**Factual and Interpretative Geotechnical Report**

**Former Shredded Wheat Factory, Broadwater Road, Welwyn Garden City, AL7 3AX**

**For Spen Hill Developments Ltd**

**Delta-Simons Project No. 2342.18_G V2**

**Issued: January 2015**

**EXECUTIVE SUMMARY**
**FACTUAL AND INTERPRETIVE GEOTECHNICAL REPORT**
**FORMER SHREDDED WHEAT FACTORY, BROADWATER ROAD, WELWYN GARDEN CITY, AL7 3AX**
**DELTA-SIMONS PROJECT NO: 2342.18_G V2**

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| **Purpose** | Delta-Simons was instructed by Spen Hill Developments Limited (the 'Client') to undertake a geotechnical assessment at the above Site. The purpose of completing this investigation is to provide detailed information on the ground conditions at the Site in order to aid foundation design of the proposed residential redevelopment including hotel provision. |
| **Current Site Status** | The Site is irregular in shape occupying an area of approximately 9 hectares (ha). The Site is situated at approximately 85 m AOD and is generally flat.<br><br>The Site is currently occupied by a former cereal factory located to the north and south of Hydeway which bisects the Site, and open derelict ground to the south of the Site, the former location of a confectionary factory and Polycell factory. |
| **Geology**<br><br>**Hydrogeology** | From British Geological Survey data, the entire Site area is indicated to be underlain by the superficial the Lowestoft Formation over Kesgrave Catchment Subgroup, overlying bedrock comprising Lewes Nodular Chalk Formation and Seaford Chalk Formation (undifferentiated).<br><br>Resting groundwater levels encountered during previous investigations were encountered in the shallow chalk aquifer recorded between 20.0 m and 26.0 m bgl. |
| **Ground Conditions** | The ground conditions encountered generally comprised Made Ground (to depths between 0.30 m and 3.50 m bgl), overlying superficial deposits of clay and sand to depths between 8.40 m and 16.60 m bgl. Bedrock Lewes Nodular Chalk Formation and Seaford Chalk Formation (undifferentiated) was then proven to a maximum depth of 30.00 m bgl in the deepest boreholes.<br><br>Resting groundwater levels recorded during the return monitoring visits were between 21.23 m and 22.62 m bgl.<br><br>In BH407 mixed soils with low density were identified from 13.80 m bgl to 17.10 m bgl with possible voiding between 14.8 m to 16.0 m bgl. Also in BH414 a void was identified from 12.95 m bgl to 16.95 m bgl and low density clayey gravelly sand to 17.50 m bgl (the base of this feature was not identified). The evidence observed in boreholes BH407 and BH414 has been tentatively identified as being caused by dissolution features. Given the sparse distribution of boreholes it is very unlikely that borehole BH407 and BH414 have encountered the only dissolution features, or the worst cases of loose ground, within the development area. |
| **Geotechnical Recommendations** | A piled foundation solution using traditional bored or CFA piles transferring loads to competent geology may be suitable for the proposed loads.<br><br>Pile design would need to take into account the presence of potential dissolution features, which may include further investigation, design and construction mitigations, spanning affected areas following discovery and capping, pre-pile probing, grouting, and use of different factors of safety and engineering redundancy.<br><br>It is recommended that further investigation is undertaken across the Site to provided coverage of previously un-investigated areas and further investigate potential dissolution features at proposed building/ pile locations. It is recommended to obtain early specialist subcontractor involvement.<br><br>Resting groundwater would not be anticipated to be encountered during basement excavation and other shallow excavations and trenches.<br><br>In-situ CBR testing indicates that a conservative value of 5% should be adopted for shallow soils for preliminary pavement design.<br><br>The conditions of the Soils at the Site would be classified as Design Sulphate Class DS-1 and ACEC Class AC-1d. |
| **This Executive Summary is intended as a summary of the assessment of the Site based on information received by Delta-Simons at the time of production.** | |

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**FOR SPENHILL DEVELOPMENTS LIMITED**
**DELTA-SIMONS PROJECT NO. 2342.18_G V2**

# 1.0 INTRODUCTION

## 1.1 Instruction

Delta-Simons was instructed by Spen Hill Developments Limited (the 'Client') to undertake a Factual and Interpretive Geotechnical Report at land located at the Former Shredded Wheat Factory, Broadwater Road, Welwyn Garden City, AL7 3AX (hereafter referred to as the 'Site'). The Client's structural engineering consultant for the project is Icis Design Limited.

A Site location map is included as Figure 1.

## 1.2 Scope and Objectives

The purpose of completing a Factual and Interpretative Geotechnical Report at the Site is to provide information regarding the strength and chemical characteristics of the underlying geological deposits in order to aid foundation design of the proposed redevelopment of the Site.

Delta-Simons has completed a Desktop Geotechnical Assessment [Ref.1] for the Site reference 2342-17, dated December 2013, which has been used to inform this Report where appropriate.

Delta-Simons has also completed a Phase I Environmental Assessment [Ref.2] for the Site, reference 2342-17 dated December 2013.

This investigation has been completed in general accordance with BS5930:1999 +A2:2010, Code of Practice for Site Investigations [Ref. 3]. Delta-Simons mutually agreed the scope of this investigation with the Client, which reflects the current Site status with building and other Site features remaining. As such, further investigation is likely to be required.

## 1.3 Site Description

The Site is irregular in shape occupying an area of approximately 9 hectares (ha). The Site is situated at approximately 85 m above Ordnance Datum (AOD) and is generally flat. The Site is currently occupied by a former cereal factory located to the north and south of

Hydeway which bisects the Site and open derelict ground to the south of the Site, the former location of a confectionary factory and Polycell factory. The Site is located within a mixed industrial, commercial and residential area and the current layout is shown on Figure 2.

## 1.4 Proposed Development

All existing buildings are to be demolished (with the exception of listed facades and structures to be retained), and the Site redeveloped for a mixed use including residential, retail, office, hotel, gym and community hub.

## 1.5 Limitations

Any other issues not listed in the scope of works, but subsequently identified during the completion of the Site investigation and reported herein (such as the potential presence of contamination, Japanese Knotweed, flood assessment studies or ecological surveys) are provided for information only and fall outside the scope of this assessment. The Report does not constitute an environmental, archaeological or ecological assessment, nor does it constitute an asbestos inspection or flood assessment.

During the preparation of this assessment, Delta-Simons obtained, reviewed and evaluated information in preparing this Report from the Client, SI Drilling, Lankelma, Professional Soils Laboratory (PSL), Derwentside Environmental Testing Services and Chemtest Limited. Delta-Simons' conclusions, opinions and recommendations are based upon this information and the information obtained during the investigation. Delta-Simons does not warrant the accuracy of the information provided to it and will not be responsible for any opinions that Delta-Simons has expressed, or conclusions which it has reached in reliance upon information which is subsequently proven to be inaccurate.

The recommendations contained in this assessment represent our professional opinions. These opinions were arrived at in accordance with currently accepted industry practices and hydrological and engineering practices at this time and location and, as such, are not a guarantee that the Site is free of hazardous or potentially hazardous materials or conditions.

This assessment was prepared by Delta-Simons for our Client. Any third party using this assessment does so entirely at their own risk. Delta-Simons makes no warranty or representation whatsoever, express or implied, with respect to the use by a third party of any information contained in this assessment or its suitability for any purpose. Delta-Simons assumes no responsibility for any costs, claims, damages or expenses (including any consequential damages) resulting from the use of this assessment or any information contained in this assessment by a third party, with the exception of those named on the cover.

# 2.0 SITE SETTING

## 2.1 Geology

Ground conditions encountered across the Site during previous investigation (as reviewed in the Desktop Geotechnical assessment [Ref.1]), have been summarised in Table 1.

**Table 1 – Generalised Ground Conditions**

| Strata | Generalised Description of Strata | Depth |
|---|---|---|
| Made Ground - Hardstand | Where present, predominantly comprising concrete, sometimes asphalt. | From surface to 0.3 metres below ground level (m bgl) |
| Made Ground | Variable inconsistent stratum.<br>Clay, silty sand, gravel, bricks, ash, slag and concrete. | Base of strata between 0.6 m and 2.0 m bgl |
| **Superficial:** Kesgrave Catchment Subgroup and the Lowestoft Formation | Variable sometimes inconsistent strata across the Site.<br>Clayey sands and gravels, and/ or gravelly clay. | Base of strata between 8.0 m and 18.0 m bgl |
| **Bedrock:** Lewes Nodular Chalk Formation and Seaford Chalk Formation (undifferentiated) | Initially weathered white putty chalk grading to chalk bedrock. | Proven to a maximum depth of 30.0 m bgl |

The Site is located in a lower probability radon area, as less than 1% of homes are above the action level, as such no radon protective measures are considered necessary in the construction of new dwellings or extensions.

The Site is not located within a Coal Mining Affected Area.

The Site is underlain by chalk formations which may be susceptible to chemical weathering and the formation of underground dissolution features.

There are 35 No. Natural Cavity entries with 1 km of the Site (11 No. within 250 m of the Site), all refer to sinkholes or solution pipes. The closest entry is located approximately 40 m south of the Site for a sinkhole (no further details are available with respect to these features).

## 2.2 Hydrogeology

Resting groundwater was encountered in the chalk formation circa 22 m to 23 m bgl; any shallow perched groundwater is likely to be limited in extent and discontinuous and unlikely to represent a significant constraint to development.

# 3.0 FIELD AND LABORATORY STUDIES

## 3.1 Walkover Survey

A representative of Delta-Simons carried out a walkover survey at the start of the works in order to confirm the location of the proposed exploratory holes.

## 3.2 Ground Investigation

The fieldwork was undertaken between the 1st October and 29th October 2014, and comprised the following items:

- Δ Supervision of all works by a Delta-Simons geo-environmental engineer. All boreholes were logged to BS5930:1999 +A2:2010, Code of Practice for Site Investigations [Ref. 3] and with reference to CIRIA C574: Engineering in Chalk [Ref. 4];
- Δ Service avoidance exercise;
- Δ Drilling of sixteen cable percussion boreholes (BH401 to BH416) to a maximum depth of 30.00 m bgl;
- Δ Advancement of ten truck mounted cone penetration tests (CPT), CPT401 to CPT408A and B, to a maximum depth of 20.21 m bgl;
- Δ Pre-drilling of selected CPT test locations utilising rotary auger techniques;
- Δ Installation of sixteen 50 mm internal diameter groundwater monitoring wells;
- Δ Standard penetration tests (SPTs) within the cable percussion boreholes at 1.00 m intervals to 5.00 m bgl and then at 1.50 m intervals thereafter;
- Δ Undisturbed samples in cohesive strata were recovered from the cable percussive boreholes at intervals selected by the supervising geologist;
- Δ Excavation of seven trial pits TP401, 401A to TP403, TP405 to TP407, and recording of basement features;
- Δ Seven dynamic cone penetrometer California bearing ratio tests (DCP CBR) tests;
- Δ Collection of disturbed soil samples from all intrusive locations for subsequent laboratory testing;
- Δ Collection of groundwater samples from installed boreholes on two occasions; and
- Δ Two rounds of gas and groundwater level monitoring.

## 3.3 Ground Investigation Factual Data

Delta-Simons engineer verified borehole logs are presented as Appendix I, the SPT Calibration Certificate (in accordance with BS EN ISO 22476-3:2005 incorporating corrigendum No. 1 2007), Geotechnical investigation and testing - Field testing - Part 3: Standard penetration test [Ref. 5]) for the cable percussive SPT trip hammers are presented as Appendix II (certificate EQ576 for BH402, BH403, BH405 to BH407, BH412, BH413, BH415 and BH416, and certificate EQU438 for BH401, BH404, BH408 to BH411 and BH414). The trial pit logs are included as Appendix III.

The gas and groundwater monitoring results are presented as Appendix IV. A borehole location plan is presented as Figure 4.

## 3.4 In-situ Testing and Sampling

Where undisturbed sampling was not attempted, SPTs were undertaken at 1.00 m intervals for the first 5.00 m and then at 1.50 m intervals thereafter in all cable percussive boreholes. The results of these tests are presented in the borehole logs included as Appendix I.

Sampling comprised disturbed tub and bulk bag samples generally taken at 1.00 m intervals as detailed on the borehole logs. Undisturbed samples (steel lined U100s) were taken in cohesive strata as detailed on the borehole logs.

The results of the DCP CBR tests, with interpretation in accordance with Design Manual for Roads and Bridges (DMRB) Vol. 7 Section 3 Part 2 HD29/08 [Ref. 7], are included in Appendix V.

The results of the CPT tests and predrilled borehole logs are included as Appendix VI and VII respectively.

## 3.5 Laboratory Investigation

Following the ground investigations, a schedule of geotechnical and chemical laboratory testing was prepared by Delta-Simons.

### 3.5.1 Geotechnical Testing

The geotechnical testing was carried out by a UKAS accredited laboratory (PSL), in accordance with BS 1377 - Parts 2 to 9:1990 Methods of test for soils for civil engineering purposes [Ref. 6] which comprised:

- Twenty moisture content;
- Ten liquid and plastic limits;
- Eight particle size distribution;

- Five unconsolidated undrained triaxial tests; and
- One saturated moisture content/ intact dry density.

Copies of the geotechnical laboratory test results are presented in Appendix VIII.

### 3.5.2 Soil Geo-chemical Analysis

Soil chemical analysis on the Made Ground, superficial deposits and chalk was undertaken by Derwentside Environmental Testing Services, which comprised:

- Five BRE SD1 full suite (pH, total sulphur, water soluble sulphate, acid soluble sulphate, magnesium, chloride and nitrate); and
- Five BRE SD1 short suite (pH, water soluble sulphate, magnesium, chloride and nitrate).

Copies of the soil chemical laboratory test results for the above are presented in Appendix VIII.

### 3.5.3 Groundwater Chemical Testing

Two rounds of groundwater chemical testing were undertaken by a UKAS accredited laboratory (Chemtest) and relevant testing comprised:

- Eighteen pH and sulphate.

Copies of the groundwater chemical test results are presented in Appendix IX.

# 4.0 GROUND SUMMARY

This section outlines the ground conditions encountered during the investigation.

## 4.1 Geology

### 4.1.1 Geology Summary

Details of the ground conditions identified by the investigation have been summarised in Table 2 below and geological cross sections presented as Figures 5a to 5e (the orientations of which are presented on Figure 4).

**Table 2: Generalised Geology Strata**

| Strata | Description of Strata | Depth Range of Strata Base (m bgl) |
|---|---|---|
| Made Ground – Hardstand | Encountered in borehole locations BH401, BH403 to BH406, BH408 to BH411 and BH413 to BH415.<br>Tarmacadam, concrete or reinforced concrete. | 0.00 m to 0.40 m bgl |
| Made Ground | Encountered in all borehole locations.<br>Made Ground typically comprised brown sandy gravelly clay or gravelly sand in varying fractions. Gravels included brick, flint and concrete. Rare brick cobbles. | 0.3 m to 3.50 m bgl |
| Lowestoft Formation | Encountered above the Kesgrave Catchment Subgroup in all locations.<br>Typically comprised inconsistent layers of orange brown and light brown sandy gravelly clay, with sand and gravel in varying fractions.<br>In BH404, BH412, BH414 and BH416 this formation comprised clayey sandy gravel and clayey gravelly sand. | 3.20 m to 10.50 m bgl |
| Kesgrave Catchment Subgroup | Subgroup encountered in all borehole locations.<br>Typically comprised of orange brown sandy gravel and gravelly sand. Gravels are fine to coarse angular to rounded flint. | 8.4 m to 16.6 m bgl |
| Lewes Nodular Chalk Formation and Seaford Chalk Formation (Undifferentiated) | Encountered in all locations.<br>Predominantly recovered as structureless chalk composed of slightly gravelly silt, Grade Dm.<br>Grade Dc structureless chalk was encountered in BH407 between 17.10 m bgl and 20.00 m bgl.<br>SPT N values indicate more competent grades of chalk than indicated by description of recovered soils due to drilling method destroying weak rock chalk structures. | Proven to a maximum depth of 30.00 m bgl |

### 4.1.2 Basement Investigation

The suspected basements in the south-west and south-east of the Site were investigated with a mechanical excavator and breaker. The basement in the south-west was approximately 1600 $m^2$ and found to be flooded. There was also a number of large service corridors/ ducts in this area, one of which contained lagged pipes labelled as asbestos. The suspected basement in the south-east of similar size appears to have been filled with demolition waste with substructure possibly left in-situ. Encountered basement locations are shown on Figure 4. There is a possibility of further basements and service duct structures being encountered in other areas of the Site.

## 4.2 Groundwater

Resting groundwater levels recorded during the return monitoring visits were between 21.23 m and 22.62 m bgl. (62.23 m and 63.75 m AOD).

Groundwater contour plots using groundwater elevation data have been produced, and are presented as Figure 6.

# 5.0 GROUND CONDITIONS AND MATERIAL PROPERTIES

A plot of uncorrected SPT ‘N’ values against depth for all strata is presented as Figure 7 and a plasticity chart for the Lowestoft Formation is presented as Figure 8.

## 5.1 Summary of Geotechnical Parameters

A summary of the derived geotechnical parameters for each strata are summarised in Table 3.

**Table 3: Summary of Geotechnical Parameters**

| | Made Ground | Lowestoft Formation - Cohesive | Lowestoft Formation - Granular | Kesgrave Catchment Subgroup | Lewes Nodular Chalk Formation and Seaford Chalk Formation (Undifferentiated) |
|---|---|---|---|---|---|
| Moisture Content - w | - | 11% - 25% | 4.2% -17% | 32% | 22% - 34% |
| Liquid Limit - $w_L$ | - | 24% - 50% | 29% | - | - |
| Plastic Limit - $w_P$ | - | 14% - 19% | 16% | - | - |
| Plasticity Index - $I_P$ | - | 10% - 31% | 13% | - | - |
| Bulk Density - ρ | - | 2.00-2.20 Mg/m$^3$ | - | - | - |
| Bulk Unit Weight[1] - γ | - | 19.62 – 21.58 kN/m$^3$ | - | - | - |
| Uncorrected SPT N[4] | 22 - 50 | 4 – 50 | 13 - 45 | 2 - 50 | 1 - 50 |
| Corrected[2,4] SPT $N_{60}$ | 22 - 50 | 4 – 50 | 13 - 45 | 2 - 50 | 1 - 50 |
| Undrained Shear Strength[3] - $c_u$ | - | 23 – 58 kPa | - | - | - |
| Saturated Moisture Content (one sample) | - | - | - | - | 26% |
| Dry Density | - | - | - | - | 1.58 Mg/m$^3$ |

1. Bulk unit weight (kN/m$^3$) = 9.81 x bulk density (Mg/m$^3$ - as determined by laboratory testing)
2. SPT N values corrected for energy delivered to drive rods utilising the determined energy ratio ($E_r$): $N_{60} = (E_r \times N) / 60$ after BS EN ISO 22476-3:2005 [Ref. 5]
3. From laboratory test results.
4. Lower SPT values are likely to originate from BH407 and BH414.

## 5.2 Geochemical Testing

Geochemical analysis was undertaken on ten soil samples and eighteen groundwater samples on two return visits, tested for selective contaminants (BRE Special Digest 1:2005 (3rd Edition), Concrete in Aggressive Ground [Ref. 8]), the results of which are summarised in Table 4.

**Table 4: BRE SD1 Test Result Summary**

| | No. of Tests | Minimum | Maximum |
|---|---|---|---|
| Soil - pH | 10 | 6.90 | 10.80 |
| Soil - Total Sulphur | 5 | 0.01% | 0.53% |
| Soil - Magnesium | 10 | 0.01 g/l | 0.02 g/l |
| Soil – Total Sulphate | 5 | 0.03 % | 0.24% |
| Soil - Water Soluble Sulphate | 10 | 0.01 g/l | 0.33 g/l |
| Groundwater - pH | 18 | 6.00 | 8.10 |
| Groundwater - Sulphate | 18 | 7.00 mg/l | 310.00 mg/l |

# 6.0 GEOTECHNICAL ASSESSMENT

## 6.1 Summary of Development Proposals

Full details of the development proposals have not been supplied to Delta-Simons due to the early stage of the project. Indicative details have been provided by the structural engineer, with column loads expected between approximately 1,500 kN to 6,000 kN from multiple storey structures. The structural engineer expects that columns loads and ground slabs will be supported by pile groups of up to four per cap. It is also understood that single storey basements are being considered for some of the structures.

A proposed development layout is presented as Figure 3.

## 6.2 Ground Conditions

By inspection of the records, the ground conditions generally comprised Made Ground (to depths between 0.30 m and 3.50 m bgl), overlying superficial deposits of clay and sand to depths between 8.40 m and 16.60 m bgl. Bedrock Lewes Nodular Chalk Formation and Seaford Chalk Formation (undifferentiated) was then proven to a maximum depth of 30.00 m bgl in the deepest boreholes.

Resting groundwater levels recorded during the return monitoring visits were between 21.23 m and 22.62 m bgl.

In BH407 mixed soils with low density were identified from 13.80 m bgl to 17.10 m bgl with possible voiding between 14.8 m to 16.0 m bgl. Also in BH414 a void was identified from 12.95 m bgl to 16.95 m bgl and low density clayey gravelly sand to 17.50 m bgl (the base of this feature was not identified). The evidence observed in boreholes BH407 and BH414 has been tentatively identified as being caused by dissolution features which are described in CIRIA C574[Ref. 4]. It is recognised that there was no evidence of the possible effects of loose ground in the existing structures on the Site during the fieldwork (such as distressed masonry), but it is not known whether previous construction methods were affected. The evidence of the existing development and surrounding area does suggest that dissolution features do not represent significant risk to overall land stability, but are at least likely to affect localised areas.

Chalk is a soft and fractured limestone that is susceptible to erosion by sediment charged and potentially acid groundwater. High rates of erosion occurred prior to, and during, deposition of glacial superficial deposits. Erosion can produce a locally highly variable interface between the Chalk and overlying sediments on different scales of magnitude, and sometimes open pipe features in limestone bedrock that are suspected widening of intersecting joints. Voids are typically infilled by later deposits and a loosely infilled near-vertical pipe in the Chalk can cause a corresponding loose column of soil in overlying sediments. Often there is no surface expression of the loose soils due to bridging between layers and individual particles or being masked by recent construction. Settlement events can be caused by compaction/movement of the infilled soil, usually associated with large water releases such as burst water mains or concentrated discharges through shallow soakaways. Dissolution features may represent a hazard to construction as a loss of support to significant structures and utilities.

Given the sparse distribution of boreholes it is very unlikely that borehole BH407 and BH414 have encountered the only dissolution features, or the worst cases of loose ground, within the development area.

The distribution of dissolution features may appear to be random in terms of depth and lateral extent, but there may not be sufficient sampling points to determine any pattern. Dissolution feature occurrence may be able to be zoned over a wider development area, or at least the frequency better understood. There are options for identification of dissolution features using geophysics, but often these have mixed results particularly on brownfield land due to constraints of structures, made ground and utilities. Boreholes and penetration tests are considered to be the most effective way of identification of dissolution features over a wide area where geophysics cannot be used.

A typical risk for piling into a dissolution feature could be poor construction of individual piles/groups and a consequent loss of concrete and pile load capacity at a critical location.

## 6.3 Foundations

### 6.3.1 Spread Foundations

The Made Ground and Lowestoft Formation are considered to be too variable, weak and compressible in their existing condition for conventional shallow foundations given the expected foundation loads for the proposed multi-storey structures.

However, the Lowestoft Formation is used as a bearing strata for typical low-rise construction such as housing or other small structures of low complexity and a general allowable bearing pressure of 100 $kN/m^2$ is recommended where settlements are required to be less than 25 mm in total.

Where traditional foundations are proposed, further investigation is recommended to check for loose ground under footings.

### 6.3.2 Ground Improvement Techniques

At this stage, ground improvement techniques for foundation construction are considered unsuitable on the Made Ground and Lowestoft Formation, given the expected large design loads, the depth to competent strata and the presence of dissolution features. Ground improvement may be able to be used as a part of a general approach to land development in reduction of risk.

### 6.3.3 Piled Foundation

The top of the Chalk Formation was identified from depths between 8.4 m and 16.6 m bgl, proven to a maximum depth of 30 m bgl.

A piled foundation solution using traditional bored or continuous flight auger (CFA) piles transferring loads to competent geology may be suitable for the expected design loads, utilising both skin friction and end bearing capacity.

The precise method of pile installation and applicability of proprietary systems, diameters and depths required would need to be informed based on the results of this investigation, by discussions with a piling contractor with suitable experience in chalk.

For preliminary design purposes, the following allowable CFA pile loads have been assessed based on commonly accepted methods and in accordance with CIRIA C574 [Ref. 4] for determining pile base resistance and skin friction/ adhesion (utilising a bulk Factor of Safety of 2.5, or 3.5 for the base and 1.5 for the skin); any negative skin friction effects associated with Made Ground strata has been ignored. Pile groups are assumed to be free standing with a spacing no less than three pile diameters. Commercial pile designers may use different parameters, design factors or safety factors than published methods. The values in the table below are likely to be significantly lower where loose soils or voiding associated with dissolution features are encountered.

**Table 5 – Estimated Allowable Pile Capacities (Bored Piles)**

| Typical Pile Size | | Single Pile | Pile Group of 4 |
|---|---|---|---|
| 0.6 m diameter | 15 m | 720 kN | 2,170 kN |
| | 20 m | 1,210 kN | 3,640 kN |
| | 25 m | 2,040 kN | 6,120 kN |
| 0.75 m diameter | 15 m | 970 kN | 3,120 kN |
| | 20 m | 1,620 kN | 5,190 kN |
| | 25 m | 2,800 kN | 8,960 kN |
| 0.9 m diameter | 15 m | 1,250 kN | 4,266 kN |
| | 20 m | 2,070 kN | 7,050 kN |
| | 25 m | 3,650 kN | 12,430 kN |

Individual pile/pile group loads will be a function of the surface area of the piles to be employed at the Site and their method of construction.

It is recommended that during demolition of the building, all relict basements, foundations, piles caps and other obstructions are removed and the locations of existing piles (if present) recorded to avoid in-ground obstructions during piling.

Normal static and dynamic load testing (including uplift tests) should be considered to achieve satisfactory quality control/assurance in accordance with good practice.

There will be a requirement for the placement of a suitably engineered piling mat, which should be designed and validated by a suitably qualified and experienced engineer.

Pile design would need to take into account the presence of potential dissolution features and this may be achieved by various methodologies which are beyond the scope of this report to recommend. However, typical practical strategies/combinations may be as follows:

- Further investigation, design and construction mitigations;
- Spanning affected areas following discovery and capping;
- Pre-construction probing;
- Grouting; and

- Use of different factors of safety and engineering redundancy.

It is recommended that further investigation is undertaken across the Site to provided coverage of previously un-investigated areas and further investigate potential dissolution features at proposed building/ pile locations.

Possible ground treatments may require specific geotechnical parameters to be obtained, and it may be efficient to obtain early specialist contractor involvement.

### 6.3.4 Basement Construction

Basements are also assumed to use piled foundations including sheet piled or secant/contiguous bored pile walls assumed to be designed as integrated with the above ground column loads so as not to interfere with each other.

The retaining structure should be designed to take account of likely lateral/external loads presented by the retained soil, and any vertical structural load to be applied, to determine suitable embedment depths and pile diameters.

Resting groundwater would not be anticipated to be encountered during basement excavation.

Based on groundwater monitoring results (corrected to ordnance datum), resting groundwater may be expected to be encountered between (62.23 m and 63.75 m AOD). Specific basement construction details are not known, however, the base of a single storey basement may be expected circa 80 m AOD.

Resting groundwater would not be anticipated to be encountered during basement excavation.

### 6.3.5 Volume Change Potential

The volume change potential should be considered in any foundation schedule for structures and services located within the influence zone of trees or bushes (proposed, existing or to be removed) and appropriate precautions and/ or founding depths should be designed accordingly.

## 6.4 Roads and Pavements

In-situ DCP CBR test results are presented as Appendix V. It is recommended that a conservative CBR value of 5% should be adopted for the shallow soils, for preliminary pavement design for external areas.

It is recommended that plate CBR tests are undertaken at formation level after demolition prior to finalising pavement design.

## 6.5 Drainage

Due to the presence of chalk dissolution features, it is recommended that the soakaways are not used as part of a drainage scheme.

It is recommended that all underground water utilities are constructed so as to resist leakage over time to a degree sufficient to protect the development from settlement events caused by dissolution features.

## 6.6 Excavations

It is expected that conventional mechanical excavators will readily remove the Made Ground and superficial soils likely to be encountered in shallow excavations, although a breaker will be required to remove any existing concrete hardstand covering and relict basements and foundations.

All shallow foundation or services excavations at the Site should be considered unstable, therefore, temporary support of all excavations should be considered when excavating on-Site.

## 6.7 Groundwater

Resting groundwater levels recorded during the return monitoring visits were between 21.23 m and 22.62 m bgl (62.23 m and 63.75 m AOD).

Significant groundwater would not be anticipated during the excavations required as part of the proposed development. Should any perched groundwater be encountered, then local dewatering via sump and pump should be suitable, however, treatment prior to disposal to sewer may be required.

## 6.8 Chemical Attack on Buried Concrete

In accordance with the recommendations of BRE Special Digest 1, 'Concrete in Aggressive Ground' 2005 [Ref. 8], the conditions of the soils at the site would be classified as Design Sulphate Class DS-1 and ACEC Class AC-1d, when considering the most appropriate type of concrete to be used at the site in order to resist chemical attack from elevated sulphate present in the soils (assuming mobile groundwater in non-pyritic soils).

# 7.0 CONCLUSIONS AND RECOMMENDATIONS

## 7.1 General

This investigation has identified ground conditions similar to previous reports and mapped records. However, geotechnical hazards have been identified in the form of potential dissolution features. Figures showing cross sections of the geology have been provided.

## 7.1 Summary of Geotechnical Recommendations

- ∆ The Made Ground and Lowestoft Formation are considered to be too variable, weak and compressible in their existing condition for conventional shallow foundations given the expected foundation loads, however, the Lowestoft Formation may be suitable for small discrete buildings;
- ∆ A piled foundation solution using traditional bored or CFA piles transferring loads to competent geology may be suitable for the proposed development;
- ∆ Pile design would need to take into account the presence of potential dissolution features, which may include further investigation, design and construction mitigations, spanning affected areas following discovery and capping, pre-pile probing, grouting, and use of different factors of safety and engineering redundancy;
- ∆ It is recommended that further investigation is undertaken across the Site to provided coverage of previously un-investigated areas and further investigate potential dissolution features at proposed building/ pile locations. It is recommended to obtain early specialist subcontractor involvement;
- ∆ The precise method of pile installation and applicability of proprietary systems, diameters and depths required would need to be informed based on the results of this investigation, by discussions with a piling contractor with suitable experience in chalk;
- ∆ Resting groundwater would not be anticipated to be encountered during basement excavation and other shallow excavations and trenches;
- ∆ In-situ CBR testing indicates that a conservative value of 5% should be adopted for shallow soils for preliminary pavement design; and
- ∆ The conditions of the Soils at the Site would be classified as Design Sulphate Class DS-1 and ACEC Class AC-1d.

# 8.0 REFERENCES

**Ref. 1** Delta Simons, Desktop Geotechnical Assessment, Broadwater Road, Welwyn Garden City, reference 2342-17, dated December 2013.

**Ref. 2** Delta Simons, Phase 1 Environmental Assessment, Broadwater Road, Welwyn Garden City, reference 2342-17, dated December 2013.

**Ref. 3** BS5930:1999 +A2:2010, Code of Practice for Site Investigations.

**Ref. 4** CIRIA C574: Engineering in Chalk, 2002.

**Ref. 5** BS EN ISO 22476-3:2005 (incorporating corrigendum No. 1 2007), Geotechnical investigation and testing - Field testing - Part 3: Standard penetration test

**Ref. 6** BS1377 – Parts 2 to 9:1990 Methods of test for soils for civil engineering purposes.

**Ref. 7** Design Manual for Roads and Bridges, Volume 7, Section 3, Part 2 HD29/08, dated May 2008

**Ref. 8** BRE Special Digest 1:2005 (3rd Edition), Concrete in Aggressive Ground.

The factual and interpretative parts of this Report were prepared by:

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Date

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29.1.15

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Date

**Project Manager**

This report was authorised by:

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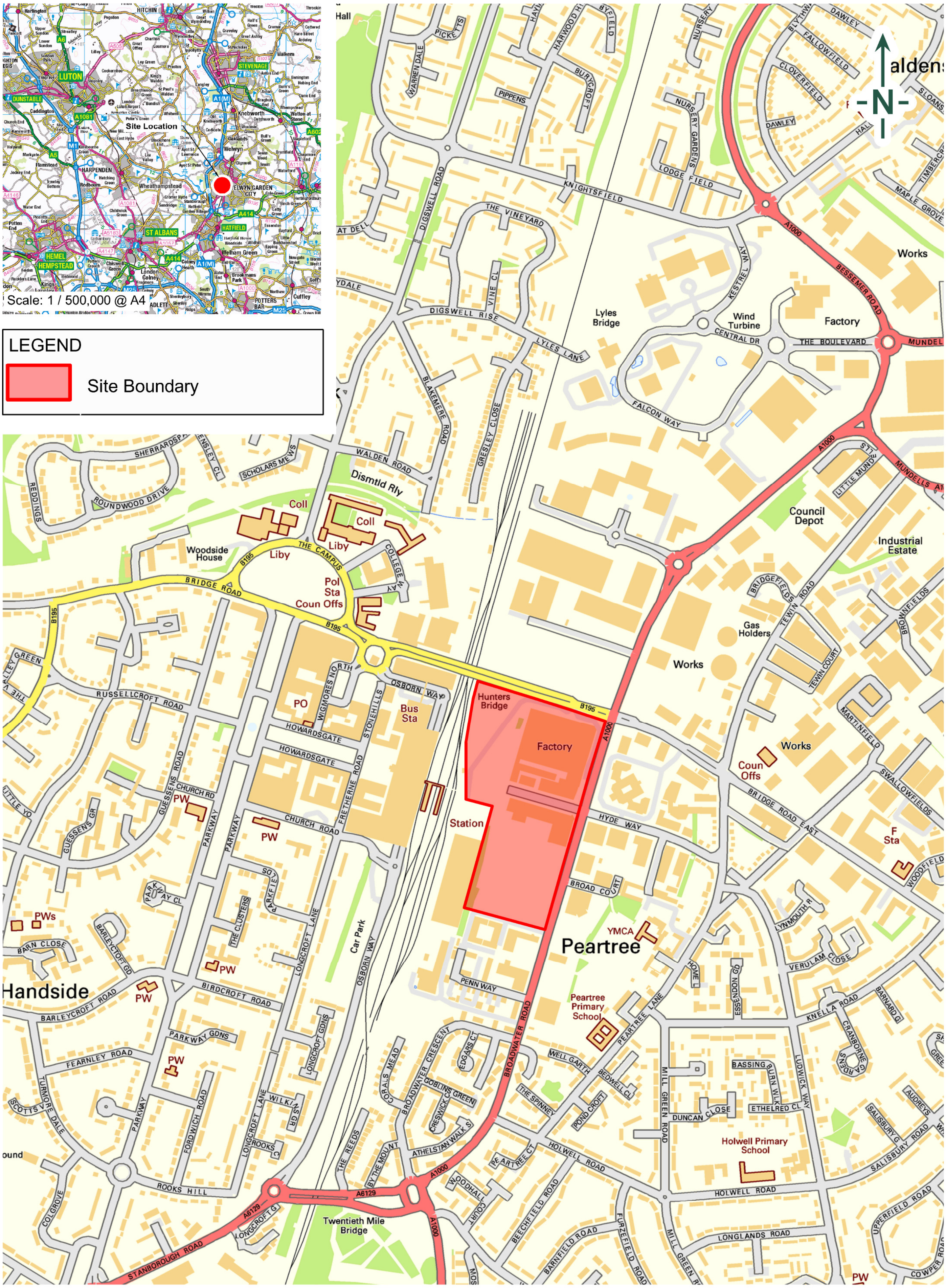
29th Jan 2015

James Harrison

Date

**Technical Director**

# Figures




Scale: 1 / 10,000 @ A4



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|---|---|---|---|---|
| deltasimons | TITLE: Site Location Map<br>Broadwater Road<br>Welwyn Garden City | DWN: DE | SCA: To Scale@A4 | PROJECT NO.: 2342.18 |
| | | CHK: SS | REV: 1 | FIGURE NO.: 1 |
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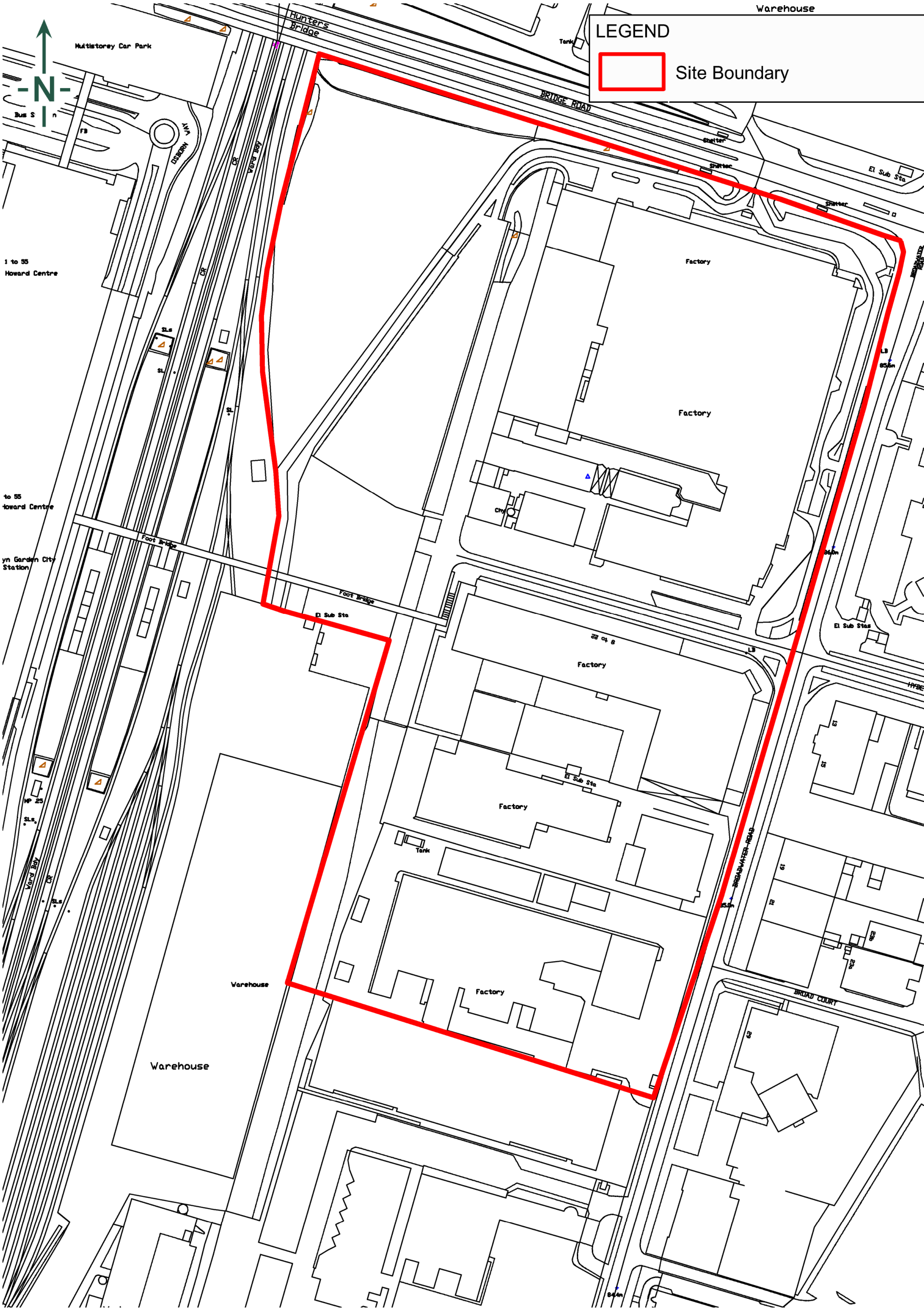


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|---|---|---|---|---|
| deltasimons | TITLE: Site Layout Plan<br>Broadwater Road<br>Welwyn Garden City | DWN: DE | SCA: Not to Scale | PROJECT NO.: 2342.18 |
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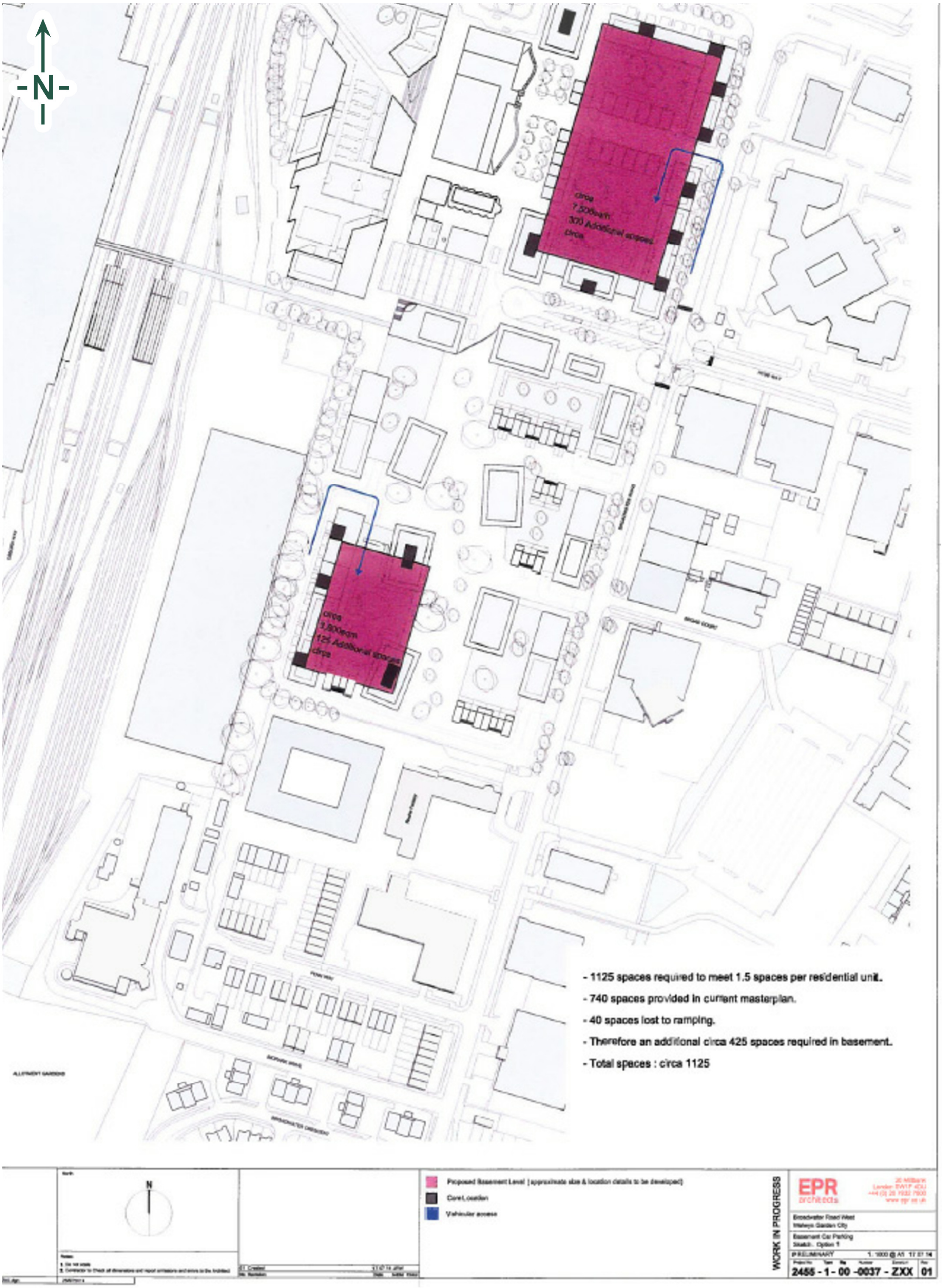
| | | DATE: 13 November 2014 | | FIGURE NO.: 2 |

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|---|---|---|---|
| deltasimons | TITLE: Proposed Development Plan<br>Broadwater Road<br>Welwyn Garden City | DWN: DP | SCA: Not to Scale |
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| | | DATE: 13 November 2014 | |
| | | PROJECT NO.: 2342.18 | FIGURE NO.: 3 |