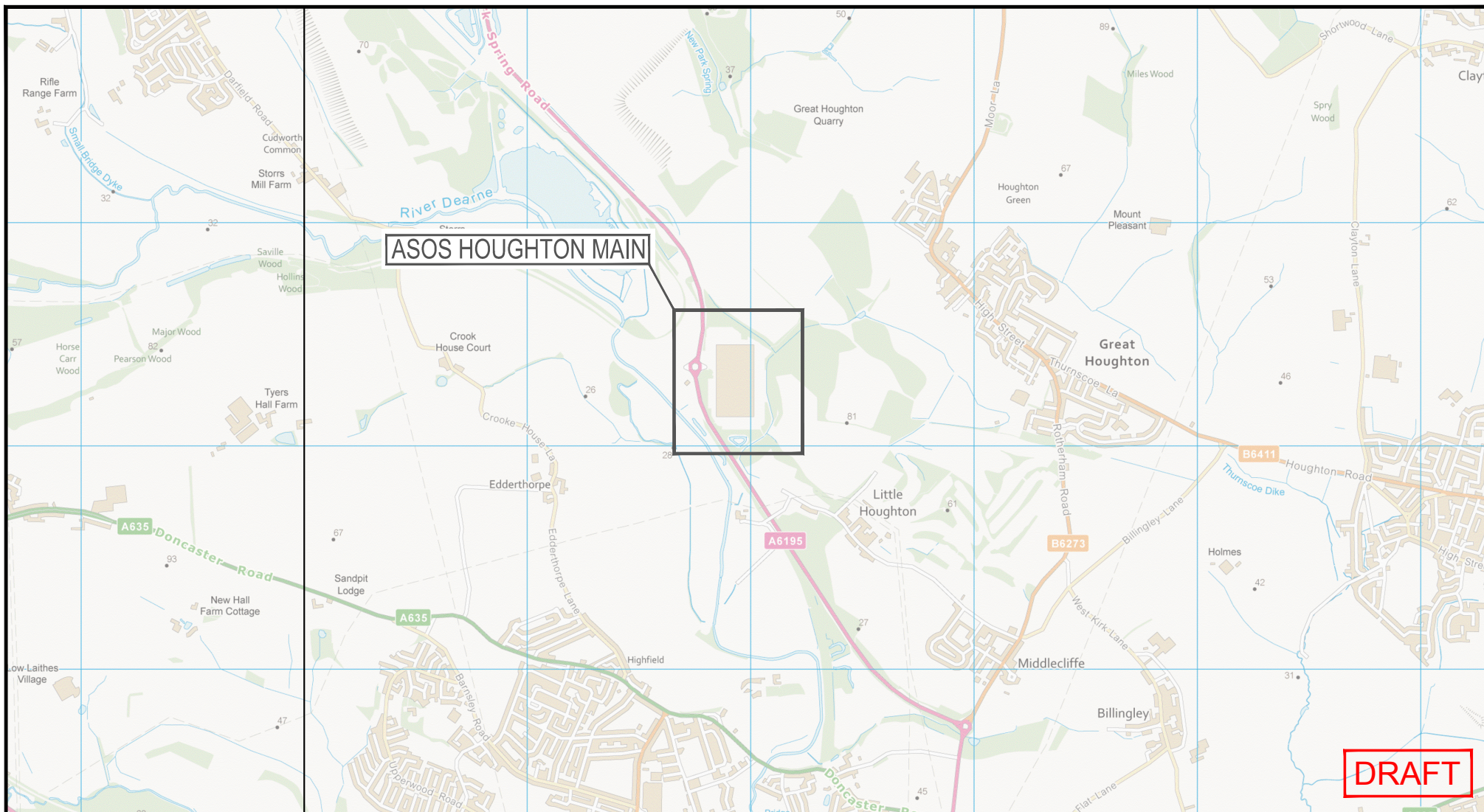
11 GEOTECHNICAL RISK REGISTER

The following register outlines all the anticipated geotechnical risk to the proposed development and where appropriate provides mitigation measures and further action required to reduce the risk.

No	Date	Description of Hazard and Risk Exposure	Mitigation of Risk (Potential or Achieved)	Further Action
1	08/02/17	Unforeseen ground conditions, variation in properties of the opencast fill. Leading to under-design of foundation resulting in foundation failure and building damage.	Ground conditions have been confirmed by investigation. Moderately conservative parameters for design have been selected based on the results of the investigation and past experience of similar materials. Designs have been carried out in accordance with the requirements of BS EN 1997 (Eurocode 7). 1% of piles are to be tested to verify the foundation design.	Implementation by contractor
2	08/02/17	Unforeseen ground conditions, variation in soil properties. Leading to underestimation of magnitude of creep settlement leading to unacceptable damage to structures.	Foundation has been designed such that movements of the extension due to creep, match movements of the existing building due to creep. Consequently, the magnitude of creep does not present an issue to the design providing that differential settlement is minimised.	None
3	08/02/17	Instability of excavations in opencast fill - Collapse of excavations causing injury or death	Excavations in the opencast backfill which require man entry should be appropriately supported, shored or battered to a safe angle to prevent collapse. Personnel, plant and equipment shall	Implementation by contractor

No	Date	Description of Hazard and Risk Exposure	Mitigation of Risk (Potential or Achieved)	Further Action
			not be permitted at the long side of excavations unless appropriate safety measures against collapse are in place.	
4	08/02/17	Vibration from piling operation causing disruption to operations, causing damage to structures or nuisance to personnel	Piles to be installed using low vibration bored or CFA techniques.	Design and construct CFA or bored piles for building.
5	08/02/17	Instability of pile bores in opencast fill - Collapse of bore causing injury to personnel in close proximity to bore.	Piles are to be installed using open boring or CFA techniques. Where open boring is to be undertaken, holes shall be cased to prevent collapse of the bore.	To be implemented by contractor
6	08/02/17	Damage to buried services during the works. Injury to personnel, repair costs, effects on programme	Piles to be positioned to avoid services. Prior to works the contractors should locate all services in the building footprint and protect or relocate during the works.	To be implemented by contractor
7	08/02/17	Possibility of personnel encountering contaminated soils during the site works leading to health issues	Where possible an assessment of contamination in the soils should be undertaken to determine risk to personnel. Good hygiene practices should be adopted such as wearing gloves to handle soils and appropriate use of hand washing facilities.	To be implemented by contractor

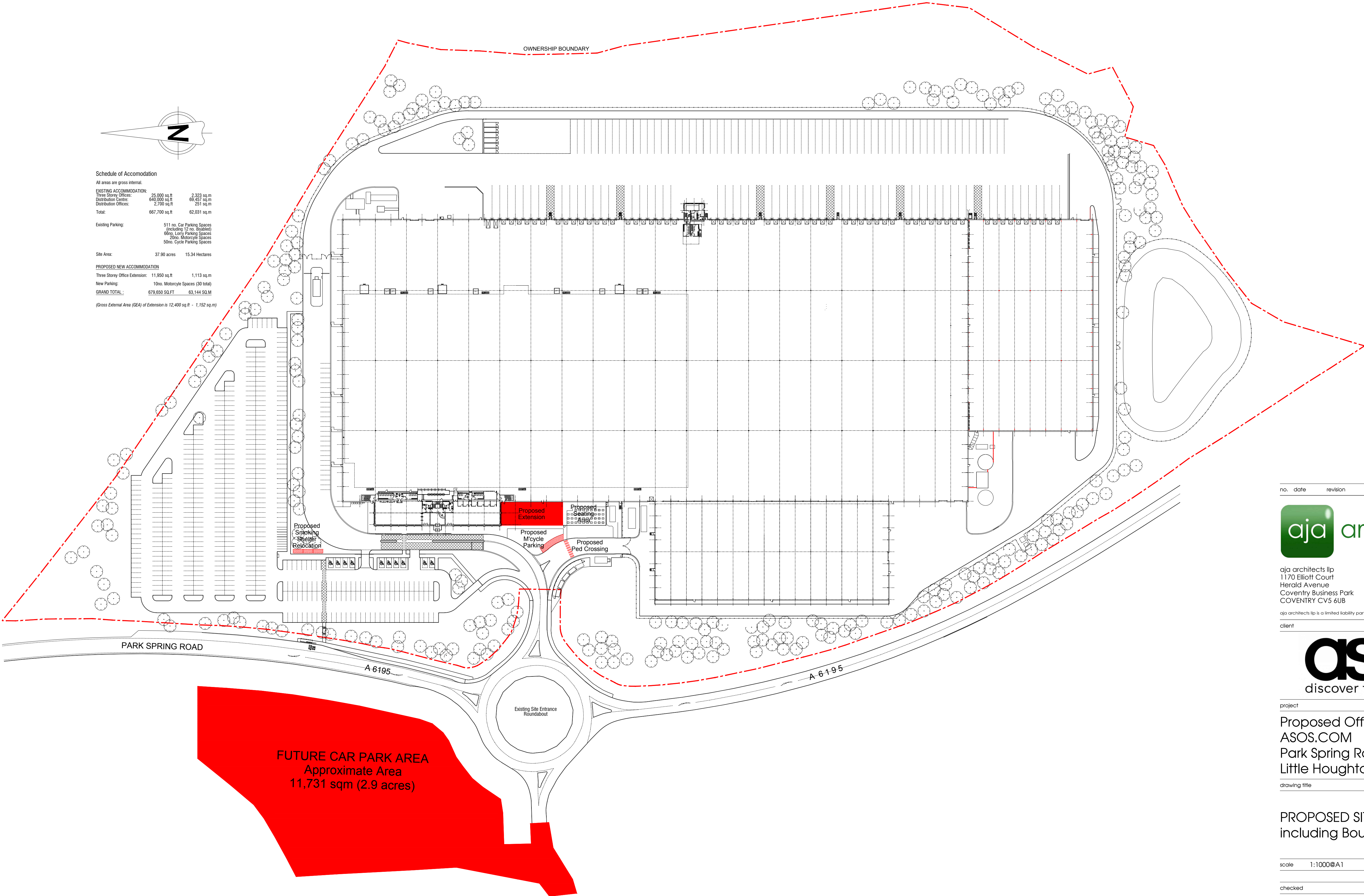
Drawings



	drawn	MLH	 A TETRA TECH COMPANY	client: BUCKINGHAM GROUP	
	date	07/02/2017		project: HOUGHTON MAIN OFFICE EXTENSION	
	scale	NTS		title: LOCATION PLAN	
	original size	A4		project no: 1975AB	fig no: 1975AB_D_001

IF IN DOUBT PLEASE ASK!

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no.	date	revision	by
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aja architects llp
1170 Elliott Court
Herald Avenue
Coventry Business Park
COVENTRY CV5 6UB
T: 024 7625 3200
F: 024 7625 3210
E: aja@aja-architects.com
W: www.aja-architects.com

aja architects llp is a limited liability partnership registered in England No. OC326721

client



project

Proposed Office Extension
ASOS.COM
Park Spring Road
Little Houghton, Yorkshire

drawing title

PROPOSED SITE PLAN
including Boundary Fence

scale 1:1000@A1 drawn RPW

checked date 15.12.16

drawing no

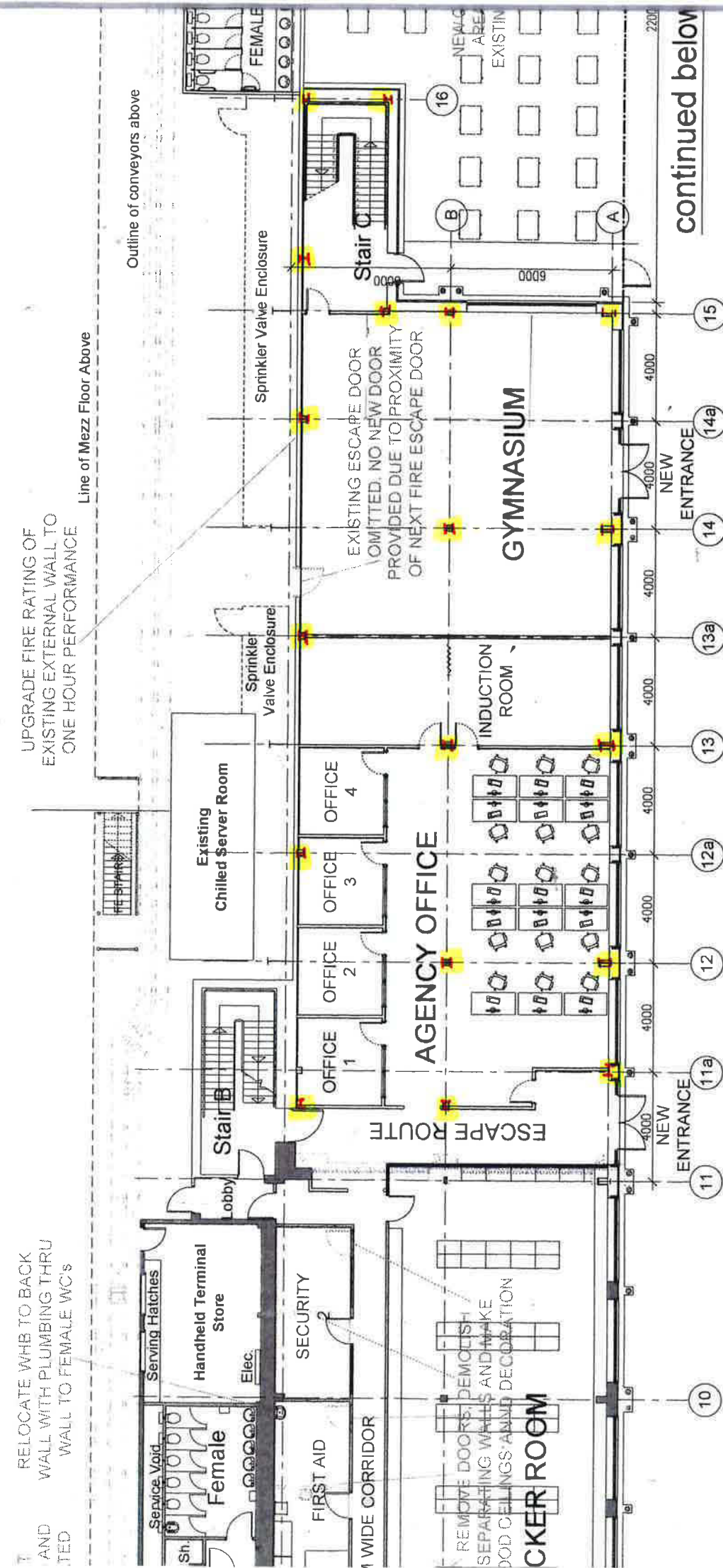
6010 - 025

Appendix A

RPS Column Loads

Job No: NRO18527	Project Ref: ASUS OFFICE EXTENSION	Drawing Ref:
Subject: PRELIMINARY COLUMN LOADS (SLS) (EXCL. FOUND. SWT.)		
By: JF	Date: 26/1/17	Checked by: Date:

Ref	Calculations		Output
	GRND	DEAD (KW)	IMPOSED (KW)
	B-12/13,	535	650
	A-12/13/14	350	325
	C-12a	350	325
	A-15	200	150
	B-15	325	350
	C-13a/14a	350	450
	A-11a	220	210
	C-11a	290	275
	B-11a	370	450



Appendix B

Previous Project Reports 01975AA_L_001A_DS and 01975AA_R_001A_DS

Our Ref: 01975AA_L_001A_DS_Ground Improvement (A)
Your Ref:

Date 25th January 2013

Buckingham Group Contracting Limited
Silverstone Road
Stowe
Buckingham
MK18 5LJ

For the attention of: Rob Miller

Dear Sir

RE: Houghton Main – Ground Improvement

Further to our meeting on 10th January 2013 we write to you in reference to the proposals for ground improvement at the proposed extensions to the distribution warehouse on the Houghton Main site near Great Houghton, Barnsley.

This letter provides a brief summary of the pertinent information relating to the onsite conditions, outlines the issues envisaged as part of the construction works, and provides a selection of potential options for the backfill below the proposed development.

1 LIMITATIONS

The findings of this report are based on information received from third parties. While it is believed that the information from these sources is correct Coffey accepts no responsibility for any omission or inaccuracy within this information.

In preparation of this document we have reviewed the following information;

- “RSA Geotechnics Ltd – *Ground Investigation Report*” (Ref – 9735), RSA, Dated August 2005
- “RSA Geotechnics Ltd – *Interpretive Geotechnical Report*” (Ref – 13378GI), RSA, Dated December 2012
- “RPS Civil and Structural Requirements *ASOS Warehouse Extension, Barnsley*” (Ref – NK014833) November 2012.
- “RJB Mining (UK) Ltd Correspondence – *Houghton Main Site – Treatment to shafts*” (Ref – JNA/HS), Dated 18th September 2000.
- “The Coal Authority – Coal Mining Report - *Houghton Main Colliery Shafts, Darfield, South Yorkshire*” (Ref – 00007712-06) Dated 21st March 2006.
- “The Coal Authority – Non Residential Coal Mining Report - *Site at Houghton Main Barnsley*” (Ref – 710000290861002) Dated 30th October 2012.

Coffey Geotechnics Limited

9/11 Hornbeam Square South, Hornbeam Park, Harrogate, North Yorkshire HG2 8NB United Kingdom
T (+44) (0) 1423 878 130 F (+44) (0) 1423 878 131 coffey.com

Registered Office: 1 Northfield Road Reading Berkshire RG1 8AH United Kingdom
Registered in England No. 06328315

2 PROJECT BACKGROUND

The existing warehouse is constructed on a former UK Coal owned site which had been subject to opencast mining.

The records reviewed indicate that the opencast site was excavated and backfilled between 1997 and 2001. The approximate thickness of fill generally varies between 10m at the south west side of the site to 25m at the north east side of the site. A 5m high highwall is present in the south west corner of the site where the fill thickness changes from 10m thick to 5m thick.

The specification for reinstatement of then former opencast site required the backfill to achieve a safe bearing capacity of 110kPa or better for a 2m by 2m foundation, without producing more than 20mm of settlement.

The existing distribution warehouse was constructed in 2006. Dynamic compaction was carried out prior to construction within the proposed building footprint, which extended to 5m beyond the proposed build line. The ground improvement was designed to compact the top 6m of fill in order to reduce settlements from the building load.

2.1 Proposed Scheme

The scheme currently proposed, comprises the construction of extensions to the west and south side of the existing warehouse building. Each extension is approximately 0.6ha in area.

Information contained in RSA Geotechnics Ltd – *Interpretive Geotechnical Report* indicates the approximate depth of fill at the open cast site varies between 10m to 16m at the location of the proposed southern extension and 13m to 21m bgl at the location of the proposed western extension.

The buried highwall for the opencast site crosses the proposed southern extension at its south western corner. The highwall is approximately 5m high, changing the fill thickness from greater than 10m thick for the majority of this extension to 5m thick in its southwest corner.

RPS initial civil and structural requirements were that the proposed footprint of the extensions are to be ground improved by dynamic compaction to achieve an allowable foundation pressure of 150kPa with gross and differential settlements of 25mm and 10mm respectively.

Without ground improvement, based on plasticity index of 20% and a moderately conservative SPT N value of 20 (recorded in RSA Geotechnics Ltd – Ground Investigation Report” Ref – 9735) a coefficient of volume compressibility (Mv) of 0.09m²/MN can be adopted for the backfill

Settlement on this site is likely to be a function of settlement by creep and settlement by imposed load. Without ground improvement a 2m x 2m shallow foundation for the proposed extension carrying an imposed load of 150kPa could settle as a result of imposed load alone by as much as 14mm, whilst the settlement of the floor slab would be a maximum of the order of 50mm.

The fill on site is also likely to be undergoing settlement due to creep because of its composition and compacted state. Following guidance set out in “Specifications for and performance of compacted opencast backfills” (Hodgetts et al), a prediction of the likely creep settlement of the fill since its placement can be estimated and a rate of future settlement can be predicted. At the time of writing this report 12 years have passed since the placement of the opencast backfill.

This guidance indicates that the creep settlement of an backfilled opencast site can vary from 0.1% to 0.2% of the fill thickness per log cycle of time for an engineered fill, to 0.65% of the fill thickness per log cycle of time for an uncompact fill.

Assuming an optimistic 0.2% creep settlement for an engineered fill, a minimum 24mm of creep settlement is possible in the 20m thick fill and up to 12mm of creep settlement is possible in the 10m thick fill over the design life of 60 years. This would represent a total differential settlement of 12mm (minimum) by creep alone across the whole site (inclusive of below the existing building) with no settlement allowance for imposed load.

With respect to the location of the quarry highwall at the southern extension, the differential creep settlement between the 5m thick fill at the top of the wall and the 10m thick fill at the bottom of the wall alone is expected to be 6mm.

Across the highwall the differential settlement is likely to be more pronounced, in comparison with settlements across the rest of the building, due to the short distance over which it takes place.

Ground improvement by dynamic compaction was initially recommended as a means of limiting the settlement of the fill below the proposed structure. However, dynamic compaction is unlikely to be possible in close proximity to the existing building and services due to potential of damage through vibration. Therefore, the current proposal is to use vibro compaction in place of dynamic compaction to achieve a similar level of compaction while minimising the level of vibration in close proximity to the existing building.

The RPS specification requires the floor slab for the extensions to be continuous from the existing warehouse and to be in accordance with Concrete Society Technical Report 34. Steps in the floor due to differential settlement are not permitted.

From existing mining records a former mineshaft is present in the footprint of the proposed southern warehouse. The shaft is recorded as being 752m deep and 6.1m in diameter.

Based on the information received to date, two phases of stabilisation of the shaft have been carried out. The stabilisation works comprised;

- Filling of the shaft in an operation which included construction of a plug between 228.67m and 234.77m. Carried out in 1994
- Capping the shaft at rockhead with a 10m diameter, 150mm thick reinforced concrete cap. The cap was extended 500mm beyond the extent of the outer casing of the shaft. The shaft was capped in 2000 by RJB Mining.

The requirement for treating the mineshaft below the southern extension, if any, has not been confirmed. The preliminary proposal was to grout the shaft. However, it is understood that this issue is in abeyance pending discussion with the relevant authorities.

3 ANTICIPATED DESIGN ISSUES

3.1 Ground Improvement

The site is underlain by recompacted / engineered opencast backfill which generally varies in thickness between 10m and 20m. At the location of the quarry highwall the 10m thickness of fill reduces to 5m over a short distance in comparison with the rest of the site.

It is our opinion that the assessment above which represents a best case for creep settlement along with settlements induced by imposed loading makes it impossible to comply with the overall foundation settlement requirements of the specification. It is also unlikely that the specified settlement requirements for the floor can be met by ground improvement. This concurs with a similar assessment carried out by RSA in their Interpretative Geotechnical Report (December 2012).

Previously dynamic compaction was carried out within the existing building footprint to 5m beyond the building extent which compacted the top 6m of the fill. The specification requires that improvement is carried out within the footprint of the proposed extensions in order to gain a similar level of compaction as with the existing building. Vibro compaction was proposed as an alternative to dynamic compaction as it causes less vibration. However, due to the stiffness of the engineered backfill pre drilling of the vibro locations will be required to ensure suitable penetration of the vibro flots to the required depth of treatment.

3.2 Shaft Stabilisation

As detailed in section 2.1 the shaft on site has been filled plugged and capped to prevent shaft collapse preventing surface features. The current proposal is to grout the shaft to eliminate the possibility of material collapse.

The depth to which the shaft is to be grouted is still to be decided and it is understood at the time of writing this report that discussions are underway with UK Coal to determine what course of action to take for this issue.

To carry out the grouting, the shaft cap would need to be located by probing with drilling equipment and penetrated. The drill would need to continue drilling to an agreed treatment depth. Grout would then need to be injected into the resulting hole to ensure all voids in the fill are closed. As the shaft is already filled, this operation would need to be repeated at a number of locations within the shaft to ensure that the fill is fully stabilised.

4 GROUND IMPROVEMENT OPTIONS

It should be borne in mind that it is unlikely that shallow ground improvement by any method will significantly reduce total and differential settlements of the fill below the proposed extension caused by creep. Settlement due to creep is expected to vary between 24mm and 12mm.

As stated in section 2.1, without ground improvement the likely imposed load settlements have been estimated as 14mm for foundations and up to 50mm for the floor.

RSA Geotechnics Ltd – Interpretive Geotechnical Report (December 2012) adopts a minimum post dynamic compaction young's modulus of 22.5 MN/m^2 for the fill which correlates to a coefficient of volume compressibility (M_v) of $0.04 \text{ m}^2/\text{MN}$. Based on 6m of improvement RSA state and we concur that the settlement below foundations would be of the order of 7mm and the settlement below the floor would be a maximum of 30mm from imposed load only.

If the stiffness of the fill is increased to 22.5 MN/m² to a depth of 3m depth using top down compaction we anticipate settlements of 8mm for foundations and 40mm for the floor from imposed load.

The table below shows a comparison of the estimated combined imposed and creep settlements for each of the treatment depths.

Improvement option	Foundation settlement (imposed)	Floor settlement (imposed)	Creep Settlement	Foundation settlement (imposed & creep)	Floor settlement (imposed & creep)
No Improvement	14mm	50mm	20mm	34mm	70mm
Landpac / RIC (3m improvement)	8mm	40mm	20mm	28mm	60mm
Dynamic Compaction / Vibro (6m improvement)	7mm	30mm	20mm	27mm	50mm

The results show that there is less than 10mm improvement in the total floor and foundation settlements between each depth of treatment. In addition to this, the results show that in all cases the specified settlement criteria are exceeded.

While ground improvement may not be effective in reducing settlement to an acceptable level, the shallow ground improvement will help to reduce the potential for shallow localised differential settlement beneath the proposed foundations and floor slab by creating a stiff surface crust of fill material.

The degree of compaction obtained from ground improvement is dictated largely by the energy input to the ground. The higher the energy input the higher the degree of improvement. However, the energy input to the ground causes vibration and the higher the energy input, the higher the vibration.

The ground improvement for the site will be required up to within 5m of the existing building. With this in mind the selection of the appropriate ground improvement to adopt will need to be considered on the basis of what will provide the best compaction as close as possible to the existing building without affecting it.

4.1 Vibro compaction

Vibro compaction of the top 6m of the fill using concrete or stone fills is possible to within 10m of the existing structure.

However, investigation carried out in the fill showed the material to have a typical SPT N value of 20 which is indicative of a firm to stiff soil.

As the fill material already exhibits strength properties which are not indicative of a material requiring improvement, it is unclear what measureable improvement can be achieved by this method. It is also unclear whether such a small improvement is worthwhile given that each vibro location will need to be pre drilled as the vibro flot will not penetrate the stiff ground.

If a stone fill is used for the vibro compaction (vibro stone column) this would add an additional risk to the fill from inundation settlement as the stone column would provide a pathway for water to significantly penetrate the fill and increase the speed of the creep of the fill. In addition, as the method differs significantly from the method used on the existing building, the new extensions may experience a different settlement pattern from the main warehouse.

4.2 Top down compaction

Alternative top down ground improvement methods to dynamic compaction are available including RIC (Rapid Impact Compaction) and Landpac. The vibration associated with these methods is not as severe as for dynamic compaction but mitigation measures / monitoring for vibration may be necessary where they are intended for use in close proximity to buildings. Using these methods may be possible to carry out work within 5 to 10m from the existing structure.

Using an alternative top down method it may be possible to improve the top 3m of the fill to provide a surface compacted crust of material which is suitably dense to prevent shallow localised differential settlement of the extension. This is a similar but lower energy approach to dynamic compaction.

4.3 Do nothing

As discussed in section 2, the combination of imposed load from the building and creep settlement will exceed the allowable total and differential settlements as outlined in the specification.

The imposed load from the floor will affect a depth of fill significantly deeper than any of the improvement techniques outlined in this above can reach, due to its large area. Therefore improving the surface 3m or even 6m of fill will make little difference to the overall settlement by imposed load or creep.

5 SHAFT STABILISATION OPTIONS

The shaft has already undergone three phases of stabilisation. Grouting of the shaft represents a final solution to the stabilisation of the shaft. Therefore, there are no alternative options to grouting except to not to grout

Ground level lies approximately 13m above the level of the top of the shaft. As the shaft has been filled, collapse of the side walls of the shaft is unlikely to result in loss of volumes of material into the shaft large enough to be detected at surface.

Taking into account that the shaft is also capped with a reinforced concrete cap, it is expected that should a localised collapse occur, the shaft cap will bridge any possible voids below the cap (where voiding is most likely to occur) reducing the potential for this material loss.

Taking into account all of the factors outlined above it is anticipated that there is a low risk of surface effects resulting from the partial collapse of the shaft

6 CONCLUSIONS

Each of the improvement and stabilisation methods described in this report has a varying level of associated risk and varying potential benefits which should be carefully considered. In terms of the impact that the shallow ground improvement will have on the structure of the underlying fill the improvement expected is not likely to prevent unacceptable settlements occurring below the building. It

is likely that the imposed load from the floor will affect a depth of fill significantly deeper than any of the improvement techniques possible for this development.

It is therefore unlikely that the settlement requirements of the current specification can be met. As the specification is unlikely to be achievable, a compromise should be made on the specification for the extensions which will provide a fit for purpose, safe working end product constructed to a lesser but achievable end specification.

Ground investigation undertaken at the site indicates that the fill is already well compacted. As this is the case there is little advantage in terms of prevention of settlement, to significant stabilisation of the fill.

We recommend that some shallow ground improvement using landpac and or rapid impact compaction (RIC) is carried out to ensure that the shallow fill to approximately 3m has a uniform compaction, reducing the potential for localised differential settlement directly below the building.

Yours sincerely
For and on behalf of Coffey Geotechnics Limited

David Stone
Design Engineer

7 REFERENCES

- “RSA Geotechnics Ltd – *Ground Investigation Report*” (Ref – 9735), RSA, Dated August 2005
- “RSA Geotechnics Ltd – *Interpretive Geotechnical Report*” (Ref – 13378GI), RSA, Dated December 2012
- “RPS Civil and Structural Requirements *ASOS Warehouse Extension, Barnsley*” (Ref – NK014833) November 2012.
- “RJB Mining (UK) Ltd Correspondence – *Houghton Main Site – Treatment to shafts*” (Ref – JNA/HS), Dated 18th September 2000.
- “The Coal Authority – Coal Mining Report - *Houghton Main Colliery Shafts, Darfield, South Yorkshire*” (Ref – 00007712-06) Dated 21st March 2006.
- “The Coal Authority – Non Residential Coal Mining Report - *Site at Houghton Main Barnsley*” (Ref – 710000290861002) Dated 30th October 2012.



01975AA HOUGHTON MAIN - INSITU COMPACTION COMPLETION REPORT

Buckingham Group Ltd

01975AA_R_001A_DS Validation Report
October 2013

Coffey Geotechnics Limited

9/11 Hornbeam Square South, Hornbeam Park, Harrogate, North Yorkshire HG2 8NB United Kingdom
T (+44) (0) 1423 878 130 F (+44) (0) 1423 878 131 coffey.com

Registered Office: 1 Northfield Road Reading Berkshire RG1 8AH United Kingdom
Registered in England No. 06328315

Document Verification Page 1 of 1					
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		Name			
		Signature			

Coffey Geotechnics Limited

9/11 Hornbeam Square South, Hornbeam Park, Harrogate, North Yorkshire HG2 8NB United Kingdom
 T (+44) (0) 1423 878 130 F (+44) (0) 1423 878 131 coffey.com

Registered Office: 1 Northfield Road Reading Berkshire RG1 8AH United Kingdom
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Figures

Figure 1: Extent of Treated Area

Figure 2: Test Location and Site Features Plan

Appendices

Appendix A: LANDPAC - BGCL ASOS Logistics Houghton HEIC-CIR-CIS ZLT Testing Report (Ref: LP1225)

1 INTRODUCTION

1.1 Site Description

The site is occupied by an existing operational warehouse near Great Horton in Barnsley. The existing warehouse is constructed on a former UK Coal owned site which had been subject to opencast mining. Records reviewed for the site indicates that the opencast site was excavated and backfilled between 1997 and 2001. The specification for reinstatement of the former opencast site required the backfill to achieve a safe bearing capacity of 110kPa or better for a 2m by 2m foundation, without producing more than 20mm of settlement.

Investigation of the site indicates the approximate thickness of fill generally varies between 10m at the south west side of the site to 25m at the north east side of the site. A 5m high highwall is present in the south west corner of the site where the fill thickness changes from 10m thick to 5m thick.

The existing distribution warehouse was constructed in 2006. To facilitate construction, dynamic compaction was carried out prior to construction within the building footprint, and extended to 5m beyond the build line. The ground improvement was designed to compact the top 6m of fill in order to reduce settlements from the building load.

1.2 Proposed Scheme

The proposed scheme comprises the construction of extensions to the west and south side of the existing warehouse building. Each extension is approximately 0.6ha in area.

In order to reduce the risk to the proposed development from differential settlement between the existing building and proposed extension, it was decided that High Energy Impact Compaction (HEIC) should be carried out below the proposed extension locations. The intention of the compaction works was to treat the top 3m of the proposed extension footprint to mimic the previous compaction works undertaken below the existing building within the constraints of the site. The extent of the compaction works is shown in the attached Figure 1.

1.3 Scope and object of the report.

This report presents a comparison between the in-situ compaction works undertaken on site and the requirements of the specification for in-situ compaction. In preparation of this document a review of the results of validation testing undertaken on site has been carried out with respect to the requirements of the specification.

Where the specified working method had to be modified to suit site conditions, or any departures have been made from the specification, this report provides reasons for the departures and any subsequent action undertaken.

1.4 Limits of the area covered by the report.

This report provides a review of information pertaining to the in-situ compaction works only and does not cover any additional up-filling works carried out to raise the general site level to facilitate construction.

2 SPECIFICATION REQUIREMENTS

2.1 Works Proposed

The works were to be carried out in accordance with Coffey Geotechnics "Specification for in-situ Compaction" Ref: COF_S_DS_001_PDF (Coffey Ref: 01975AA_S_DS001B), dated 12th July 2013. The purpose of the works was to in-situ compact the top 3m of the site to mimic the ground conditions below the existing building.

The validation testing of the in-situ compaction was carried out using a series of 2m by 2m zone load tests at ground surface in accordance with BS1377: Part 9, Section 4.2, "Methods of test for Soils for Civil Engineering purposes In-situ Tests".

Prior to commencement of the compaction works, Zone Load Testing (ZLT) was carried out within 5m of the existing building on the existing treated area to provide load settlement criteria to calibrate against testing on the newly compacted surface. Following the testing the proposed extension footprints were compacted.

Continuous impact response (CIR) monitoring was carried out as compaction was undertaken, to assess any variation of relative density in the compacted soils. On completion of the compaction works the results of the continuous impact response (CIR) monitoring were provided to Coffey.

On receipt of the CIR monitoring, Coffey selected locations where the underlying soils had the lowest relative density for validation Zone Load tests. These locations were likely to perform worst under zone load testing. A validation ZLT test was carried out in each of the compacted areas at the locations agreed with Coffey.

2.2 Bearing Capacity and Settlement Criteria

The ground improvement was to be carried out to provide a minimum bearing capacity of 110 kN/m² at the site surface following compaction, with a corresponding settlement of less than 20mm for a 2m by 2m foundation.

2.3 Differential Settlement Validation Criteria

The settlements from all zone load testing, both calibration ZLT and validation ZLT were compared to each other. The specification required that the differential settlement between any two ZLT undertaken on site should not exceed 10mm.

3 WORKS UNDERTAKEN

The in-situ compaction works were carried out between 10th July 2013 and 07th August 2013. The works were undertaken in line with Landpac Site Specific Method Statement (HEICMS-LP1225-REV001), dated 04th July 2013 and Coffey Geotechnics "Specification for in-situ Compaction" Ref: COF_S_DS_001_PDF (Coffey Ref: 01975AA_S_DS001B), dated 12th July 2013.

3.1 Works Constraints and Departures from Specification.

During the dynamic compaction works a number of issues were encountered where it was necessary to vary the works from the specification. This section reviews these constraints and reports actions undertaken to deal with these constraints.

3.1.1 Buried obstructions and open trenches

During the compaction works a number of drainage trenches were open around the eastern side of the southern extension (Area 1). Direct compaction of the ground surrounding these areas was not possible due to the potential for collapse of the trenches.

A surface water drain was present in the footprint of the western extension which ran from the north side of the extension to the south side. This drain was allowed to remain during the course of the compaction works to provide drainage for the site. Following the compaction works the storm drain was excavated and removed.

Following completion of the compaction works all the trenches were backfilled in accordance with RPS "Concrete Works & Substructures Specification" RPS_SPC_0001_PDF.

3.1.2 Soft spots

During the works execution any visually soft spots encountered were identified and remediated by excavating and replacing the unsuitable material with site won granular fill.

Two areas were identified where remediation was required. These comprised a strip at the south side of the southern extension (Area 1) and a strip at the south west side of the western extension (Area 2).

The backfilling of these areas was carried out in accordance with RPS "Concrete Works & Substructures Specification" RPS_SPC_0001_PDF.

The remediated areas are indicated on Figure 2 enclosed.

3.2 ZLT Validation Testing

The results of the Continuous Impact Response (CIR) monitoring and Zone Load (ZLT) validation testing are presented in Landpac handover report "BGCL ASOS Logistics Houghton HEIC-CIR-CIS ZLT Testing Report (Ref: LP1225), dated 08th August 2013. The report is enclosed as Appendix A.

In accordance with the specification, calibration Zone Load Testing (ZLT) was undertaken at two locations adjacent to and within 5m of the existing warehouse building. Test ZLT101 was carried out at the southern extension location (Area 1) and ZLT102 was carried out at the western extension location (Area 2). The results of the Zone Load Testing indicated a settlement of 10.6mm at test location ZLT101 and 12.09mm at test location ZLT101 under a 110kN/m2 load.

Following HEIC treatment the results of the CIR were reviewed and locations for validation ZLT's were selected in the least dense areas. Validation testing was carried out at one location in the footprint of each of the proposed extensions. Test ZLT103 was carried out at the southern extension location and ZLT104 was carried out at the western extension location. The results of the Zone Load Testing indicated a settlement of 13.53mm at test location ZLT101 and 15.50mm at test location ZLT101 under a 110kN/m2 load.

The Locations of the ZLT's are indicated on Figure 2.

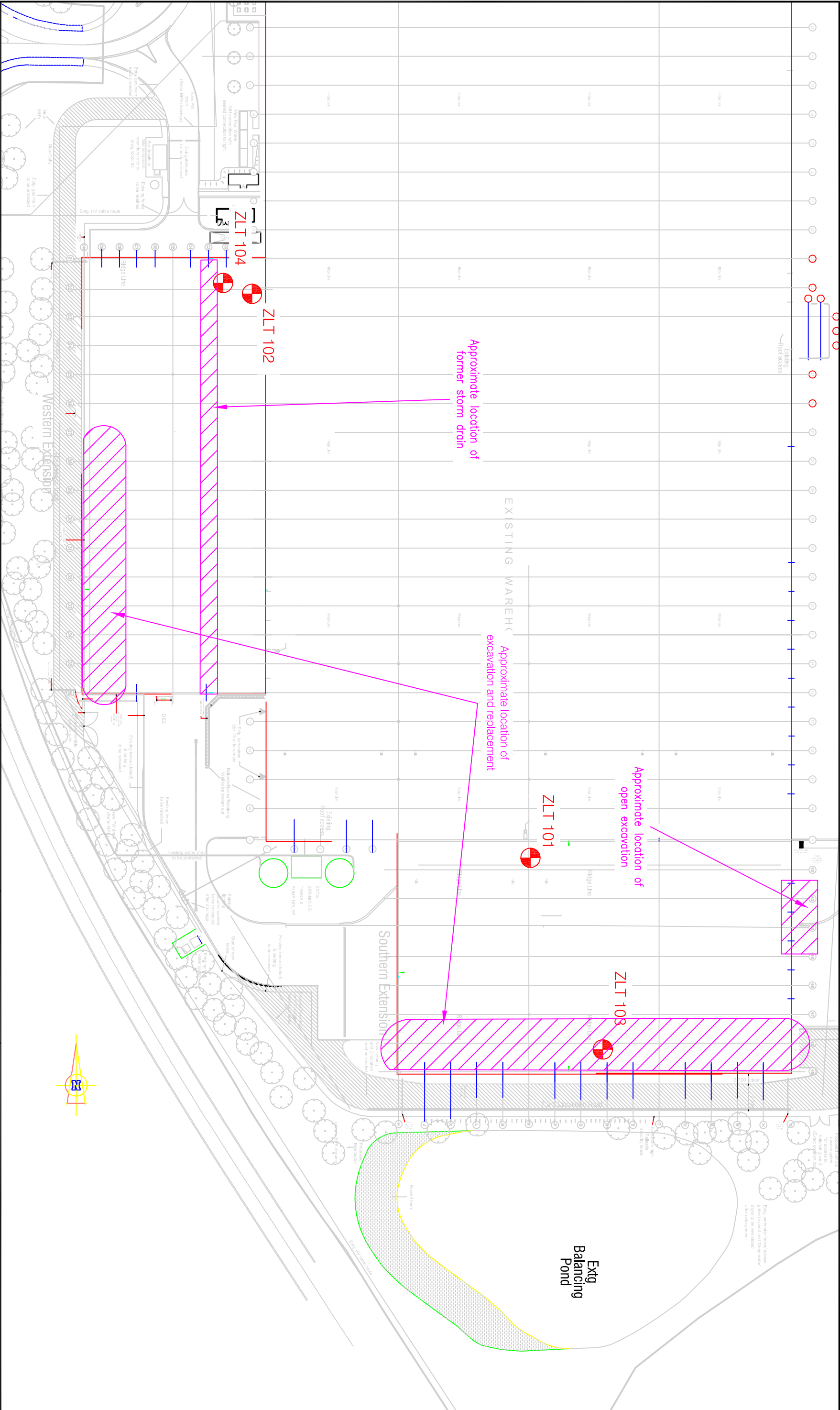
4 CONCLUSIONS

The in-situ compaction works have been executed in accordance with Coffey Geotechnics "Specification for in-situ Compaction" Ref: COF_S_DS_001_PDF (Coffey Ref: 01975AA_S_DS001B),

dated 12th July 2013. The results of the zone load testing indicate that the specified load settlement criteria and differential settlement criteria have been met.

Where soft-spots and trenches were present these have been excavated (where required) and backfilled in accordance with RPS "Concrete Works & Substructures Specification" RPS_SPC_0001_PDF.

Figures



Notes:

1. Do not scale from this drawing.
2. All dimensions are in metres unless otherwise stated.

KEY

ZLT 101



Zone Load Test Locations

drawn	DAS	client:	BUCKINGHAM GROUP LTD	
date	14/10/13	project:	HOUGHTON MAIN, BARNSELEY	
scale	NTS	title:	TEST LOCATION AND SITE FEATURES PLAN	
original size	A3	project no:	01975AA	fig no: Figure 2

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