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Flat 1, 29 Croftdown Rd London NW5

Structural Engineers Report in Support of the Planning Submission for the Proposed Subterranean Works

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1.0 INTRODUCTION

1.1 Conisbee has been appointed to provide Structural Engineering advice for the proposed works to Flat 1, 29 Croftdown Road, London NW5. The following report is in support of the Planning Application for the proposed subterranean works to property.

1.2 The property is a four-storey semi-detached structure with Flat 1 on the Lower Ground and Ground floor. The upper floors consist of the First and Second floors being the full footprint of the building and the Third floor being in the attic space. No 27 is the attached property and No 31 being separated by a passageway.

1.3 This report focuses on the structural engineering aspects of the proposed works associated with the subterranean works.

2.0 EXISTING BUILDING AND FOUNDATIONS

2.1 29 Croftdown Road is a four-storey semi-detached property constructed in the early 1900's as confirmed by the desk study of the historic Ordnance Survey Maps. The adjacent properties were also constructed at a similar time and follow the same form as No 29.

2.2 The property is of traditional construction and consists of a loose cut timber roof, with gables to the south and east elevation and a dormer to the rear. The internal floors to the upper levels are timber which span from the front elevation to the rear. Intermediate support is provided by the spine wall located in the centre of the property. To the lower levels the spine wall is solid possible timber studs which are infilled with the uppermost floors being timber stud. The existing Lower Ground floor is a ground bearing concrete slab. The external envelop is solid masonry 330mm thick.

2.3 The floor in the room to the front of the property of the Lower Ground floor has previously been lowered. The Party Wall with No 27 has been underpinned for its whole length as has the spine wall, the front elevation and the front half of the flank wall. The lowering of the floor provided a floor to ceiling height of 2270mm.

2.4 The lower ground floor to No 27 has previously been lowered for the whole of the foot print of the Lower Ground Floor and these works permitted the lowering to the front part of No 29.

3.0 PROPOSED DEVELOPMENT

3.1 The proposed subterranean works include the underpinning of the existing load bearing walls to the rear section of the main house to allow for the room to the rear to be lowered by 350mm to the level of the room to the front of the property. The hallway will also be lowered with the lower flight of the timber stair being replaced.

3.2 A new lightwell is to be formed in the front garden to provide natural light and ventilation into the property via new opening being formed in the existing bay window structure.

4.0 SITE INVESTIGATION

4.1 A desk study, ground investigation and BIA has been undertaken by GEA, their report referenced J21224 Rev No 0 dated 16 November 2021.

4.2 The brief of the investigation was to confirm the soil conditions, hydrology the impact of the proposed basement on the local hydrogeology and the provision of advice with respect the design of the new foundations and retaining walls as well as meeting the requirement to comply with the London Borough of Camden Planning Guidance relating to the requirements for a Basement Impact Assessment.

4.3 As part of the assessment the historic underpinning construction information was reviewed.

4.4 A bore hole was undertaken in the front garden adjacent to the proposed new light well this confirmed that the top 1.2m is made ground over various bands of clay. The bore hole extended to a depth of 9.5m below the existing garden level.

4.5 Ground water was not encountered during the investigation, a standpipe was installed, and a monitoring visit undertaken. Ground water was recorded at a depth of 4.29m below the existing ground level, a further monitoring visit is planned.

4.6 A trial pit was excavated to the rear elevation this confirmed that the footing is formed of two brick corbels founded in the made ground approximately 170mm above the clay.

4.7 The investigation confirms that there are no subterranean tunnels that will be impacted by the works. Similarly, there are no recorded public services that will be impeded by the subterranean construction.

4.8 The BIA confirms that there are no slope stability concerns.

4.9 The (perched) ground water from the made ground is to be further monitored, dewatering measures such as pumping are to be allowed for during the construction phase and the detailing of waterproofing of the new structure.

5.0 STRUCTURAL DESIGN OF THE NEW SUBSTRUCTURE

5.1 Underpinning Design

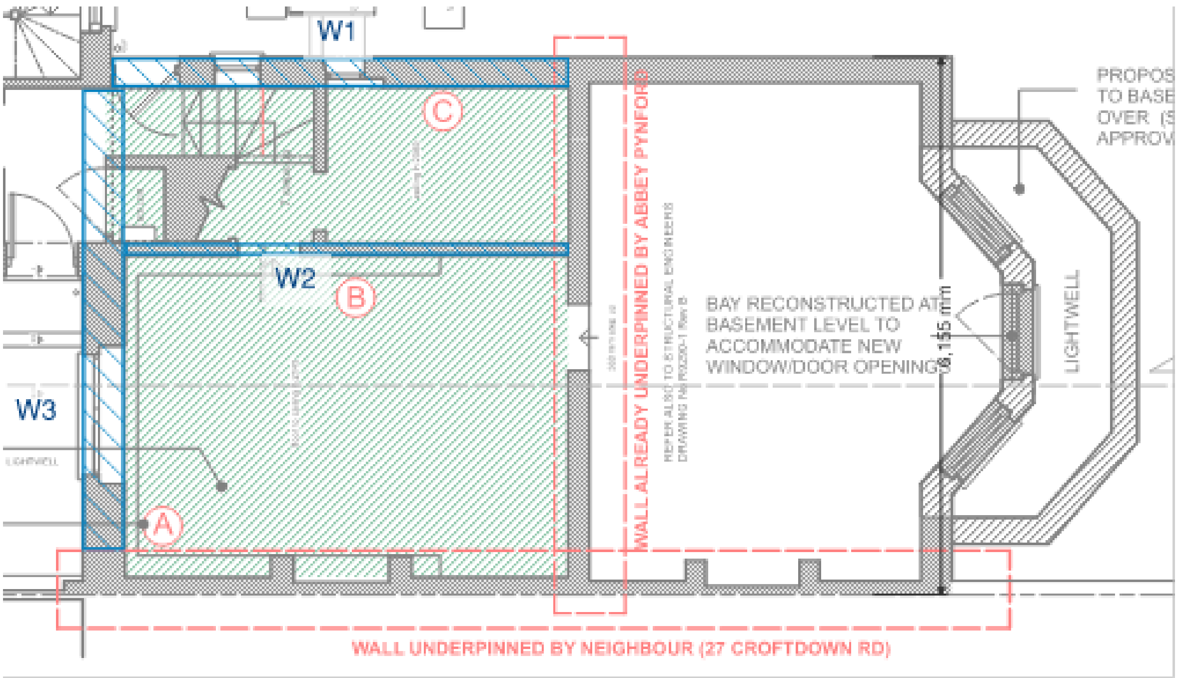
As outlined in the Section 2 of this report the property is a four-storey semi-detached structure of traditional construction. At lower ground floor level, the floor at the front of the property has previously been lowered, and the Party Wall with No 27 has been underpinned for its whole length as has the spine wall, the front elevation and the front half of the flank wall.

5.1.1 To suitably lower the rear section of the lower ground floor the masonry walls which have not yet been underpinned will need to be underpinned to facilitate the change in level.

5.1.2 The loads acting on the structure can be summarised as per the table below.

	Dead Load (kN/m^2)		**Imposed Load (kN/m^2)**	
Pitched Roof	Slates, timber battens & felt	0.50	Snow and access	0.75
	Rafters	0.15		
	Plaster Board	0.15		
	Ceiling, finishes and services	0.20		
	Total	**1.00**	**Total**	**0.75**
Internal Timber Floor	Timber boards/plywood	0.20	Residential	1.5
	Timber joists	0.15		
	Finishes and services	0.25		
	Total	**0.60**	**Total**	**1.5**
215mm Thick Brick Wall	Bricks	3.87		
	Plaster (2 sides)	0.4		
	Total	**4.27**	**Total**	**0.0**
330mm Thick Brick Wall	Bricks	5.94		
	Plaster (1 sides)	0.2		
	Total	**6.14**	**Total**	**0.0**

5.1.3 The total loads acting on the foundations of the 3 walls (labelled as W1, W2 and W3 in the figure below) can be established based on the applied loading summarised above.



5.1.4 The unfactored loads acting at the base of each of the three walls has been summarised below based on the defined loads and geometry of the structure.

	Unfactored Dead Load (kN/m)	Unfactored Imposed Load (kN/m)	Total Unfactored Load (kN/m)
Wall 1	64.37	6.56	70.93
Wall 2	53.27	7.45	60.73
Wall 3	76.67	12.43	89.10

5.1.5 The bearing capacity of the soil as per the site investigation report is 150kPa, which is recommended value with adequate factors of safety applied. Providing a 600mm wide strip footing will result in the bearing pressures beneath each of the three walls being less than the allowable 150kPa as can be seen in the table below.

	Total Unfactored Load (kN/m)	Self-Weight of Underpin (kN/m)	0.6m Strip Footing - Stress (kPa)
Wall 1	70.93	9.00	118.21
Wall 2	60.73	9.00	101.21
Wall 3	89.10	9.00	148.51

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5.2 Internal RC Slab

5.2.1 The internal RC slab at the rear of the property is to be lowered by 350mm. The slab itself has been designed to be 175mm thick with A252 mesh top and bottom, this will be cast atop 150mm of well compacted hardcore and a 50mm layer of blinding below.

5.2.2 In addition to the self-weight of the slab the ground bearing slab has been designed to resist a super imposed dead load of 1.5kN/m^2 to allow for finishes and a variable load of 1.5kN/m^2. These loads have been summarised in the table below.

Description	**Dead Load (G_k) – kN/m^2**	**Imposed Load (Q_k) – kN/m^2**
0.175m thick slab self weight Reinforced concrete density – 25kN/m^3	4.4	-
Screed and other finishes	1.5	-
Residential Load	-	1.5
Total	**5.9**	**1.5**

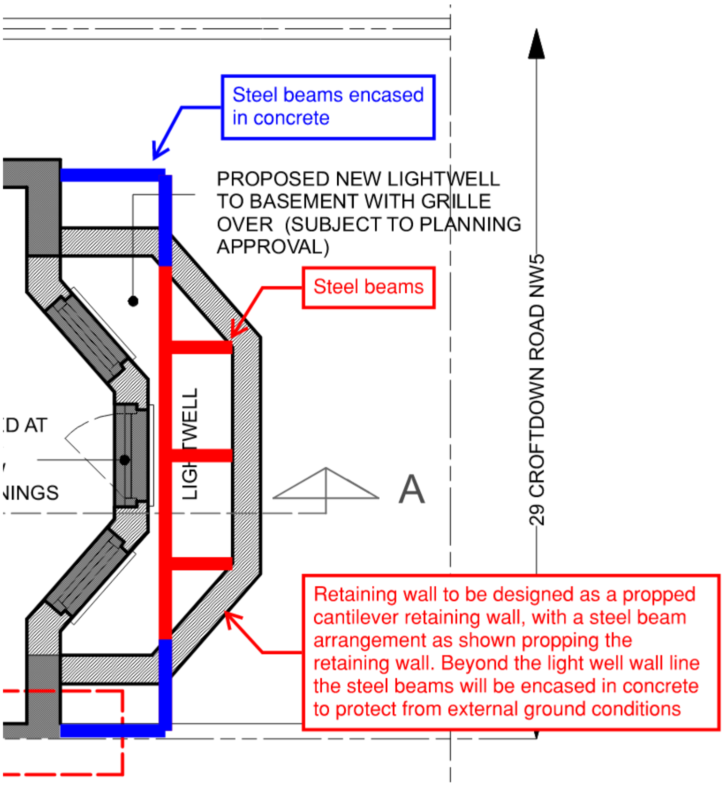
5.2.3 The total factored load based on the loads in the table above are 10.215kN/m^2, this results in the applied bearing pressure being significantly less than the allowable bearing pressure of 150kN/m^2.

5.3 Lightwell Retaining Wall Design

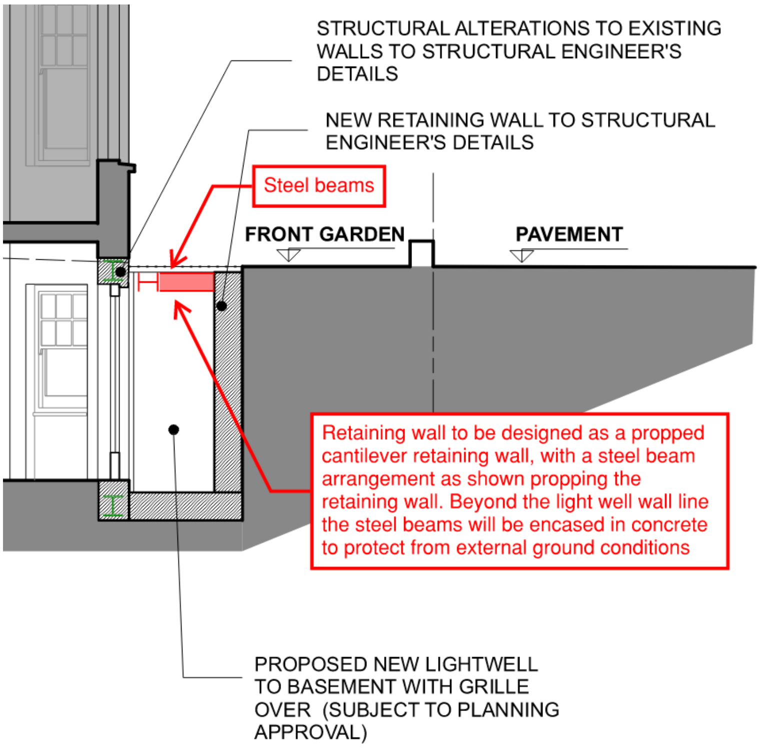
5.3.1 The proposed lightwell is to be formed in the front garden space beyond the front wall of the existing house. The lightwell will have a total depth of 2.6m and be formed as an RC retaining wall, with the retained and base soil taken as stiff clay with a bearing capacity of 150kPa in line with the site investigation report.

5.3.2 Furthermore, whilst no ground water has been identified, the site investigation report states that a groundwater table should be assumed at two-thirds of the retained height to mitigate the risk of any ground or surface water collecting behind the retaining walls. To alleviate the hydrostatic pressure weep holes should be provided in the retaining wall. Additionally, the base slab of the lightwell wall will be tied into the existing lower ground floor slab along the line of the front of house bay window, this will act as a prop at the base of the wall.

5.3.3 The design of the retaining wall will be carried out as a cantilever retaining wall propped at the head of the wall. The propping at the head of the wall will be in the form of a steel beam arrangement as shown in the images below.



Plan View of Lightwell Retaining Wall



Section Through Lightwell Retaining Wall

5.3.4 Based on this the central section of the retaining wall has been designed with the calculations shown on the following pages and the two return segments will follow the same design principles. The key design aspects can be summarised as follows:

- 400mm thick RC base with a 2350mm high stem
- 900mm long toe tied into 400mm minimum of the existing slab, 400mm thick stem with a total length of 1700mm
- RC 32/40 concrete with 50mm cover to all sides of the stem and base
- Base reinforcement:
  - H16 rebar at 200 centres in both orthogonal directions in the top mat
  - H16 rebar at 200 centres in both orthogonal directions in the bottom mat
- Stem reinforcement:
  - H12 rebar at 200 centres to the front face – the face not in contact with soil
  - H16 rebar at 200 centres to the rear face – the retaining face
  - H12 rebar at 200 centres transverse reinforcement

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.21

Analysis summary

Design summary

Overall design utilisation; 0.923

Overall design status; Pass

Description	Unit	Capacity	Applied	F o S	Result
Bearing pressure	kN/m^2	150	65.9	2.276	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Shear resistance	kN/m	30.5	158.7	0.192	PASS
Stem p0 - Shear resistance	kN/m	158.7	60.0	0.378	PASS
Stem p1 front face - Flexural reinforcement	mm^2/m	565.5	522.0	0.923	PASS
Stem p1 - Shear resistance	kN/m	158.7	8.2	0.052	PASS
Stem p2 front face - Flexural reinforcement	mm^2/m	565.5	522.0	0.923	PASS
Stem p2 - Shear resistance	kN/m	158.7	20.5	0.129	PASS
Base bottom face - Flexural reinforcement	mm^2/m	1005.3	537.8	0.535	PASS
Base - Shear resistance	kN/m	158.7	30.5	0.192	PASS
Min. transverse stem reinf.	mm^2/m	565.5	400.0	0.707	PASS
Min. transverse base reinf.	mm^2/m	1005.3	201.1	0.200	PASS

Retaining wall details

Stem type; Propped cantilever

Stem height; h_{stem} = **2350** mm

Prop height; h_{prop} = **2350** mm

Stem thickness; t_{stem} = **400** mm

Angle to rear face of stem; α = **90** deg

Stem density; γ_{stem} = **25** kN/m^3

Toe length; l_{toe} = **1300** mm

Base thickness; t_{base} = **400** mm

Base density; γ_{base} = **25** kN/m^3

Height of retained soil; h_{ret} = **2350** mm

Angle of soil surface; β = **0** deg

Depth of cover; d_{cover} = **0** mm

Retained soil properties

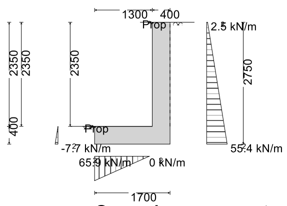
Soil type;	Stiff clay
Moist density;	γ_{mr} = **19.5** kN/m^3
Saturated density;	γ_{sr} = **19.5** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Stiff clay
Soil density;	γ_b = **19.5** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **150** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **2.5** kN/m^2



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem}$ = **1700** mm
Moist soil height;	$h_{moist} = h_{soil}$ = **2350** mm
Length of surcharge load;	$l_{sur} = l_{heel}$ = **0** mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **1700** mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **2750** mm

- Distance to horizontal component; $x_{sur\_h} = h_{eff} / 2 =$ **1375** mm

Area of wall stem; $A_{stem} = h_{stem} \times t_{stem} =$ **0.94** m^2

- Distance to vertical component; $x_{stem} = l_{toe} + t_{stem} / 2 =$ **1500** mm

Area of wall base; $A_{base} = l_{base} \times t_{base} =$ **0.68** m^2

- Distance to vertical component; $x_{base} = l_{base} / 2 =$ **850** mm

For undrained soils - Annex C.1(2)

At rest pressure coefficient; $K_0 =$ **1.000**

Passive pressure coefficient; $K_P =$ **1.000**

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} =$ **23.5** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} =$ **17** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} =$ **40.5** kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_0 \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} =$ **6.8** kN/m

Moist retained soil; $F_{moist\_h} = K_0 \times \cos(\delta_{r.k}) \times \gamma_{mr} \times h_{eff}^2 / 2 =$ **72.8** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 =$ **-1.5** kN/m

Total; $F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{pass\_h} =$ **78.1** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} =$ **35.3** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} =$ **14.5** kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h} =$ **-9.3** kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **-66.8** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sur} + M_{moist} =$ **-26.4** kNm/m

Check bearing pressure

Propping force to stem; $F_{prop\_stem} =$ **15.6** kN/m

Propping force to base; $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} =$ **62.4** kN/m

Moment from propping force; $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) =$ **43** kNm/m

Distance to reaction; $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} =$ **410** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 =$ **-440** mm

Loaded length of base; $l_{load} = 3$ ´ $\bar{x} =$ **1229** mm

Bearing pressure at toe; $q_{toe} = 2$ ´ $F_{total\_v} / l_{load} =$ **65.9** kN/m^2

Bearing pressure at heel; $q_{heel} =$ **0** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) =$ **2.276**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.21

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C32/40

Characteristic compressive cylinder strength; $f_{ck} =$ **32** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube} =$ **40** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8\ N/mm^2 =$ **40** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck} / 1\ N/mm^2)^{2/3} =$ **3.0** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} =$ **2.1** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22\ kN/mm^2 \times (f_{cm} / 10\ N/mm^2)^{0.3} =$ **33346** N/mm^2

Partial factor for concrete - Table 2.1N; $\gamma_C =$ **1.50**

Compressive strength coefficient - cl.3.1.6(1); $\alpha_{cc} =$ **0.85**

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **18.1** N/mm^2

Maximum aggregate size; $h_{agg} =$ **20** mm

Ultimate strain - Table 3.1; $\varepsilon_{cu2} =$ **0.0035**

Shortening strain - Table 3.1; $\varepsilon_{cu3} =$ **0.0035**

Effective compression zone height factor; $\lambda =$ **0.80**

Effective strength factor; $\eta =$ **1.00**

Bending coefficient k_1; $K_1 =$ **0.40**

Bending coefficient k_2; $K_2 = 1.00$ ´ $(0.6 + 0.0014/\varepsilon_{cu2}) =$ **1.00**

Bending coefficient k_3; $K_3 =$**0.40**

Bending coefficient k_4; $K_4 = 1.00$ ´ $(0.6 + 0.0014/\varepsilon_{cu2}) =$**1.00**

Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} =$ **500** N/mm^2

Modulus of elasticity of reinforcement; $E_s =$ **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S =$ **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S =$ **435** N/mm^2

Cover to reinforcement

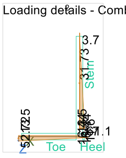
Front face of stem; $c_{sf} =$ **50** mm

Rear face of stem; $c_{sr} =$ **50** mm

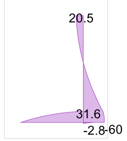
Top face of base; $c_{bt} =$ **50** mm

Bottom face of base; $c_{bb} =$ **50** mm

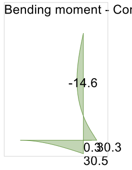
Loading details - Combination No.1 - kN/m

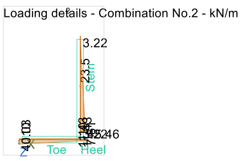


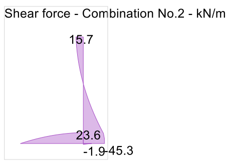
Shear force - Combination No.1 - kN/m

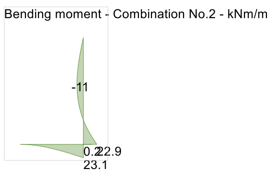


Bending moment - Combination No.1 - kNm/m









Check stem design at 1228 mm

Depth of section; h = **400** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **14.6** kNm/m

Depth to tension reinforcement; $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **332** mm

$K = M / (d^2 \times f_{ck})$ = **0.004**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C) \times (1 - \lambda \times (\delta - K_1)/(2 \times K_2)) \times (\lambda \times (\delta - K_1)/(2 \times K_2))$

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **315** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **42** mm

Area of tension reinforcement required; $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{106}$ mm²/m

Tension reinforcement provided; 12 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sfM.prov} = \pi \text{ ´ } \phi_{sfM}^2 / (4 \text{ ´ } s_{sfM}) = \mathbf{565}$ mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sfM.min} = \max(0.26 \text{ ´ } f_{ctm} / f_{yk}, 0.0013) \text{ ´ } d =$ **522** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sfM.max} = 0.04 \text{ ´ } h = \mathbf{16000}$ mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.923}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio; $\rho = A_{sfM.req} / d = \mathbf{0.000}$

Required compression reinforcement ratio; $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N; $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17; $K_s = \min(500\ N/mm^2 / (f_{yk} \text{ ´ } A_{sfM.req} / A_{sfM.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{40}$

Actual span to depth ratio; $h_{prop} / d = \mathbf{7.1}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{10.2}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sfM.prov} \text{ ´ } z) = \mathbf{57.4}$ N/mm²

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \text{ ´ } (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{119500}$ mm²/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.0}$ N/mm²

Reinforcement ratio; $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} = \mathbf{0.005}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

Bond property coefficient; $k_1 = \mathbf{0.8}$

Strain distribution coefficient; $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \text{ ´ } c_{sf} + k_1 \text{ ´ } k_2 \text{ ´ } k_4 \text{ ´ } \phi_{sfM} / \rho_{p.eff} =$ **601** mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.103}$ mm

$w_k / w_{max} = \mathbf{0.345}$

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section; $h = \mathbf{400}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{18.3}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{342}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.005}$

$K' = (2 ´ \eta ´ \alpha_{cc}/\gamma_C) ´ (1 - \lambda ´ (\delta - K_1)/(2 ´ K_2)) ´ (\lambda ´ (\delta - K_1)/(2 ´ K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 ´ (1 - 2 ´ K / (\eta ´ \alpha_{cc} / \gamma_C))^{0.5}, 0.95) ´ d = \mathbf{325}$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = \mathbf{43}$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{129}$ mm²/m

Tension reinforcement provided; 16 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi ´ \phi_{sr}^2 / (4 ´ s_{sr}) = \mathbf{1005}$ mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 ´ f_{ctm} / f_{yk}, 0.0013) ´ d = \mathbf{538}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 ´ h = \mathbf{16000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.535}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = \mathbf{0.000}$

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N; $K_b = \mathbf{1}$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} ´ A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{40}$

Actual span to depth ratio; $h_{prop} / d = \mathbf{6.9}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{12.9}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} ´ z) = \mathbf{39.4}$ N/mm²

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 ´ (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{119083}$ mm²/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.0** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.008**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.998**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 =$ **3.4**

$k_4 =$ **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3$ ´ $c_{sr} + k_1$ ´ k_2 ´ k_4 ´ $\phi_{sr} / \rho_{p.eff} =$

492 mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k =$ **0.058** mm

$w_k / w_{max} =$ **0.194**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **60** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = \min(1 + \sqrt{}(200 \text{ mm} / d), 2) =$ **1.765**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) =$ **0.003**

$v_{min} = 0.035$ N$^{1/2}$/mm ´ $k^{3/2}$ ´ $f_{ck}^{0.5} =$ **0.464**

N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c}$ ´ k ´ (100 N^2/mm^4 ´ ρ_l ´

$f_{ck})^{1/3}$, $v_{min})$ ´ d

$V_{Rd.c} =$ **158.7** kN/m

$V / V_{Rd.c} =$ **0.378**

PASS - Design shear resistance exceeds design shear force

Check stem design at 1000 mm

Depth of section; h = **400** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **13.7** kNm/m

Depth to tension reinforcement; $d = h - c_{sf} - \phi_{sx} - \phi_{sf1} / 2 =$ **332** mm

$K = M / (d^2 \times f_{ck}) =$ **0.004**

K' = (2 ´ η ´ α_{cc}/γ_C) ´ (1 - λ ´ (δ - K_1)/(2 ´

K_2)) ´ (λ ´ (δ - K_1)/(2 ´ K_2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5 + 0.5 ´ (1 - 2 ´ K / (η ´ α_{cc} /

γ_C))$^{0.5}$, 0.95) ´ d = **315** mm

Depth of neutral axis; $x = 2.5 \times (d – z) =$ **42** mm

Area of tension reinforcement required; $A_{sf1.req} = M / (f_{yd} \times z) =$ **100** mm^2/m

Tension reinforcement provided; 12 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{sf1.prov} = \pi$ ´ ϕ_{sf1}^2 / (4 ´ s_{sf1}) = **565** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sf1.min}$ = max(0.26 ´ f_{ctm} / f_{yk}, 0.0013) ´ d = **522** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sf1.max}$ = 0.04 ´ h = **16000** mm^2/m

max($A_{sf1.req}$, $A_{sf1.min}$) / $A_{sf1.prov}$ = **0.923**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio; ρ_0 = √(f_{ck} / 1 N/mm^2) / 1000 = **0.006**

Required tension reinforcement ratio; ρ = $A_{sf1.req}$ / d = **0.000**

Required compression reinforcement ratio; ρ' = $A_{sf1.2.req}$ / d_2 = **0.000**

Structural system factor - Table 7.4N; K_b = **1**

Reinforcement factor - exp.7.17; K_s = min(500 N/mm^2 / (f_{yk} ´ $A_{sf1.req}$ / $A_{sf1.prov}$), 1.5) = **1.5**

Limiting span to depth ratio - exp.7.16.a; min(K_s × K_b × [11 + 1.5 × √(f_{ck} / 1 N/mm^2) × ρ_0 / ρ + 3.2 × √(f_{ck} / 1 N/mm^2) × (ρ_0 / ρ - 1)$^{3/2}$], 40 × K_b) = **40**

Actual span to depth ratio; h_{prop} / d = **7.1**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **9.6** kNm/m

Tensile stress in reinforcement; σ_s = M_{sls} / ($A_{sf1.prov}$ ´ z) = **54** N/mm^2

Load duration; Long term

Load duration factor; k_t = **0.4**

Effective area of concrete in tension; $A_{c.eff}$ = min(2.5 ´ (h - d), (h - x) / 3, h / 2)

$A_{c.eff}$ = **119500** mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff}$ = f_{ctm} = **3.0** N/mm^2

Reinforcement ratio; $\rho_{p.eff}$ = $A_{sf1.prov}$ / $A_{c.eff}$ = **0.005**

Modular ratio; α_e = E_s / E_{cm} = **5.998**

Bond property coefficient; k_1 = **0.8**

Strain distribution coefficient; k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max}$ = k_3 ´ c_{sf} + k_1 ´ k_2 ´ k_4 ´ ϕ_{sf1} / $\rho_{p.eff}$ = **601** mm

Maximum crack width - exp.7.8; w_k = $s_{r.max}$ × max(σ_s – k_t × ($f_{ct.eff}$ / $\rho_{p.eff}$) × (1 + α_e × $\rho_{p.eff}$), 0.6 × σ_s) / E_s

w_k = **0.097** mm

w_k / w_{max} = **0.325**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **8.2** kN/m

$C_{Rd,c}$ = 0.18 / γ_C = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.776}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr1.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} ˘ k^{3/2} ˘ f_{ck}^{0.5} = \mathbf{0.469}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} ˘ k ˘ (100 \text{ N}^2/\text{mm}^4 ˘ \rho_l ˘ f_{ck})^{1/3}, v_{min}) ˘ d$

$V_{Rd.c} = \mathbf{155.6}$ kN/m

$V / V_{Rd.c} = \mathbf{0.053}$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section; h = **400** mm

Rectangular section in shear - Section 6.2

Design shear force; V = **20.5** kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.776}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr2.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} ˘ k^{3/2} ˘ f_{ck}^{0.5} = \mathbf{0.469}$ N/mm²

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} ˘ k ˘ (100 \text{ N}^2/\text{mm}^4 ˘ \rho_l ˘ f_{ck})^{1/3}, v_{min}) ˘ d$

$V_{Rd.c} = \mathbf{155.6}$ kN/m

$V / V_{Rd.c} = \mathbf{0.132}$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req} = \max(0.25 ˘ A_{sr.prov}, 0.001 ˘ t_{stem}) = \mathbf{400}$ mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = \mathbf{400}$ mm

Transverse reinforcement provided; 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi ˘ \phi_{sx}^2 / (4 ˘ s_{sx}) = \mathbf{565}$ mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; h = **400** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **24.3** kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{342}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.006}$

$K' = (2 ˘ \eta ˘ \alpha_{cc}/\gamma_C) ˘ (1 - \lambda ˘ (\delta - K_1)/(2 ˘ K_2)) ˘ (\lambda ˘ (\delta - K_1)/(2 ˘ K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5$ ´ $(1 - 2$ ´ $K / (\eta$ ´ $\alpha_{cc} /$
$\gamma_C))^{0.5}, 0.95)$ ´ $d =$ **325** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **43** mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) =$ **172** mm^2/m

Tension reinforcement provided; 16 dia.bars @ 200 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi$ ´ $\phi_{bb}^2 / (4$ ´ $s_{bb}) =$ **1005** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26$ ´ $f_{ctm} / f_{yk}, 0.0013)$ ´ $d =$
538 mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04$ ´ $h =$ **16000** mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} =$ **0.535**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 =$ **0.6**

Serviceability bending moment; $M_{sls} =$ **17.1** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov}$ ´ $z) =$ **52.4** N/mm^2

Load duration; Long term

Load duration factor; $k_t =$ **0.4**

Effective area of concrete in tension; $A_{c.eff} = \min(2.5$ ´ $(h - d), (h - x) / 3, h / 2)$

$A_{c.eff} =$ **119083** mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.0** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} =$ **0.008**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.998**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 =$ **3.4**

$k_4 =$ **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3$ ´ $c_{bb} + k_1$ ´ k_2 ´ k_4 ´ $\phi_{bb} / \rho_{p.eff} =$
492 mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k =$ **0.077** mm

$w_k / w_{max} =$ **0.258**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V =$ **30.5** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **1.765**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) =$ **0.003**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm}$ ´ $k^{3/2}$ ´ $f_{ck}^{0.5} =$ **0.464**
N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **158.7** kN/m

$V / V_{Rd.c}$ = **0.192**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

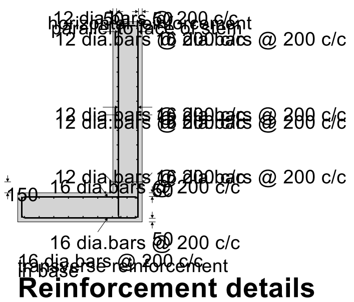
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **201** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3); s_{bx_max} = **450** mm

Transverse reinforcement provided; 16 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **1005** mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

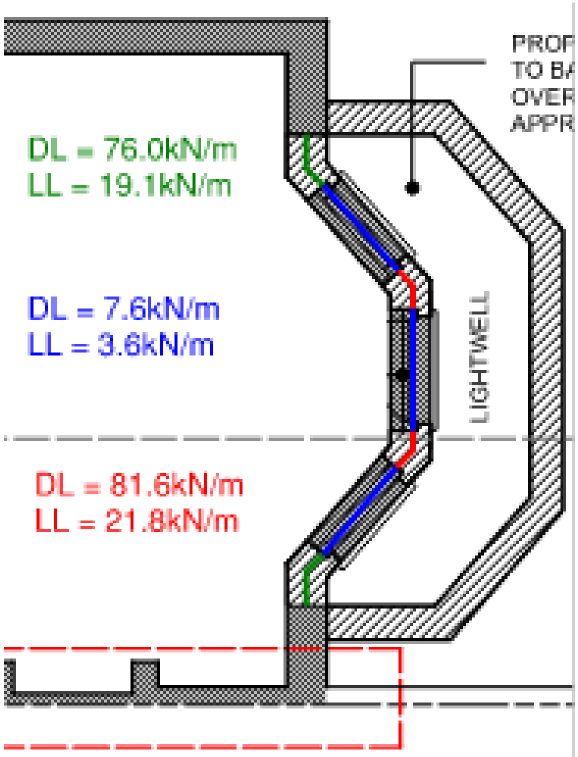
5.4 Reconstructed Bay for Window

5.4.1 Following the construction of the lightwell the existing wall at the front of house at lower ground floor level will be needled and propped to facilitate the formation of openings for the bay window construction.

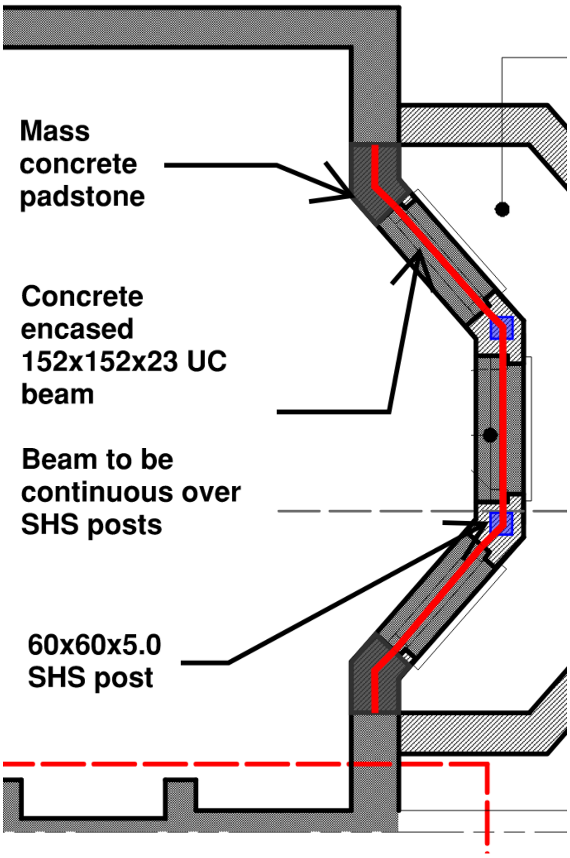
5.4.2 To form the structural openings, it is necessary to install a steel beam spanning over the openings to support the masonry walls above. The masonry walls above follow the same layout with large bay window openings at the same location as those which have been proposed.

5.4.3 Based on the structural loads defined in Section 5.1 the loads acting at the top of the reconstructed bay window line can be established. Owing to the presence of large openings the loads have been taken to spread into the masonry piers either side of the windows which are continuous throughout the height of the property. Only the dead and live loads supported at ground floor level have been taken to act over the line of openings and supported directly onto the spanning beam.

5.4.4 The loads acting along the reconstructed bay window line have been established as shown in the figure below.

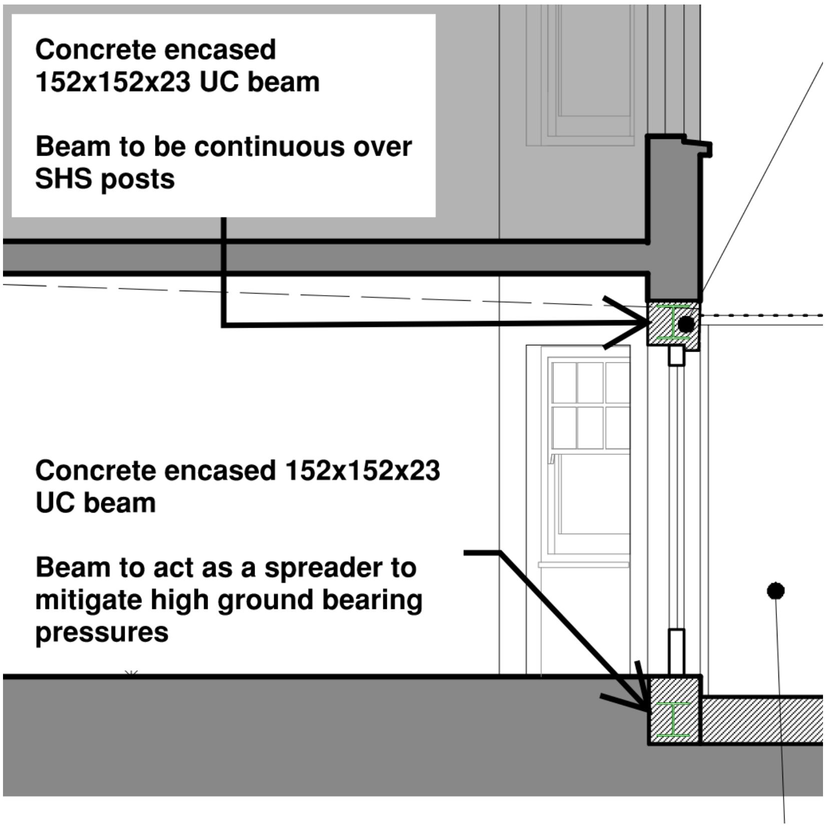


5.4.5 Owing to the presence of slender piers in the central span SHS posts have been introduced to provide suitable vertical support to the spanning beam. The beam itself has been designed as a 152x152x23UC member encased in concrete, at the ends of the bay the beam will be supported on mass concrete padstones. This arrangement is illustrated in the figure below.



5.4.6 Alternatively, the 152 UC beam could be designed with a flat plate on the top flange to provide suitable support to the brickwork above, this would allow the concrete encasement to be omitted and suitable architectural details can be established for this arrangement.

5.4.7 At the lower ground floor slab level to mitigate the impact of high point loads resulting from the SHS posts and masonry piers at the ends of the bay, a spreader beam has been introduced. The structural arrangement of this will follow the line of the bay window and this can be a similar concrete encased 152x152x23 UC or alternatively the beam could be installed as a reinforced concrete beam. This arrangement can be seen in the annotated figure below.


Concrete encased
152x152x23 UC beam

Beam to be continuous over
SHS posts

Concrete encased 152x152x23
UC beam

Beam to act as a spreader to
mitigate high ground bearing
pressures

6.0 CONSTRUCTION SEQUENCE

6.1 The final construction sequence will be produced by the appointed contractor and this will be reviewed and approved by Conisbee prior the works being undertaken. The actual sequence will be based upon a copy of this report, the construction issue drawing and specification produced by Conisbee and the BIA report prepared by GEA.

6.2 Lowering of Internal Slab

6.2.1 The suggested sequence for the underpinning of the wall to the rear elevation and the rear section of the flank wall which was not part of the works previously undertaken. The underpinning is to be undertaken in bays no greater than 1.0m wide on a hit and miss basis with no more that 20% of a wall being underpinned at one time, see drawing 210485-SK-S-001. The pins are to be cast in accordance with the profile detailed on drawing 210485-SK-S-001. The pins are to be reinforced with a suitable shear key between pins.

6.2.2 On completion of the underpinning the remaining sections of the of the lower ground floor slab are to be removed and the ground excavated to the required level. The new slab is to be cast to the required level and profile incorporating the required insulation and waterproofing as detailed by the Architect.

6.3 Retaining Wall Construction

6.3.1 On completion of the underpinning and the Lower Ground floor slab the front garden is to be excavated to allow for the construction of the new reinforced retaining wall that will form the lightwell. The excavation and installation of the retaining wall should also be carried out in bays no greater than 1.0m, following the same process as the underpinning noted in Section 6.2.1. The retaining wall is to be excavated and cast in accordance with the retaining wall geometry outlined in Section 5.3, with the installation sequence shown on drawing 210485-SK-S-001.

6.3.2 To facilitate the continuity of and suitable anchorage/lapping of reinforcement, horizontal reinforcement provided for the retaining wall to be driven into the soil adjacent to the underpin being installed prior to pouring the concrete.

6.4 Bay Window Construction

6.4.1 On completion of the lightwell needles are to be installed and propped on to the ground bearing concrete slab to permit the formation of the new opening and the bay window construction.

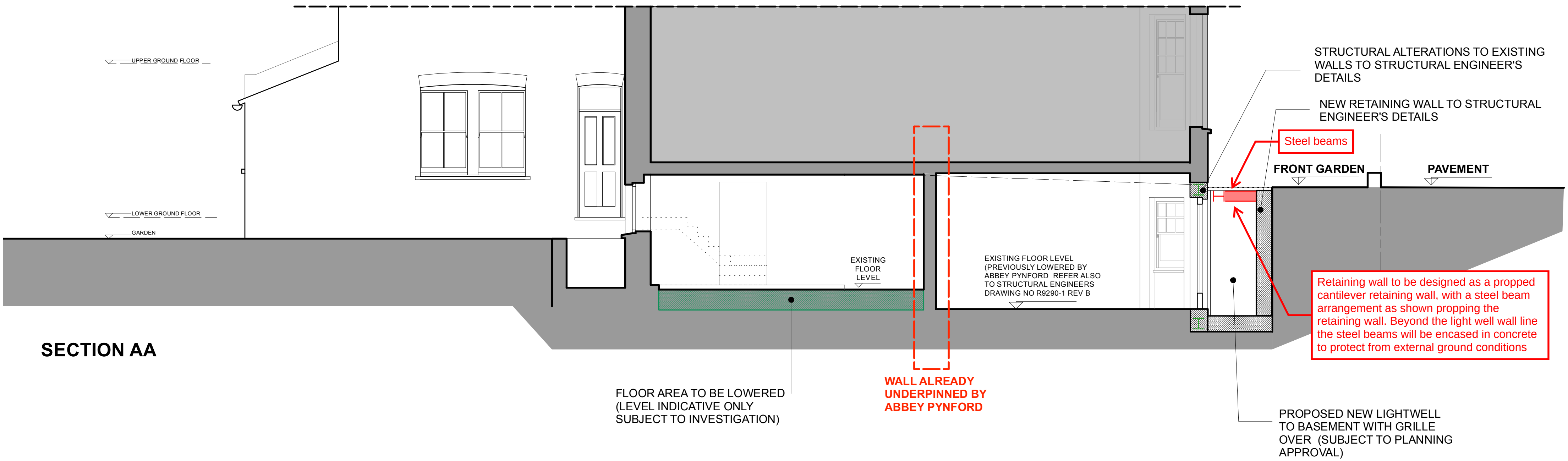
7.0 WATERPROOFING STRATEGY

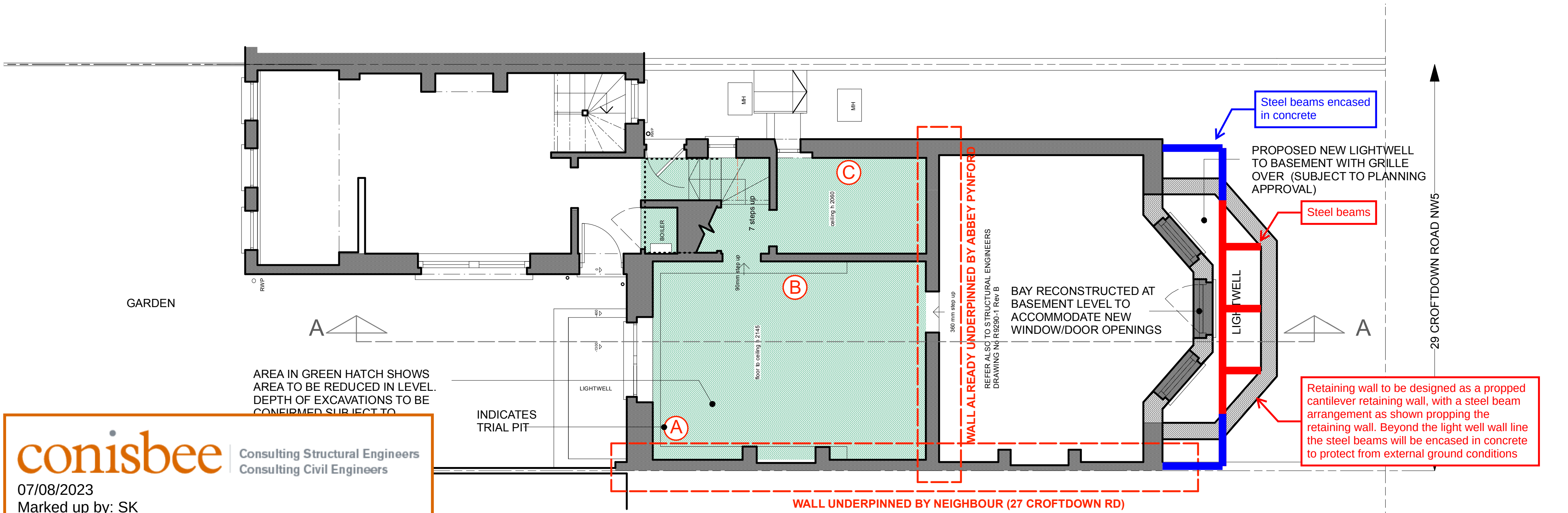
7.1 Details of the waterproofing strategy are within the Architects scope of work.

8.0 IMPACT OF THE SUBTERRANEAN DESIGN AND CONSTRUCTION TO ADJOINING PROPERTIES

8.1 A ground movement analysis has been undertaken and the recommendation provided by GEA will be adopted in the undertaken of the works. The reported impact on the adjoining properties in negligible, category 0 using the Burland scale. Monitoring will be put in place with agreed trigger levels with the adjoining owners to ensure stability at all times.

8.2 The BIA concludes that the proposed development is unlikely to result in any specific land or slope stability issues, ground or surface water issues.





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07/08/2023
Marked up by: SK