9 MARESFIELD GARDENS
LONDON
NW3 5SJ

STRUCTURAL CALCULATIONS

ICT/8972/Rev.B SEPTEMBER 2016

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CONTENTS

1. INTRODUCTION

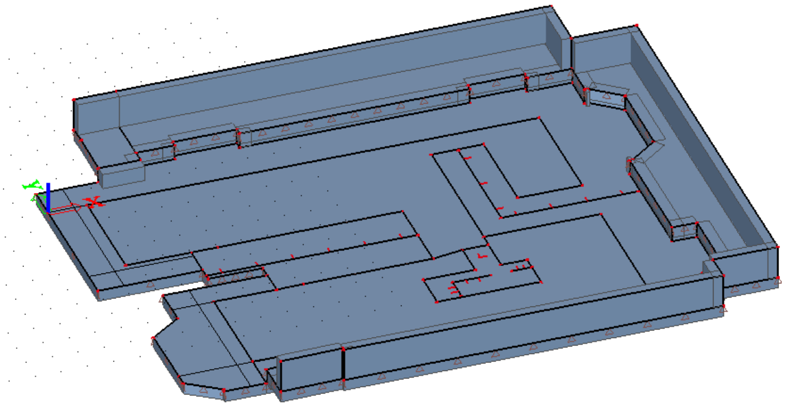
We have been appointed as structural engineers for the basement extension works at 9 Maresfield Gardens. Existing structure is a mosonry building with 2 storeys and a loft floor. Proposed structural works consists of RC underpinning, RC basement slab construction, and extension of masonry to the new basement level. We have designed new steel beams to support the masonry walls where required.

RC ground retaining elements of the structure has been designed considering the soil loads and ground water pressure 1m below the existing ground level.

Ground has been modeled using springs. We have used 10000kN/m^3 stiffness for soil modelling considering a maximum consolidation value of 10mm and 100kN/m^2 maximum ground bearing stress.

We have carried out additional checks for heave effects under the new foundation load distribution. We have reduced the soil support area in the model considering possible heave form installation under ground bearing slab.

SCIA Engineer Version 16 and Tedds 2016 has been used for the part of the calculations carried out in this document.



Structural analysis model

Standards and Reference Documents

BS EN 1991-1-1: General Actions – Densities, self-weight, imposed loads for buildings

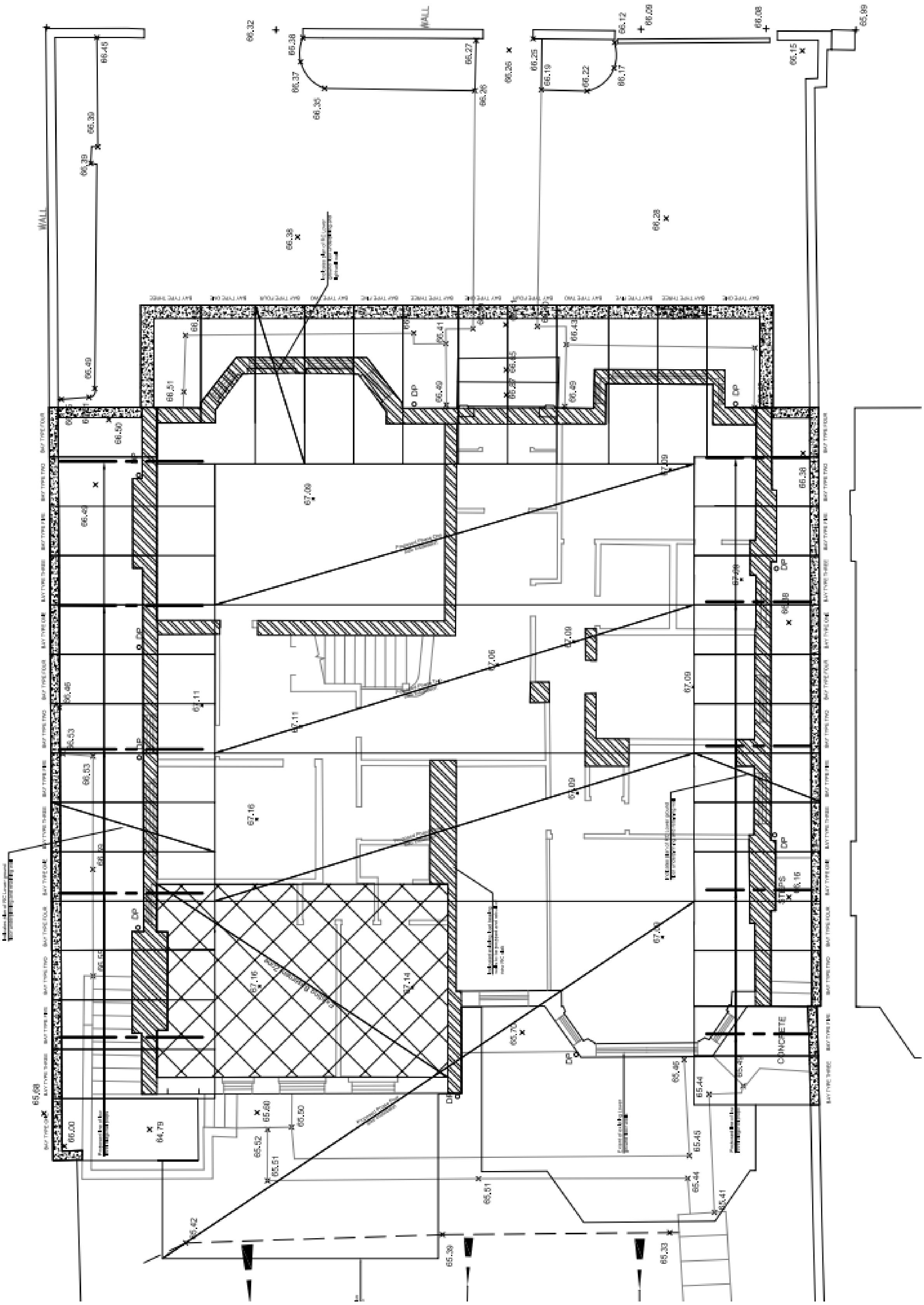
BS EN 1992-1-1: Design of Concrete Structures

BS EN 19931-1: Design of Steel Structures

Along with the above structural codes, UK national annexes have been considered where relevant.

2. STRUCTURAL DRAWINGS

Drawings at this section is to help with calculation reading. Please request latest drawing set for more details.



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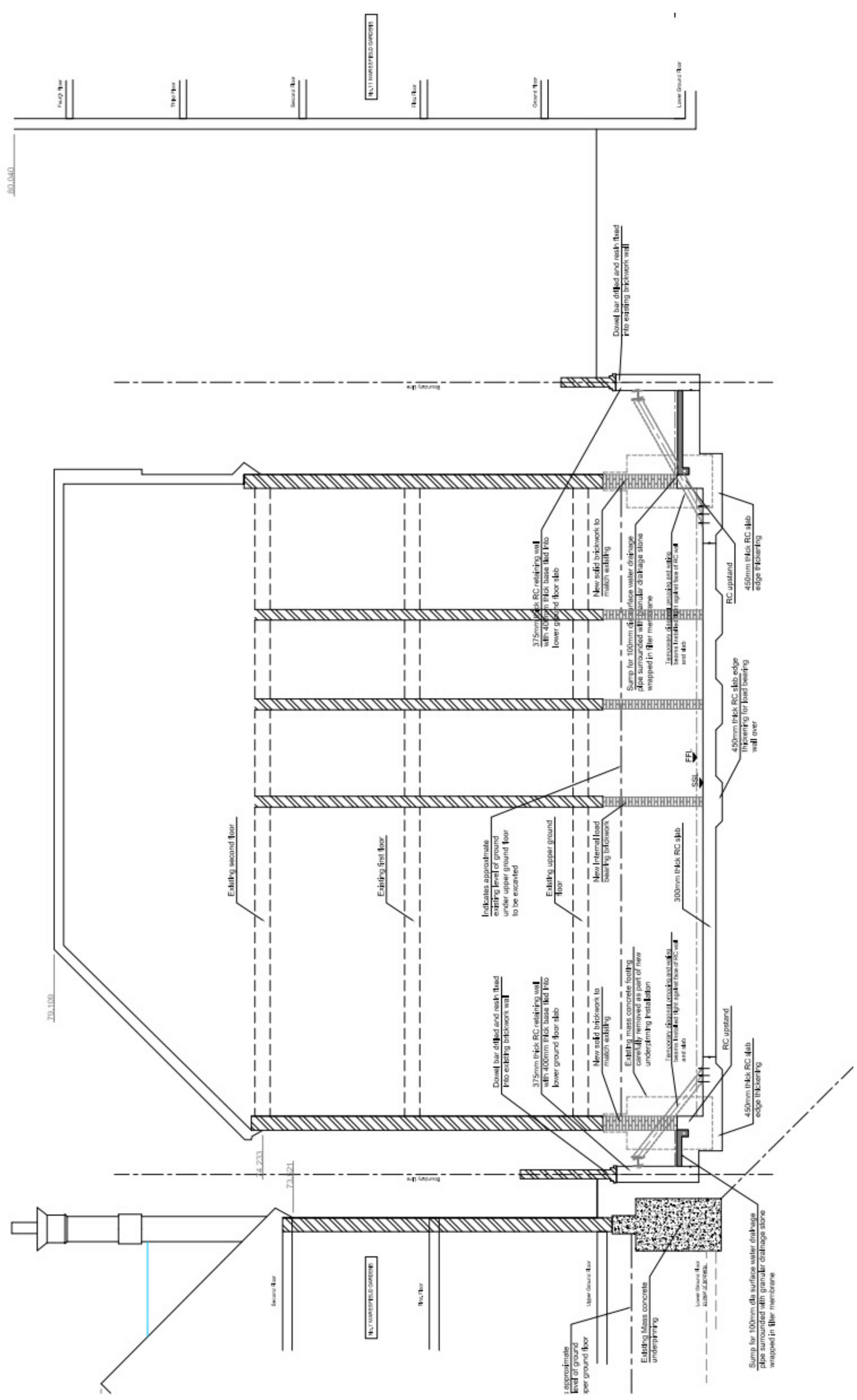

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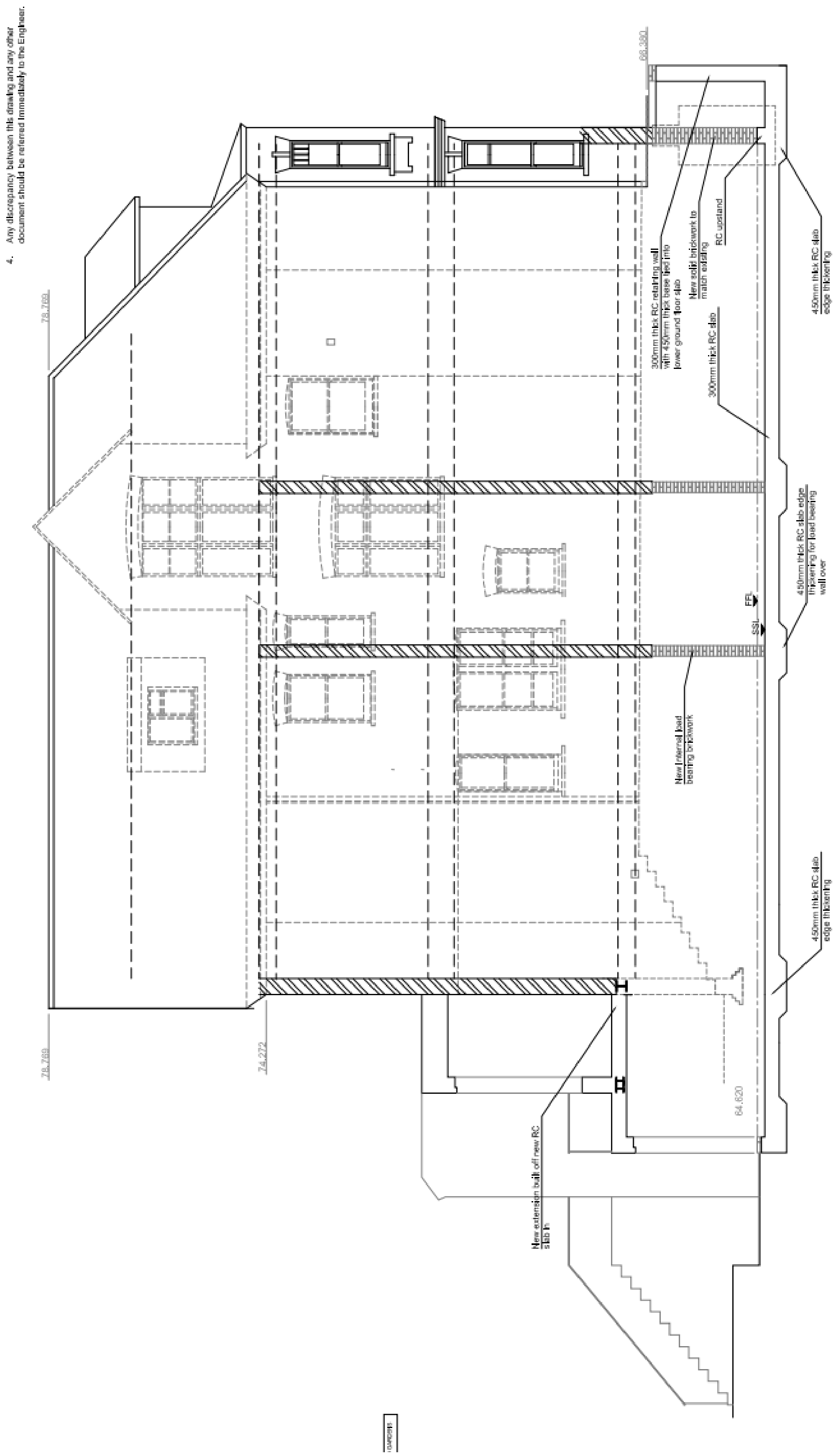
8972	**9 Maresfield Gardens**	Date:	**28.09.2016**
		By:	**I. Tozluoglu**
		Checked:	**U. Mizrahi**

taylor whalley spyra



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3. LOADING

3.1. DEAD LOADS

Existing Timber Floors

Boards + Joists	: $0.35kN/m^2$
Soffite	: $0.15kN/m^2$
Partitions	: $1.00kN/m^2$
Total	: $1.5kN/m^2$

Existing Roof

Tiles	: $0.6kN/m^2$
Battens and feld	: $0.1kN/m^2$
Rafters	: $0.15kN/m^2$
Joists + insulation	: $0.15kN/m^2$
Soffite	: $0.15kN/m^2$
Total	: $1.15kN/m^2$

310mm Thick Masonry Walls

Brick	: $0.310m \times 18kN/m^3 = 5.58kN/m$
Plaster	: 0.25kN/m
Total	: 5.83kN/m

225mm Thick Masonry Walls

Brick	: $0.225m \times 18kN/m^3 = 4.05kN/m$
Plaster for 2 faces	: 0.5kN/m
Total	: 4.6kN/m

100mm Thick Masonry Walls

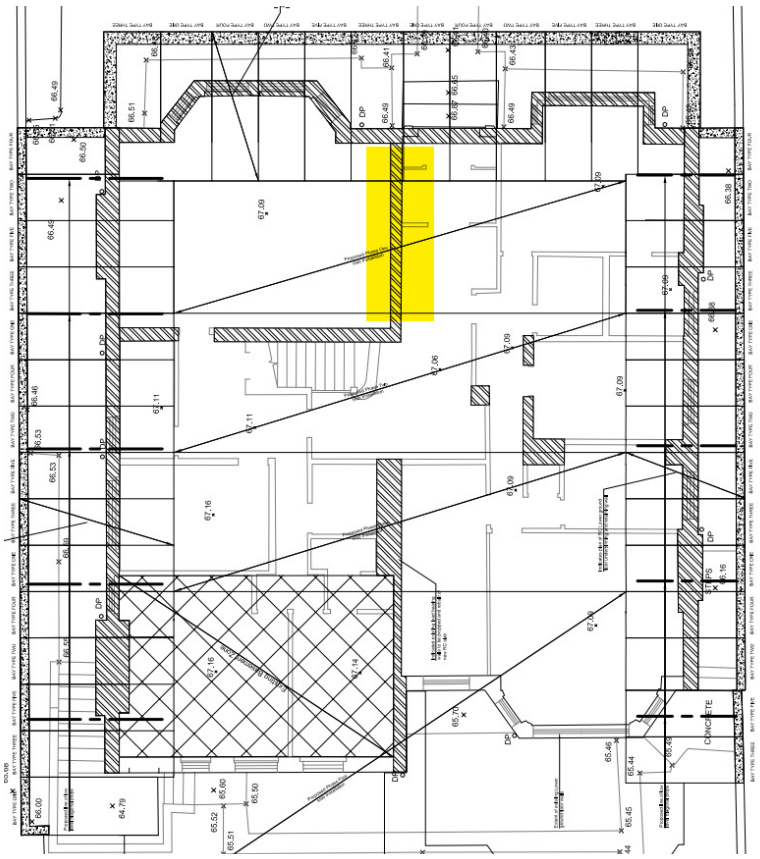
Brick	: $0.102m \times 18kN/m^3 = 1.84kN/m$
Plaster for 2 faces	: 0.5kN/m
Total	: 2.3kN/m

3.2. LIVE LOADS

Residential areas	: $1.5kN/m^2$
Communial Areas	: $3.0kN/m^2$
Roof	: $0.6kN/m^2$

$10kN/m^2$ surcharge load has been considered for the retaining wall design.

4. JUSTIFICATION OF NEW LOADING ON THE FOUNDATIONS



For the marked area:

Existing Load Under Ground Floor Internal Wall Line:

7m (load span) x [2 floors x 1.5kN/m^2 + 1.15kN/m^2 (roof)] + 11 m (wall height) x 4.6kN/m^2 = 79.7kN/m DL

7m (load span) x [2 floors x 1.5kN/m^2 + 0.60kN/m^2 (roof)] = 25.2kN/m DL

Proposed Load Under New Basement Internal Wall Line:

7m (load span) x [3 floors x 1.5kN/m^2 + 1.15kN/m^2 (roof)] + 14.5m (wall height) x 4.6kN/m^2 = 106.3kN/m DL

7m (load span) x [3 floors x 1.5kN/m^2 + 0.60kN/m^2 (roof)] = 35.7kN/m DL

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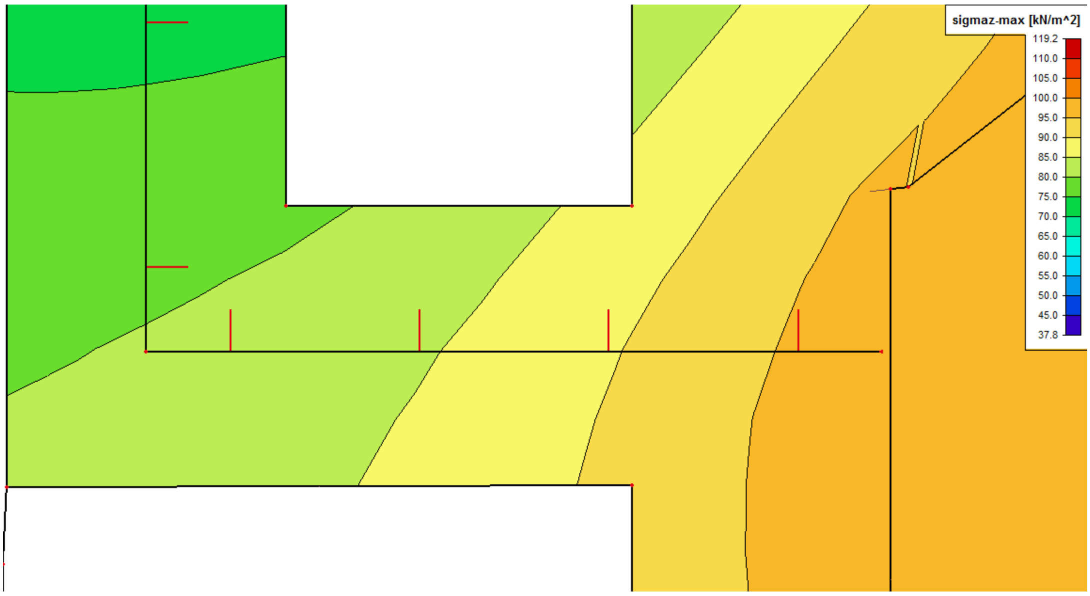
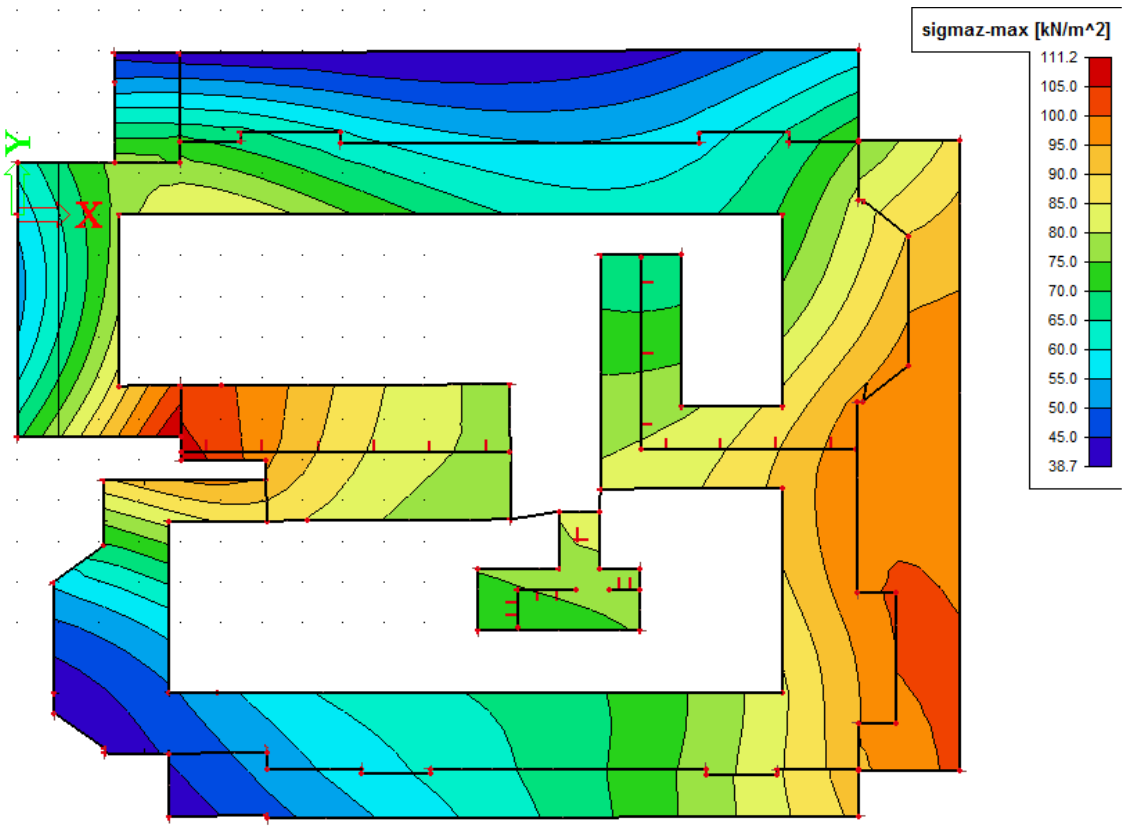
New Load Distribution

Loading Comb.: Dead Load + Live Load + Soil Pressure + Water Pressure



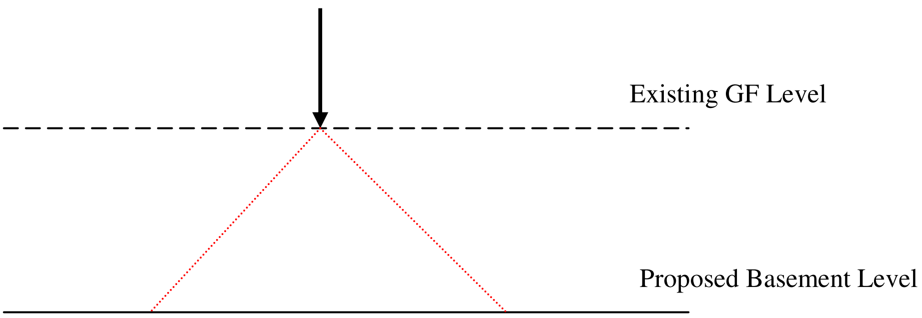
Ground pressure is aprox.100kN/m^2 behind the wall ➔ OK



Overall maximum ground pressure is aprox.111kN/m^2. Area exceeding the assumed ground bearing pressure 100kN/m^2 is a local area and will be redesigned in detail before construction.

HEAVE FORCES

Load at the the new basement level in existing situation



(79.7kN/m+25.2kN/m) / 6m (distribution for 3m deep) + 1.5kN/m^2 (GF floor DL) + 1.5 (GF LL) + 0.15mx20kN/m^3 (overcast concrete) + 2.2mx18kN/m^3 (soil load) ≈ 63kN/m^2

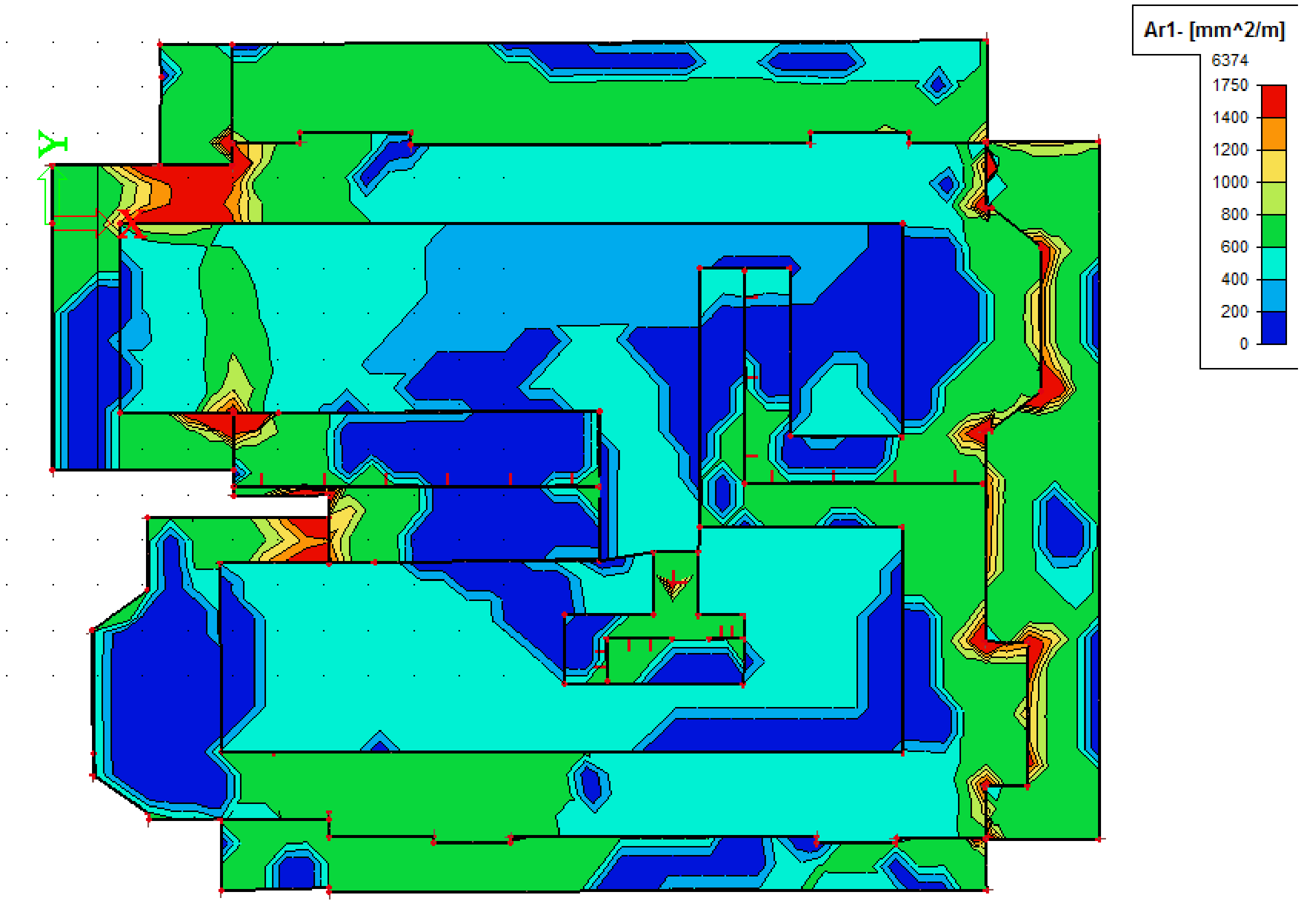
New load distribution approximately retains the existing load distribution at the new basement level after the construction. For this reason we don't expect considerable heave effects on the basement slab.

5. BASEMENT SLAB DESIGN

Preliminary reinforcement design is showing the buildability of the new basement.

Loading Comb.: 1.35Dead Load + 1.50xLive Load + 1.35xSoil Pressure + 1.50Water Pressure

Bottom Reinforcement Ar1-



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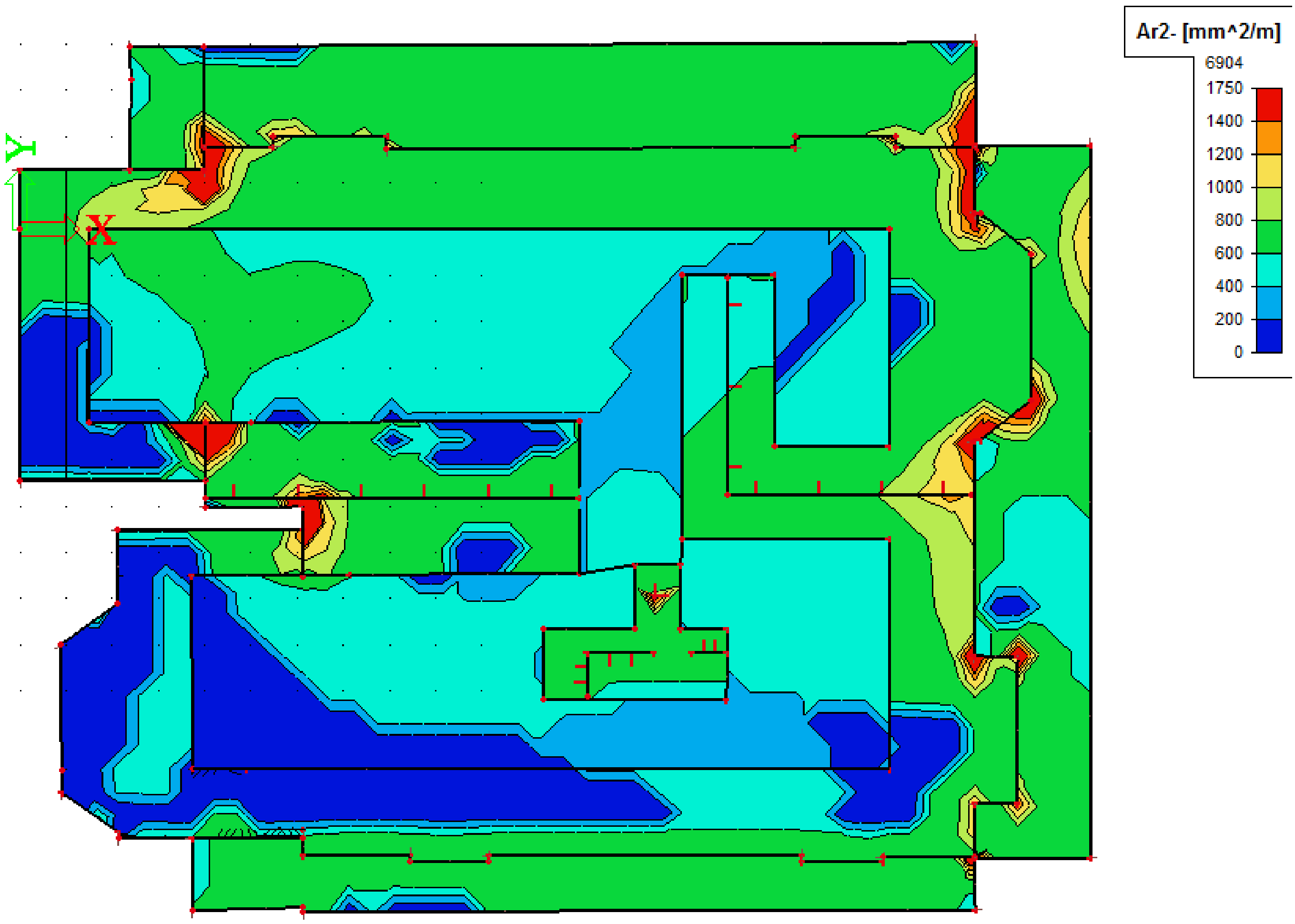
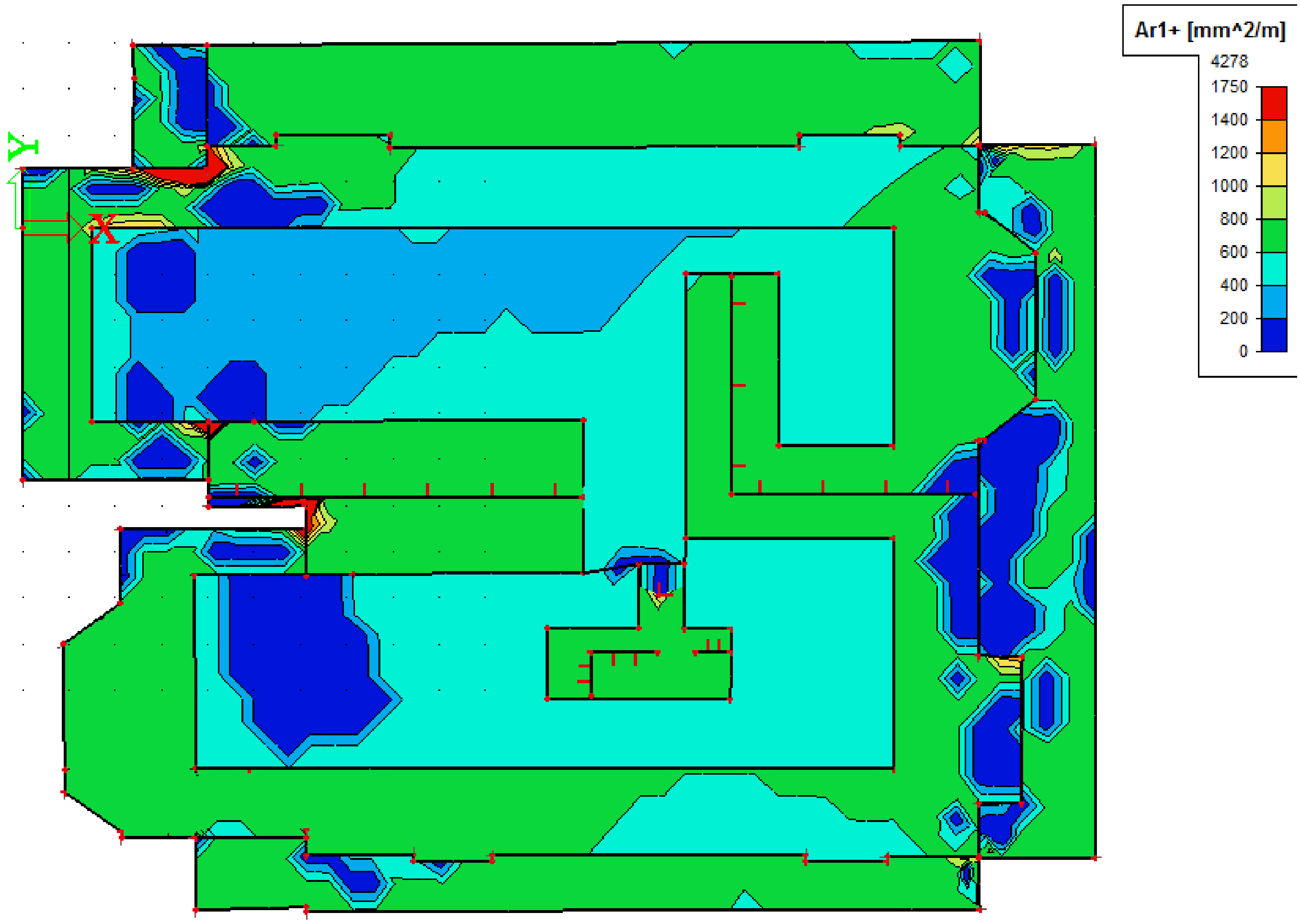
Bottom Reinforcement Ar2-



Top Reinforcement Ar1+



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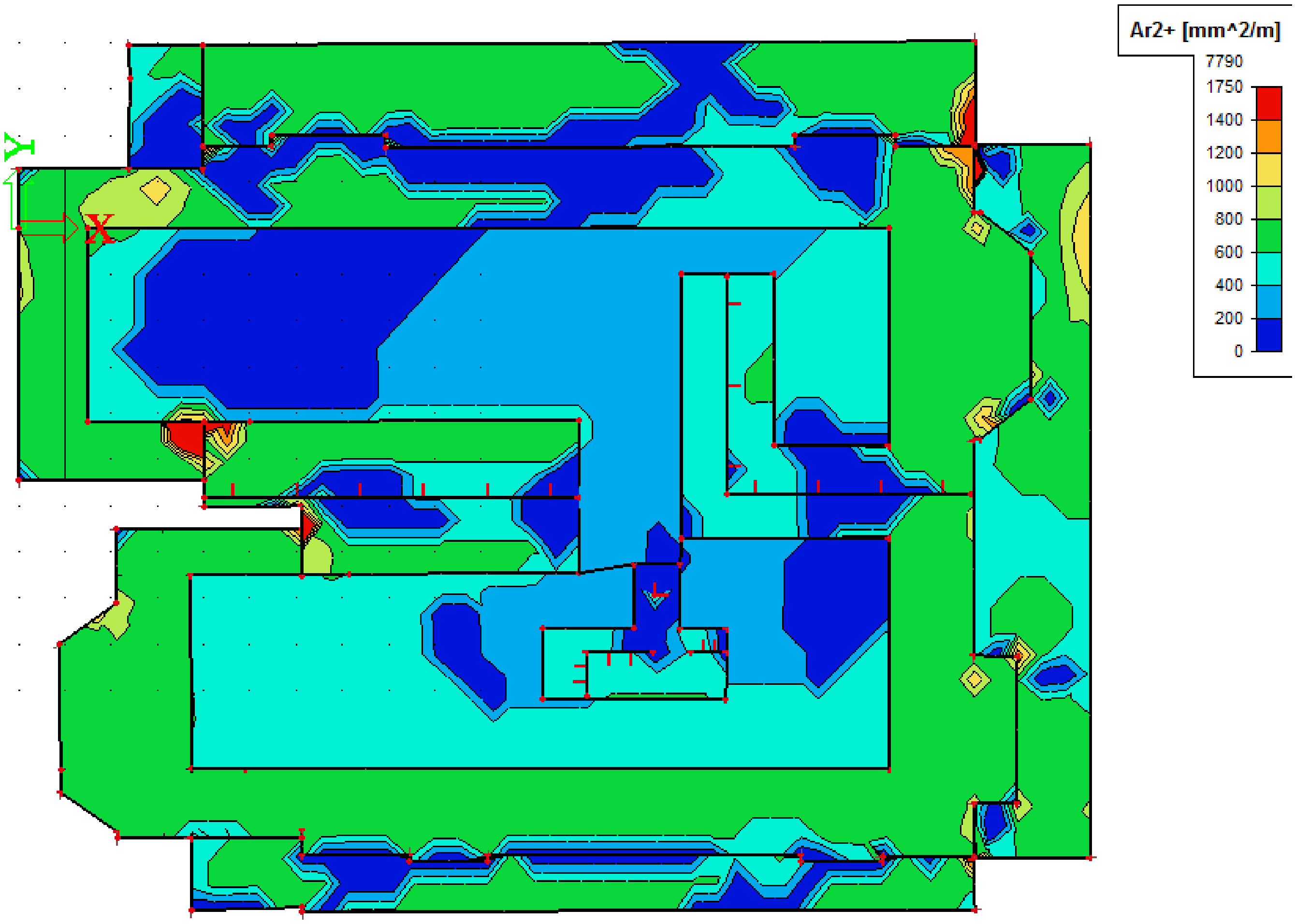
Top Reinforcement Ar2+



6. DESIGN OF MASONRY SUPPORT BEAMS

Loading:

Masonry	8.4m x 5.83kN/m^2 x 0.75	= 36.7kN/m (DL)
Roof	2m x 1.15kN/m^2	= 2.30kN/m (DL)
Floors	3 x 3.3m x 1.5kN/m^2	= 14.9kN/m (DL)

Live Load 3 x 1.5m x 1.5kN/m^2 = 6.75kN/m (LL)

STEEL MEMBER 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

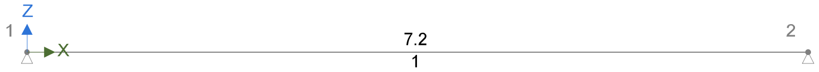
Tedds calculation version 4.1.01

ANALYSIS

Tedds calculation version 1.0.17

Geometry

Geometry (m)



Material - Steel (EC3)

Density	7850 kg/m^3	Youngs Modulus	210 kN/mm^2
Shear Modulus	80.8 kN/mm^2	Thermal Coefficient	0.000012 °C^{-1}

Section type - UC 305x305x240

Area	306 cm^2		
Major moment of inertia	64203 cm^4	Shear area A_y	216 cm^2
Minor moment of inertia	20315 cm^4	Shear area A_z	81 cm^2

Support conditions

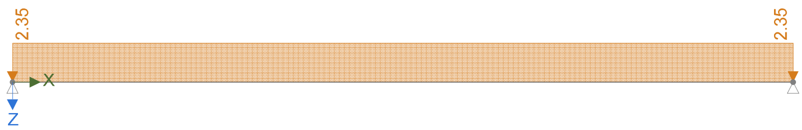
Support 1	X Fixed	Z Fixed	Rotationally Free
Support 2	X Fixed	Z Fixed	Rotationally Free

Spans

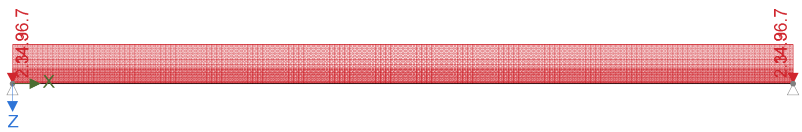
Span 1 7.2 m

Loading

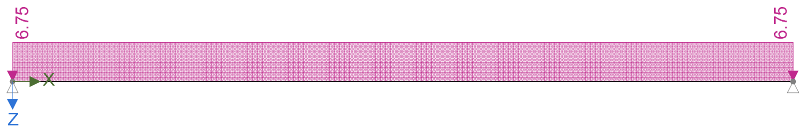
Self Weight - Loading



Permanent - Loading



Imposed - Loading



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00
1.0G + 1.0ψ_2Q (Quasi)	1.00	1.00	0.30

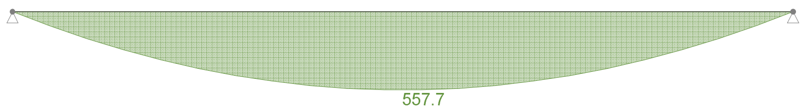
Member UDL loads

Member	Load case	Position Type	Start	End	Load (kN/m)	Orientation
Beam	Permanent	Ratio	0	1	36.7	GlobalZ
Beam	Permanent	Ratio	0	1	2.3	GlobalZ
Beam	Permanent	Ratio	0	1	14.9	GlobalZ
Beam	Imposed	Ratio	0	1	6.75	GlobalZ

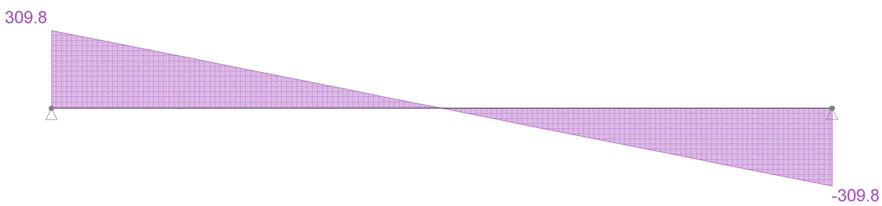
Results

Forces

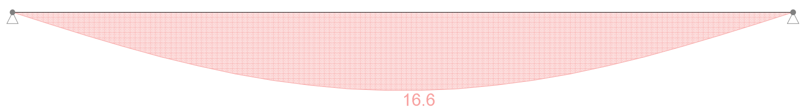
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



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Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1**

Resistance of members to instability; γ_{M1} = **1**

Resistance of tensile members to fracture; γ_{M2} = **1.1**

Beam - Span 1

Section details

Section type; UC 305x305x240 (BS4-1)

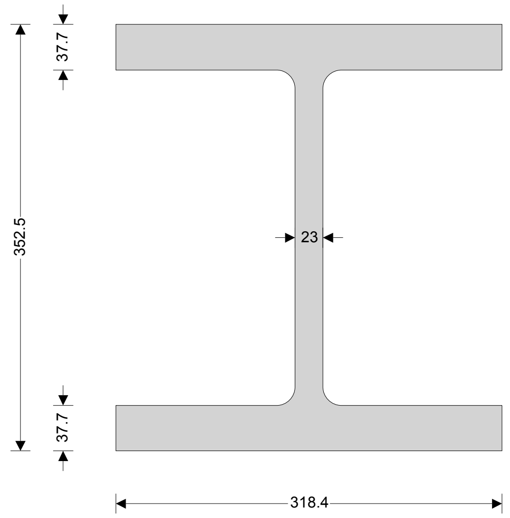
Steel grade - EN 10025-2:2004; S275

Nominal thickness of element; $t_{nom} = \max(t_f, t_w) = \mathbf{37.7}$ mm

Nominal yield strength; $f_y = \mathbf{265}$ N/mm^2

Nominal ultimate tensile strength; $f_u = \mathbf{410}$ N/mm^2

Modulus of elasticity; $E = \mathbf{210000}$ N/mm^2



UC 305x305x240 (BS4-1)
Section depth, h, 352.5 mm
Section breadth, b, 318.4 mm
Mass of section, Mass, 240 kg/m
Flange thickness, t_f, 37.7 mm
Web thickness, t_w, 23 mm
Root radius, r, 15.2 mm
Area of section, A, 30579 mm^2
Radius of gyration about y-axis, i_y, 144.899 mm
Radius of gyration about z-axis, i_z, 81.507 mm
Elastic section modulus about y-axis, $W_{el.y}$, 3642697 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 1276042 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 4247074 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 1950585 mm^3
Second moment of area about y-axis, I_y, 642025279 mm^4
Second moment of area about z-axis, I_z, 203145868 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

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

Width of section; $c = d = \mathbf{246.7}$ mm

$c / t_w = 10.7 = 11.4 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \mathbf{132.5}$ mm

$c / t_f = 3.5 = 3.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \mathbf{277.1}$ mm; $\eta = \mathbf{1.000}$

$h_w / t_w = 12 = 12.8 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \mathbf{309.8}$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{8585}$ mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \mathbf{1313.5}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.236}$

PASS - Design shear resistance exceeds design shear force

Check design 3600 mm along span

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \mathbf{557.7}$ kNm

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Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **1125.5** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.496**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

Poissons ratio; $\nu =$ **0.3**

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

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_T} =$ **7200** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **3574.3** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.561**

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

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

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} =$ **0.34**

Correction factor for rolled sections; $\beta =$ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.645**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.934**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.973**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.960**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **1080.3** kNm

$M_{y,Ed} / M_{b,y,Rd} =$ **0.516**

PASS - Design buckling resistance moment exceeds design bending moment

<u>Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)</u>

<u>Check design 3600 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y =$ **16.6** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1\_s1} / 360 =$ **20** mm

$\delta_y / \delta_{y,Allowable} =$ **0.829**

PASS - Allowable deflection exceeds design deflection

7. UNDERPINNING

Load distribution under the underpinning toe during temporary condition:

Existing brick wall load:	4.6kN/m^2 x 2.25m	= 10.6kN/m
RC underpinning wall	25kN/m^3 x 0.375m x 1.5m	= 14.1kN/m
RC underpinning base	25kN/m^3 x 0.4m x 2.9m	= 29kN/m
Total		= 53.7kN/m

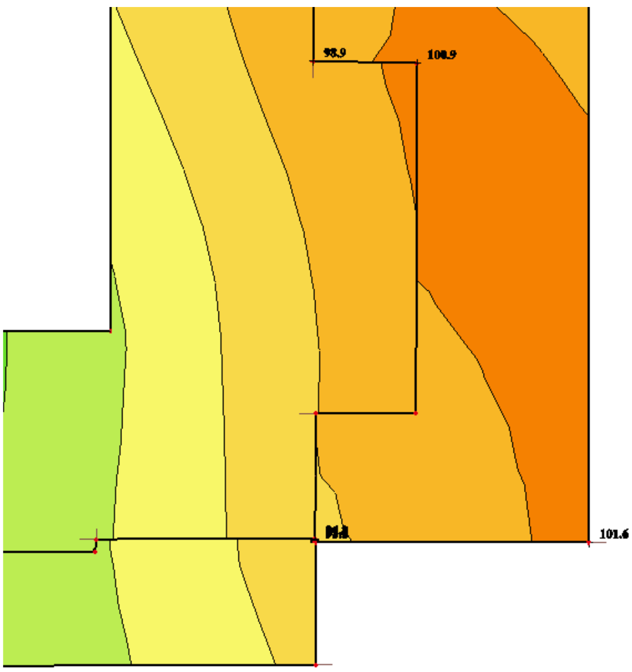
Soil pressure under the underpinning base during temporary condition:

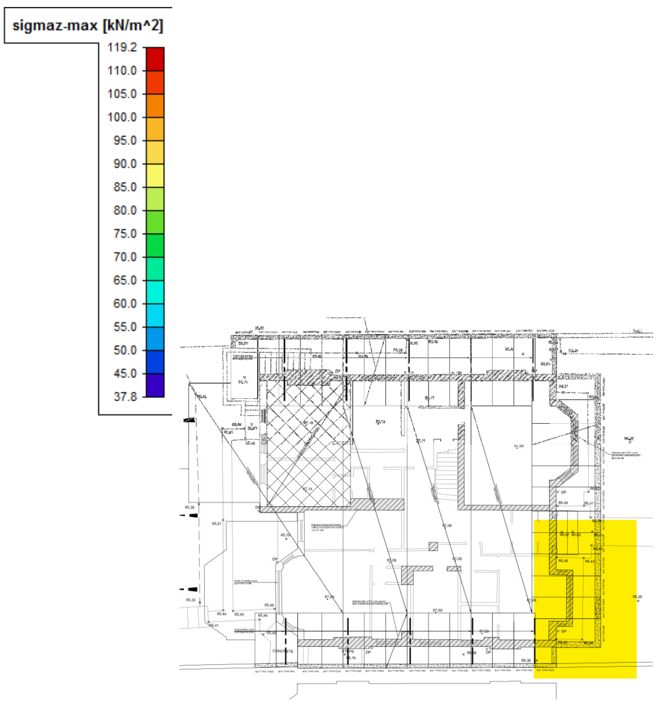
53.7kN/m / 2.9m = 18.5kN/m^2

After the construction of all underpinning stages and completion of the basement slab all RC members below ground slab level will work as a monolythic element.

Maximum Soil Pressure Area Under Underpinning Base in Permanent Condition

Loading Combination: Dead Load + Live Load + Soil Pressure + Water Pressure





8. RETAINING WALL ANALYSIS AND DESIGN

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 2.6.11

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **2255** mm
Stem thickness;	t_{stem} = **300** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **2500** mm
Base thickness;	t_{base} = **400** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **2255** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **0** mm
Height of water;	h_{water} = **1255** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

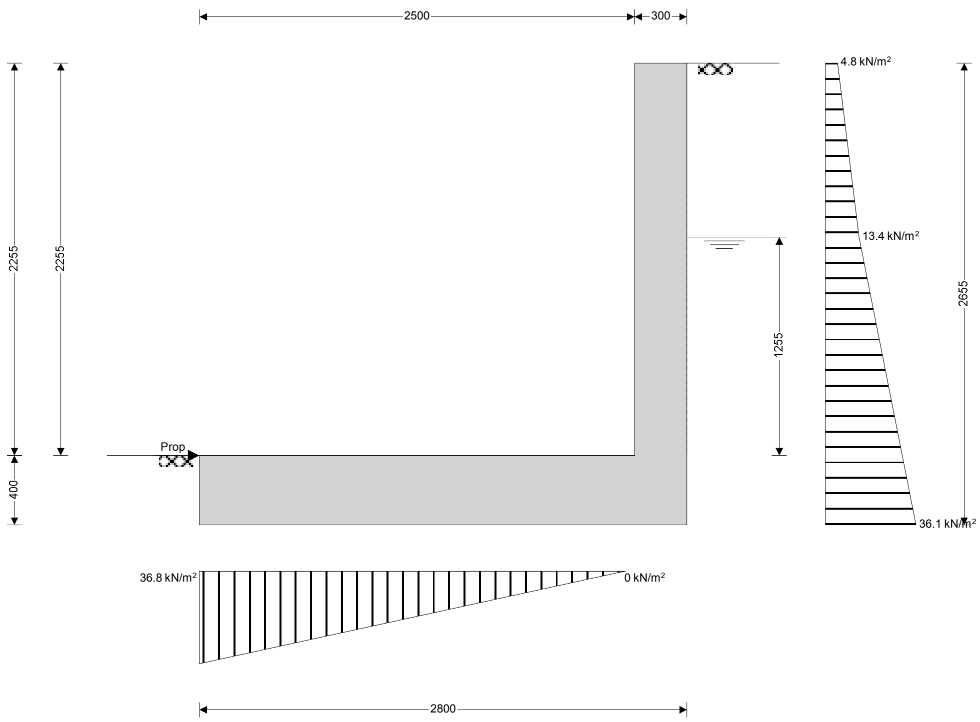
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** 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;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} =$ **2800** mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover} =$ **1255** mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water} =$ **1000** mm
Length of surcharge load;	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **2800** mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **2655** mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 =$ **1327** mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} =$ **0.677** m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **2650** mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} =$ **1.12** m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 =$ **1400** mm

Using Coulomb theory

Active pressure coefficient;	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) =$ **0.483**
Passive pressure coefficient;	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) =$ **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem;	$F_{stem} = A_{stem} \times \gamma_{stem} =$ **16.9** kN/m
Wall base;	$F_{base} = A_{base} \times \gamma_{base} =$ **28** kN/m
Total;	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} =$ **44.9** kN/m

Horizontal forces on wall

Surcharge load;	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} =$ **12.7** kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{5.4}$ kN/m

Water; $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{13.4}$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{18.5}$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-3.4}$ kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{46.6}$ kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} = \mathbf{44.8}$ kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} = \mathbf{39.2}$ kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-16.8}$ kNm/m

Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-3}$ kNm/m

Water; $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-7.4}$ kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-20.3}$ kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = \mathbf{36.5}$ kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h} = \mathbf{46.6}$ kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v} = \mathbf{814}$ mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 = \mathbf{-586}$ mm

Loaded length of base; $l_{load} = 3 \times \bar{x} = \mathbf{2441}$ mm

Bearing pressure at toe; $q_{toe} = 2 \times F_{total\_v} / l_{load} = \mathbf{36.8}$ kN/m^2

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

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{2.717}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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 2.6.11

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

Concrete strength class; C30/37

Characteristic compressive cylinder strength; $f_{ck} = \mathbf{30}$ N/mm^2

Characteristic compressive cube strength; $f_{ck,cube} = \mathbf{37}$ N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm^2 $\times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm^2 $\times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2

Partial factor for concrete - Table 2.1N; $\gamma_C = \mathbf{1.50}$

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

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2

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

Reinforcement details

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

Modulus of elasticity of reinforcement; $E_s = \mathbf{200000}$ N/mm^2

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S = \mathbf{1.15}$

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

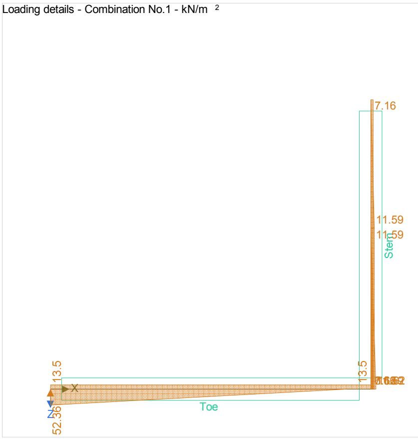
Cover to reinforcement

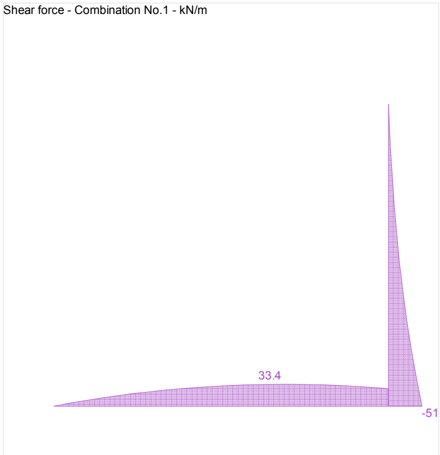
Front face of stem; $c_{sf} = \mathbf{40}$ mm

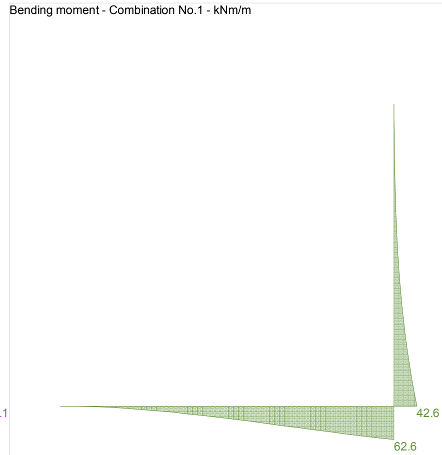
Rear face of stem; $c_{sr} = \mathbf{50}$ mm

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

Bottom face of base; c_{bb} = **75** mm







Check stem design at base of stem

Depth of section; h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **42.6** kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2$ = **244** mm

$K = M / (d^2 \times f_{ck})$ = **0.024**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **232** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **31** mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z)$ = **423** mm^2/m

Tension reinforcement provided; 12 dia.bars @ 150 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **754** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **368** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h$ = **12000** mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.561**

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

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**

Required tension reinforcement ratio; $\rho = A_{sr.req} / d$ = **0.002**

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

Structural system factor - Table 7.4N; K_b = **0.4**

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

Limiting span to depth ratio - exp.7.16.a; $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **55.5**

Actual span to depth ratio; h_{stem} / d = **9.2**

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} = **25.4** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **145.2** 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) = **89833** mm²/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

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

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **6.091**

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; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **413** 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 = **0.18** mm

w_k / w_{max} = **0.6**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **51.1** kN/m

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

k = min(1 + √(200 mm / d), 2) = **1.905**

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

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.504** N/mm²

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

$V_{Rd.c}$ = **123** kN/m

$V / V_{Rd.c}$ = **0.415**

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); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **300** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2); s_{sx_max} = **400** mm

Transverse reinforcement provided; 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** 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 = **62.6** kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2$ = **317** mm

$K = M / (d^2 \times f_{ck})$ = **0.021**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **301** mm

Depth of neutral axis; x = 2.5 × (d – z) = **40** mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z)$ = **478** mm²/m

Tension reinforcement provided; 16 dia.bars @ 150 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **477** mm²/m

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

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.357**

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

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} = **44.5** kNm/m
Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **110.2** N/mm^2
Load duration;	Long term
Load duration factor;	k_t = **0.4**
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **120125** mm^2/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio;	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.011**
Modular ratio;	$\alpha_e = E_s / E_{cm}$ = **6.091**
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;	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **499** 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 = **0.165** mm
	w_k / w_{max} = **0.55**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

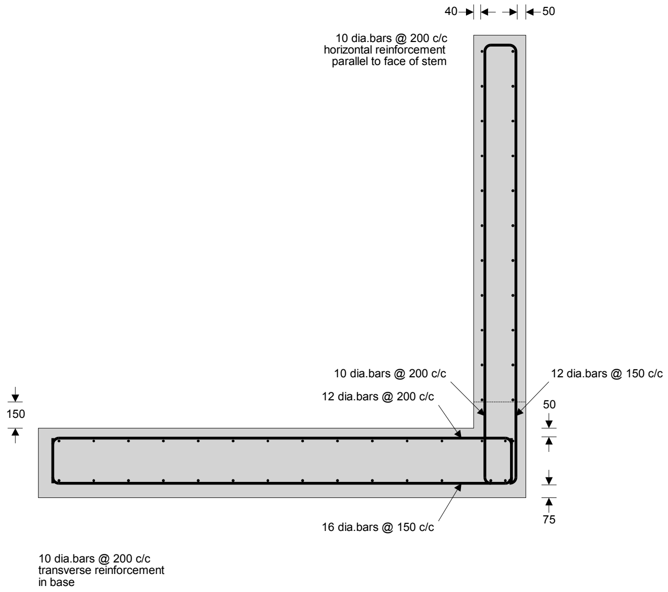
Design shear force;	V = **33.4** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.794**
Longitudinal reinforcement ratio;	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.461** N/mm^2
Design shear resistance - exp.6.2a & 6.2b;	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **159.2** kN/m
	$V / V_{Rd.c}$ = **0.210**

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);	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **268** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3);	s_{bx_max} = **450** mm
Transverse reinforcement provided;	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided;	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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



Reinforcement details