Our ref: SCL/PC/9194

Your ref:

Date: 18th December 2017

Mr R Lester
Planning Officer
London Borough of Camden
2nd Floor, 5 Pancras Square
London N1C 4AG

Dear Sir,

re: **1 Lyndhurst Road, London, NW3 5PX**
Ref: 2017/1822/NEW
Basement Impact Assessment Audit Ref 12466-98

We are in receipt of Campbell Reith's Basement Impact Assessment Audit 12466-98 dated October revision D1 which has requested some clarification and additional information for our Basement Impact Assessment.

We have reviewed the above and attach our response and the additional information as requested.

We requested that GCG undertake a hydrogeological assessment and attach their letter dated 14th November which confirms that the proposed basement extension is not expected to have adverse effects on the local hydrology.

Attached is an outline construction programme.

We have reviewed Camden Geological, Hydrogeological and Hydrological study maps and attach the relevant maps with the site location indicated. A review of the maps confirms the proposed basement will not have any adverse effects on geological, hydrogeological or hydrological aspects.

We have reviewed Camden Strategic Flood Risk Assessment and attach the relevant maps which confirm the site is outside any critical drainage area.

We have updated our calculation package as attached which provides retaining wall design, deflections at construction and permanent stages, Ground Movement Assessment and Damage Impact Assessment.

Yours faithfully,
TAYLOR WHALLEY SPYRA

SIMON LANE

Encs:
Taylor Whalley Spyra responses to check list items 1 to 28 of Campbell Reith Basement Impact Assessment Audit 12466-98 dated October revision D1.
Taylor Whalley Spyra calculations package.
Geotechnical Consulting Group Hydrogeological Assessment dated 14th November 2017.
Camden Geological, Hydrogeological and Hydrological Study Maps extracts 1 to 4, 8, & 10 to 18.
Camden Strategic Flood Risk Assessment Maps 2, 4iv & 6.
Construction programme.

3 Dufferin Avenue,
Barbican, London, EC1Y 8PQ
T: 020 7253 2626
F: 020 7253 2767
E: tws@tws.uk.com
W: www.tws.uk.com

1 LYNDHURST ROAD
CAMDEN
LONDON NW3 5PX

TAYLOR WHALLEY SPYRA RESPONSES TO CHECK LIST ITEMS 1 TO 28 OF CAMPBELL REITH BASEMENT IMPACT ASSESSMENT AUDIT 12466-98 DATED OCTOBER REVISION D1

Report Ref	Revision	Notes	Issued by	Date
9194_GB	2.0	Issued for Planning	GB	18th December 2017

ITEM 1 - ARE BIA AUTHOR(S) CREDENTIALS SATISFACTORY?

The BIA report is authored by Simon Lane who is qualified as BSc(Eng), CEng, FICE, FIStructE.

The attached Hydrogeological Assessment is authored by Dr J. Skipper who is qualified as BSc, PhD, DIC, CGeol.

ITEM 2 - IS DATA REQUIRED BY CL.233 OF THE GSD PRESENTED?

An outline construction programme is attached.

Utilities companies have not been approached as initial desktop review would indicate the works are located at the rear of the property and no services are located to the side or rear in the raised garden, the adjacent rear garden basement of 2 Lyndhurst Road extends further back so would obstruct any below ground services .

Network Rail and TFL have confirmed that they have no infrastructure in the area of the works.

ITEM 3 - DOES THE DESCRIPTION OF THE PROPOSED DEVELOPMENT INCLUDE ALL ASPECTS OF TEMPORARY AND PERMANENT WORKS WHICH MIGHT IMPACT UPON GEOLOGY, HYDROGEOLOGY AND HYDROLOGY?

No further comments.

ITEM 4 - ARE SUITABLE PLANS/MAPS INCLUDED?

The attached Geotechnical Consulting Group Hydrological Assessment dated 14th November 2017 has reviewed Camden Maps/SFRA Maps and concludes that the proposed basement extension is not expected to have adverse effects on the local hydrology.

Attached is a set of Camden GH&HS maps and Camden SFRA maps with the site location indicated.

ITEM 5 - DO THE PLANS/MAPS SHOW THE WHOLE OF THE RELEVANT AREA OF STUDY AND DO THEY SHOW IT IN SUFFICIENT DETAIL?

Attached is a set of Camden GH&HS maps and Camden SFRA maps with the site location indicated.

Refer to item 4 above.

ITEM 6 - LAND STABILITY SCREENING: HAVE APPROPRIATE DATA SOURCES BEEN CONSULTED? IS JUSTIFICATION PROVIDED FOR ‘NO’ ANSWERS?

Attached is a set of GH&HS maps and Camden SFRA maps with the site location indicated.

ITEM 7 - HYDROGEOLOGY SCREENING: HAVE APPROPRIATE DATA SOURCES BEEN CONSULTED? IS JUSTIFICATION PROVIDED FOR ‘NO’ ANSWERS?

Attached is a set of GH&HS maps and Camden SFRA maps with the site location indicated.

Refer to item 4 above.

ITEM 8 - HYDROLOGY SCREENING: HAVE APPROPRIATE DATA SOURCES BEEN CONSULTED? IS JUSTIFICATION PROVIDED FOR ‘NO’ ANSWERS?

Reference to Camden GH&HS maps confirms the site is outside of the Hampstead Heath Surface Water Catchments and Drainage.

Reference to Camden SFRA confirms the site is not within a Critical Drainage Area, see attached SFRA maps Figures 2, 3iv and 6.

ITEM 9 - IS A CONCEPTUAL MODEL PRESENTED?

No further comments.

3 Dufferin Avenue, Barbican, London, EC1Y 8PQ: T: 020 7253 2626 F: 020 7253 2767 E: tws@tws.uk.com W: www.tws.uk.com

ITEM 10 - LAND STABILITY SCOPING PROVIDED? IS SCOPING CONSISTENT WITH SCREENING OUTCOME?

Attached is a set of Camden maps with the site location indicated which confirm the site is not in a slope angle area greater than 7 deg. The site and surrounding area are less than 5 degs as noted in the BIA.
The Arboricultural Report dated 16th March 2017 confirms the Silver Birch (T1) as having a shallow root system and sets out tree and root protection measures that will be adopted as part of the construction process involving supervised excavation by an Arboriculturist.

The report confirms that the works will have a neutral effect on the trees.

The tree has a shallow roots zone and is located at a higher raised garden level of 50.4 above the existing ground floor of 50.0, so will not have any effect on the proposed basement which is 46.95. The existing extension has a 700mm void under with a brick retaining wall between (see BIA & TWS drg 9194_BIA02 section 3a _ 3a).

ITEM 11 - HYDROGEOLOGY SCOPING PROVIDED? IS SCOPING CONSISTENT WITH SCREENING OUTCOME?

The attached Geotechnical Consulting Group Hydrological Assessment dated 14th November 2017 has reviewed the Claygate member and River Tyburn and concludes that the proposed basement extension is not expected to have adverse effects on the local hydrology.

Attached is a set of Camden GH&HS maps and Camden SFRA maps with the site location indicated.

ITEM 12 - HYDROLOGY SCOPING PROVIDED? IS SCOPING CONSISTENT WITH SCREENING OUTCOME?

The attached Geotechnical Consulting Group Hydrological Assessment dated 14th November 2017 has reviewed Camden GH&HS Maps and SFRA Maps and concludes that the proposed basement extension is not expected to have adverse effects on the local hydrology.

Attached is a set of Camden GH&HS Maps and SFRA Maps with the site location indicated.

ITEM 13 - IS FACTUAL GROUND INVESTIGATION DATA PROVIDED?

No further comments.

ITEM 14 - IS MONITORING DATA PRESENTED?

No further comments.

ITEM 15 - IS THE GROUND INVESTIGATION INFORMED BY A DESK STUDY?

No further comments.

ITEM 16 - HAS A SITE WALKOVER BEEN UNDERTAKEN?

No further comments.

ITEM 17 - IS THE PRESENCE/ABSENCE OF ADJACENT OR NEARBY BASEMENTS CONFIRMED?

No further comments.

ITEM 18 - IS A GEOTECHNICAL INTERPRETATION PRESENTED?

Refer to the attached Taylor Whalley Spyra calculation package.

ITEM 19 - DOES THE GEOTECHNICAL INTERPRETATION INCLUDE INFORMATION ON RETAINING WALL DESIGN?

The design of the retaining walls has been undertaken using SCIA Engineer 17 and TEDDS. These are based on geotechnical information from the site SI and information from an adjoining site SI in Lyndhurst Road and from previous experience of undertaking basements in the area. Refer to the attached Taylor Whalley Spyra calculation package.

3 Dufferin Avenue, Barbican, London, EC1Y 8PQ: T: 020 7253 2626 F: 020 7253 2767 E: tws@tws.uk.com W: www.tws.uk.com

ITEM 20 - ARE REPORTS ON OTHER INVESTIGATIONS REQUIRED BY SCREENING AND SCOPING PRESENTED?

The Arboricultural Report dated 16th March 2017 confirms the Silver Birch (T1) as having a shallow root system and sets out tree and root protection measures that will be adopted as part of the construction process involving supervised excavation by an Arboriculturist.

The report confirms that the works will have a neutral effect on the trees. The tree has a shallow roots zone and is located at a higher raised garden level of 50.4 above the existing ground floor of 50.0, so will not have any effect on the proposed basement which is 46.95. The existing extension has a 700mm void under with a brick retaining wall between (see TWS drg 9194_BIA02 section 3a _ 3a).

For Ground Movement Assessment and Damage Impact Assessment refer to the attached Taylor Whalley Spyra calculation package.

ITEM 21 - ARE BASELINE CONDITIONS DESCRIBED, BASED ON THE GSD?

No further comments.

ITEM 22 - DO THE BASE LINE CONDITIONS CONSIDER ADJACENT OR NEARBY BASEMENTS?

No further comments.

ITEM 23 - IS AN IMPACT ASSESSMENT PROVIDED?

No further comments.

ITEM 24 - ARE ESTIMATES OF GROUND MOVEMENT AND STRUCTURAL IMPACT PRESENTED?

For Ground Movement Assessment and Damage Impact Assessment refer to the attached Taylor Whalley Spyra calculation package attached.

ITEM 25 - IS THE IMPACT ASSESSMENT APPROPRIATE TO THE MATTERS IDENTIFIED BY SCREEN AND SCOPING?

The additional review of the hydrogeological assessment, Camden GH&HS Maps and SFRA Maps, does not change the BIA Assessment and the temporary works sequence, construction sequence and monitoring will remain unchanged.

ITEM 26 - HAS THE NEED FOR MITIGATION BEEN CONSIDERED AND ARE APPROPRIATE MITIGATION METHODS INCORPORATED IN THE SCHEME?

Movement and damage has been assessed within the calculations and the proposed form of construction and the additional review of the hydrogeological assessment, Camden GH&HS Maps and SFRA Maps, does not change the BIA Assessment and the temporary works sequence, construction sequence and monitoring will remain unchanged.

ITEM 27 - HAS THE NEED FOR MONITORING DURING CONSTRUCTION BEEN CONSIDERED?

The recommendations for monitoring are based on the form of construction and the movement criteria as designed in the calculations and ground movement. The risks are considered to be minimal.

ITEM 28 - HAVE THE RESIDUAL (AFTER MITIGATION) IMPACTS BEEN CLEARLY IDENTIFIED?

The additional review of the hydrogeological assessment, Camden GH&HS Maps and SFRA Maps does not change the BIA Assessment and the temporary works sequence, construction sequence and monitoring will remain unchanged. The risks are considered to be minimal.

3 Dufferin Avenue, Barbican, London, EC1Y 8PQ: T: 020 7253 2626 F: 020 7253 2767 E: tws@tws.uk.com W: www.tws.uk.com

1 LYNDHURST ROAD

CAMDEN, LONDON
NW3 5PX

PRELIMINARY CALCULATIONS

ICT/9194/Rev.B DECEMBER 2017

3 Dufferin Avenue,
Barbican, London, EC1Y 8PQ
T: 020 7253 2626
F: 020 7253 2767
E: tws@tws.uk.com
W: www.tws.uk.com

CONTENTS

1. INTRODUCTION

We have carried out structural calculations for 1 Lyndhurst Road. Generally these preliminary calculations aims to present buildability of the proposed scheme within safety criterias of structural codes and common construction practice.

Existing building is a masonry structure with timber floors. Proposed scheme supports the rear corner wall with two double steel beams to build the new lower rear ground level extension. Reinforced concrete extension box will be built according to the defined sequence, details of which can be seen in related TWS drawing. A list of the loads used in this calculation can be seen at section 2 of this document.

Steel support beams has been designed individually and isolated from the interaction of the slab to stay at the safe side for wall deflections and beam sizing. We took a staged analysis and design approach for the design of the RC box. Lateral wall deflections in this document doesn't consider support of the new lower rear ground floor level slab. Reinforcement design section of the document shows a buildable reinforcement amount achieved at the end of the design process.

We have carried out a damage assessment for the existing 1 Lyndhurst Road building and adjasent buildings. Damage assessment is generally following the principles of Ciria C760 however due to the nature of the phased construction method, type of ground retaining structure being a concrete box walls and excavation being done in claygate and made ground soils we have used a structural deformation based method. We have considered an assumed ground movement for the excavation stage and superimposed the structural deformationsof the walls based on our FEM analysis.

Heave section is showing how loads are balanced between new and old load distribution at the excavation level. As the result of our calculation we don't expect any significant heave effect on the structure. For the more detailed final design we can use heave forms to reduce heave effects if required.

We have used Tedds for the individual design elements and SCIA Engineer 17 for the finite element modeling analysis and structural design of the concrete box.

Standards and regulations

BS EN 1991-1-1: General Actions – Densities, self-weight, imposed loads for buildings
BS EN 1992-1-1: Design of Concrete Structures
BS EN 1997-1: Geotechnical Design
Along with the above structural codes, UK national annexes have been considered where relevant.

2. LOADING

2.1. Dead Loads

Timber Floors	
Timber Stud Light Partitions	: 1.00kN/m^2
Boads + Joists	: 0.35kN/m^2
Soffite	: 0.15kN/m^2
Total	*: 1.50kN/m^2*

250mm RC Slab	
Finishes	: 0.50kN/m^2
250mm RC Slab	: 6.25kN/m^2
Insulation	: 0.25kN/m^2
Total	: 7.00kN/m^2

Green Roof	
Green Roof Soiland Plants	: 18x0.75= 13.5
250mm RC Slab	: 6.25kN/m^2
Insulation	: 0.25kN/m^2
Total	: 20kN/m^2

Roof	
Tiles	: 0.6kN/m^2
Battens + Felt	: 0.1kN/m^2
Rafters	: 0.15kN/m^2
Total	: 0.85kN/m^2

External Walls	
Plaster	: 0.25kN/m^2
315mm Brick Wall	: 18kN/m^3 x 0.315 = 5.7kN/m^2
Total	*: 5.95kN/m^2*

2.2. Live Load

Residential : 1.5kN/m2

2.3. Ground Loads

Soil investigation report didn't encounter water during the inspections. In this document ground water table has been considered 1m below the ground level and water pressure applied to walls and ground slab. Ground water loading considered as an accidental case.

Ground loads has been applied to walls where applicable considering 18kN/m^3 soild density. For the calculation purposes coefficient of active pressure K_a has been considered as 0.30.

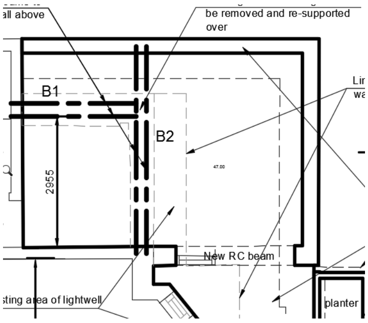
We have considered 5kN/m^2 surcharge load at the ground level for all walls.

2.4. Combinations

Combinations Name	Type	Load cases	Coeff. [-]
ULS - Construction	Linear - ultimate	SW - construction	1.35
		DL - construction	1.35
		LL - construction	1.5
		GL - construction	1.35
		WL - construction	1.05
SLS - Construction	Linear - serviceability	SW - construction	1
		DL - construction	1
		LL - construction	1
		GL - construction	1
		WL - construction	1
ULS - permanent	Linear - ultimate	SW - permanent	1.35
		DL - permanent	1.35
		LL - permanent	1.5
		GL - permanent	1.35
		WL - permanent	1.05
SLS - permanent	Linear - serviceability	SW - permanent	1
		DL - permanent	1
		LL - permanent	1
		GL - permanent	1
		WL - permanent	1
SLS - permanent without water uplift	Linear - serviceability	SW - permanent	1
		DL - permanent	1
		LL - permanent	1
		GL - permanent	1

SW : Self weight
DL : Dead Load
LL : Live Load
GL : Ground Load
WL : Ground Water Load

3. DESIGN OF STEEL SUPPORT BEAMS



3.1. BEAM B1

Loads on Beam B1

<u>Dead Loads</u>

Ground floor	: 1.5kN/m^2 x 4.7m / 2 = 3.5kN/m
Ground F. Green Roof	: 20kN/m^2 x 1.2m / 2 = 12.0kN/m
1st Floor	: 1.5kN/m^2 x 3.0m / 2 = 2.3kN/m
2nd Floor	: 1.5kN/m^2 x 3.0m / 2 = 2.3kN/m
3rd Floor (penthouse)	: 1.5kN/m^2 x 3.0m / 2 = 2.3kN/m
External Wall	: 5.7kN/m^2 x 9.3m = 53kN/m
Roof	: 0.85kN/m^2 x 4.0m / 2 = 1.7kN/m
Total	*: 77.1kN/m*

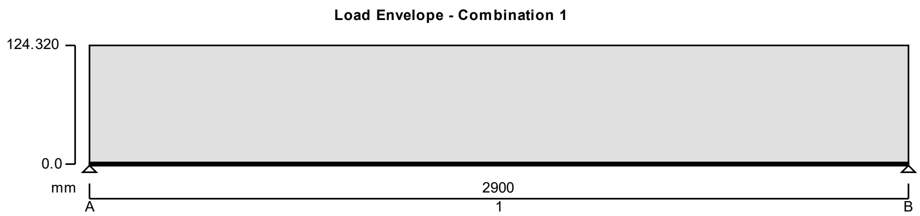
<u>Live Loads</u>

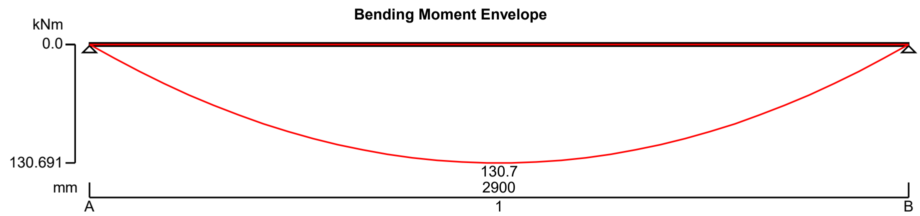
Ground floor	: 1.5kN/m^2 x 4.7m / 2 = 3.5kN/m
1st Floor	: 1.5kN/m^2 x 3m / 2 = 2.3kN/m
2nd Floor	: 1.5kN/m^2 x 3m / 2 = 2.3kN/m
3rd Floor (penthouse)	: 1.5kN/m^2 x 3m / 2 = 2.3kN/m
Roof	: 0.60kN/m^2 x 4.0m / 2 = 1.2kN/m
Total	: 11.6kN/m

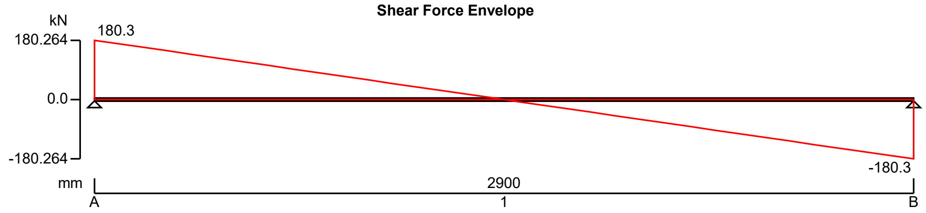
<u>STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)</u>

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 77.1 kN/m
	Variable full UDL 11.6 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

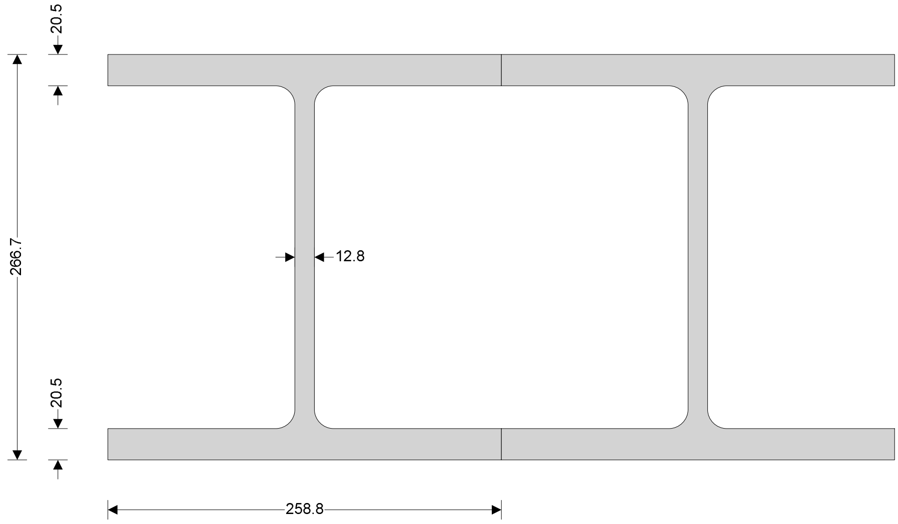
Maximum moment;	M_{max} = **130.7** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **180.3** kN;	V_{min} = **-180.3** kN
Deflection;	δ_{max} = **1.1** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **180.3** kN;	R_{A_min} = **180.3** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **114.8** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **16.8** kN	
Maximum reaction at support B;	R_{B_max} = **180.3** kN;	R_{B_min} = **180.3** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **114.8** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **16.8** kN	

Section details

Section type;	**2 x UKC 254x254x107 (Tata Steel Advance)**
Steel grade;	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element;	$t = \max(t_f, t_w)$ = **20.5** mm
Nominal yield strength;	f_y = **265** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1.00**

Resistance of members to instability; γ_{M1} = **1.00**

Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = **1.000**

Effective length factor in minor axis; K_z = **1.000**

Effective length factor for torsion; $K_{LT.A}$ = **1.000**;

$K_{LT.B}$ = **1.000**;

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.94**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; c = d = **200.3** mm

$c / t_w = 16.6 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 5.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f$ = **225.7** mm

Shear area factor; η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = max(abs(V_{max}), abs(V_{min}))$ = **180.3** kN

Shear area - cl 6.2.6(3); $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3811** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **1166** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **130.7** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **786.7** kNm

3 Dufferin Avenue, Barbican, London, EC1Y 8PQ
T: 020 7253 2626 **F:** 020 7253 2767
E: tws@tws.uk.com **W:** www.tws.uk.com

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio; ν = **0.3**

Shear modulus; $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length; $L = 1.0 \times L_{s1}$ = **2900** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **3193.9** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.351**

Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ < $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250$ = **11.6** mm

Maximum deflection span 1; $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **1.137** mm

PASS - Maximum deflection does not exceed deflection limit

3.2. BEAM B2

Loads on Beam B2

Dead Loads

Ground floor	: 6.5kN/m^2 x 3.9m / 2 = 12.7kN/m
1st Floor Extension Roof	: 0.85kN/m^2 x 3.9m / 2 = 1.7kN/m
External Wall	: 5.7kN/m^2 x 9.3m = 53kN/m
Roof	: 0.85kN/m^2 x 4.0m / 2 = 1.7kN/m
Total	*: 69.1kN/m*

Point Load from Beam B1 : 112.7kN

Live Loads

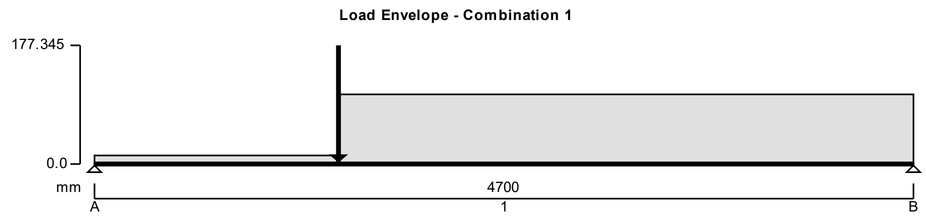
Ground floor	: 1.5kN/m^2 x 3.9m / 2 = 2.9kN/m
1st Floor extension roof	: 0.60kN/m^2 x 3.9m / 2 = 1.2kN/m
Roof	: 0.60kN/m^2 x 4.0m / 2 = 1.2kN/m
Total	: 5.3kN/m

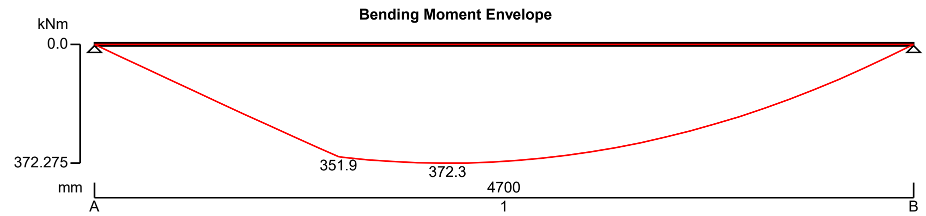
Point Load from Beam B1 : 16.8kN

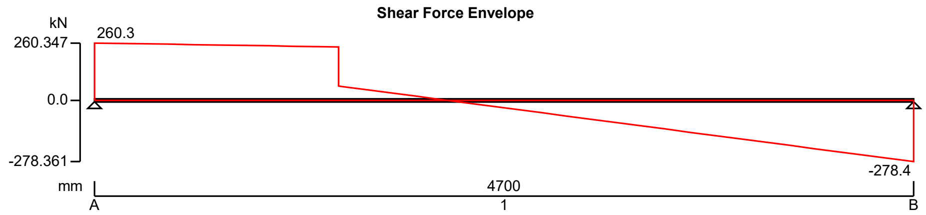
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13







Support conditions

Support A	Vertically restrained
	Rotationally free

Support B	Vertically restrained	
	Rotationally free	

Applied loading

Beam loads	Permanent self weight of beam × 1	
	Permanent partial UDL 69.1 kN/m from 1400 mm to 4700 mm	
	Variable partial UDL 5.3 kN/m from 1400 mm to 4700 mm	
	Permanent point load 112.7 kN at 1400 mm	
	Variable point load 16.8 kN at 1400 mm	
	Permanent partial UDL 3.5 kN/m from 0 mm to 1400 mm	
	Variable partial UDL 3.5 kN/m from 0 mm to 1400 mm	

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

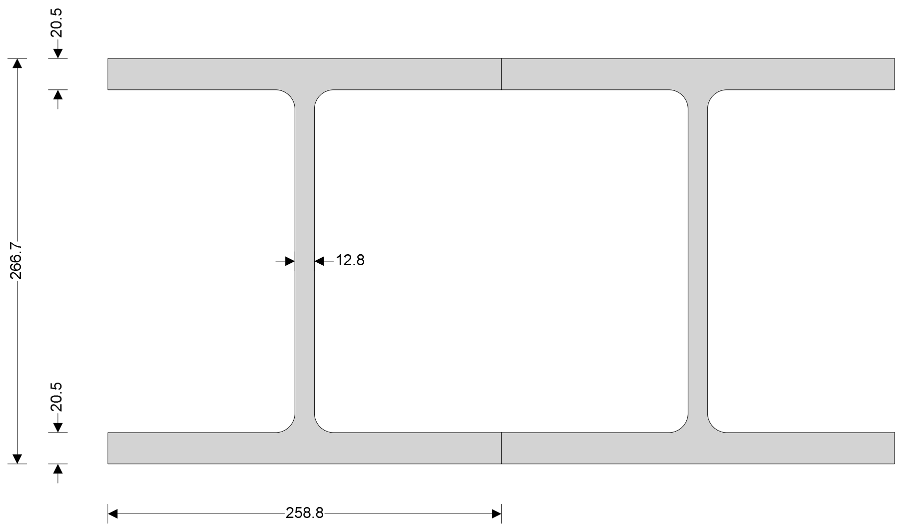
Maximum moment;	M_{max} = **372.3** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **260.3** kN;	V_{min} = **-278.4** kN
Deflection;	δ_{max} = **8.4** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **260.3** kN;	R_{A_min} = **260.3** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **168.3** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **22.1** kN	
Maximum reaction at support B;	R_{B_max} = **278.4** kN;	R_{B_min} = **278.4** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **187.2** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **17.1** kN	

Section details

Section type;	**2 x UKC 254x254x107 (Tata Steel Advance)**
Steel grade;	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element;	$t = \max(t_f, t_w)$ = **20.5** mm
Nominal yield strength;	f_y = **265** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1.00**

Resistance of members to instability; γ_{M1} = **1.00**

Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = **1.000**

Effective length factor in minor axis; K_z = **1.000**

Effective length factor for torsion; $K_{LT.A}$ = **1.000**;

$K_{LT.B}$ = **1.000**;

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.94**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section; c = d = **200.3** mm

$c / t_w = 16.6 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 5.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f$ = **225.7** mm

Shear area factor; η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **278.4** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3811** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **1166** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **372.3** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **786.7** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio; ν = **0.3**

Shear modulus; $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2

Unrestrained length; $L = 1.0 \times L_{s1}$ = **4700** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **1551.6** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.504**

Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; α_{LT} = **0.34**

Correction factor for rolled sections; β = **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.613**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.959**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.975**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.983**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl.y} \times f_y / \gamma_{M1}$ = **773.4** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

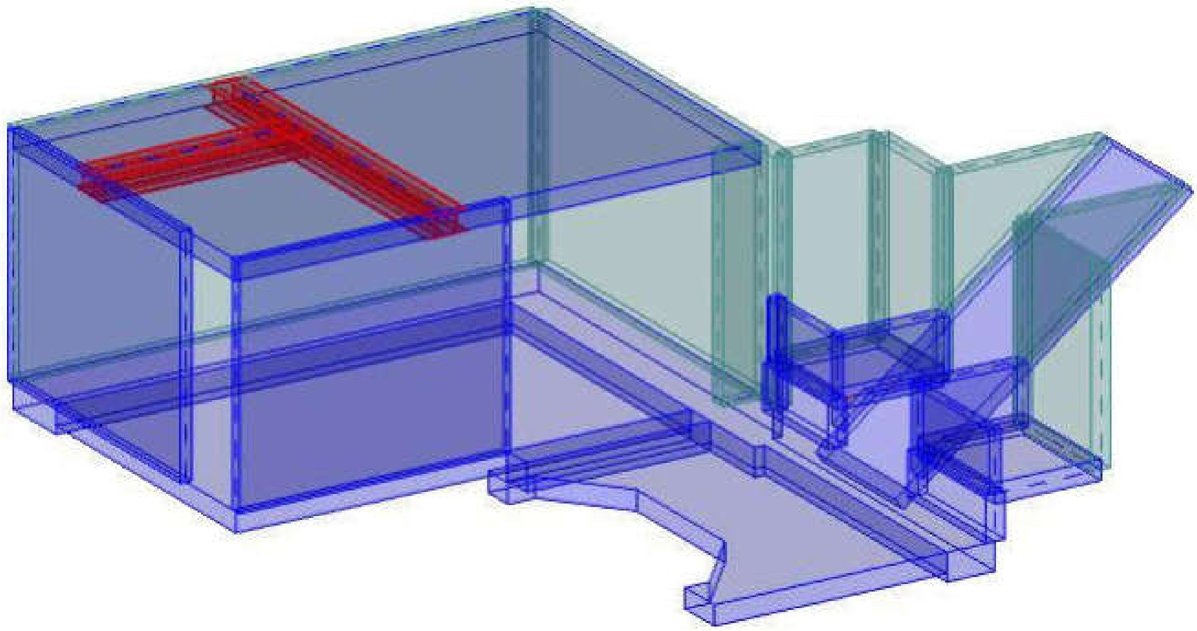
Consider deflection due to permanent and variable loads

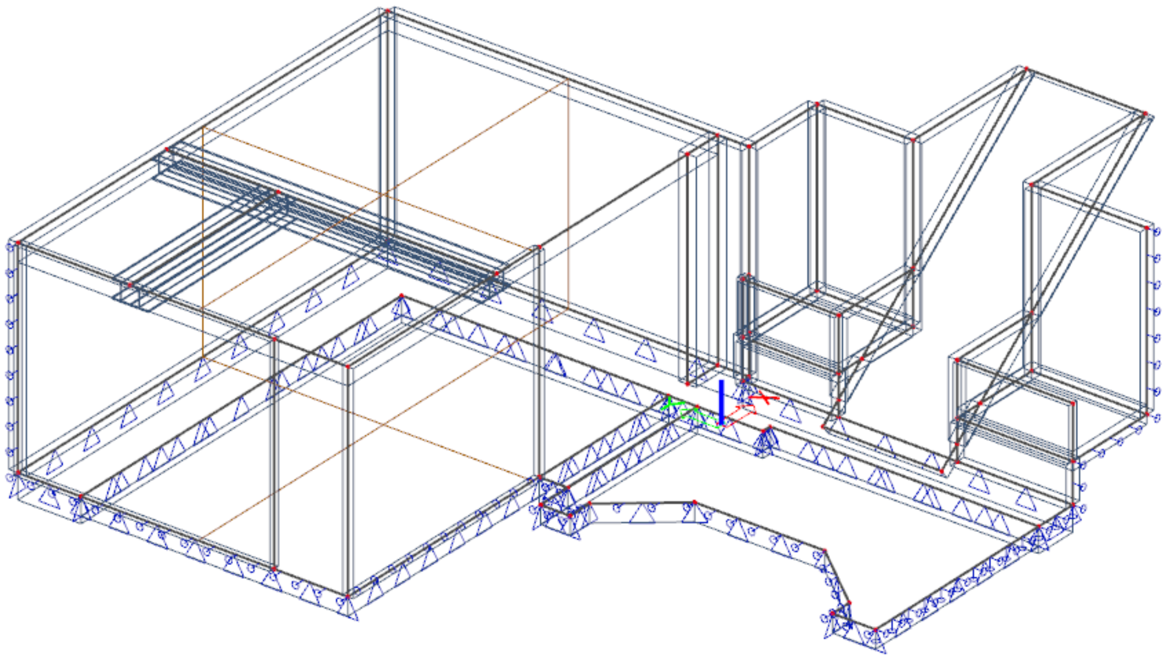
Limiting deflection; $\delta_{lim} = L_{s1} / 250$ = **18.8** mm

Maximum deflection span 1; $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **8.448** mm

PASS - Maximum deflection does not exceed deflection limit

4. DESIGN MODEL

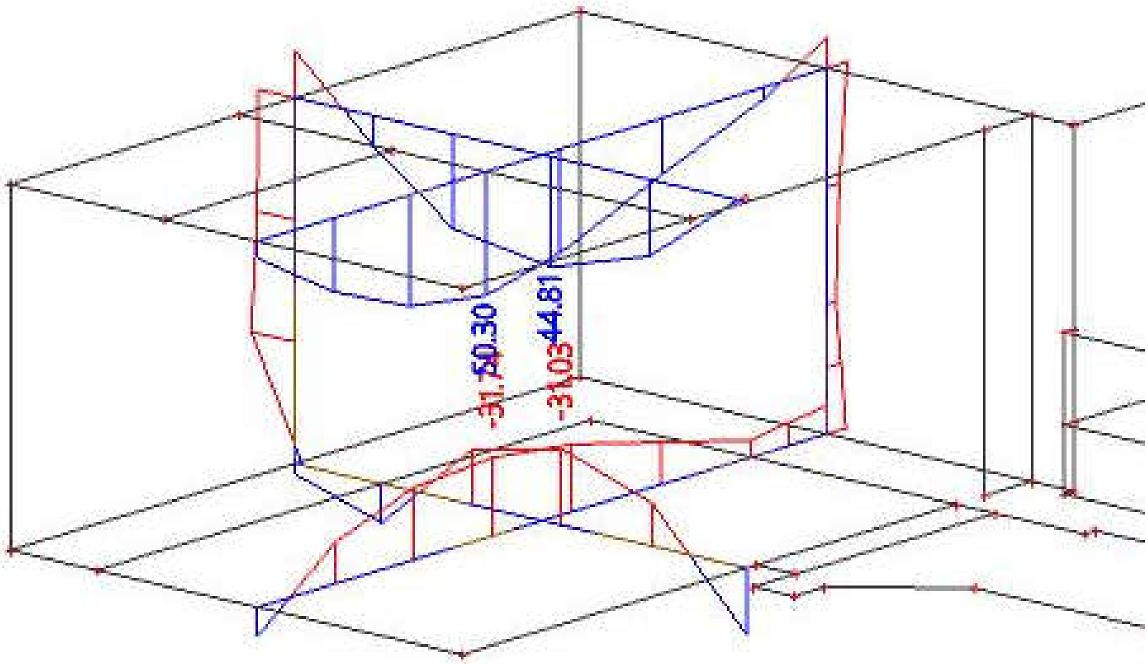




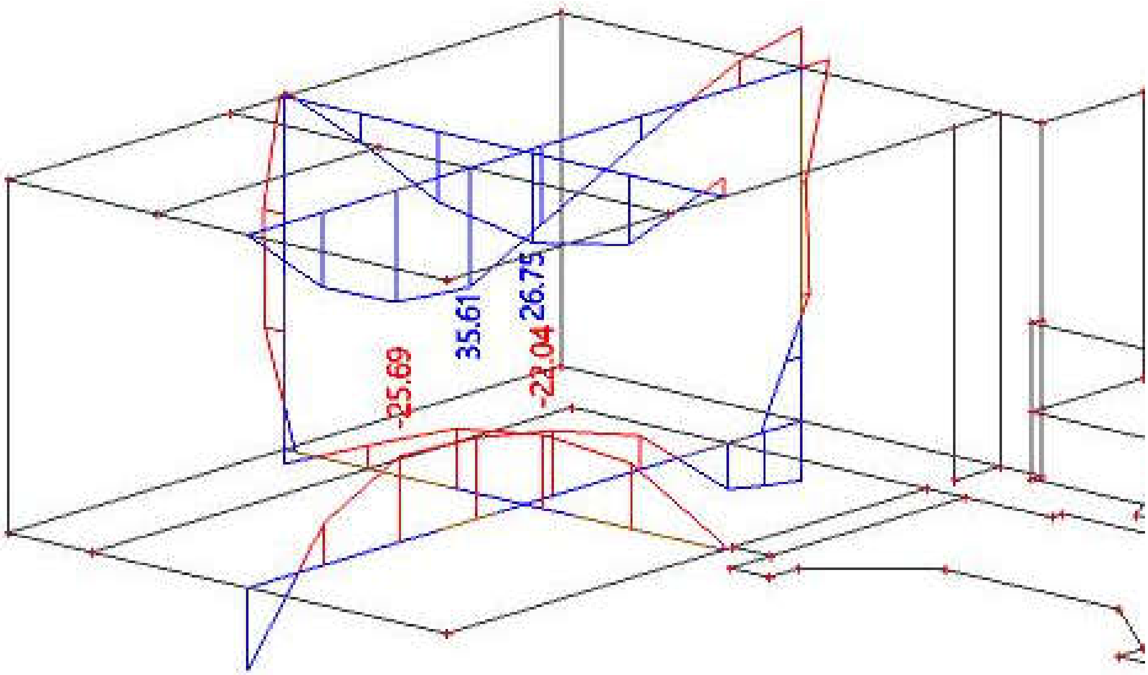
5. ANALYSIS RESULTS

5.1. Moments

Internal forces; my

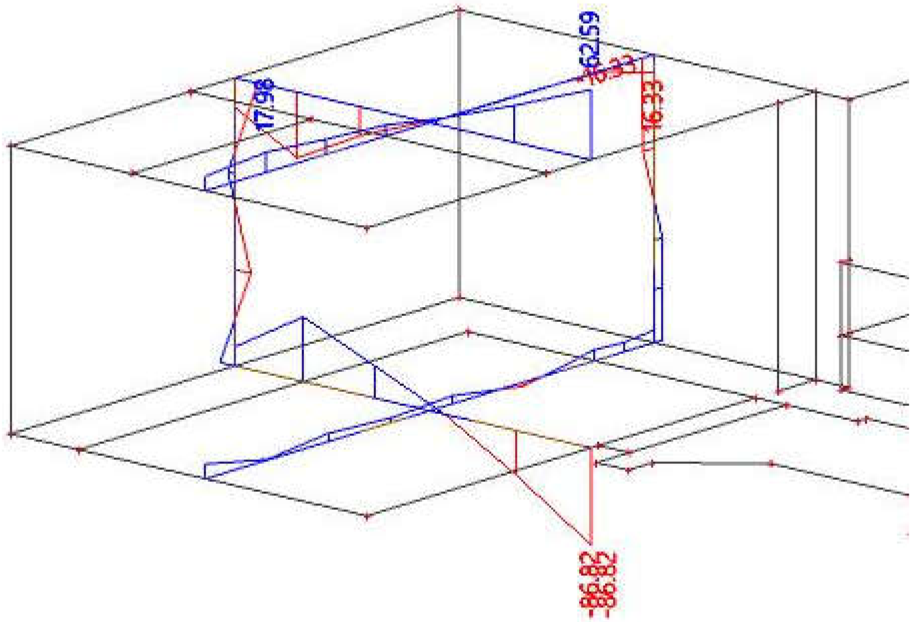


Internal forces; mx

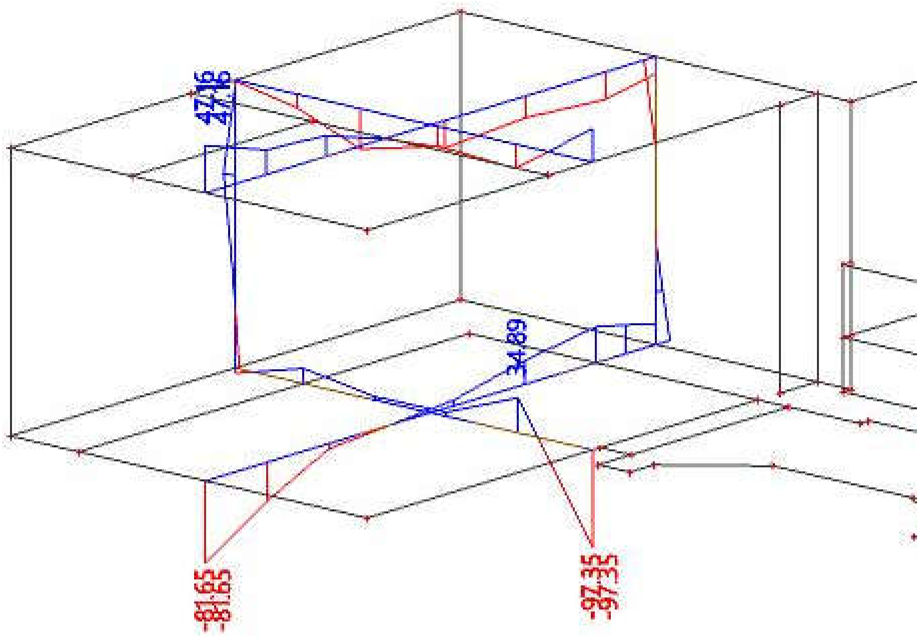


5.2. Shear

Internal forces; vy

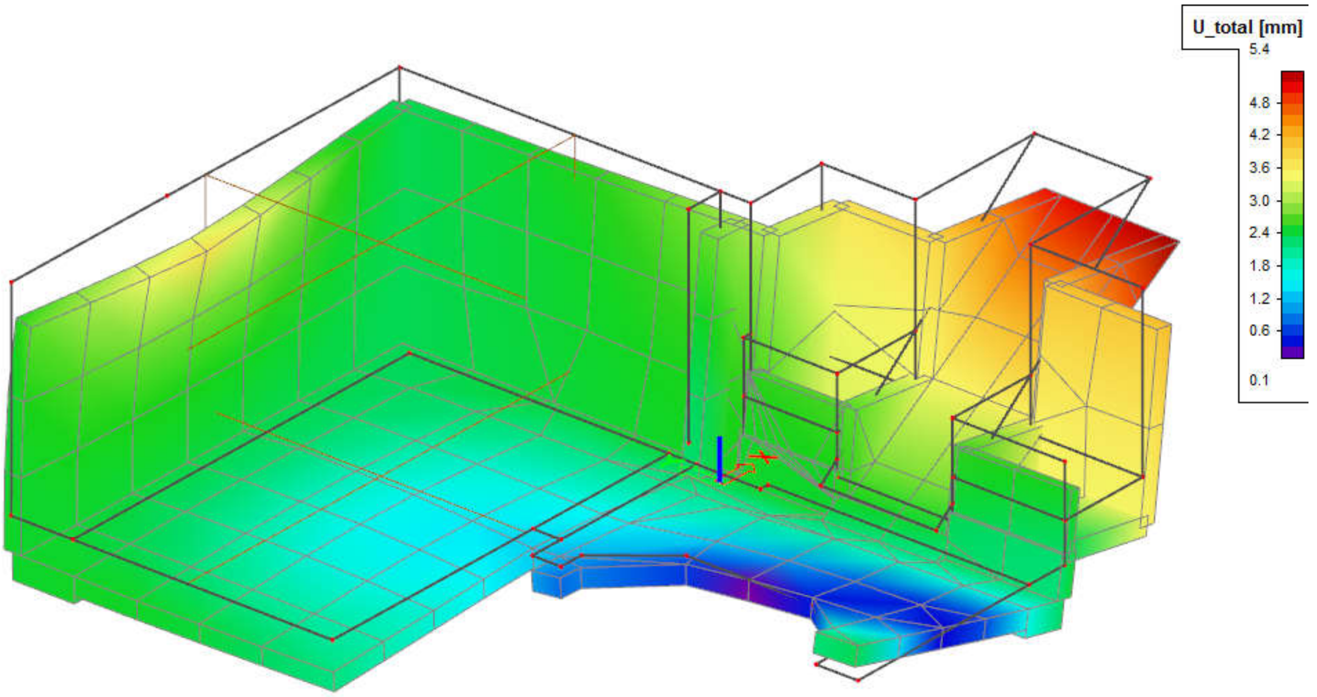


2D member - Internal forces; vx

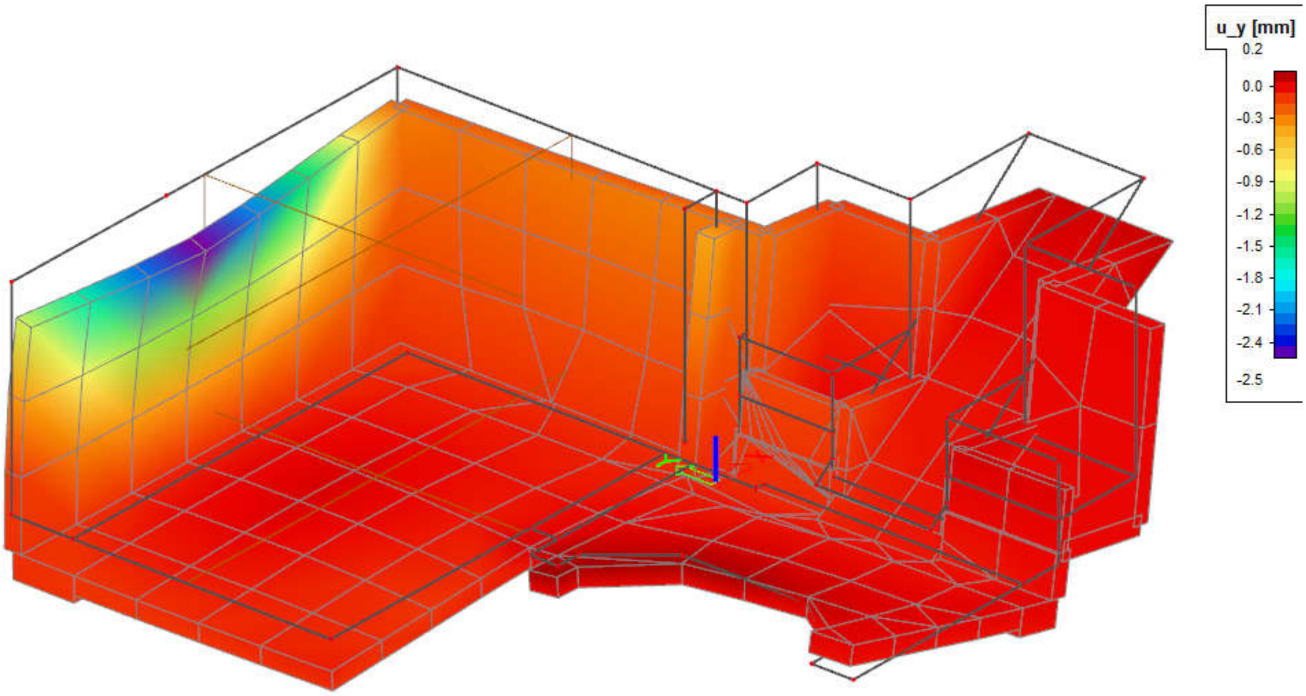


5.3. DEFLECTIONS

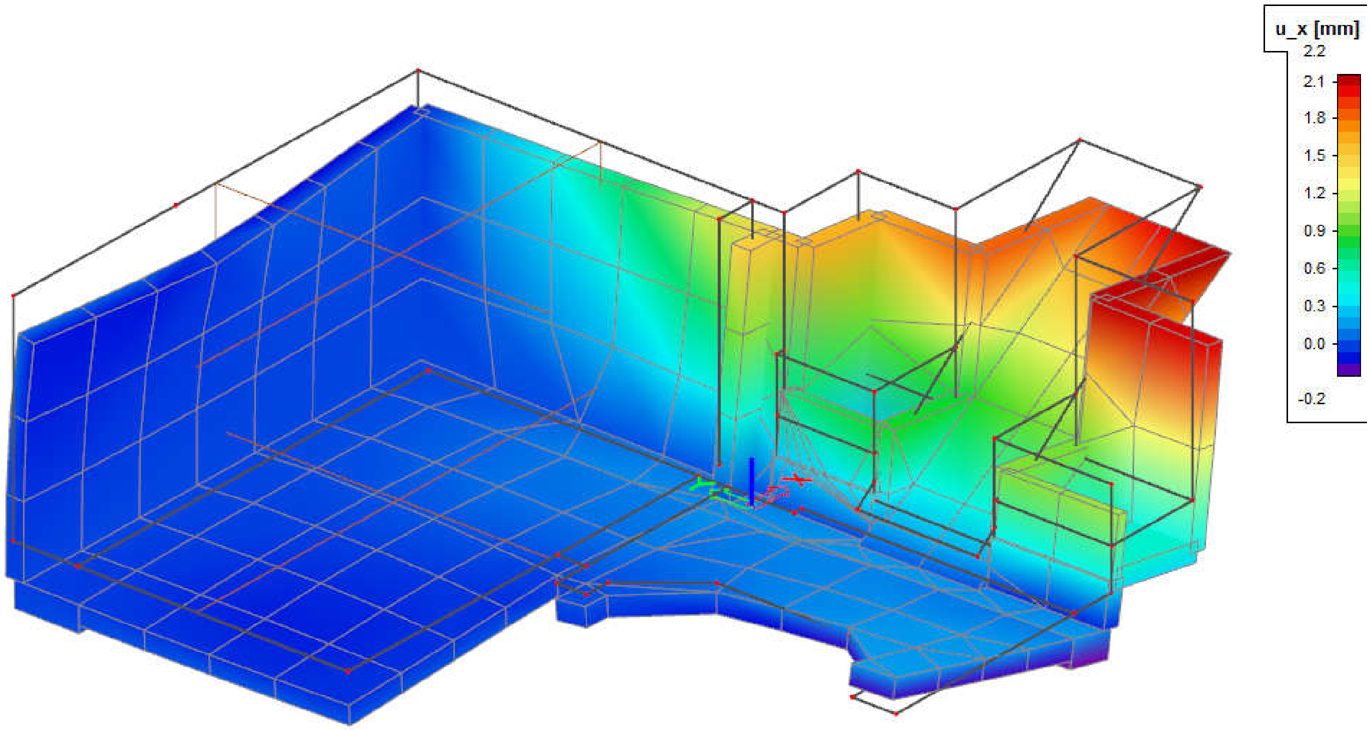
Total Deflections at Construction Stage



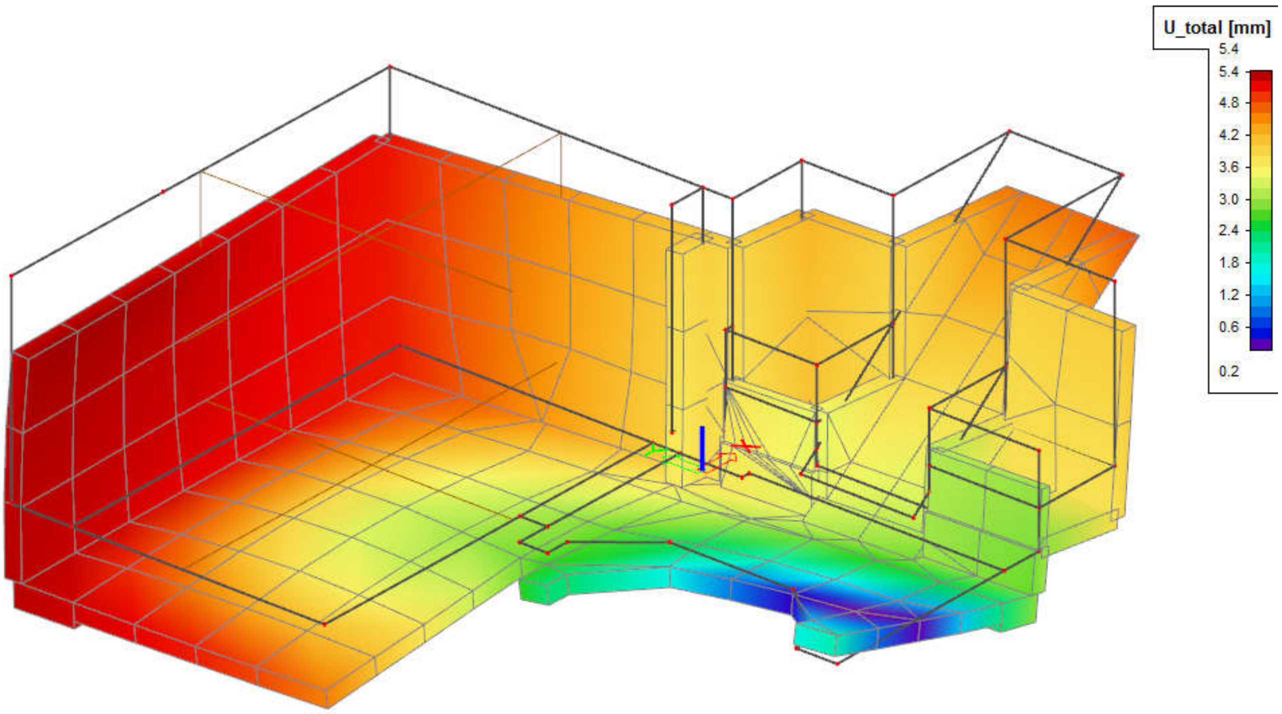
Deflections at Construction Stage (y direction only)



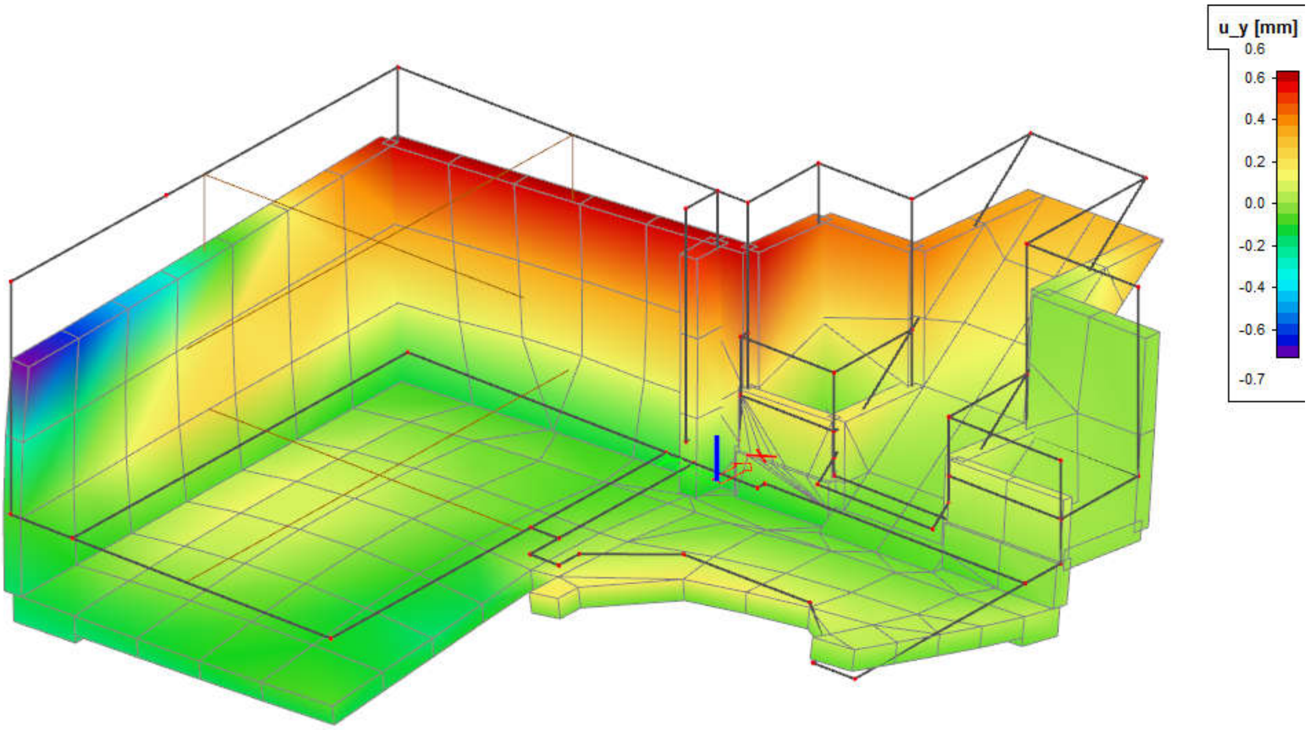
Deflections at Construction Stage (x direction only)



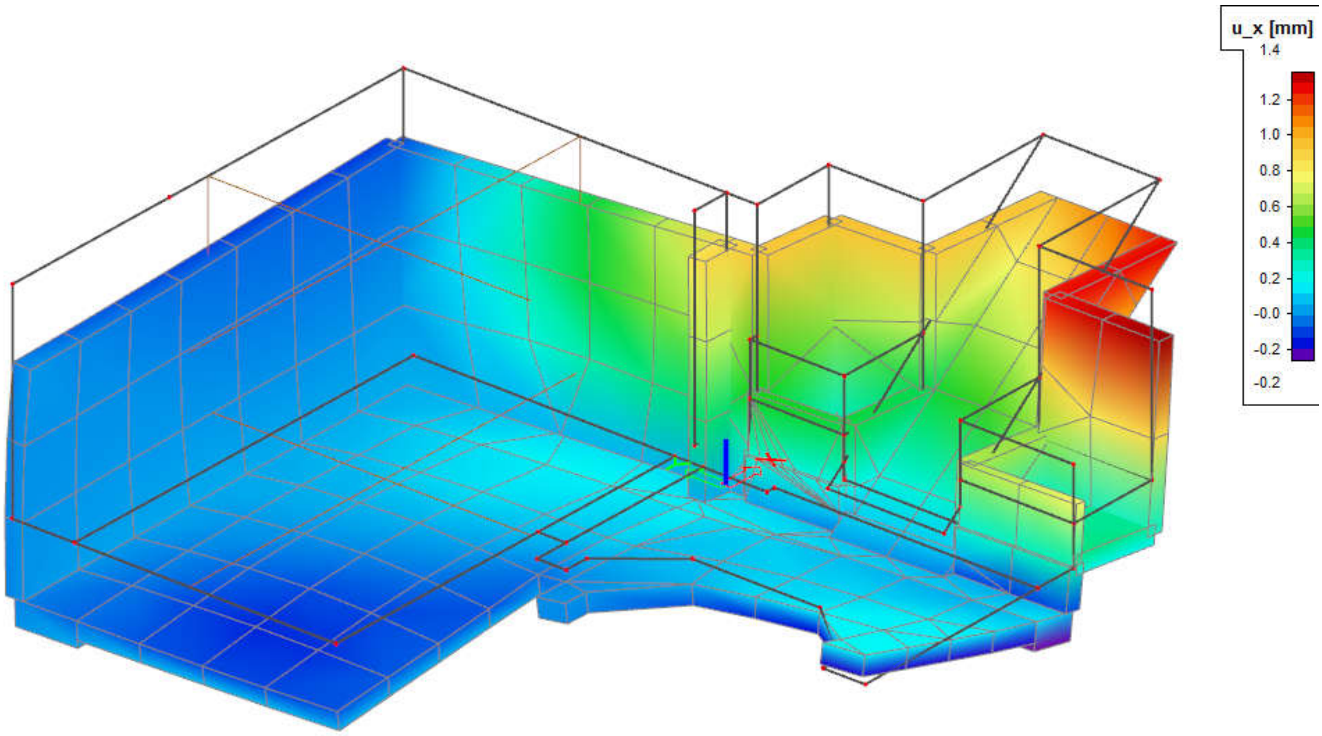
Total Deflections at Permanent Stage



Deflections at Permanent Stage (y direction only)

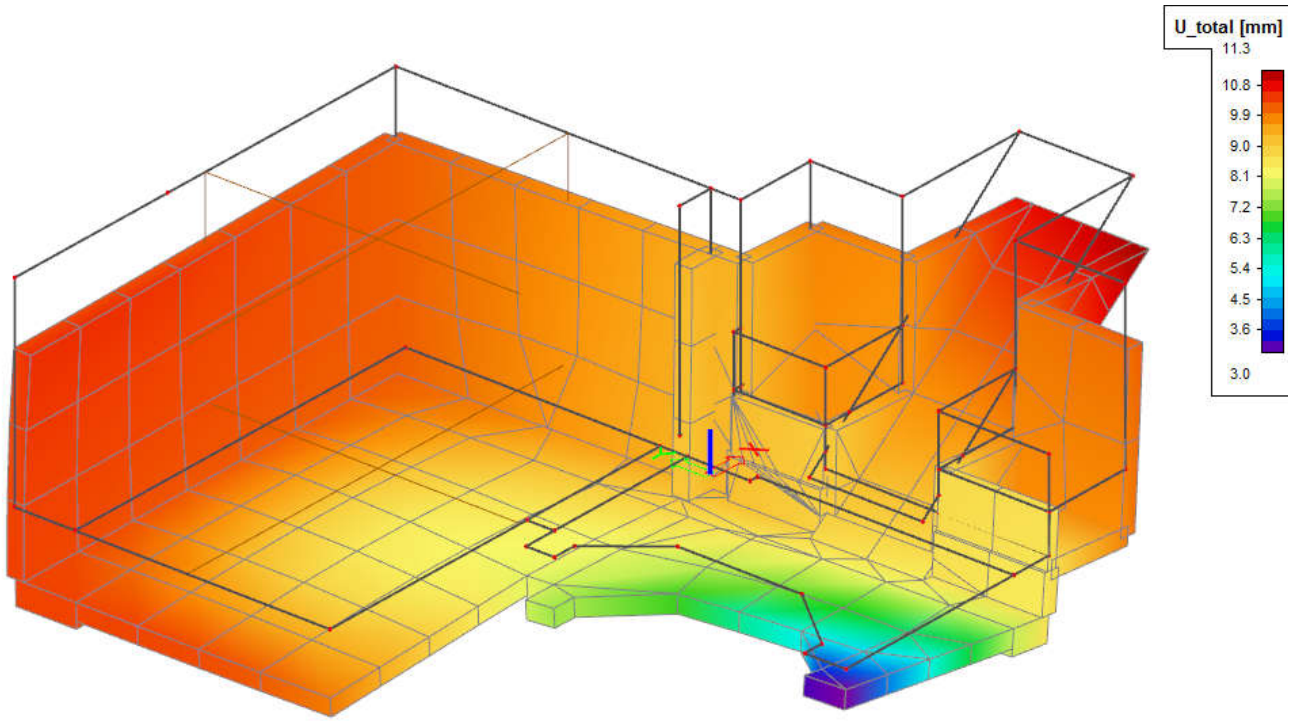


Deflections at Permanent Stage (x direction only)

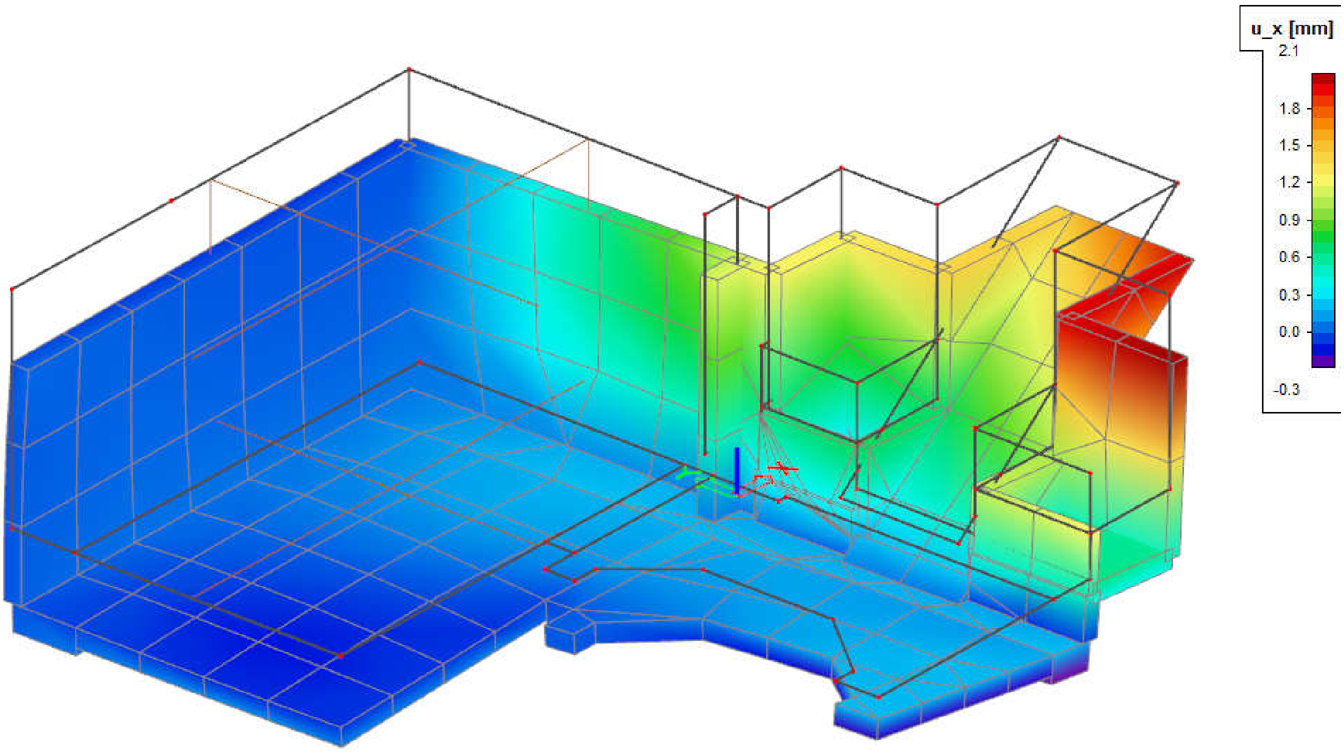


Job No	Description	Page:	19
9194	1 Lyndhurst Road, Camden, London	Date:	14.12.2017
		By:	I. Tozluoglu
		Checked:	U. Mizrahi

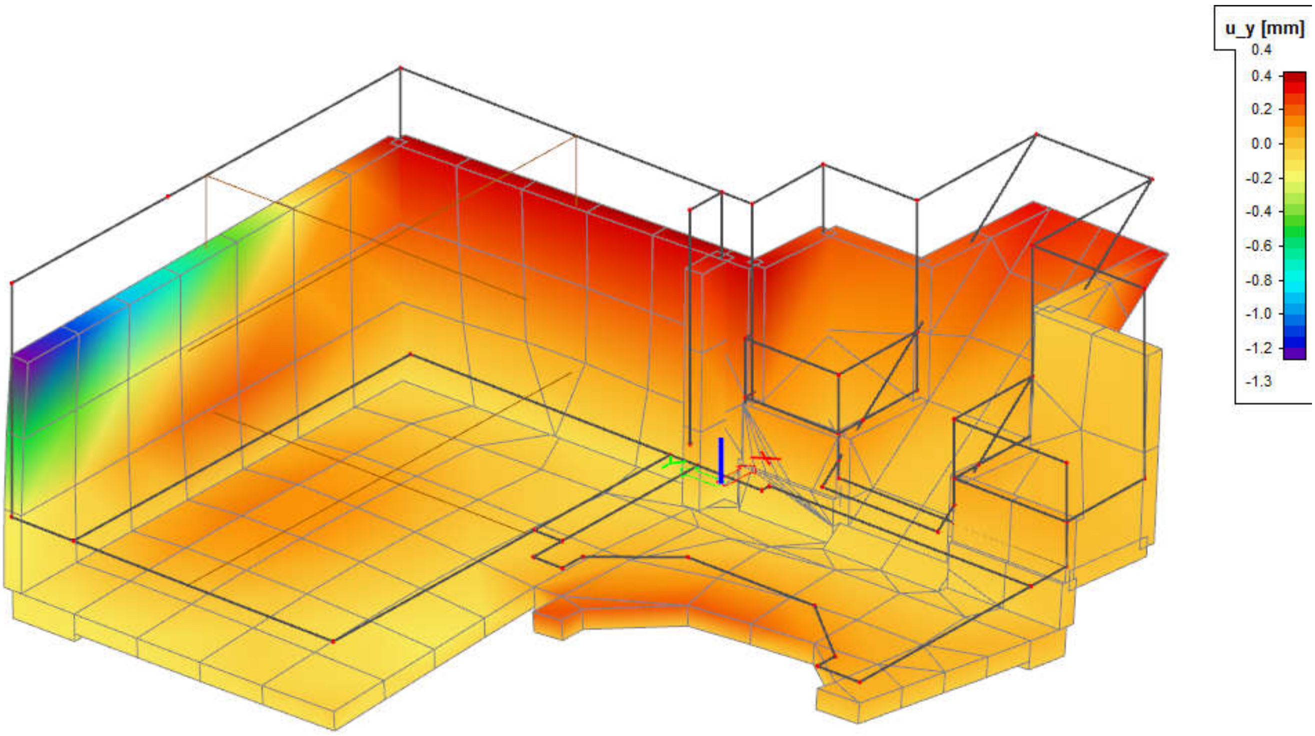
Total Deflections at Permanent Stage (without water pressure)



Deflections at Permanent Stage (without water pressure - x direction only)



Deflections at Permanent Stage (without water pressure - y direction only)



6. BUILDING DAMAGE ASSESMENT

In our design we are proposing a carefully sequenced phased construction method for construction of the proposed basement. Excavation widths will be approximately 1.7m and proposed excavation will be carried out with conventional temporary trench sheets with regular propping. Due to the proposed construction method ground movements will be minimised during excavation stage and workmanhip will be the leading factor of ground movements. Rest of the ground deflection will take part because of the structural deflections of the basement box system. For damage assesment we have considered ground movements equal to 3mm temporary deflection plus structural deformations. Methods described in Ciria C760 doesn't fit for the purpose of this analysis both due to the soil type we are excavating in, the phased excavation method we are using and also retaining structure used for supporting the ground. However we refered to the damage assesment method instructed in C760.

Expected Movements for Adjoining Buildings

Table 6.4 Classification of visible damage to walls (after Burland et al, 1977, Boscardin and Cording, 1989, and Burland, 2001)

Category of damage	Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain, ε_{lim} (%)
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible	<0.1	0.0 to 0.05
1 Very slight	Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection	<1	0.05 to 0.075
2 Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	<5	0.075 to 0.15
3 Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable lining. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5 to 15 or a number of cracks >3	0.15 to 0.3
4 Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Services pipes disrupted.	15 to 25, but also depends on number of cracks	>0.3
5 Very severe	This requires a major repair, involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	Usually >25, but depends on numbers of cracks	

Notes

1 In assessing the degree of damage, account must be taken of its location in the building or structure.

2 Crack width is only one aspect of damage and should not be used on its own as a direct measure of it.

** Extracts from Ciria C760

No1 Lyndhurst Road:

Adjacent Building Height	: 9m
Adjacent Building Length	: 8m
Distance from excavation	: 0m
Excavation Depth	: 2.4m
Horizontal movement	: 0mm
ε (Sagging)	: 0.0006
δ – Wall maximum displacement	: NA
Max tensile strain (%)	: 0.001
Approximate crack width	: <0.8mm
Damage Category	: Cat.1 : Very Slight

No2 Lyndhurst Terrace:

Adjacent Building Height	: 9m
Adjacent Building Length	: 8m
Distance from excavation	: 2.5m
Excavation Depth	: 2.4m
Horizontal movement	: 2.4+3mm
ε (Sagging)	: 0.00032
δ – Wall maximum displacement	: 2.4mm
Max tensile strain (%)	: 0.03
Approximate crack width	: <0.1mm
Damage Category	: Cat.0 : Negligible

No2 Lyndhurst Road:

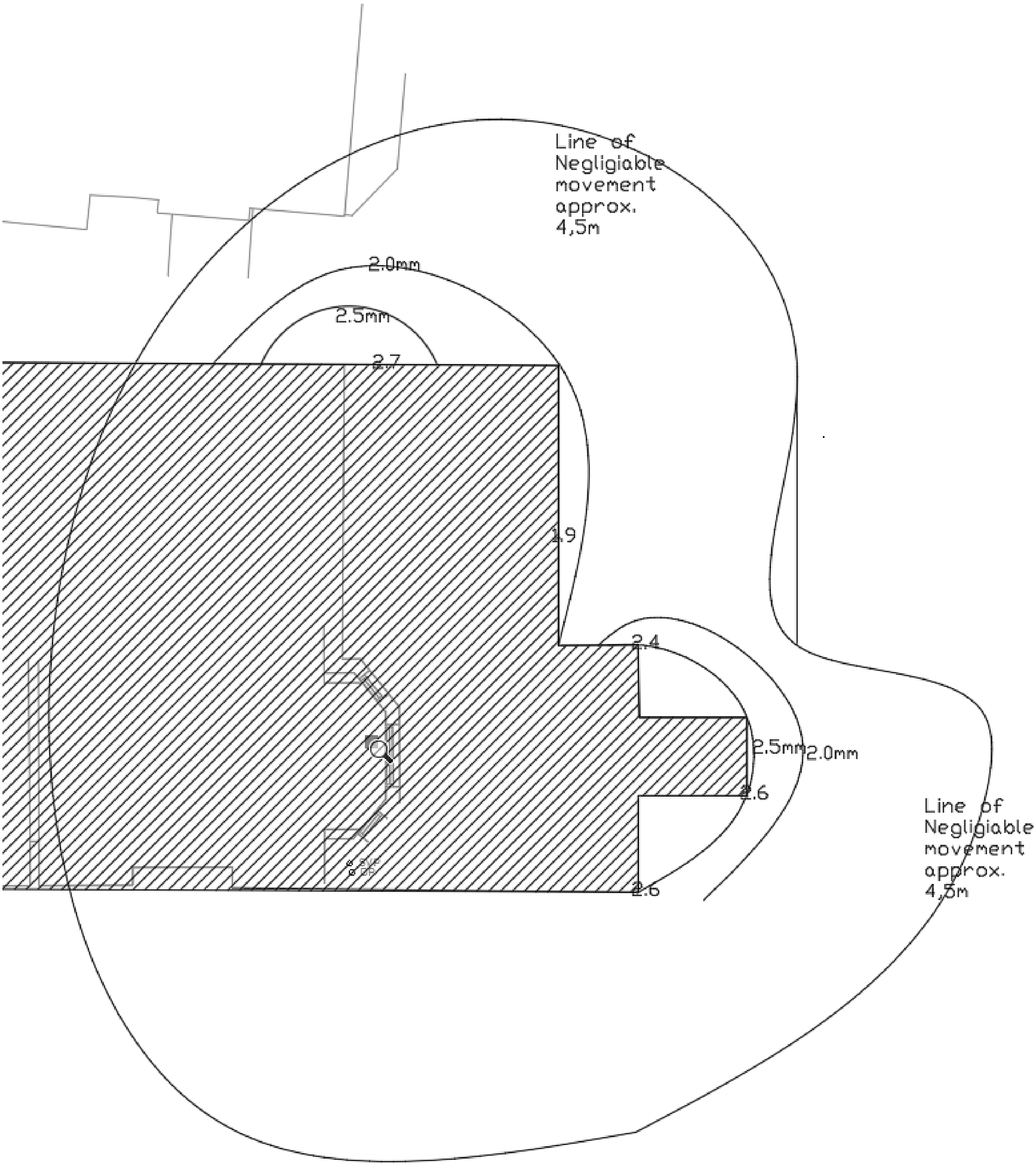
Adjacent Building Height	: 9m
Adjacent Building Length	: 8m
Distance from excavation	: 0m
Excavation Depth	: 2.4m
Horizontal movement	: 0mm
ε (Sagging)	: 0.0006
δ – Wall maximum displacement	: NA
Max tensile strain (%)	: 0.001
Approximate crack width	: <0.8mm
Damage Category	: Cat.1 : Very Slight

3 Dufferin Avenue,
Barbican, London, EC1Y 8PQ

T: 020 7253 2626
F: 020 7253 2767

E: tws@tws.uk.com
W: www.tws.uk.com

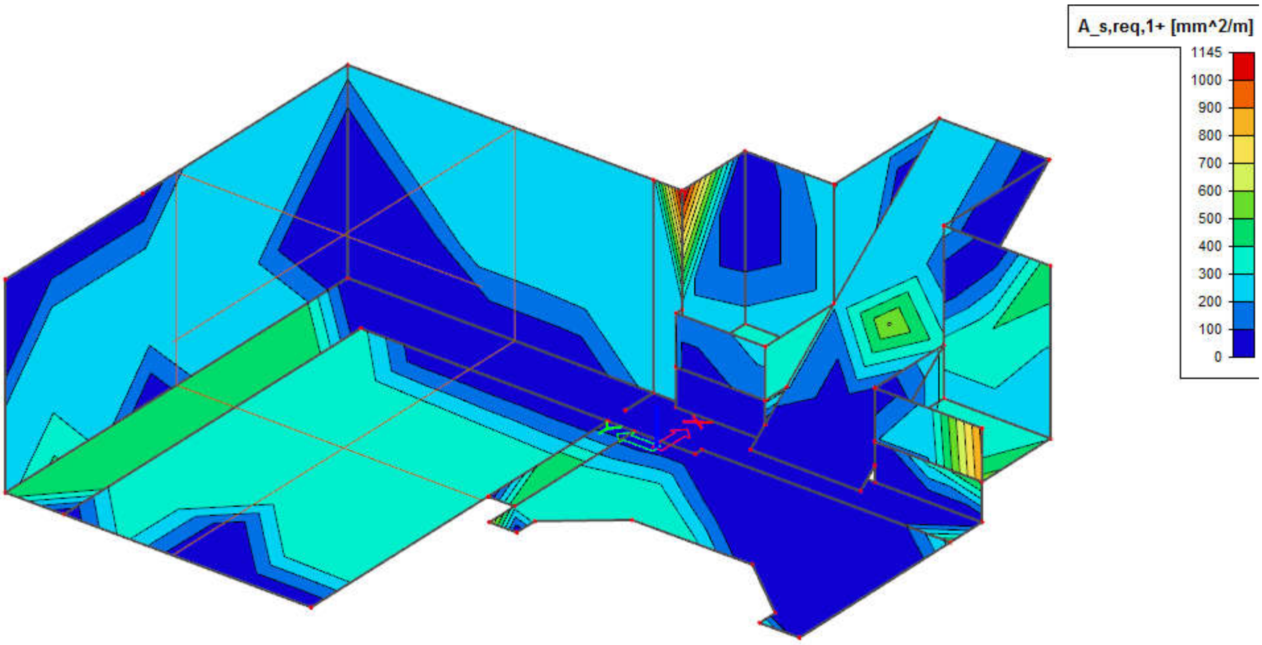
Settlement isolines at ground level



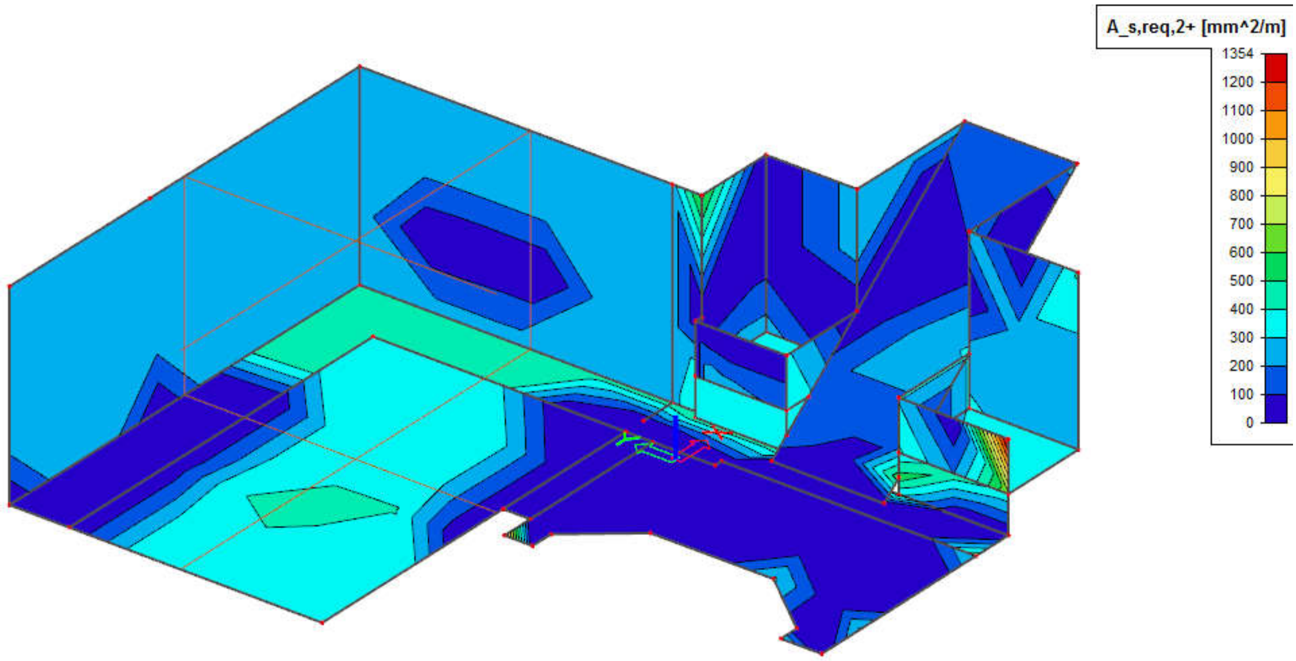
7. REINFORCEMENT DESIGN

Construction Stage

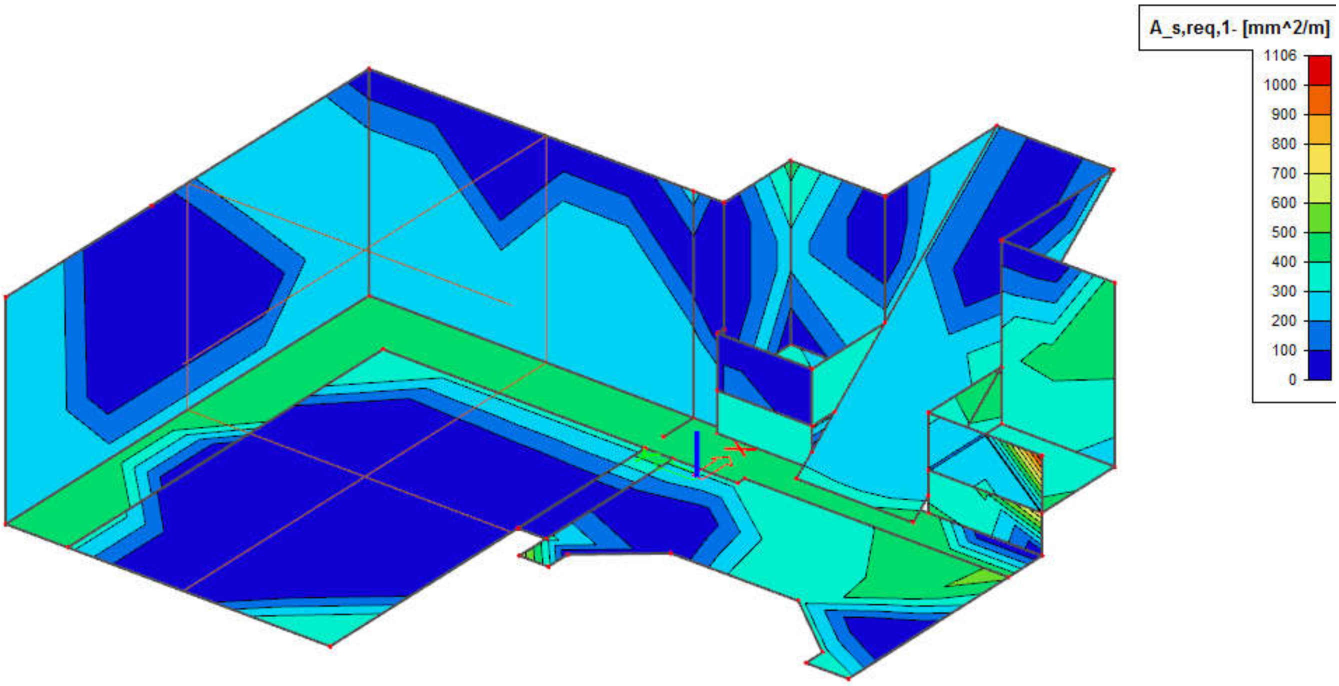
X Direction Top Reinforcement



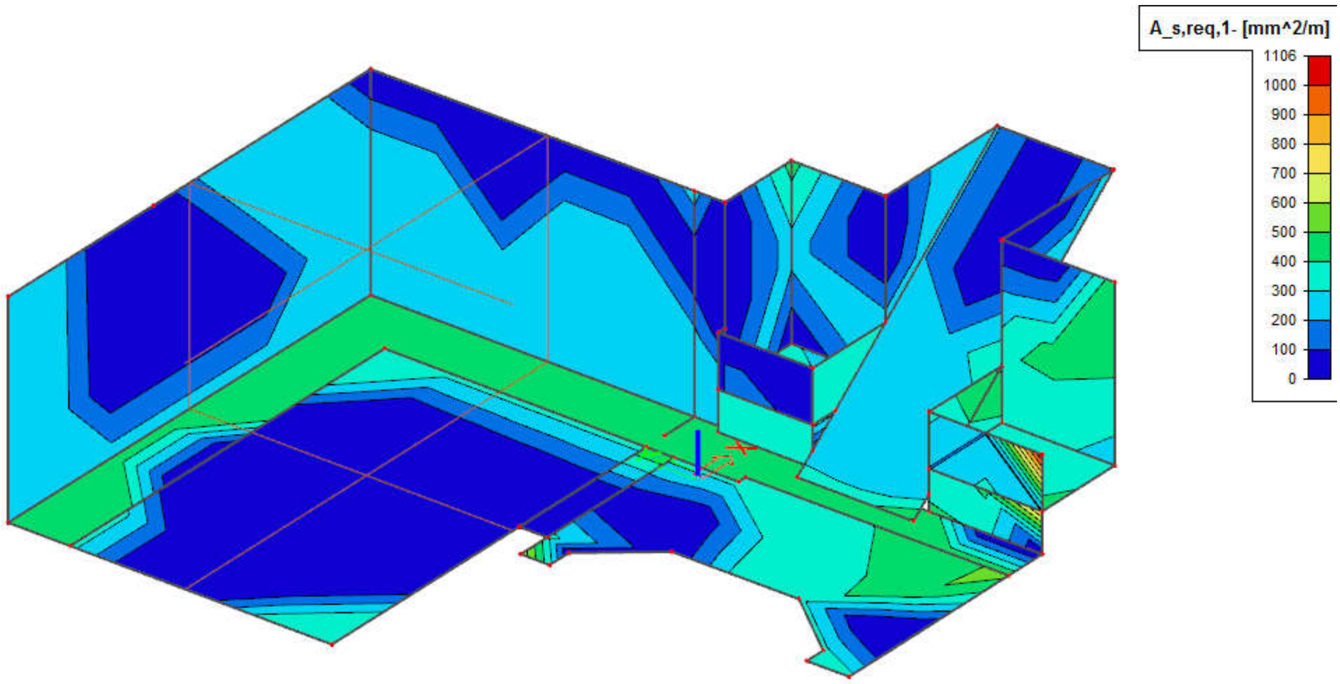
Y Direction Top Reinforcement



X Direction Bottom Reinforcement

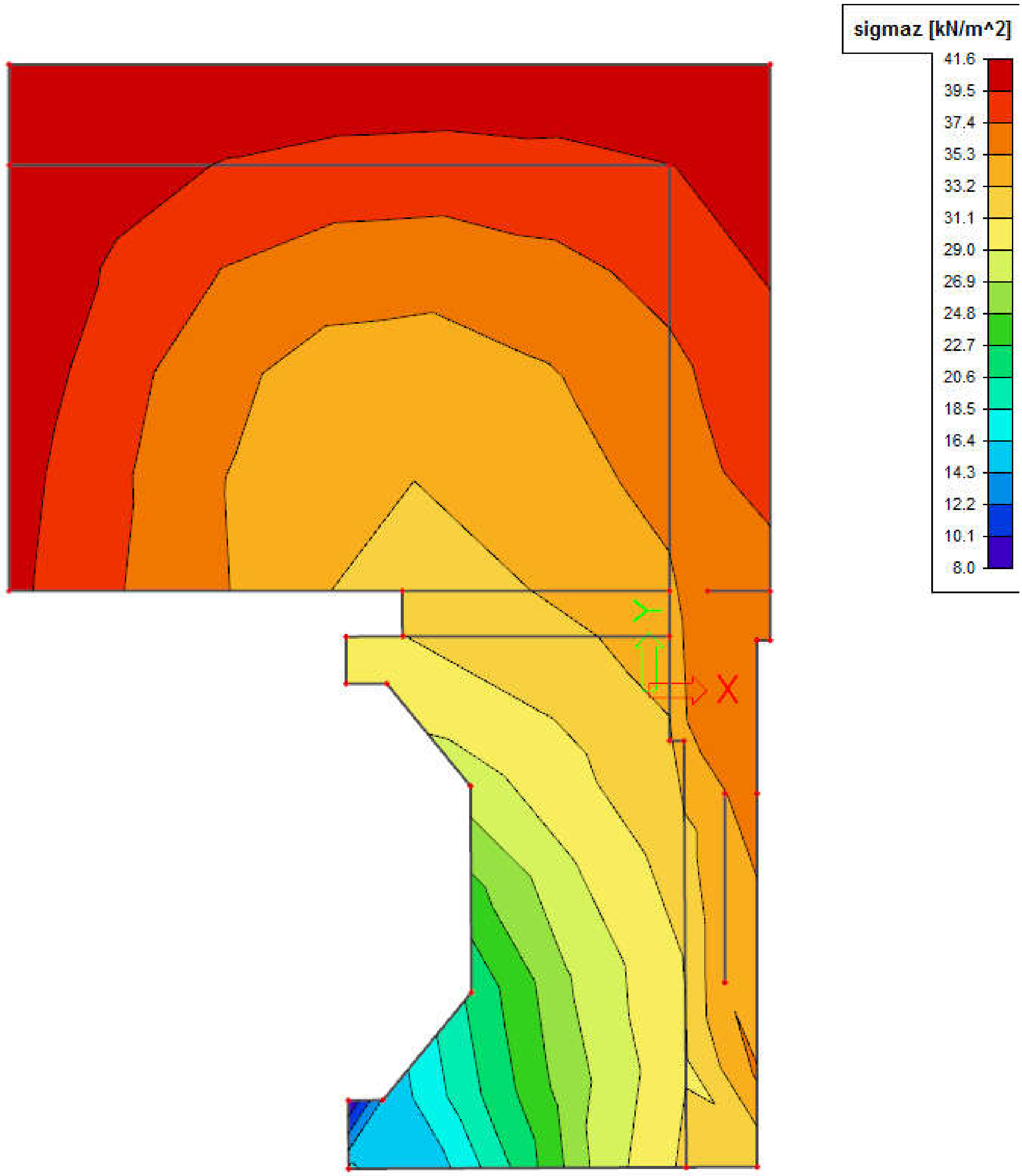


Y Direction Bottom Reinforcement



8. GROUND STRESS VALUES

Loading: DL + LL
Unit: kN/m^2
Allowable safe bearing stress: 100kN/m^2 at 3.0-3.5m depth. (Risk Management SI Report)
We have allowed 25mm for the maximum settlement based on the SI Report and soil stiffness parameter has been considered 4000kN/m^3 for this calculation.



9. HEAVE

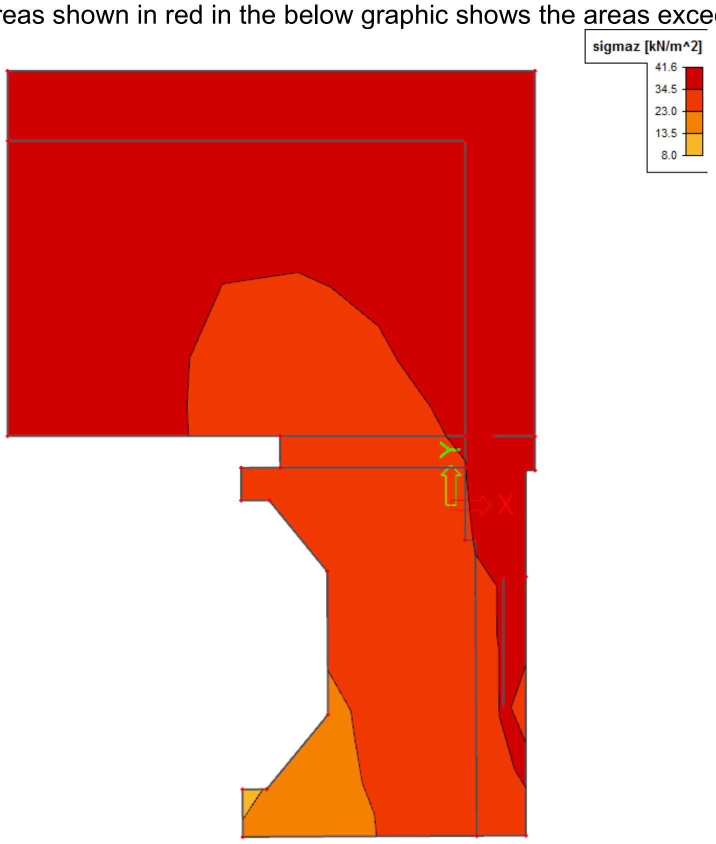
Loads to be removed from the excavation depth of new lower rear ground floor will be as follows:

49.22 - 47.00 + 0.35 = 2.57m

2.57m x 18kN/m^3 = 46.26kN/m^2

Proposed foundation stress levels under the new extension varies between 8kN/m^2 to 42kN/m^2

Areas shown in red in the below graphic shows the areas exceeding 46kN/m^2.



%75 of 46 = 34.5kN/m^2
%50 of 46 = 23.0kN/m^2

Approximately 2/3 of the ground area will be loaded with ground pressure values above 75%of the original 46kN/m^2 pressure.

Considering the evaluation above and the load reduction from the existing light well we expect heave effects will be minimal and will not cause any significant effect on structure.

For the more detailed final design we can use heave forms to minimize the effects of heave if required.

GEOTECHNICALCONSULTING GROUP

52A Cromwell Road, London,SW75BE,United Kingdom.

Tel: +44(0)2075818348 Email: admin@gcg.co.uk

Fax: +44(0)2075840157 Web: www.gcg.co.uk

Ouref: Youref:

Mr. M. Magid

Ryla Ltd
5 South Hill Park Gardens
London, NW3 2TD

14 November 2017

Dear Mr Magid,

1 LYNDHURST ROAD – HYDROGEOLOGICAL ASSESSMENT

Thank you for asking us to review the hydrogeological conditions of 1 Lyndhurst Road and assess the potential impact of the proposed lower ground extension on the local hydrogeology.

We understand that, as part of the refurbishment of the house, the existing light wells at the rear of the house are to be extended to form an open patio on the north-eastern section of the house and a new room on the western side, above which a new glass conservatory is to be formed at ground level.

The existing light wells at the rear of the house will be extended northwards by approximately 2.5m. A 1m wide strip to the side of the house will also be excavated to extend the rear part of the house to match the width of the front part. New stairs will be created to connect the new patio area to the garden at the rear. The levels of the existing light wells will be slightly lowered by less than 0.5m to reach the level of the lower ground floor across the rest of the house.

The structural plan and sections of the proposal are shown in the drawing 9194 BIA_02 provided by Taylor Whalley Spyra (TWS), who are the structural engineers for the works.

Based on Ordnance Survey (OS) mapping, the site lies at about +93m OD, with the ground surface sloping locally to the south at a gradient of around 1:16 and to the east at a gradient of 1:20. The level of the rear garden is approximately 2m above the street level at the front of the property.

The British Geological Survey (BGS) map of North London (Sheet 256) shows that the site is underlain by Claygate Member of the London Clay Formation. The London Clay outcrops at approximately 300m away from the site, along the contour level of +75mOD. Considering the gentle southward dip of the surface level of the London Clay across the area, its outcrop level suggests that the Claygate Member at the site could be approximately 15m thick.

Further to the north, approximately 300m from the site, the Bagshot Beds overlay the Claygate Member. However, a BGS map of England and Wales Edition 1920 shows the Bagshot Beds to extend over the location of the site.

Window sample boreholes were carried out in the rear garden to confirm the stratigraphy of the site. Below the top soil, the ground includes sand with pockets of clay up to the investigated depth of 8m. Layers of clay less than 1m thick were also identified at approximate depths of 1.5m and 4m depth.

The site is above an aquifer designated as 'Secondary Aquifer A'. No groundwater was encountered during the site investigation, except in a trial pit, where the observed water was attributed to a pipe leakage. No signs of instability were noted in the boreholes or in the trial pits that could suggest the presence of free water and the standpipes installed in the boreholes were found to be dry up to 7m depth.

The soil investigation and the groundwater measurements were carried out during summer months and therefore the groundwater measurements might not be representative of the maximum groundwater levels at the site. However the sandy nature of the soil at the site suggests that the groundwater is deep below ground and it is not surprising that the maximum groundwater level at the site is deeper than 7m. Perched groundwater could be present above the thin layers of clay, but it is likely to be limited and localised.

The BGS 1920 map shows the onsets of three streams within a radius of 600m from the site and the map of the Lost Rivers of London shows a tributary of the Tyburn River at less than 100m to the west of the site. The presence of these water features at close distance from the site indicates that preferential ways for the groundwater flow exist across the area. The site is over 100m away from the Hampstead Ponds.

The site is not in the list of primary or secondary locations at risk of surface water flooding. Lyndhurst Gardens, less than 300m to the east and south of the site was affected by flooding in 1975. However, the topographical profile of the area suggests that the risk of pluvial flooding at the site is very low.

The site is in an area designated by the Environment Agency to be at Flood Risk 1, with low risk of flooding from sea, river or reservoirs.

The existing lower ground floor of the house is above the level of the upper aquifer and the proposed northwards extension of the existing lower ground floor will remain approximately at the same level and will not intercept the groundwater level.

The proposed extension of the existing lower ground floor will not change slopes at the property boundary and will not alter the pre-existing situation of water run-off into the property.

In summary, having reviewed the local hydrogeology, we conclude that the proposed basement extension at 1 Lyndhurst Road is not expected to have adverse effects on the local hydrogeology.

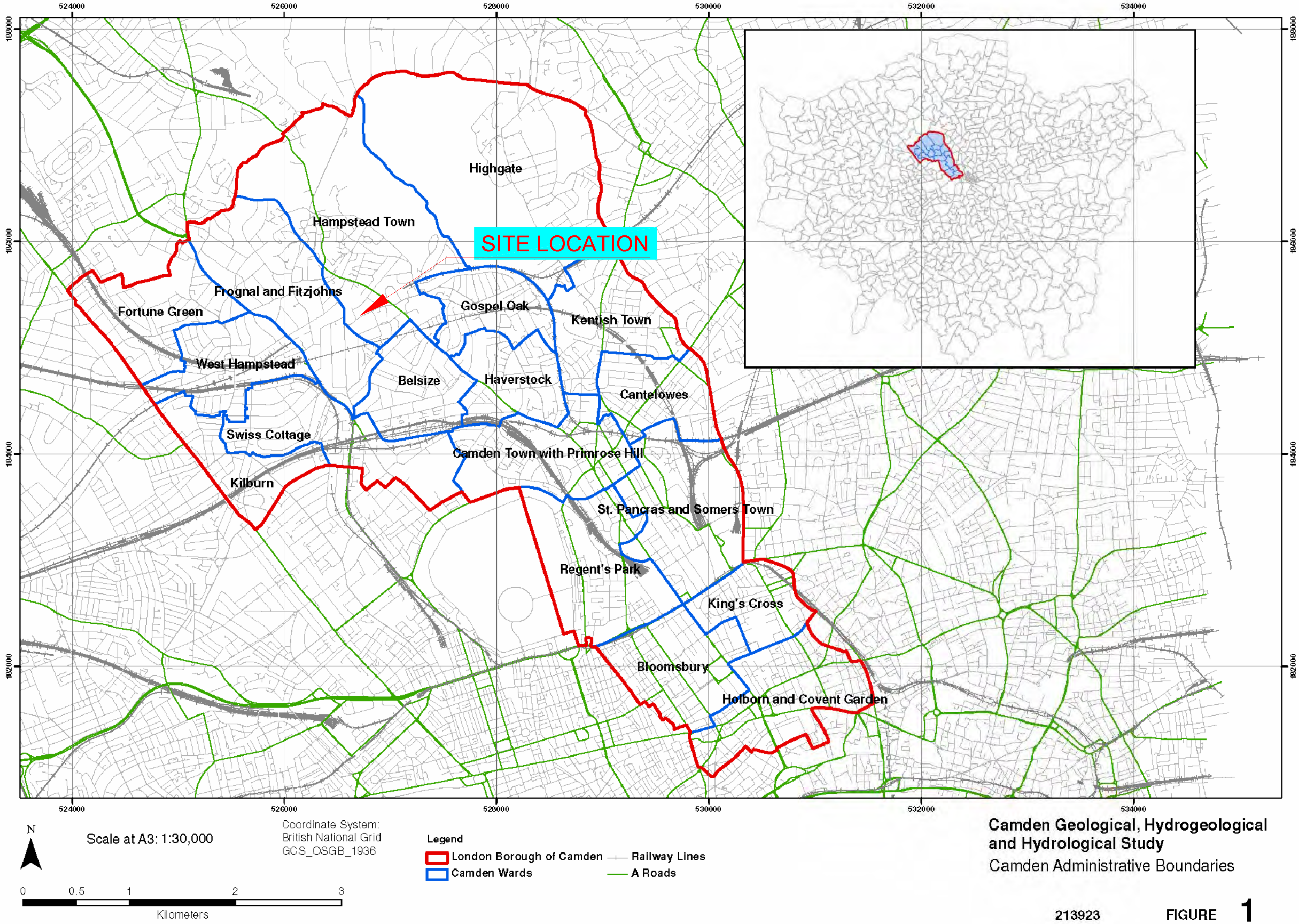
Yours sincerely,
For Geotechnical Consulting Group,

Dr Apollonia Gasparre
Dott. Ing. PhD DIC CEng MICE

Dr. J. Skipper
BSc PhD DIC CGeol

Disclaimer

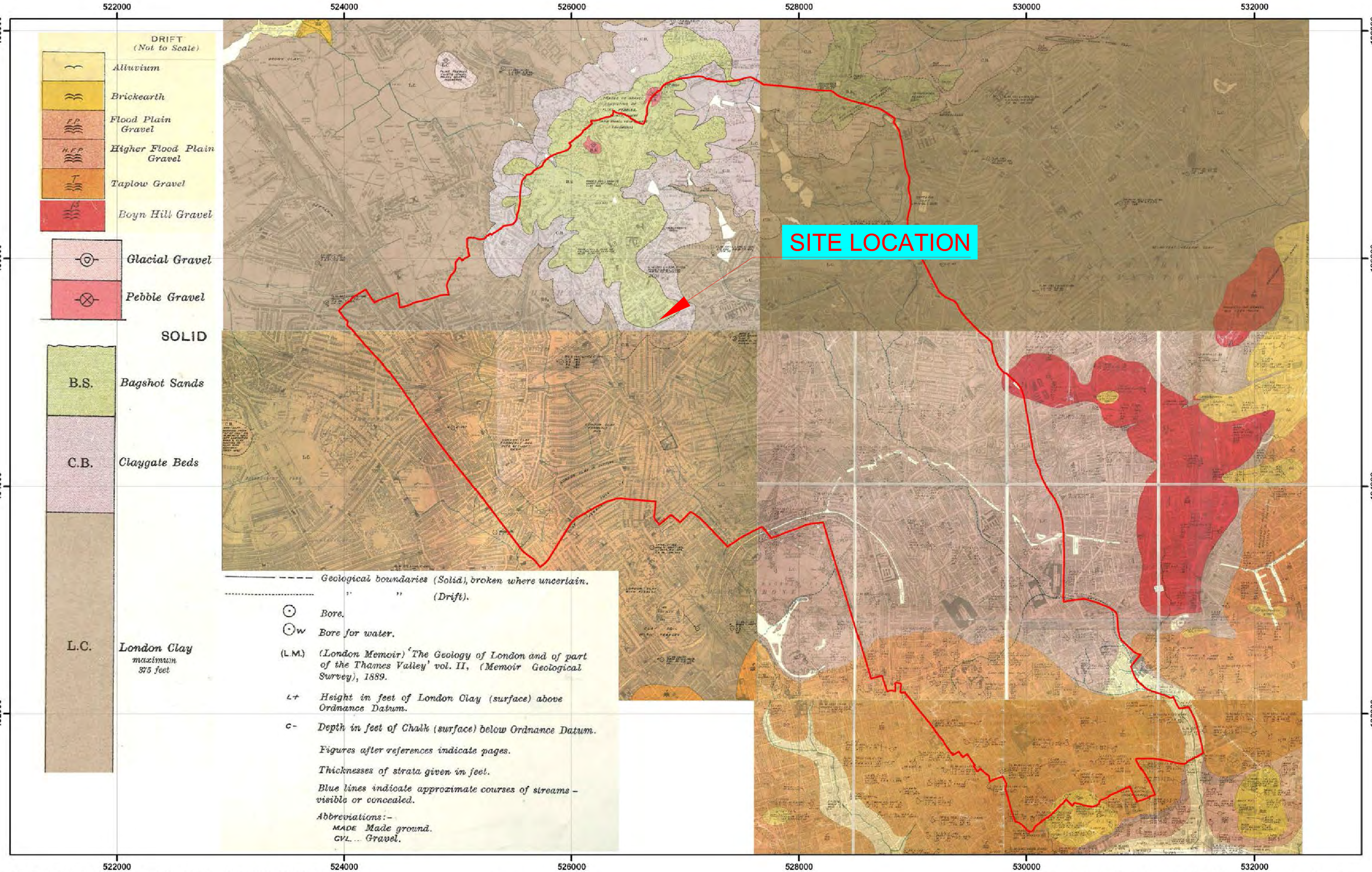
This letter (as well as any letters, information, opinions and advice provided to you) is the sole property of Geotechnical Consulting Group LLP and is and must remain strictly private and confidential at all times. The possession of this document does not, in any manner, constitute a right to reproduce or disclose the whole or any part of it to any third party. Neither the report nor any information contained in it should be used by anyone other than Michael Magid and can only be used by Michael Magid for the purpose for which it was originally proposed. Geotechnical Consulting Group LLP is not responsible for information used in this report which has been supplied to it by Michael Magid or any other third party. This report does not constitute or represent verification for purpose. The report should not be reproduced (in whole or in part), referred to in any other document or made available to any third party (in any format) without the prior written consent of Geotechnical Consulting Group LLP.



Camden Geological, Hydrogeological and Hydrological Study

Camden Administrative Boundaries

213923 FIGURE **1**



BGS 1:10,560 Map Sheets 1SE, 2SW, 4NE, 5SW, 5NW 1920 Edition)

N

Scale at A3: 1:30,000

0 0.4 0.8 1.6 2.4
Kilometers

Coordinate System:
British National Grid
GCS_OSGB_1936

Legend

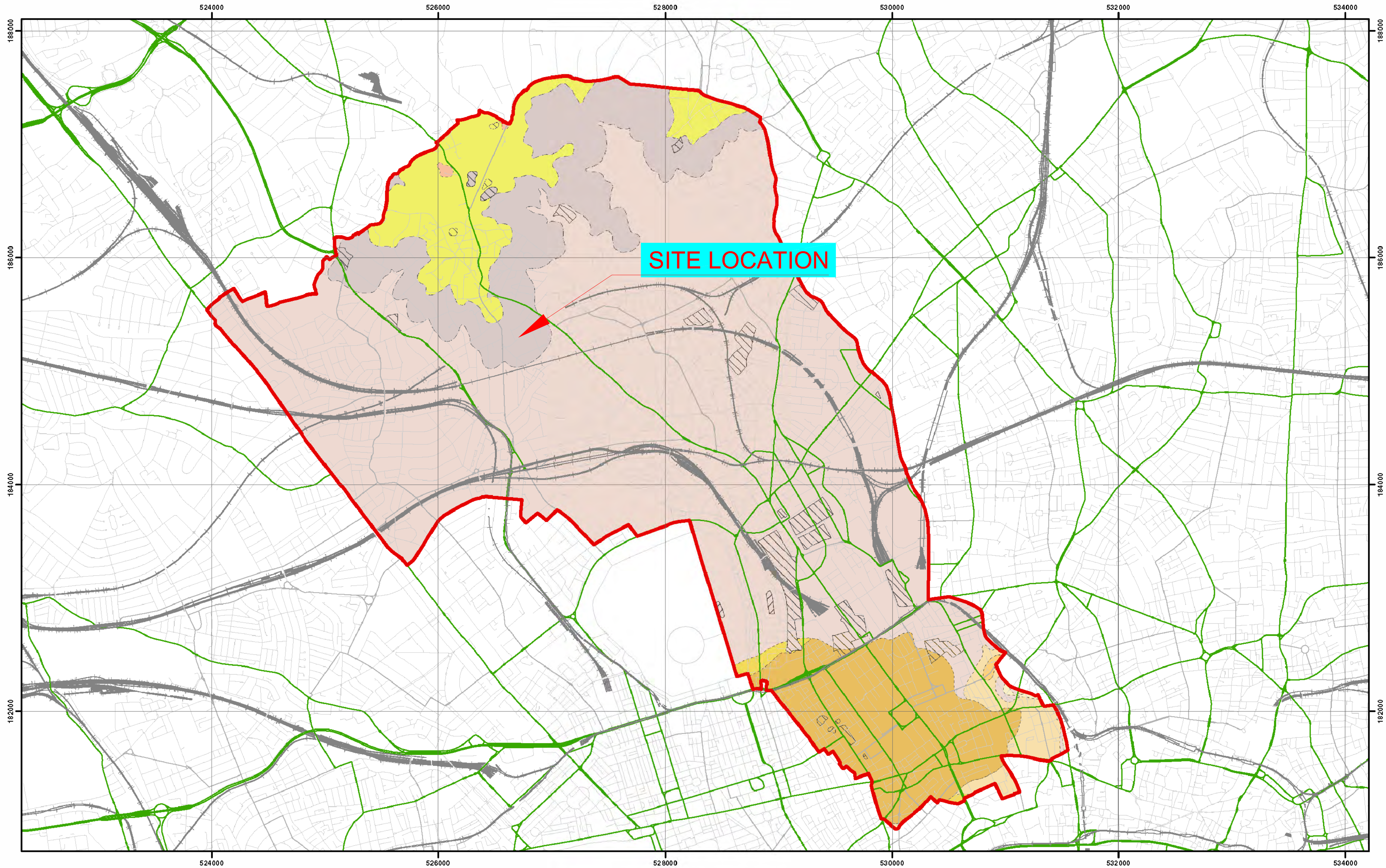
London Borough of Camden

Camden Geological, Hydrogeological and Hydrological Study

Camden 1:10,560 Geological Map (1920)

213923

FIGURE **2**



Data source - BGS Mapping - Scale 1:10,000

N

Scale at A3: 1:30,000

Coordinate System:
British National Grid
GCS_OSGB_1936

0 0.5 1 2 3
Kilometers

Legend

London Borough of Camden
Railway Lines
A Roads

BGS 1:10K Artificial Ground
MADE GROUND
WORKED GROUND

BGS 1:10K Drift Geology
ALLUVIUM
HACKNEY GRAVEL FORMATION
LANGLEY SILT FORMATION
LYNCH HILL GRAVEL FORMATION
STANMORE GRAVEL FORMATION

BGS 1:10K Solid Geology
BAGSHOT FORMATION
CLAYGATE MEMBER
LAMBETH GROUP
LONDON CLAY FORMATION

NB: Geological boundaries are largely indicative based on available geological mapping data

Camden Geological, Hydrogeological and Hydrological Study

Camden Geological Map

213923

FIGURE 3