45 Maresfield Gardens

Basement Impact Assessment
Part 3 of 3

Knapp Hicks and Partners

with Doyle Town Planning and Urban Design

for Mr Shai Greenberg/ Hestia Developments

March 2015

APPENDIX H

45 Maresfield Gardens

Structural and Temporary Works Drawings & Calculations

Prepared By Malachy Walsh and Partners

And

Martin Redston Associates

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX ✓
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

CALCULATIONS
FOR
45 Maresfield Gardens
NW3 London

14.073 July, 2014

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date Feb 2014
Eng. RS
Job No. 14 073.
Sheet 1

45 Maresfield Gdns
NW3, London

Flat extention beams: on new erected flat plan.
B2.1 → above kitchen: length: 4,2m

D. Load: Pitched Roof: 1,25 kN/m² · 2,5m = 3,2 kN/m
Reinforced concrete floor 8,5 kN/m² · 2,85 m = 25,0 kN/m
Timber floor: 5m · 0,5 · 0,5 kN/m² = 1,25 kN/m
cavity wall: 1,5 m · 4,5 kN/m² = 6,8 kN/m
flat roof: 0,75 kN/m² · 1,6m = 1,2 kN/m
38, kN/m

V. loads: Roof (Pitched + flat) (2,5m + 1,6m) · 0,75 kN/m² = 3,7 kN/m
Residential L. (floor) (3,45m + 2,5m) · 1,5 kN/m² = 9 kN/m
12,7 kN/m

B1.1 → supporting back kitchen wall at 2nd level, [flat]
Length: 3,3m
DL: cavity wall 3,0m · 4,5 kN/m² = 13,5 kN/m
sw eight: 0,6 kN/m / 14,1 kN/m
$R_L = R_R = \frac{14,1 \text{ kN/m} \cdot 3,3 \text{ m}}{2} = 23,9 \text{ kN}$ } reactions at ends of beam

B1.2 → supporting front kitchen wall at 2nd fl. level (flat)
the same load & span as B1.1.

BY TEDDS calculations, sheet no T1. Use UC 152 × 152 × 37

Beam 1.3.
length: 3,3m

Dead L (inner leaf):
Flat Roof: 1,6 m · 0,75 kN/m² = 1,2 kN/m
B & B floor = 3,0 kN/m² · 2,8 m = 8,4 kN/m
cavity wall (internal leaf only) = 2,0 kN/m² · 4,0m = 8 kN/m
timber floor: 2m · 0,5 kN/m² = 1 kN/m
Pitched Roof: 2,5m · 1,25 kN/m² = 3,2 kN/m
Concrete slab 24,23 kN/m
Σ 46,0 kN/m

Inner leaf [VL]: Residential 1,5 kN/m² · (5,6m + 2m) = 11,4 kN/m
Pitched roof 0,75 kN/m² · 2,5 m = 1,9 kN/m
flat Roof: 1,2 kN/m
14,5 kN/m

BY TEDDS calculations, sheet no T2
Use 203 × 203 UC 71

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date Feb 2014
Eng. RS
Job No. 14.073
Sheet No. 2

45 Maresfield Gdns
NW3, London

Beam B1.3
Outer leaf [3,3m span design]
[DL]
cavity outer leaf (2x external block)
$1,5 m \cdot 5,1 kN/m^2 = 7,7 kN/m$
cavity outer leaf (1x external brick)
$2,5 m \cdot 2,5 kN/m^2 = 6,25 kN/m$ — 14,0 kN/m

Point load (B 1.2) at 0,6 m
Dead Load Only:
$R_{12} = 23,9 kN$

Use 203 x 133 UB30 + 6mm top plate
or 203 x 203 UC46
TEDDS calculations, sheet no T3

Beam B 1.5
[supporting staircase leading to flat above]
Length: 3,3 m.
D.L: 100x block: $1,5kN/m^2 \cdot 2,5m = 3,8 kN/m$
RC Slab $2,8 m \cdot 6,5 kN/m^2 = 18,2 kN/m$
(+ [illegible]) B & B floor: $3,0 kN/m^2 \cdot 2,8 m = 8,4 kN/m$
30,2 kN

VL: Residential:
$2,8m \times 2 \cdot 1,5 kN/m^2 = 8,4 kN/m$
Partition wall:
$2,5 m \cdot 0,5 kN/m^2 = 1,3 kN/m$
Σ 9,7 kN/m

Use 203 x 203 UC46 in reference to
TEDDS calculations, sheet no T4

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date	Feb 2014
Eng.	M.S.
Job No.	14.073
Sheet No.	3

45 Meresfield gardens
NW3 London

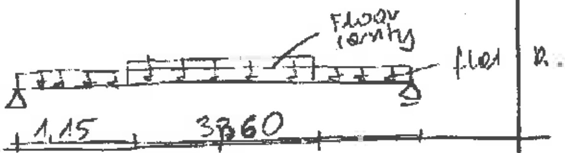
BG1 -> Beam Replacing cranked beam under Bay window:

length 5,45m

D. Loads: Flat Roof: 0,75m · 0,75 kN/m² = 0,6 kN/m
Cavity wall: 8" b/wk + Block: 7,5 kN/m² · 3m = 22,5 kN/m
Cavity wall: 13" b/wk + Block: 10 kN/m² · 0,6m = 6 kN/m
RC floor: 1m · 0,25m · 25 kN/m³ = 6,25 kN/m

Σ 35,4 kN/m

V. Loads: Flat R: 0,6 kN/m
Floor: 1,5 kN/m
Σ 2,1 kN/m



USE 254 x 254 UC 167

BG3 -> Box Frame: [length = 5,45m]

D.L: Flat Roof: 0,8m · 0,75 kN/m² = 0,6 kN/m
Glass Roof: 1,5 kN/m² · 0,4m = 0,6 kN/m

VL: Roof: 2,3m · 0,75 kN/m² = 1,73 kN/m

Use 203 x 133 UB 30 [Beam on ground floor level, see drawing no 2]
SEE TEDDS calculations, sheet no T5

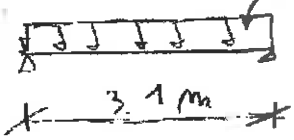
BG4 -> supporting folding door – design as lintel.
length: 2,400 m
Cavity wall: 3,8m · 4,5 kN/m² = 17,1 kN/m
Use UC 203 x 133 x 30
SEE TEDDS calculations, sheet no T6

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date Feb 2014	Sheet No. 4
Eng. RS	
Job No. 14.073	

45 Haresfield Gardens
NW3 London

BG2 - supporting b/w above
length = 3,10m
Dead L: 3,8m · 4,5kN/m² = 17,1kN/m



BY TEDDS calculations, sheet no 7;

Use UC 203 × 203 × 52
reaction at each end: 32 kN (perm only)

~~Beam BG5 length = 1,9m~~
~~Dead: Floor · 6,25 kN/m² · (2,6m + 1,9m) = 28,2 kN/m~~
~~Variable: Residential: 1,5 kN/m² · 3,5m = 5,3 kN/m~~

~~Point Load (at 1,0m)~~
~~Beam BG2, support B~~
~~permanent: 257,3 kN~~
~~variable: 40,0 kN~~

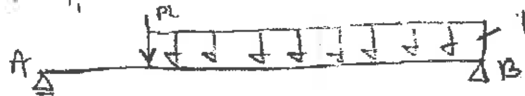
~~203 × 203 × 86~~

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date 23 July 2014	Sheet No. 5
Eng. RS	
Job No. 14.073	

45 Mansfield Gdns
NW3 London

Beam RC 3 - Revised
length: 4,55 m



UDL loads

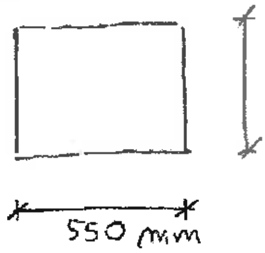
D.L: 9" blck + 140 block = 0,230m · 18kN/m³ + 0,14m · 24 kN/m³ =
→ 7,73 kN/m² - weight of wall

7,73 kN/m² · 3,8m = 30 kN/m UDL loads

$$PL = \frac{30\,kN/m \cdot 2{,}1\,m}{2} = 32\,kN$$ } Part of Ring Beam housing point load on Beam RC 3

For analysis - see TEDDS calculations, sheets T8 - T9

Reinforced Beam



z = 0,95d
d = h - cnom - Ø10-Ø20
d: 340 mm
z = 0,95 · 306mm
$F_{yd} - \frac{f_{yk}}{1{,}15}$
SIKA Carbodur : f_{yds} = 0,65 · 2800 MPa = 1820 MPa
f_{yds} - design Tensile strength

M_{max} = 158 kNm
Max shear - support A = 109 kN = V_{Amax}
Max shear - support B = 128 kN = V_{Bmax}

$$K = \frac{158\,kNm \cdot 10^6\,Nmm}{550\,mm \cdot (340\,mm)^2 \cdot 30\,N/mm^2} = 0{,}083$$

$k' = 0{,}598 \cdot 1 - 0{,}181 \cdot 1^2 - 0{,}21 = 0{,}207$ $k' > k$ - compression reinforcement not needed

Span : $A_{req} = \frac{M_{max}}{f_{yds} \cdot z} = 285\,mm^2$
Use 3x S812 SikA carbodur plates.
96 mm² · 3 = 288 mm² > A_{req}

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date 23 July 2014
Eng. RS
Job No. 14.073
Sheet No. 6

45 Maresfield Gdns
NW3 CONTROL

Support Moment → $M_{max} = \beta \cdot 153\,kNm = 38\,kNm$ — $\beta = 0,25$

$k' = 0,598 \cdot 1 - 0,181 \cdot 1 - 0,21 = 0,207$

$k = \frac{40\,kNm}{350mm \cdot (340mm)^2 \cdot 30\,MPa} = 0,021$

$k' > k$ - no compression reinforcement required

$A_{s\,req} = \frac{40\,kNm}{1820\,MPa \cdot 306mm} = 72,0\,mm^2$ } TENSION [TOP] REINFORCEMENT

APPLY FOR BOTH SUPPORTS.

Use 1x S812 Sika Carbodur plates. Apply for Both supports

$A_{s\,prov} = 86mm^2 > A_{s\,req}$

Shearing reinforcement (span)

$$A_{sv\,min} = \frac{0,08\,MPa \cdot 350mm \cdot \left(\frac{30\,MPa}{1\,MPa}\right)^{0,5}}{1820\,MPa} = 132\,mm^2/m_{min}$$

Use S812 plates @ 300 c/c

supports:

$V_{Ed\,max} = 128,0\,kN$

$V_{Ed} = \frac{128,0\,kN}{550mm \cdot 306mm} = 0,761\,MPa$

$A_{s\,req} = \frac{0,761\,MPa \cdot 550mm}{1820\,MPa} = 230\,mm^2/m_{min}$

Use S812 plates @ 300 c/c

Martin Redston Associates
Consulting Civil & Structural Engineers

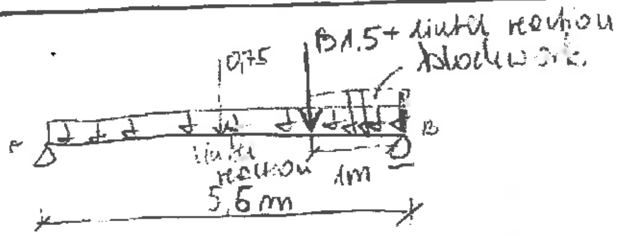
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date Feb 2014	Sheet No. 7
Eng. RS	
Job No. 14.073	

45 Maresfield Gardens
NW3, London



DL sweight : 6,5 kN/m

blockwork : $1,5\ kN/m^2 \cdot 2,5m = 3,8\ kN/m$

reactions of lintels : 0,75 kN at each end of lintel.

Point load of beam 1.5 :

Perm : 50,6 kN + lintel reaction = 51,4 kN

Variable : 16 kN

$\Sigma M_{A\,perm} = -2,8m \cdot 6,5kN/m \cdot 5,6m - 51kN \cdot 4,6m -$
$-(4,6m + 0,5m) \cdot 3,8kN \cdot 1 + 5,6m \cdot R_B = 0$

$\Rightarrow$ $R_B = 64\ kN$ - perm value

$R_A = 28\ kN$ - perm value

$\Sigma M_{P\,var} = -16kN \cdot 4,6m + R_{B\,var} \cdot 5,6m = 13,0kN$

$R_B = 13,0\ kN$ variable

$R_A = 2,9\ kN$ Variable.

Max shearing $\Rightarrow$ support B : $V_{max} = 77\ kN$ unfactored

$V_{max} = 105,9 \rightarrow$ factored.

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date Feb 2014
Eng. RS
Job No. 14 073
Sheet No. 8

45 Maresfield Gardens
NW3, London

Max moment at 4,6m [unfactored value]

$M_{max} = 4{,}6m \cdot (-2{,}3m) \cdot 6{,}5 kN/m + 4{,}6m \cdot (29kN + 2{,}9kN) =$ 77,97 kN/m

Factored: $= -68{,}77 \cdot 1{,}35 + 133{,}4 kNm \cdot 1{,}35 +$
$+ 133{,}4 kNm \cdot 1{,}5 = 110 kNm$

Reinforcing Slab:

$A_{s,req} = \frac{110 kNm}{f_{yds} \cdot z}$

150mm
450mm

$z = 150mm = h$

$f_{yds} = 1820 MPa$

$A_{s,req} = \frac{110 kNm}{1820 MPa \cdot 0{,}15 m} = 402 mm^2$

Use 3× Sika CarboDur S1512

$3 \times 180 mm^2 \Rightarrow$ 1,2 mm
$= 540 mm^2 > A_{s,req}$ 150 mm

Shearing reinforcement:

$V_{max} = 105{,}8 kN$

$V_{ED} = \frac{105{,}8 kN}{300mm \cdot 150mm} = 2{,}4 MPa$

$A_{s,req} = \frac{2{,}4 MPa \cdot 300mm}{1820 MPa} = 395{,}6 mm^2/m$ min

Use 5× Sika CarboDur S1012 at 250 c/c

$5 \times 120 mm^2 = 600 mm > A_{s,req}$

Or L4/50/100 Sika CarboShear

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

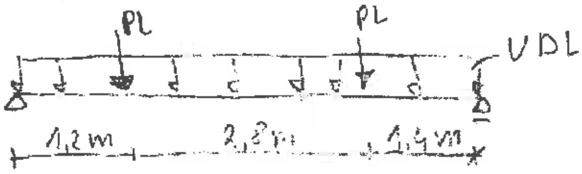
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

45 Marresfield Gdns
NW3 London

Beam RC2 - supporting cantilever

length : 5,4 m



UDL [see sheet no 5, Hand calculations] = 30 kN/m

Point load : 32 kN [see sheet no 5, Hand calculations

For reinforcement design and beam analisys see TEDDS calculations, sheet no. T10 - T16

Pos rebar :

$$A_{s,req} = \frac{30\ kN/m \cdot 5{,}4\ m}{480\ MPa \cdot 0{,}6} = 562{,}5\ mm^2$$

fud of steel

USE 8 H12 Rawplug epoxy resin to anchor cantilever to new beam

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date: June 2014
Eng.: RS
Job No.: 14.073
Sheet No. 10

45 Maresfield Gdns, London

CARBON BOND ABOVE STAIRCASE ONLY!
length : 2,65 m

Dead loads :
RC Beam sweight : $(0{,}45m \cdot 0{,}55m) \cdot 25 kN/m^2 = 6{,}2 kN/m$
Concrete floor above : $0{,}3m \cdot 2{,}85m \cdot 25 kN/m^3 = 21{,}4 kN/m$
Concrete floor above : $0{,}3m \cdot 1{,}55 m \cdot 25 kN/m^3 = 11{,}63 kN/m$
beam 1.1 : 6,9 kN/m

46,13 kN/m

Variable loads :
floors : $(1{,}55 m + 2{,}85 m) \cdot 1{,}5 kN/m^2 = 6{,}65 kN/m$

$M_{max} = 75 kNm$ – midspan
$V_{max} = 114 kN$ each support
$V_{BS1} = V_{ABS1} = 79 kN$ – span.

See TEDDS analysis sheets no T17 – T18

$z = 0{,}85\ d$
$d = h - c_{nom} - \phi_{10} - \phi_{20} = 340 mm$
$z = 306\ mm$
$F_{yd} = \frac{f_{yk}}{1{,}15}$

450
550

Sika Carbodur : $f_{yds} = 0{,}65 \cdot 2800 MPa = 1820 MPa$
Design Tensile strength

Supports :
$M_A = M_B = 25\% \cdot 75 kNm = 18{,}0 kNm$

$$k = \frac{18{,}0 kNm}{b \cdot d^2 \cdot f_{ck}} = \frac{18{,}0 kN/m}{550 \cdot 340^2 \cdot 30 MPa} = 0{,}008$$

$\phi = 45° \Rightarrow \delta = 1$
$k' = 0{,}598 \cdot 1 - 0{,}181 \cdot 1^2 - 21 = 0{,}207$
NO compression r-cment needed

Tension -> [Top] => AS612 [0,8m long]

$$\frac{18 kNm}{1820 MPa \cdot 306 mm} = 34{,}1 mm^2$$

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014	Sheet No. 11
Eng. MS	
Job No. 14.073	
45 Morestield Gdns NW3 London	

Shearing on each support:

$V_{Amax} = V_{Bmax} = V_{max} = 114kN$

$$V_{Ed} = \frac{107kN}{550 \cdot 306mm} = 0{,}677\ N/mm^2$$

$$A_{sv\,req} = \frac{0{,}677 \cdot 550mm}{1820MPa} = 204{,}6\ mm^2/m$$

3× Sika Carbo shear L 4/50/100 @ 200 mm c/c

Mid span:

$M_{max} = 71kNm$

$$A_{req} = \frac{75kNm}{1820MPa \cdot 306m} = 134{,}668$$

$2 \times 5\ 812$

Shearing:

$V_{S1} = 75kN$

$$V_{Ed} = \frac{75kN}{550mm \cdot 306mm} = 0{,}45\ N/mm^2$$

$$A_{sv\,req} = \frac{0{,}45\ N/mm^2 \cdot 550}{1820MPa \cdot} = 136\ mm^2/m$$

4 Sika Carbo Shear L 4/50/100 @ 250 mm c/c

Martin Redston Associates
Consulting Civil & Structural Engineers

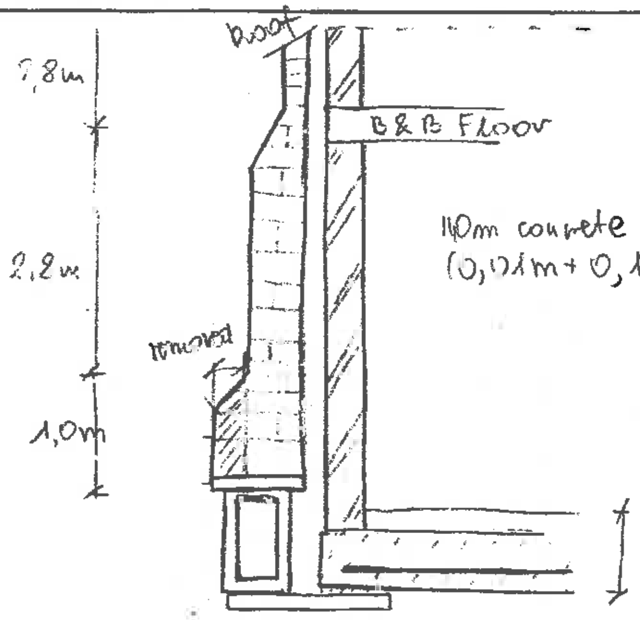
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date June 2014		Sheet No. 12
Eng. RS		
Job No. 14.073		

45 Maresfield Gdns
NW3 London



110mm concrete Blockwork
$(0{,}01m + 0{,}14m) \cdot 24\,kN/m^3 = 3{,}6\,kN/m^2$

305mm max
$8{,}5\,kN/m^2$

Outer bwk loads:

9" b/wk – $5{,}1\,kN/m^2 \cdot 3{,}8\,m = 19{,}38\,kN/m$
4" b/wk – $2{,}5\,kN/m^2 \cdot 2{,}85\,m = 7{,}13\,kN/m$

Flat roof above: $0{,}75\,kN/m^2 \cdot 1{,}6\,m = 1{,}2\,kN/m$
Floor (flat above): $0{,}5\,kN/m^2 \cdot 1{,}6\,m = 0{,}8\,kN/m$ } 2nd span only

28,55 kN/m

Beam B1.3 [above 1st floor level, at 3,7m]
(Outer leaf : 45,2 kN

VL:

Flat R above : $0{,}75\,kN/m^2 \cdot 1{,}6\,m = 1{,}2\,kN/m$
Floor (flat above): $1{,}5\,kN/m^2 \cdot 1{,}6\,m = 2{,}4\,kN/m$ } 2nd span only

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX ☑
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date June 2014	Sheet No. 13
Eng. RS	
Job No. 14.073	
45 Maresfield Gdns NW3 London	

Inner leaf:

DL: Roof: $1{,}25 kN/m^2 \cdot 2{,}45 m = 3{,}1 kN/m$

$2 \times 140mm$ b/wk $= 3{,}6 kN/m^2 \cdot 6{,}1 m = 22{,}0 kN/m$

B & Block Floor: $10{,}0 kN/m$.

concrete Floor $= 2 \cdot 2{,}85 m \cdot 7{,}32 kN/m^2 = 41{,}74 kN/m$ – above Gr. Floor & 2nd Floor

beam B1.3 82,0 [point load at 3,7m]

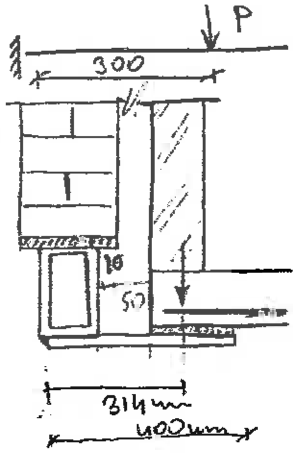
$\Sigma 77{,}0 kN/m$

VL: Roof: $0{,}75 kN/m^2 \cdot 2{,}45 m = 1{,}84 kN/m$

3 × Floors $1{,}5 kN/m^2 \cdot 3 \cdot 2{,}85 m = 12{,}83 kN/m$

Beam B1.3 = 23,8 kN

$\Sigma 14{,}7 kN/m$



$P = 102{,}5 kN/m \cdot 1{,}35 + 23{,}1 kN/m \cdot 1{,}5 =$
$= 173{,}1 kN/m$ 173,083

$M_A = 173{,}1 kN/m \cdot 0{,}315 m = 54{,}5 kNm/m$

$W_y = [1000mm \cdot 38mm^2] \cdot \frac{1}{6} = 240666{,}7 mm^3$

$\frac{54{,}5 kNm/m}{240666{,}7 mm^3/m} = 227 MPa$

f_y – nominal Yeld strength = =265

$f_y = 275 MPa \leftarrow$ max steel capacity

$M_y = \chi_{LT} \cdot W_y \cdot \frac{f_y}{\gamma_{M1}} = 0{,}9 \cdot W_y \cdot \frac{265 MPa}{1} = 57{,}4 kN$ ($\chi_{LT} \approx 0{,}9$)

$M_y > 54{,}7 kNm/m$

Weld: Transverse capacity for s=12mm fillet weld, $f_{yt} = 2{,}32 kN/mm$

applied: $P = 173{,}1 kN/m = 173{,}1 kN/1000mm = 0{,}1731 kN/mm$

$P < f_{yt}$

For outer leaf support use 400 × 200 × 16.0 + 400 mm × 38mm thick plate.

For RHS Beam design see TEDDS calculations, sheet no T20 – T23

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014
Eng. RS
Job No. 14.073
Sheet No. 14

45 Mansfield Gdns

STEEL BEAM BG1 loadings:

DL: ① RC Beam Sweight $(0{,}15 \cdot 0{,}55\,m) \cdot 25\,kN/m^2 = 6{,}2\,kN/m$

② cavity wall (0,14m block + half Brick)
$(0{,}14m \cdot 21\,kN/m^3 + 2{,}5\,kN/m^2) \cdot 2{,}3m = 13{,}0\,kN/m$

③ cavity wall (0,14m block + 9" brick):
$(0{,}14m \cdot 21\,kN/m^3 + 5{,}1\,kN/m^2) \cdot 3{,}8m = 31{,}0\,kN/m$

④ concrete floor above $2 \cdot 0{,}25m \cdot 24\,kN/m^3 \cdot 2{,}85m = 34{,}2\,kN/m$

⑤ Pitched R: $2{,}5m \cdot 1{,}25\,kN/m^2 = 3{,}2\,kN/m$ 6,5 kN/m

⑥ B&B floor above: $2{,}85m \cdot 3{,}5\,kN/m^2 = 10{,}0\,kN/m$

⑥a concrete floor $(1{,}55\,m \cdot 6{,}0\,kN/m^2) = 9{,}05\,kN/m$

⑦ Timber floor: $2{,}5m \cdot 0{,}5\,kN/m^2 = 1{,}3\,kN/m$

⑧ RR of Beam 1,3: $(77kN + 37{,}7kN)/2{,}6\,m = 44{,}1\,kN/m$

⑨ solid Partitions above: $(21\,kN/m^3 \cdot 0{,}1m) \cdot 2{,}4m = 5{,}04\,kN/m$

158,0 kN/m

Imposed load:

⑥a ④' ⑦ concrete Floor: $(2{,}85\,m \cdot 3 + 1{,}55m + 2{,}5m) \cdot 1{,}5\,kN/m^2 = 19{,}0\,kN/m$

⑤ Pitched R: $2{,}5\,m \cdot 0{,}75\,kN/m^2 = 1{,}9\,kN/m$

⑧' RR of Beam 1.3: $(23{,}9\,kN)/2{,}6m = 9{,}19\,kN/m$
left reaction

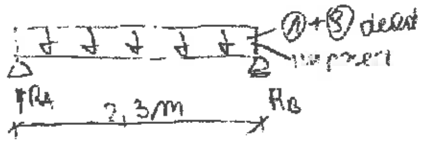
30,1 kN/m

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date 22 July 2014
Eng. RS
Job No. 14.073
Sheet No. 15

45 Maresfield Gdns
NW3 London

Beam BG1a - Revised



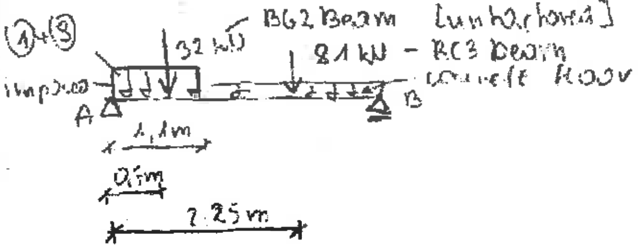
Reaction A:
perm = 180,4 kN
variable = 34,6 kN

1+8 dead = 157,5 kN
imposed = 30,1 kN

Reaction B:
perm = 180,4 kN
variable = 34,6 kN

Use UC 203 × 203 × 86
See TEDDS calculations, sheet no 24

Beam BG1b.



Reaction A:
perm = 217,4 kN
variable = 27,8 kN

Reaction B
perm = 108 kN
variable = 5,3 kN

concrete floor: $\frac{1}{2}$ 7,55m · 0,25 · 24 kN/m² = 27,2 kN
variable : 7,55m · $\frac{1}{2}$ · 1,5 kN/m² = 5,7 kN

Use UC 203 × 203 × 86
See TEDDS calculations, sheet no 25

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

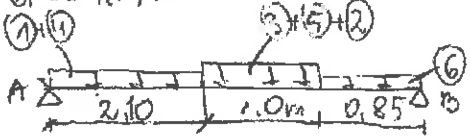
Date Feb 2014
Eng. R.S.
Job No. 14.073
Sheet No. 16

45 Maresfield Gardens
NW3 London

BLG 1 – Supporting kitchen wall above
length: 3,85m

D.L ① cavity wall : 0,6m · 4,5 kN/m² = 2,7 kN/m
② cavity wall : 2,75 m · 4,5 kN/m² = 12,2 kN/m

③ Distributed Reaction of beam BG4 on 0,85m — $\frac{20.9 kN}{0,85m}$ = 24,6 kN/m
④ glass-> folding doors : 1 kN/m² · 2,5m = 2,5 kN/m
⑤ : 3,7 m · 4,5 kN/m² = 16,7 kN/m - cavity above
⑥ : Pathway : 1m · 6,25 kN/m² = 6,25 kN/m



①+④ = 5,2 m
③ + ⑤ + ② = 24,6kN/m +
+ 16,7kN/m + 12,2kN/m = 53 kN/m

Variable ⑥ residential (on pathway) - 1,5 kN/m² · 6,25 kN/m²

For BLG1
Use UC 203×203UC71, TEDDS, Sheet no T26

BLG 2 :
length : 2,55m Dead L : Cavity wall · 0,3m · 4,5 kN/m² = 1,35 kN/m
glazed balustrade : 1,1m · 0,5 kN/m² = 0,55 kN/m
Use 203 × 133 UB25 + 6mm bottom plate
TEDDS calculations, sheet no T27

BLG 3a ; length : 5,0m
D.L : Floor joists : 0,6kN/m² · 3,9m = 2,4 kN/m

V.L : Residential : 1,5 kN/m² · 3,9m = 5,85 kN/m

Use UC 203 × 203 × 46 for BLG a, TEDDS, sheet no T28

BLG 3b -> The same load as BLG 3a, length:
Use double joists, minimum 2No 200×50 Gr. C24
Timbers
TEDDS calculations, sheet no T29

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

BLG4:
length: 2,35 m:
D.L. floor joists: 0,6 kN/m² · 2,65 m = 2,4 kN/m
Folding doors: 1 kN/m² · 2,5 m = 2,5 kN/m
RC Patio Floor: 6.25 kN/m² · 2 m = 12,5 kN/m — 17.4 kN/m
VL: Residential: (Floor): 1,5 kN/m² · 3,9 m = 5,85 kN/m
Point Load of Beam BLG1 support A (at 2,15 m)
Permanent: 68,1 kN
Variable: 3,8 kN
USE 203 × 203 × 60 TEDDS calculations, sheet no T30

BLG5
length 2,3 m
D.L: Cavity wall: 4,5 kN/m² · 3,5 m = 15,8 kN/m
Distributed load of Beam BG1
permanent: $\frac{62,3\ kN}{1,4\ m}$ = 44,5 kN/m
variable: $\frac{5,7\ kN}{1,4\ m}$ = 4,1 kN/m
Use 2 × 203 × 102 UB 23,
For BLG5, TEDDS calculations, sheet no T31

BLG6 & BLG10 [worst case = BLG10 => longer]
. length: 3,5 m of BLG10, 2,4 m for BLG6
DL: 190 mm blockwork: 0,14 m · 21 kN/m³ · 2,8 m = 8,4 kN/m
Floor joists: 2,3 m · 0,6 kN/m² = 1,4 kN/m

VL: Residential: (2,7 m + 2,3 m) · 1,5 kN/m² = 7,5 kN/m

Use UB 203 × 133 × 25, TEDDS calculations,
sheet no T32 for BLG6
sheet no T33 for BLG10

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014	Sheet No. 18
Eng. RS	
Job No. 14.073	

15 Mansfield Gdns
NW3, London

Column C1:

Beam RHS, support C:
$R_{Cmax} = 522\,kN$

Beam RC, support A:
$R_A = 107\,kN$

Beam BLG3a, support b:
$R_b = 31,6\,kN$

Beam BLG3b, support a
$R_a = 7,9\,kN$

$N_A \quad \Sigma\ 670\,kN$

$M_y = (522kN + 31,6kN - 7,9kN - 107 N)\cdot(0,087+0,1) = 85,0\,kNm$

Use RHS 300×150×16, TEDDS, sheet no T34 – T35

Foundation under Column C1:

Beam RHS, support C
perm = 325,4 kN
variable = 58,9 kN

Beam KBG1
perm = 69 kN
var = 9 kN

Beam BLG3a, support b:
perm = 7,1 kN
variable = 14,6 kN

Beam BLG3b, support a
perm = 1,61 kN
variable = 3,2 kN

Foundation weight = $2,5 \cdot 2m \cdot 1,5m \cdot 7kN/m^3 = 52,5\,kN$

$\Sigma\ 460,2\,kN$

$\frac{524,47kN}{125\,kPa} = 4,2\,m^2 \Rightarrow \sqrt{4,2m^2} = 2,05\,m$

Use 2,0 × 2,1 × 1,2 m thick concrete pad foundation

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date May 2014
Eng. RS
Job No. 14 073
Sheet No. 18

45 Moresfield Gardens
London

Foundation under wall between staircase

Beam BLG 3b → permanent : 1,6 kN
variable : 3,8 kN

$0{,}215m \cdot 2{,}5\,m \cdot 2m \cdot 24\,kN/m^3 = 26\,kN$

Beam BLG 8 → permanent : 14,3 kN
variable : 10,1 kN

Foundation weight : $2{,}3 \times 2{,}3 \times 0{,}9m \cdot (24\,kN/m^3 - 17\,kN/m^2) = 28\,kN$

Σ 84,0 kN

$\frac{84{,}0\,kN}{125\,kPa} = 0{,}67\,m^2$

Use 500 × 450 thick concrete strip foundation under wall between staircase

Column under support B, Beam RHS: [Column C2]

Factored loads on column:

- Support B, RHS Beam
  $R_B = 1290\,kN$
- Beam BG 4, support B:
  $R_B = 28{,}2\,kN$
- Beam BLG 4, support B
  $R_B = 119{,}6\,kN$
- Beam BLG 3a, support A
  $R_A = 31{,}6\,kN$

Σ 1445,0 kN

$M_y = [(-119{,}6 + 31{,}6\,kN) \cdot (0{,}12m + 0{,}1m)] = 19{,}4\,kNm$

$M_z = 28{,}2\,kN \cdot (0{,}12m + 0{,}1m) = 6{,}2\,kNm$

Use UC 254 × 254 × 89

TEDDS calculations, sheet no T36 – T37

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014	Sheet No. 20
Eng. RS	
Job No. 14.073	

45 Mansfield gardens
London

Column C3
length (height) = 6,5 m

Reaction on column: Beam BG 1a, support A:
perm = 190,4 kN
var = 35 kN
$R_{max} = 308,4$ kN

Beam BG AC 1, support B:
perm = 68 kN
variable = 9 kN
$R_{max} = 107$ kN

Beam BG 8, support B:
perm = 10 kN
variable = 8,1 kN
$R_{max} = 25,7$ kN

R_{max} = factored value

Beam BG 6, support A
perm = 3,8
variable 7,8
$R_{max} = 17$ kN

Σ perm : 216,7 kN } unfactored
variable : 53,0 kN

factored: 460 kN

$M_R = (0,1m + 0,203m \cdot 0,5) \cdot (223\ kN + 17\ kN) = 48,40\ kNm$

$M_L = (0,1m + 0,203m \cdot 0,5) \cdot (107\ kN + 25,7\ kN) = 26,74\ kNm$

Use UC 203 x 203 x 46 as column C3

TEDDS, sheet no T38 - T38

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014
Eng. RS
Job No. 14.073
Sheet No. 21

45 Maresfield Gardens
London

Column C4
length (height of column C4) = 6,5m

Reaction on column: Beam BG1a, support B:

(L) perm = 182,1 kN
variable = 35,0 kN
R_{max} = 297,8 kN

Beam BG1b, support A
R perm = 217,4 kN
variable = 32,2 kN
$R_{A\,max}$ = 341,0 kN

R_{max} = factored value

(L) Beam BLG5, support B
perm = 12,3 kN
variable = 7,8 kN
R_{max} = 28,5 kN

(R) Beam BLG10, support A
perm = kN
variable = kN
R_{max} = 42 kN

factored Σ 715 kN

$M_L = (0,1m + 0,204m \cdot 0,5) \cdot 326,4\,kN = 65,3\,kNm$

$M_R = (0,1m + 0,204m \cdot 0,5) \cdot 383,0\,kN = 77,4\,kNm$

Use UC 203 × 203 × 52 as column C4
TEDDS, sheet no T40 – T41

Reaction B_1, Beam BG1b
perm + variable = 124,7 + 14 kN = 145 kN

padstone under:

$$\frac{145\,kN}{1,0\,MPa \cdot 1,5} = 10000\ mm^2$$

Use 1000 × 100 × 215dp concrete lintel

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date March 2014	Sheet No.
Eng. RS	22
Job No. 14 073	

46 Mansfield Gdns
NW3, London

Peer check (foundation)

D.L: Reaction A of RHS Beam permanent + variable : 84,2 kN	
Peer above: 0,55 m · 0,35 m · 3,1 m · 10 kN/m = (ground floor)	5,9 kN
Peer above: (lower ground floor) 1,6 m · 0,73 · 2,7 m · 10 kN/m	31 kN
Reaction of Beam BLG5	71 kN
Reaction of Beam BLG4	23,1 kN
V.L: R. A of RHS Beam → variable	8,0
R. of Beam BLG5	4,7 kN
R. of Beam BLG4	6,9 kN
Self wt of foundation: (1.5 m · 2 m · 2 m) · (24 kN/m³ − 17 kN/m³) =	42 kN
=	276,8 kN

$$\frac{276,8\,kN}{125\,kPa} = 2,21\,m^2 ;$$

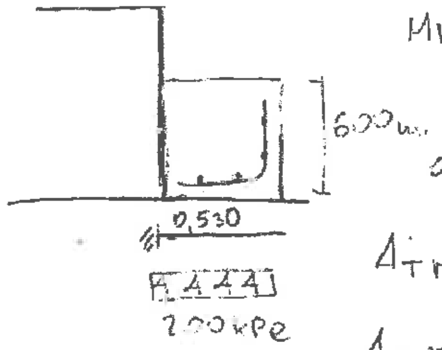
Foundation under peer L to be minimum
3 m × 1,5 m × 1,0 m deep.

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014	Sheet No. 23
Eng. RS	
Job No. 14.073	

45 MANESFIELD Gardens

Enlarging foundation under column C2



$w = 200\,kN/m$

$M_{max} = 200\,kN/m \cdot \frac{(0{,}53\,m)^2}{2} =$

$= 28{,}1\,kNm$

$d = 600mm - 40mm - 10mm - \frac{16mm}{2} = 542mm$

$A_{T\,req} = \frac{28{,}1\,kNm}{542mm \cdot 435\,MPa} *$

$A_{T\,req} = 119{,}20\,mm^2$

Use H12 @ 400 c/c as tension reinforcement

$V_{max} = 200 \cdot 0{,}53 = 110\,kN$

$v_{Ed} = \frac{110\,kN}{1m \cdot 542\,mm} = 0{,}203\,MPa$

$A_{sv\,req} = \frac{0{,}203\,MPa \cdot 1m}{435\,MPa} = 466{,}6\,mm^2/m$

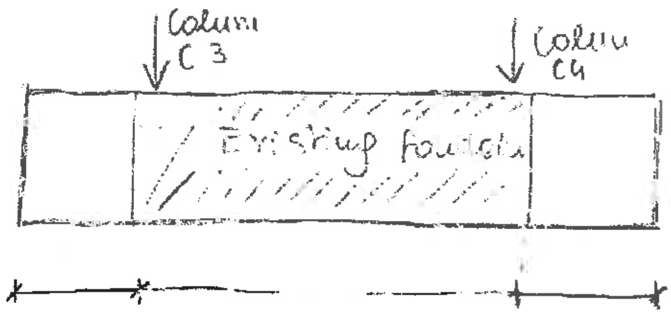
use H16 @ 350 mm as shearing

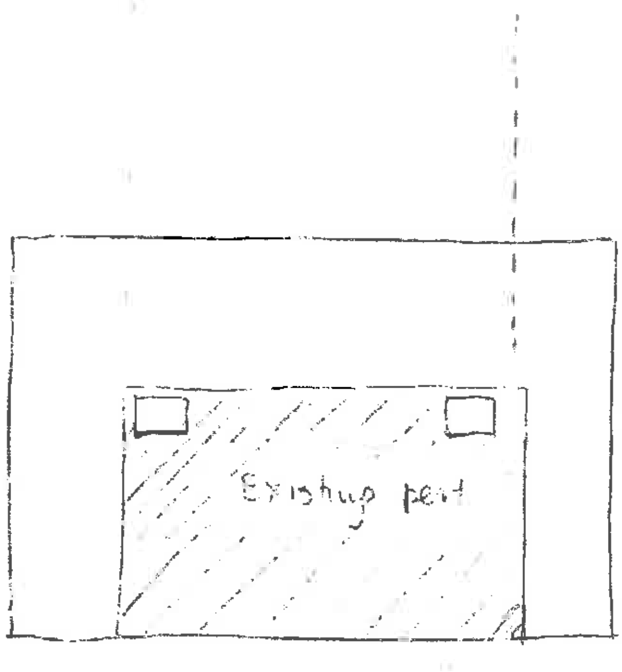
Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014	Sheet No.
Eng. RS	24
Job No. 14.073	

45 Maresfield Gdns
London

Foundation under columns C3 & C4:





Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014
Eng. RS
Job No. 14.073
Sheet No. 25

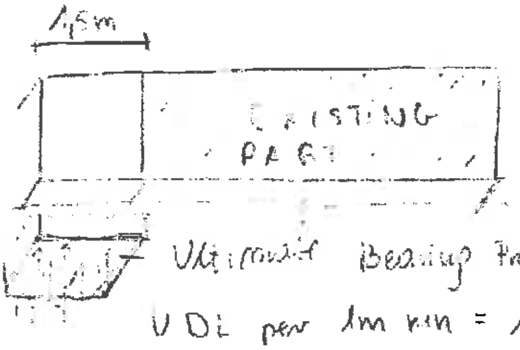
45 Maresfield Gardens London

Reaction from Colum C3:
permanent = 216,7 kN
variable = 53.0 kN

Reaction from Column C4
permanent = 513 kN
variable = 83 kN

For Foundation analysis see TEDDS calculations, sheet no T42–T43

1,5m



Ultimate Bearing Pressure: 135 kPa · 1,4 = 188 kPa

UDL per 1m run = 188 kPa

$M_{max} = (188\ kN/m \cdot 1{,}5m) \cdot 1{,}5m / 2 = 213\ kNm$

$d = 1000mm - 40mm - 10 - \frac{16mm}{2} = 942\ mm$

$z = 0{,}9d = 847{,}8\ mm$

Tension Reinforcement

$A_{T\,req} = \frac{M_{max}}{f_{yd} \cdot z}$; $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500\ MPa}{1{,}15} = 435\ N/mm^2$

$A_{T\,req} = \frac{213 \cdot 10^6\ Nmm}{847{,}8mm \cdot 435\ MPa} = 577\ mm^2/m$

Use H12 @ 175 + H10@200 distribution

Shear reinforcement: $V_{Ed} = 189\ kN/m \cdot 1{,}5m = 284\ kN$

$V_{Ed} = \frac{284\ kN}{1000mm \cdot 847{,}8mm} = 0{,}335\ MPa$

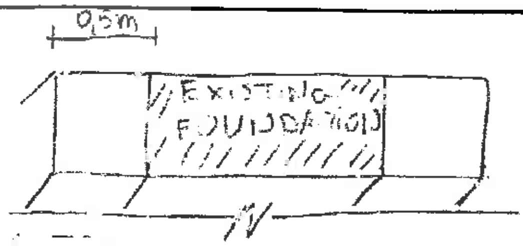
$A_{sv,req} = \frac{0{,}335\ MPa \cdot 1000\ mm}{435\ MPa} = 770{,}12\ mm^2/m$

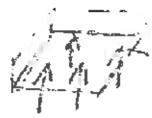
Use H16 @ 250 c/c EPOXY RESIN ANCHORS R-KEX II, All

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date June 2014
Eng. RS
Job No. 14.073
Sheet No. 26

45 Maresfield Gardens
London





UDL per 1m = 189 kPa

$M_{max} = 189\,kN/m \cdot 0{,}5m \cdot 0{,}5m/2 = 24\ kNm$

$d = 1000\,mm - 40\,mm - 10\,mm - \frac{16\,mm}{2} = 942\ mm$

$z = 0{,}9d = 847{,}8\ mm$

Tension reinforcement

$A_{T\,req} = \frac{M_{max}}{f_{yd} \cdot z} = \frac{24 \cdot 10^6\ Nmm}{847{,}8\,mm \cdot 435\,MPa} = 70\,mm^2$

Use H12 @ 400 c/c

Shearing reinforcement : $V_{max} = 189\,kN/m \cdot 0{,}5m = 95\ kN$

$V_{Ed} = \frac{95\ kN}{1000\,mm \cdot 847{,}8\,mm} = 0{,}112\ MPa$

$A_{sv\,req} = \frac{0{,}112\,MPa \cdot 1m}{435\ MPa} = 260\ mm^2/m$

Use 16 @ 300 c/c EPOXY RESIN ANCHORS
K-KEX II, AII

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Date April 2014	Sheet No. 27
Eng. RS	
Job No. 14.073	
45 Maresfield Gardens NW3, London	

New Retaining wall [front basement wall]

DL: Cavity wall : 12" blk + brick : $10\ kN/m^2 \cdot 3{,}8\ m = 38\ kN/m$

[Terrace] RC concrete floor above $0{,}4\ m \cdot 6{,}3\ kN/m^2 = 2{,}52\ kN/m$

Pitched R: $0{,}9\ m \cdot 1{,}25\ kN/m^2 = 1{,}13\ kN/m$

Point load : BLG 10, support B : perm = 201,4 kN

$$\frac{201{,}4\ kN}{4{,}5\ m} = 44{,}8\ kN/m$$

Beam supporting B & B floor : $0{,}5\ m \cdot 0{,}6\ m \cdot 25\ kN/m^3 = 7{,}5\ kN/m$ — 87,5 kN/m

VL: Terrace $0{,}4\ m \cdot 1{,}5\ kN/m^2$ $0{,}6\ kN/m$

Pitched R: $0{,}9\ m \cdot 0{,}75\ kN/m^2 = 0{,}675\ kN/m$

Point l. BLG 10, support B, var : 47,2 kN

$$\frac{47{,}2\ kN}{4{,}5\ m} = 10{,}5\ kN/m$$

Σ 11,8 kN/m

No change required [23.07.2014]

For Analysis & Design see TEDDS calculations sheets no T44 – T52

Basement wall by path [worst case] length: 6.3m

Vertical Load DL: Cavity wall : $6{,}3\ m \cdot 4{,}5\ kN/m^2 = 28{,}4\ kN/m$

Cavity wall : [future flat ext.] : $2{,}8\ m \cdot 4{,}5\ kN/m^2 = 12{,}6\ kN/m$

Flat R : $1{,}7\ m \cdot 0{,}75\ kN/m^2 = 1{,}3\ kN/m$

Floor [Timber] : $1{,}7\ m \cdot 0{,}5\ kN/m^2 = 0{,}85\ kN/m$

RC floor : $6{,}3\ kN/m^2 \cdot 1{,}7\ m = 10{,}7\ kN/m$

Sweight : $2{,}5\ m \cdot 0{,}3\ m \cdot 25\ kN/m^3 = 19{,}0\ kN/m$

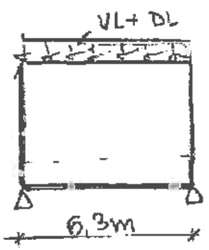
Σ 73 kN/m

VL : Flat R : $1{,}7\ m \cdot 0{,}75\ kN/m^2 = 1{,}3\ kN/m$

Floor [Timber] : $1{,}7\ m \cdot 1{,}5\ kN/m^2 = 2{,}6\ kN/m$

RC floor : $1{,}7\ m \cdot 1{,}5\ kN/m^2 = 2{,}6\ kN/m$

Σ 6,5 kN/m



$$M_{perm} = \frac{w \cdot l^2}{8} = \frac{73\ kN/m \cdot (6{,}3\ m)^2}{8} = 362{,}171\ kNm$$

$d = 1{,}5\ m$, $z = 0{,}95 \cdot d = 1425\ mm$

$$F_{yd} = \frac{f_{yk}}{1.15} = 434{,}78 \approx 435\ MPa$$

$$A_{sreq} = \frac{362{,}21\ Nm \cdot 1{,}35}{435\ MPa \cdot 1425\ mm} = 784\ mm^2$$ Use 2 × H25

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX ☑
Tel: 020 8959 1666 Fax: 020 8906 8503
Email: martin@redston.org

Horizontal load:

$w_P = 14\,kN/m^3 \cdot (2,6m)^2 \cdot \frac{1}{2} = 47,3\,kN/m$, factored 63,5 kN/m

Surcharge : 3 kN/m [horizontal & vertical]
factored : 4,5 kN/m

$k_A = 0,333$
$k_P = 4,977$

$w_{max} = 68,5\,kN/m = w_{D0}$ [+ surcharge]

$w_{D1} = \frac{2}{3} \cdot 68,5\,kN/m = 43\,kNm$

$w_{D2} = 21,5\,kN/m =$

$M_{max} = \frac{45\,kN/m \cdot (6,3m)^2}{8} = 223\,kNm = M_{D1}$

$b = 350$, $d = 350mm - 40mm - 20mm/2 = 300mm$

$z = d \cdot 0,9$

$A_{sreq1} = \frac{223\,kNm}{435\,MPa \cdot 270mm} = 1\,900\,mm^2$

Use H25 @ 250c/c up to 1,5m Height of wall.

$w_{D2} = \frac{2}{3} \cdot \frac{1}{2} \cdot 1,5m^2 \cdot 14\,kN/m^3 = 10,5\,kN/m$, factored 14,17 kN/m

$M_{D2} = \frac{18,7\,kN/m \cdot (6.3\,m)^2}{8} = 92,65$

$A_{sreq2} = \frac{92.65\,kN\,m}{435\,MPa \cdot 270mm} = 799\,mm^2$

Use H16 @ 200 c/c

Vertical Reinforcement

$w_{max} = 68,5\,kN/m$

$M_{max} = \frac{68,5\,kN/m \cdot (2,6m)^2}{8} = 58\,kNm$

$A_{sreq} = \frac{58\,kNm}{435\,MPa \cdot 285mm} = 466,9\,mm^2$

Use H12 @ 200c/c

Reaction at each end : $\frac{73\,kN/m \cdot 6,3m}{2} = 230\,kN$ - perm

$\frac{6,5\,kN/m \cdot 6,3m}{2} = 20,5\,kN$ - variable

$A_{sreq} = \frac{341,25\,kN}{435\,MPa} = 785\,mm^2$

8 H16 @ 300 c/c

A_{sprov} : 1610 mm²

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date April 2014
Eng. RS
Job No. 14.073

Sheet No. 28

45 Moresfield Gdns
London

Basic length of Bars in new wall : $\phi 40 = 640mm^2 min$,
Use 700 = length of Bars.

Bond length : $l_b = \frac{\phi}{4} \cdot f_{yd}/f_{bd} = \frac{16mm}{4} \cdot \frac{500MPa}{2.7MPa} = 740\ mm^2$

$l_{bd\,min} = 740 \frac{A_{s\,req}}{A_{s\,prov}} = 362\ mm^2$ use $l_{bd} = 550\ mm$.

R/cment by supports [Bases]
25% of tension R/cment [in span]
$A_{s\,req\,1} \cdot 0,25 = 475\ mm^2$ [H16@250]
$A_{s\,req\,2} \cdot 0,25 = 223,5\ mm^2$ [H12@250]

not needed ∴ strength provided by shearing R/cment.

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666 Fax: 020 8906 8503

Email: martin@redston.org

Date: March 2014
Eng. RS
Job No. 14.073
Sheet No. 30

45 Marefield Gdns
NW3, London

Beam BG1
Support B — Revised

$R_B = R_A = 124,7 + 14\ kN =$

$$\frac{138,0\,kN}{1,0\,MPa \cdot 1,5} = 92700\,mm^2$$

Use 100 × 100 × 215 dp concrete lintel as padstone

Beam BG2 — Revised

$C_A = 27,0\ kN$

$$\frac{27,0\,kN}{1,0\,MPa \cdot 1,5} = 18000\,mm^2$$

Use 200 × 100 × 150 dp concrete padstone

Beam BG4 supports:

$R_A = R_B = 21,1\ kN$

$$\frac{21,1\,kN}{1,0\,MPa \cdot 1,5} = 14000\,mm^2$$

Use 250 × 100 × 100 concrete padstone at each end

BLG 2

$R_A = 27\ kN$

$$\frac{27\,kN}{1,0\,MPa \cdot 1,5} = 18\,000\,mm^2$$

Use 200 × 100 × 150 dp concrete padstone

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377 Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1566 Fax: 020 8906 8503
Email: martin@redston.org

BLG5 :

$R_A = 71kN + 4,7kN = 76kN = R_B /2 = 38kN$

$\frac{76 kN/2}{1,0 MPa \cdot 1,5} = 25333,3 \ mm^2$

Use 200 × 100 × 150 concrete padstone for PS14

350 × 100 × 215 concrete padstone for PS13

Beam BLG4

$R_A - (27,8kN + 6,8kN) = 35,0 \ kN$

$\Rightarrow \frac{35,0kN}{1,0 MPa \cdot 1,5} = 23333 \ mm^2$

Use 2No 250 × 150 × 150 concrete padstones for Both supports : PS15 or 1× 500×150×150 concrete padstone

Restraining Beam
length : 2,6m
D.Load : $0,1m \cdot 21kN/m^2 \cdot 2,8 m = 5,9 \ kN/m$ - non bearing solid partition.

Use UB 203×102×23
TEDDS, sheet no T53

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 26.6 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 32.2 kN	V_{min} = -32.2 kN
Deflection	δ_{max} = 4.8 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 32.2 kN	R_{A_min} = 32.2 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 23.9 kN	
Maximum reaction at support B	R_{B_max} = 32.2 kN	R_{B_min} = 32.2 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 23.9 kN	

Section details

Section type	**UC 152x152x37 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = 32 kN — Design shear resistance $V_{c,Rd}$ = 226.5 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 26.6 kNm — Des.bending resist.moment $M_{c,Rd}$ = 84.9 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\overline{\lambda}_{LT}$ = 0.593 — Limiting slenderness ratio $\overline{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 80.4 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 δ_{lim} = 9.2 mm — Maximum deflection δ = 4.811 mm

PASS - Maximum deflection does not exceed deflection limit

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = **115.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **139.9** kN	V_{min} = **-139.9** kN
Deflection	δ_{max} = **5.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **139.9** kN	R_{A_min} = **139.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **77** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **23.9** kN	
Maximum reaction at support B	R_{B_max} = **139.9** kN	R_{B_min} = **139.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **77** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **23.9** kN	

Section details

Section type	**UC 203x203x71 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **140** kN	Design shear resistance	$V_{c,Rd}$ = **371.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **115.4** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **211.7** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\overline{\lambda}_{LT}$ = **0.455**	Limiting slenderness ratio	$\overline{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **211.7** 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	δ_{lim} = **9.2** mm	Maximum deflection	δ = **5.907** mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London			Job no. 14.073	
	Calcs for Beam B1.3 outer Leaf			Start page no./Revision T 3	
	Calcs by RS	Calcs date 20/06/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = **46.1 kNm**	M_{min} = **0 kNm**
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **36.8 kNm**	$M_{s1\_seg1\_min}$ = **0 kNm**
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **46.1 kNm**	$M_{s1\_seg2\_min}$ = **0 kNm**
Maximum shear	V_{max} = **68.7 kN**	V_{min} = **-47.7 kN**
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **68.7 kN**	$V_{s1\_seg1\_min}$ = **0 kN**
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **21.4 kN**	$V_{s1\_seg2\_min}$ = **-47.7 kN**
Deflection segment 3	δ_{max} = **6.9 mm**	δ_{min} = **0 mm**
Maximum reaction at support A	R_{A_max} = **68.7 kN**	R_{A_min} = **68.7 kN**
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **50.9 kN**	
Maximum reaction at support B	R_{B_max} = **47.7 kN**	R_{B_min} = **47.7 kN**
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **35.3 kN**	

Section details

Section type	**UB 203x133x30 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **69 kN**	Design shear resistance	$V_{c,Rd}$ = **231.4 kN**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **46.1 kNm**	Des.bending resist.moment	$M_{c,Rd}$ = **86.5 kNm**

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\bar{\lambda}_{LT}$ = **1.273**	Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **46.5 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	δ_{lim} = **9.4 mm**	Maximum deflection	δ = **6.883 mm**

PASS - Maximum deflection does not exceed deflection limit

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = 47.3 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 72.7 kN	V_{min} = -72.7 kN
Deflection	δ_{max} = 2.5 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 72.7 kN	R_{A_min} = 72.7 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 39.8 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 12.6 kN	
Maximum reaction at support B	R_{B_max} = 72.7 kN	R_{B_min} = 72.7 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 39.8 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 12.6 kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = 73 kN	Design shear resistance	$V_{c,Rd}$ = 269.5 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = 47.3 kNm	Des.bending resist.moment	$M_{c,Rd}$ = 136.8 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\overline{\lambda}_{LT}$ = 0.426	Limiting slenderness ratio	$\overline{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 136.8 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	δ_{lim} = 7.2 mm	Maximum deflection	δ = 2.503 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London					Job no. 14.073	
	Calcs for Beam BG3					Start page no./Revision T 5	
	Calcs by RS	Calcs date 03/03/2014	Checked by	Checked date		Approved by	Approved date

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 17 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 12.4 kN	V_{min} = -12.4 kN
Deflection	δ_{max} = 6 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 12.4 kN	R_{A_min} = 12.4 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 4.1 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 4.6 kN	
Maximum reaction at support B	R_{B_max} = 12.4 kN	R_{B_min} = 12.4 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 4.1 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 4.6 kN	

Section details

Section type **UB 203x133x30 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 12 kN Design shear resistance $V_{c,Rd}$ = 231.4 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 17 kNm Des.bending resist.moment $M_{c,Rd}$ = 86.5 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = 1.146 Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 54.1 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 δ_{lim} = 14 mm Maximum deflection δ = 6.034 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BG4-supporting folding door				Start page no./Revision T 6	
	Calcs by RS	Calcs date 04/06/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = 16.9 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 28.2 kN	V_{min} = -28.2 kN
Deflection	δ_{max} = 1.2 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 28.2 kN	R_{A_min} = 28.2 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 20.9 kN	
Maximum reaction at support B	R_{B_max} = 28.2 kN	R_{B_min} = 28.2 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 20.9 kN	

Section details

Section type	**UB 203x133x30 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = 28 kN	Design shear resistance	$V_{c,Rd}$ = 231.4 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = 16.9 kNm	Des.bending resist.moment	$M_{c,Rd}$ = 86.5 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\overline{\lambda}_{LT}$ = 0.640	Limiting slenderness ratio	$\overline{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 80 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	δ_{lim} = 6.7 mm	Maximum deflection	δ = 1.236 mm

PASS - Maximum deflection does not exceed deflection limit

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 32.8 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 42.6 kN	V_{min} = -36.4 kN
Deflection	δ_{max} = 2.2 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 42.6 kN	R_{A_min} = 42.6 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 31.6 kN	
Maximum reaction at support B	R_{B_max} = 36.4 kN	R_{B_min} = 36.4 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 27 kN	

Section details

Section type **UC 203x203x52 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 43 kN Design shear resistance $V_{c,Rd}$ = 297.6 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 32.8 kNm Des.bending resist.moment $M_{c,Rd}$ = 156 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = 0.480 Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 154.8 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 δ_{lim} = 8.6 mm Maximum deflection δ = 2.199 mm

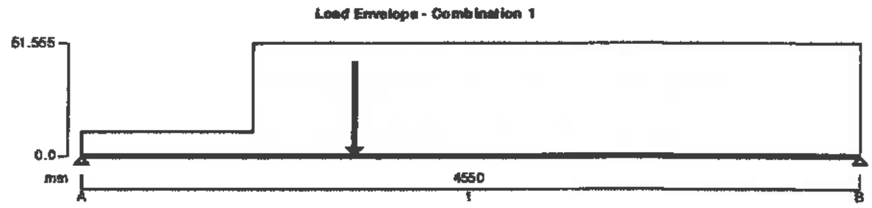
PASS - Maximum deflection does not exceed deflection limit

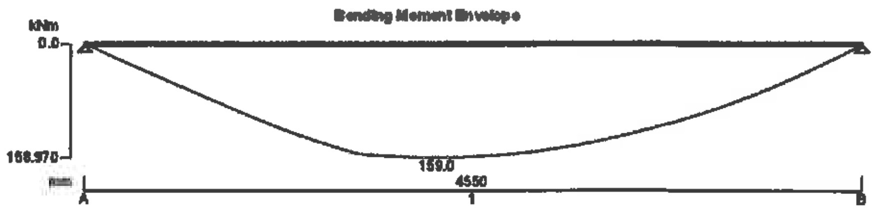
Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job Ref. 14.073	
	Section Beam RC3-analysis to design of carbon bonding				Sheet no./rev. T	
	Calc. by RS	Date 23/07/2014	Chk'd by	Date	App'd by	Date

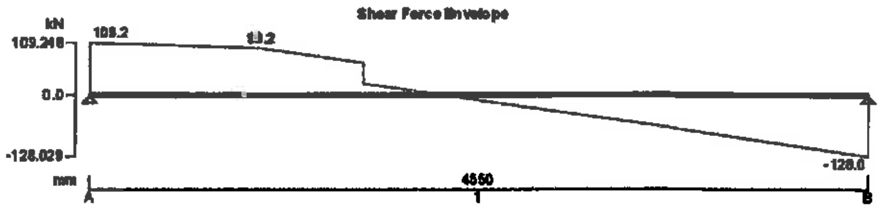
RC BEAM ANALYSIS & DESIGN (EN1992-1)

In accordance with UK national annex

TEDDS calculation version 2.1.13







Support conditions

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

Applied loading

	Permanent self weight of beam × 1
Span 1 loads	Permanent UDL 30.000 kN/m from 1000 mm to 4550 mm
	Permanent UDL 2.000 kN/m from 0 mm to 4550 mm
	Permanent point load 32.000 kN at 1600 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

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London			Job Ref. 14.073	
	Section Beam RC3-analysis to design of carbon bonding			Sheet no./rev. T	
	Calc. by RS	Date 23/07/2014	Chk'd by	Date	App'd by	Date

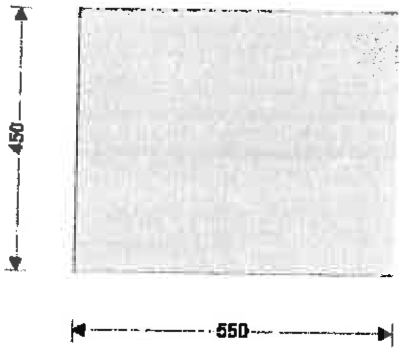
Variable × 1.50

Analysis results

Maximum moment support A;	M_{A_max} = 0 kNm;	M_{A_red} = 0 kNm;
Maximum moment span 1 at 2067 mm;	M_{s1_max} = 159 kNm;	M_{s1_red} = 159 kNm;
Maximum moment support B;	M_{B_max} = 0 kNm;	M_{B_red} = 0 kNm;
Maximum shear support A;	V_{A_max} = 109 kN;	V_{A_red} = 109 kN
Maximum shear support A span 1 at 397 mm;	$V_{A\_s1\_max}$ = 105 kN;	$V_{A\_s1\_red}$ = 105 kN
Maximum shear support B;	V_{B_max} = -128 kN;	V_{B_red} = -128 kN
Maximum shear support B span 1 at 4153 mm;	$V_{B\_s1\_max}$ = -108 kN;	$V_{B\_s1\_red}$ = -108 kN
Maximum reaction at support A;	R_A = 109 kN	
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = 81 kN	
Maximum reaction at support B;	R_B = 128 kN	
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = 95 kN	

Rectangular section details

Section width;	b = 550 mm
Section depth;	h = 450 mm



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

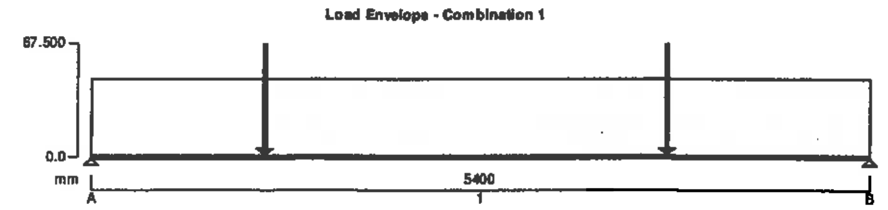
Concrete strength class;	C30/37
Characteristic compressive cylinder strength;	f_{ck} = 30 N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube}$ = 37 N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck}$ + 8 N/mm^2 = 38 N/mm^2
Mean value of axial tensile strength;	f_{ctm} = 0.3 N/mm^2 × (f_{ck}/ 1 N/mm^2)$^{2/3}$ = 2.9 N/mm^2
Secant modulus of elasticity of concrete;	E_{cm} = 22 kN/mm^2 × [f_{cm}/10 N/mm^2]$^{0.3}$ = 32837 N/mm^2
Partial factor for concrete (Table 2.1N);	γ_C = 1.50
Compressive strength coefficient (cl.3.1.6(1));	α_{cc} = 0.85
Design compressive concrete strength (exp.3.15);	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = 17.0 N/mm^2
Maximum aggregate size;	h_{agg} = 20 mm

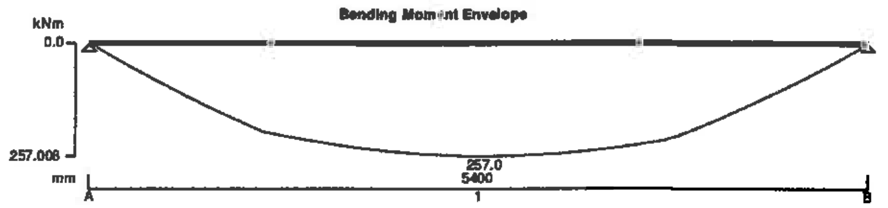
Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London			Job no. 14.073	
	Calcs for Beam RC 2 design and analisys			Start page no./Revision T 10	
	Calcs by RS	Calcs date 04/06/2014	Checked by	Checked date	Approved by	Approved date

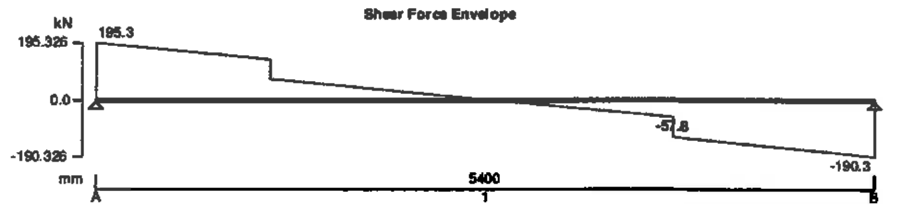
RC BEAM ANALYSIS & DESIGN (EN1992-1)

In accordance with UK national annex

TEDDS calculation version 2.1.13







Support conditions

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

Applied loading

	Permanent self weight of beam × 1
Span 1 loads	Permanent UDL 30.000 kN/m from 0 mm to 5400 mm Variable UDL 0.400 kN/m from 0 mm to 5400 mm Permanent point load 50.000 kN at 1200 mm Permanent point load 50.000 kN at 4000 mm

Load combinations

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

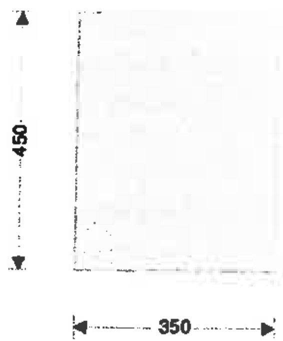
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 2754 mm	M_{s1_max} = **257** kNm	M_{s1_red} = **257** kNm
Maximum moment support B	M_{B_max} = **0** kNm	M_{B_red} = **0** kNm
Maximum shear support A	V_{A_max} = **195** kN	V_{A_red} = **195** kN
Maximum shear support A span 1 at 399 mm	$V_{A\_s1\_max}$ = **177** kN	$V_{A\_s1\_red}$ = **177** kN
Maximum shear support B	V_{B_max} = **-190** kN	V_{B_red} = **-190** kN
Maximum shear support B span 1 at 5001 mm	$V_{B\_s1\_max}$ = **-172** kN	$V_{B\_s1\_red}$ = **-172** kN
Maximum reaction at support A	R_A = **195** kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **143** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1** kN	
Maximum reaction at support B	R_B = **190** kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **140** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1** kN	

Rectangular section details

Section width	b = **350** mm
Section depth	h = **450** mm



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

Concrete strength class	**C30/37**
Characteristic compressive cylinder strength	f_{ck} = **30** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **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 × (f_{ck}/ 1 N/mm^2)$^{2/3}$ = **2.9** N/mm^2
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm^2 × [f_{cm}/10 N/mm^2]$^{0.3}$ = **32837** N/mm^2
Partial factor for concrete (Table 2.1N)	γ_C = **1.50**
Compressive strength coefficient (cl.3.1.6(1))	α_{cc} = **0.85**
Design compressive concrete strength (exp.3.15)	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel (Table 2.1N)	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

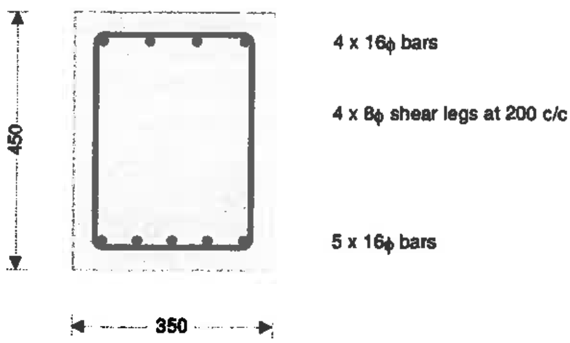
Nominal cover to reinforcement

Nominal cover to top reinforcement c_{nom_t} = **35 mm**

Nominal cover to bottom reinforcement c_{nom_b} = **35 mm**

Nominal cover to side reinforcement c_{nom_s} = **35 mm**

Support A



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1)) β_1 = **0.25**

Design bending moment $M = max(abs(M_{A\_red}), \beta_1 \times abs(M_{s1\_red}))$ = **64 kNm**

Depth to tension reinforcement $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **399 mm**

Percentage redistribution m_{rA} = **0 %**

Redistribution ratio $\delta = min(1 - m_{rA}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.038**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **379 mm**

Depth of neutral axis $x = 2.5 \times (d - z)$ = **50 mm**

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **390 mm²**

Tension reinforcement provided 4 × 16φ bars

Area of tension reinforcement provided $A_{s,prov}$ = **804 mm²**

Minimum area of reinforcement (exp.9.1N) $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **210 mm²**

Maximum area of reinforcement (cl.9.2.1.1(3)) $A_{s,max} = 0.04 \times b \times h$ = **6300 mm²**

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

Minimum bottom reinforcement at supports

Minimum reinforcement factor (cl.9.2.1.4(1)) β_2 = **0.25**

Area of reinforcement to adjacent span $A_{s,span}$ = **2513 mm²**

Minimum bottom reinforcement to support $A_{s2,min} = \beta_2 \times A_{s,span}$ = **628 mm²**

Bottom reinforcement provided 5 × 16φ bars

Area of bottom reinforcement provided $A_{s2,prov}$ = **1005 mm²**

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

Rectangular section in shear (Section 6.2)

Design shear force at support A $V_{Ed,max} = abs(max(V_{A\_max}, V_{A\_red}))$ = **195 kN**

Angle of comp. shear strut for maximum shear θ_{max} = 45 deg

Maximum design shear force (exp.6.9) $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (cot(\theta_{max}) + tan(\theta_{max}))$ = **595 kN**

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 399 mm $V_{Ed} = \max(V_{A\_s1\_max}, V_{A\_s1\_red}) = \mathbf{177}$ **kN**

Design shear stress $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{1.333}$ **N/mm²**

Strength reduction factor (cl.6.2.3(3)) $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.528}$

Compression chord coefficient (cl.6.2.3(3)) $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3)

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ **deg**

Area of shear reinforcement required (exp.6.13) $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{429}$ **mm²/m**

Shear reinforcement provided 4 × 8ϕ legs at 200 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1005}$ **mm²/m**

Minimum area of shear reinforcement (exp.9.5N) $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{307}$ **mm²/m**

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N) $s_{vl,max} = 0.75 \times d = \mathbf{299}$ **mm**

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width $w_k = \mathbf{0.3}$ **mm**

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} = \mathbf{2.9}$ **N/mm²**

Stress distribution coefficient $k_c = 0.4$

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) =$ **0.97**

Depth of tensile zone $h_{cr} = h - x = \mathbf{400}$ **mm**

Area of concrete in the tensile zone $A_{ct} = h_{cr} \times b = \mathbf{140044}$ **mm²**

Adjusted maximum bar diameter (exp.7.6N) $\phi_{mod} = \phi_{top} \times (2.9 \text{ N/mm}^2 / f_{ct,eff}) \times 2 \times (h - d) / (k_c \times h_{cr}) = \mathbf{10}$ **mm**

Maximum adjusted bar diameter $\phi_{max} = \mathbf{32}$ **mm**

Tension bar spacing $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = \mathbf{83}$ **mm**

Maximum tension bar spacing $s_{max} = \mathbf{300}$ **mm**

Minimum allowable bar spacing $s_{min} = \max(\phi_{top}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{top} = \mathbf{41}$ **mm**

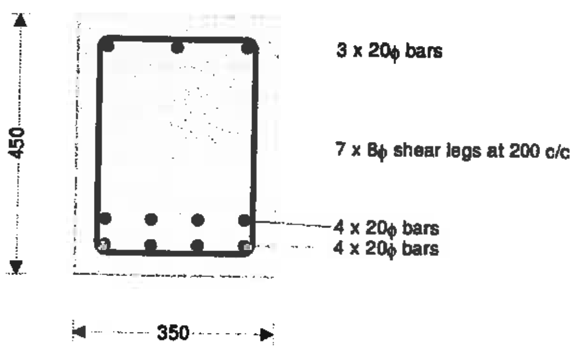
Maximum stress permitted (Tables 7.2N & 7.3N) $\sigma_s = \mathbf{280}$ **N/mm²**

Minimum area of reinforcement required (exp.7.1) $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{559}$ **mm²**

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

PASS - Actual bar spacing exceeds minimum allowable

Mid span 1



Multiple layers of bottom reinforcement

Reinforcement provided - layer 1 4 × 20ϕ bars

Area of reinforcement provided - layer 1 — A_{s_L1} = **1257** mm²

Depth to layer 1 — d_{L1} = **397** mm

Reinforcement provided - layer 2 — 4 × 20φ bars

Area of reinforcement provided - layer 2 — A_{s_L2} = **1257** mm²

Depth to layer 2 — d_{L2} = **352** mm

Total area of reinforcement — $A_{s,prov} = A_{s\_L1} + A_{s\_L2}$ = **2513** mm²

Centroid of reinforcement — $d_{bot} = (A_{s\_L1} \times d_{L1} + A_{s\_L2} \times d_{L2}) / A_{s,prov}$ = **375** mm

Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment — $M = abs(M_{s1\_red})$ = **257** kNm

Depth to tension reinforcement — $d = d_{bot}$ = **375** mm

Percentage redistribution — $m_{rs1} = M_{s1\_red} / M_{s1\_max} - 1$ = **0** %

Redistribution ratio — $\delta = min(1 - m_{rs1}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.175**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **303** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **178** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z)$ = **1949** mm²

Tension reinforcement provided — 4 × 20 φbars + 4 × 20 φbars

Area of tension reinforcement provided — $A_{s,prov}$ = **2513** mm²

Minimum area of reinforcement (exp.9.1N) — $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **197** mm²

Maximum area of reinforcement (cl.9.2.1.1(3)) — $A_{s,max} = 0.04 \times b \times h$ = **6300** mm²

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

Rectangular section in shear (Section 6.2)

Shear reinforcement provided — 7 × 8φ legs at 200 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1759** mm²/m

Minimum area of shear reinforcement (exp.9.5N) — $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **307** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N) — $s_{vl,max} = 0.75 \times d$ = **281** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear resistance (assuming cot(θ) is 2.5) — $V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **579.9** kN

Shear links provided valid between 0 mm and 5400 mm with tension reinforcement of 2513 mm²

Crack control (Section 7.3)

Maximum crack width — w_k = **0.3** mm

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

Stress distribution coefficient — k_c = 0.4

Non-uniform self-equilibrating stress coefficient — $k = min(max(1 + (300\ mm - min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.97**

Depth of tensile zone — $h_{cr} = h - x$ = **272** mm

Area of concrete in the tensile zone — $A_{ct} = h_{cr} \times b$ = **95178** mm²

Adjusted maximum bar diameter (exp.7.6N) — $\phi_{mod} = \phi_{bot,L1} \times (2.9\ N/mm^2 / f_{ct,eff}) \times 2 \times (h - d) / (k_c \times h_{cr})$ = **28** mm

Maximum adjusted bar diameter — ϕ_{max} = **32** mm

Tension bar spacing — $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot,L1}) / (N_{bot,L1} - 1)$ = **81** mm

Maximum tension bar spacing — s_{max} = **300** mm

Minimum allowable bar spacing — $s_{min} = max(\phi_{bot,L1}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot,L1}$ = **45** mm

Maximum stress permitted **(Tables 7.2N & 7.3N)** — σ_s = **160** N/mm²

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam RC 2 design and analisys				Start page no./Revision T 15	
	Calcs by RS	Calcs date 04/06/2014	Checked by	Checked date	Approved by	Approved date

Minimum area of reinforcement required (exp.7.1) $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **665** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Reference reinforcement ratio $\rho_{m0} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio $\rho_m = A_{s,req} / (b \times d) = \mathbf{0.015}$

Required compression reinforcement ratio $\rho'_m = A_{s2,req} / (b \times d) = \mathbf{0.000}$

Structural system factor (Table 7.4N) $K_b = \mathbf{1.0}$

Basic allowable span to depth ratio (7.16b) $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / (\rho_m - \rho'_m) + (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho'_m / \rho_{m0})^{0.5} / 12] = \mathbf{14.026}$

Reinforcement factor (exp.7.17) $K_s = \min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5) = \mathbf{1.289}$

Flange width factor $F1 = \mathbf{1.000}$

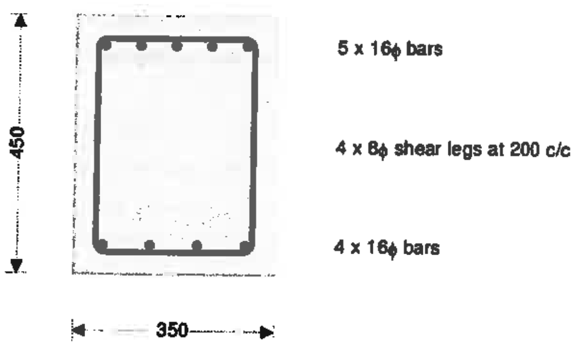
Long span supporting brittle partition factor $F2 = \mathbf{1.000}$

Allowable span to depth ratio $span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b) = \mathbf{18.086}$

Actual span to depth ratio $span\_to\_depth_{actual} = L_{s1} / d = \mathbf{14.419}$

PASS - Actual span to depth ratio is within the allowable limit

<u>Support B</u>



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1)) $\beta_1 = \mathbf{0.25}$

Design bending moment $M = \max(abs(M_{B\_red}), \beta_1 \times abs(M_{s1\_red})) = \mathbf{64}$ kNm

Depth to tension reinforcement $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2 = \mathbf{399}$ mm

Percentage redistribution $m_{rB} = \mathbf{0}$ %

Redistribution ratio $\delta = \min(1 - m_{rB}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.038}$

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21 = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) = \mathbf{379}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{50}$ mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z) = \mathbf{390}$ mm^2

Tension reinforcement provided **5 × 16ϕ bars**

Area of tension reinforcement provided $A_{s,prov} = \mathbf{1005}$ mm^2

Minimum area of reinforcement (exp.9.1N) $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{210}$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)) $A_{s,max} = 0.04 \times b \times h = \mathbf{6300}$ mm^2

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

Minimum bottom reinforcement at supports

Minimum reinforcement factor (cl.9.2.1.4(1)) $\beta_2 = \mathbf{0.25}$

Area of reinforcement to adjacent span $A_{s,span} = \mathbf{2513}\ mm^2$

Minimum bottom reinforcement to support $A_{s2,min} = \beta_2 \times A_{s,span} = \mathbf{628}\ mm^2$

Bottom reinforcement provided 4 × 16φ bars

Area of bottom reinforcement provided $A_{s2,prov} = \mathbf{804}\ mm^2$

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

Rectangular section in shear (Section 6.2)

Design shear force at support B $V_{Ed,max} = abs(max(V_{B\_max}, V_{B\_red})) = \mathbf{190}\ kN$

Angle of comp. shear strut for maximum shear $\theta_{max} = 45\ deg$

Maximum design shear force (exp.6.9) $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (cot(\theta_{max}) + tan(\theta_{max})) = \mathbf{595}\ kN$

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 5001 mm $V_{Ed} = abs(min(V_{B\_s1\_max}, V_{B\_s1\_red})) = \mathbf{172}\ kN$

Design shear stress $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{1.295}\ N/mm^2$

Strength reduction factor (cl.6.2.3(3)) $v_1 = 0.6 \times [1 - f_{ck} / 250\ N/mm^2] = \mathbf{0.528}$

Compression chord coefficient (cl.6.2.3(3)) $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3)

$$\theta = min(max(0.5 \times Asin[min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8\ deg), 45deg) = \mathbf{21.8}\ deg$$

Area of shear reinforcement required (exp.6.13) $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times cot(\theta)) = \mathbf{417}\ mm^2/m$

Shear reinforcement provided 4 × 8φ legs at 200 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1005}\ mm^2/m$

Minimum area of shear reinforcement (exp.9.5N) $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk} = \mathbf{307}\ mm^2/m$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N) $s_{vl,max} = 0.75 \times d = \mathbf{299}\ mm$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width $w_k = \mathbf{0.3}\ mm$

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} = \mathbf{2.9}\ N/mm^2$

Stress distribution coefficient $k_c = 0.4$

Non-uniform self-equilibrating stress coefficient $k = min(max(1 + (300\ mm - min(h, b)) \times 0.35 / 500\ mm, 0.65), 1) =$ **0.97**

Depth of tensile zone $h_{cr} = h - x = \mathbf{400}\ mm$

Area of concrete in the tensile zone $A_{ct} = h_{cr} \times b = \mathbf{140044}\ mm^2$

Adjusted maximum bar diameter (exp.7.6N) $\phi_{mod} = \phi_{top} \times (2.9\ N/mm^2 / f_{ct,eff}) \times 2 \times (h - d) / (k_c \times h_{cr}) = \mathbf{10}\ mm$

Maximum adjusted bar diameter $\phi_{max} = \mathbf{32}\ mm$

Tension bar spacing $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = \mathbf{62}\ mm$

Maximum tension bar spacing $s_{max} = \mathbf{300}\ mm$

Minimum allowable bar spacing $s_{min} = max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top} = \mathbf{41}\ mm$

Maximum stress permitted (Tables 7.2N & 7.3N) $\sigma_s = \mathbf{280}\ N/mm^2$

Minimum area of reinforcement required (exp.7.1) $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{559}\ mm^2$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

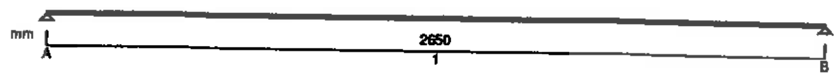
PASS - Actual bar spacing exceeds minimum allowable

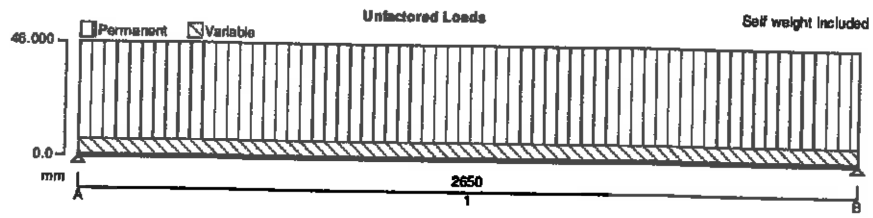
Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project: 45 Maresfield Gardens NW3 London				Job no.: 14.073	
	Calcs for: Beam BGRC1_between new staircase only				Start page no./Revision: T 17	
	Calcs by: RS	Calcs date: 19/06/2014	Checked by	Checked date	Approved by	Approved date

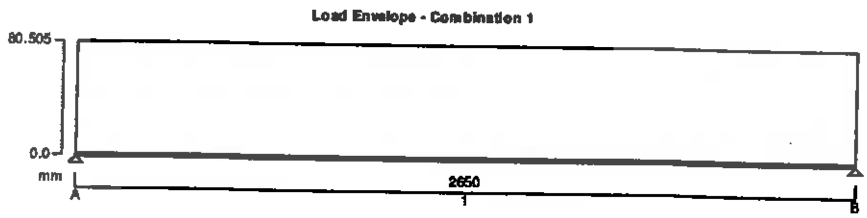
RC BEAM ANALYSIS & DESIGN (EN1992-1)

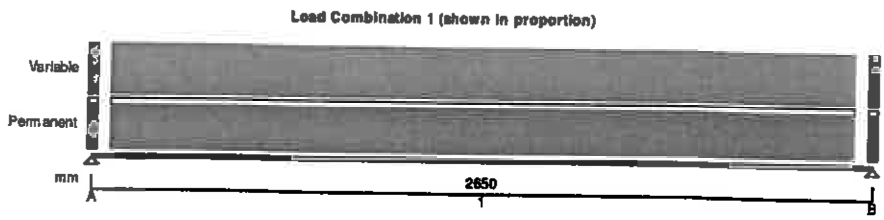
In accordance with UK national annex

TEDDS calculation version 2.1.13

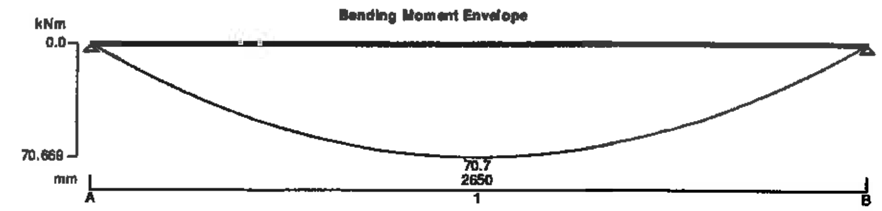


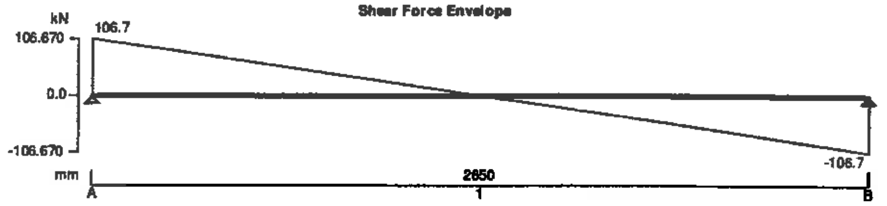






Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BGRC1_between new staircase only				Start page no./Revision T 18	
	Calcs by RS	Calcs date 19/06/2014	Checked by	Checked date	Approved by	Approved date





Support conditions

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

Applied loading

	Permanent self weight of beam × 1
Span 1 loads	Permanent UDL 46.000 kN/m from 0 mm to 2650 mm
	Variable UDL 6.700 kN/m from 0 mm to 2650 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

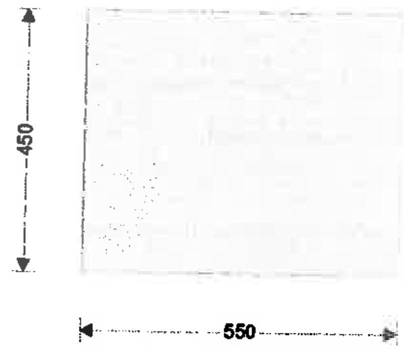
Maximum moment support A	M_{A_max} = 0 kNm	M_{A_red} = 0 kNm
Maximum moment span 1 at 1325 mm	M_{s1_max} = 71 kNm	M_{s1_red} = 71 kNm
Maximum moment support B	M_{B_max} = 0 kNm	M_{B_red} = 0 kNm
Maximum shear support A	V_{A_max} = 107 kN	V_{A_red} = 107 kN
Maximum shear support A span 1 at 397 mm	$V_{A\_s1\_max}$ = 75 kN	$V_{A\_s1\_red}$ = 75 kN
Maximum shear support B	V_{B_max} = -107 kN	V_{B_red} = -107 kN
Maximum shear support B span 1 at 2250 mm	$V_{B\_s1\_max}$ = -74 kN	$V_{B\_s1\_red}$ = -74 kN
Maximum reaction at support A	R_A = 107 kN	
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 69 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 9 kN	
Maximum reaction at support B	R_B = 107 kN	
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 69 kN	

Unfactored variable load reaction at support B $R_{B\_Variable}$ = 9 kN

Rectangular section details

Section width b = 550 mm

Section depth h = 450 mm



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

Concrete strength class	**C35/45**
Characteristic compressive cylinder strength	f_{ck} = **35 N/mm²**
Characteristic compressive cube strength	$f_{ck,cube}$ = **45 N/mm²**
Mean value of compressive cylinder strength	$f_{cm} = f_{ck}$ + 8 N/mm² = **43 N/mm²**
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm² × (f_{ck}/ 1 N/mm²)$^{2/3}$ = **3.2 N/mm²**
Secant modulus of elasticity of concrete	E_{cm} = 22 kN/mm² × [f_{cm}/10 N/mm²]$^{0.3}$ = **34077 N/mm²**
Partial factor for concrete (Table 2.1N)	γ_C = **1.50**
Compressive strength coefficient (cl.3.1.6(1))	α_{cc} = **0.85**
Design compressive concrete strength (exp.3.15)	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8 N/mm²**
Maximum aggregate size	h_{agg} = **20 mm**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500 N/mm²**
Partial factor for reinforcing steel (Table 2.1N)	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435 N/mm²**

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **35 mm**
Nominal cover to bottom reinforcement	c_{nom_b} = **35 mm**
Nominal cover to side reinforcement	c_{nom_s} = **35 mm**

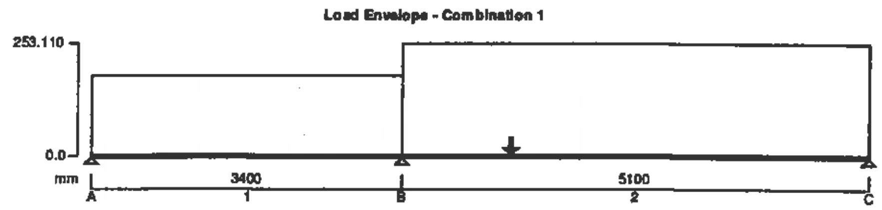
Support A

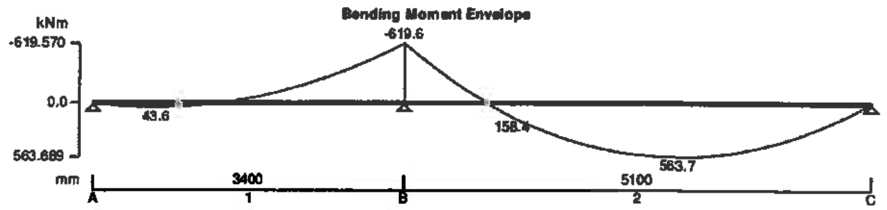
Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam RHS 400x200x16				Start page no./Revision T 20	
	Calcs by RS	Calcs date 13/06/2014	Checked by	Checked date	Approved by	Approved date

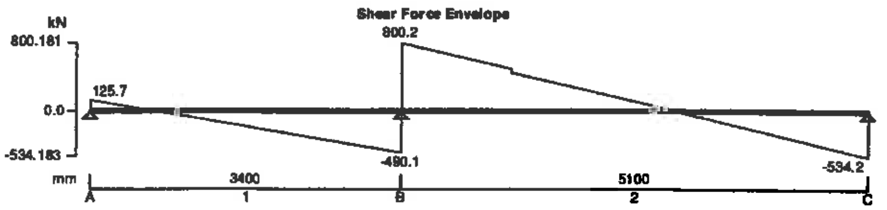
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.06







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
Span 1 loads	Permanent UDL 115 kN/m from 0 mm to 3400 mm Variable UDL 16 kN/m from 0 mm to 3400 mm
Span 2 loads	Permanent UDL 155 kN/m from 0 mm to 5100 mm Variable UDL 28 kN/m from 0 mm to 5100 mm Permanent point load 29 kN at 1200 mm Variable point load 2.9 kN at 1200 mm

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project: 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for: Beam RHS 400x200x16				Start page no./Revision: T 21	
	Calcs by: RS	Calcs date: 13/06/2014	Checked by	Checked date	Approved by	Approved date

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
	Span 2	Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = 563.7 kNm	M_{min} = -619.6 kNm
Maximum moment span 1	M_{s1_max} = 43.6 kNm	M_{s1_min} = -619.6 kNm
Maximum moment span 2	M_{s2_max} = 563.7 kNm	M_{s2_min} = -619.6 kNm
Maximum shear	V_{max} = 800.2 kN	V_{min} = -534.2 kN
Maximum shear span 1	V_{s1_max} = 125.7 kN	V_{s1_min} = -490.1 kN
Maximum shear span 2	V_{s2_max} = 800.2 kN	V_{s2_min} = -534.2 kN
Deflection	δ_{max} = 12.8 mm	δ_{min} = 1.6 mm
Deflection span 1	δ_{s1_max} = 0 mm	δ_{s1_min} = 1.6 mm
Deflection span 2	δ_{s2_max} = 12.8 mm	δ_{s2_min} = 0 mm
Maximum reaction at support A	R_{A_max} = 125.7 kN	R_{A_min} = 125.7 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 84.2 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 8 kN	
Maximum reaction at support B	R_{B_max} = 1290.3 kN	R_{B_min} = 1290.3 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 808.2 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 132.8 kN	
Maximum reaction at support C	R_{C_max} = 534.2 kN	R_{C_min} = 534.2 kN
Unfactored permanent load reaction at support C	$R_{C\_Permanent}$ = 329.8 kN	
Unfactored variable load reaction at support C	$R_{C\_Variable}$ = 59.3 kN	

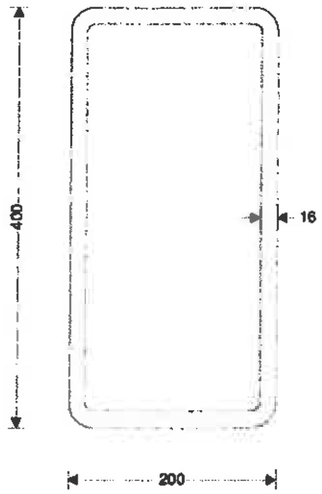
Section details

Section type	**RHS 400x200x16.0 (Corus Celsius)**
Steel grade	**S275H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = 16.0 mm
Nominal yield strength	f_y = 275 N/mm^2
Nominal ultimate tensile strength	f_u = 410 N/mm^2
Modulus of elasticity	E = 210000 N/mm^2

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam RHS 400x200x16				Start page no./Revision T 22	
	Calcs by RS	Calcs date 13/06/2014	Checked by	Checked date	Approved by	Approved date



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

Span 2 has full lateral restraint

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**

$K_{LT.C}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.92**

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

Width of section $c = h - 3 \times t$ = **352** mm

c / t = **23.8** $\times \varepsilon$ <= **72** $\times \varepsilon$ Class 1

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

Width of section $c = b - 3 \times t$ = **152** mm

c / t = **10.3** $\times \varepsilon$ <= **33** $\times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t$ = **368** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **800.2** kN

Shear area - cl 6.2.6(3) $A_v = A \times h / (b + h)$ = **11934** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **1894.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **619.6 kNm**

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **620.3 kNm**

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 $\delta_{lim} = \min(14\ mm, L_{s2} / 360)$ = **14 mm**

Maximum deflection span **2** $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **12.76 mm**

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BG1a				Start page no./Revision T 24	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = **171.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **297.8** kN	V_{min} = **-297.8** kN
Deflection	δ_{max} = **0.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **297.8** kN	R_{A_min} = **297.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **182.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **34.6** kN	
Maximum reaction at support B	R_{B_max} = **297.8** kN	R_{B_min} = **297.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **182.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **34.6** kN	

Section details

Section type	**UC 203x203x86 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **298** kN	Design shear resistance	$V_{c,Rd}$ = **469.6** kN

PASS - Design shear resistance exceeds design shear force

Combined bending and shear - Section 6.2.8

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **171.2** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **240.2** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection	δ_{lim} = **6.4** mm	Maximum deflection	δ = **0.553** mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project: 45 Maresfield Gardens NW3 London				Job no.: 14.073	
	Calcs for: Beam BG1b				Start page no./Revision: T 25	
	Calcs by: RS	Calcs date: 23/07/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = **204.3 kNm**	M_{min} = **0 kNm**
Maximum shear	V_{max} = **341.7 kN**	V_{min} = **-189.3 kN**
Deflection	δ_{max} = **1 mm**	δ_{min} = **0 mm**
Maximum reaction at support A	R_{A_max} = **341.7 kN**	R_{A_min} = **341.7 kN**
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **217.4 kN**	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **32.2 kN**	
Maximum reaction at support B	R_{B_max} = **189.3 kN**	R_{B_min} = **189.3 kN**
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **124.7 kN**	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **14 kN**	

Section details

Section type	**UC 203x203x86 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **342 kN** Design shear resistance $V_{c,Rd}$ = **469.6 kN**

PASS - Design shear resistance exceeds design shear force

Combined bending and shear - Section 6.2.8

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **204.3 kNm** Des.bending resist.moment $M_{c,Rd}$ = **205.1 kNm**

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **9.4 mm** Maximum deflection δ = **1.004 mm**

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG1 -supporting kitchen wall above				Start page no./Revision T 26	
	Calcs by RS	Calcs date 30/05/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = **65.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **38.1** kN	V_{min} = **-61** kN
Deflection	δ_{max} = **4.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **38.1** kN	R_{A_min} = **38.1** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **28.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **0.1** kN	
Maximum reaction at support B	R_{B_max} = **61** kN	R_{B_min} = **61** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **43.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.1** kN	

Section details

Section type	**UC 203x203x71 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **61** kN — Design shear resistance $V_{c,Rd}$ = **371.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **65.6** kNm — Des.bending resist.moment $M_{c,Rd}$ = **211.7** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.516** — Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **207.1** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent loads

Limiting deflection δ_{lim} = **15.8** mm — Maximum deflection δ = **4.445** mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG2				Start page no./Revision T 27	
	Calcs by RS	Calcs date 04/03/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

Support A — Vertically restrained, Rotationally free

Support B — Vertically restrained, Rotationally free

Analysis results

Maximum moment	M_{max} = 2.4 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 3.7 kN	V_{min} = -3.7 kN
Deflection	δ_{max} = 0.2 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 3.7 kN	R_{A_min} = 3.7 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 2.7 kN	
Maximum reaction at support B	R_{B_max} = 3.7 kN	R_{B_min} = 3.7 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 2.7 kN	

Section details

Section type **UB 203x133x25 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 4 kN Design shear resistance $V_{c,Rd}$ = 203.5 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 2.4 kNm Des.bending resist.moment $M_{c,Rd}$ = 70.9 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\overline{\lambda}_{LT}$ = 0.704 Limiting slenderness ratio $\overline{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 63.4 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 δ_{lim} = 7.1 mm Maximum deflection δ = 0.24 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG3a				Start page no./Revision T 28	
	Calcs by RS	Calcs date 13/06/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 39.5 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 31.6 kN	V_{min} = -31.6 kN
Deflection	δ_{max} = 7.4 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 31.6 kN	R_{A_min} = 31.6 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 7.1 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 14.6 kN	
Maximum reaction at support B	R_{B_max} = 31.6 kN	R_{B_min} = 31.6 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 7.1 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 14.6 kN	

Section details

Section type **UC 203x203x46 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 32 kN Design shear resistance $V_{c,Rd}$ = 269.5 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 39.5 kNm Des.bending resist.moment $M_{c,Rd}$ = 136.8 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = 0.709 Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 121.9 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 δ_{lim} = 13.9 mm Maximum deflection δ = 7.383 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London			Job no. 14.073	
	Calcs for Beam BLG3			Start page no./Revision T 29	
	Calcs by RS	Calcs date 09/06/2014	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.07

Analysis results

Design moment $M = 2.563$ kNm Design shear $F = 7.885$ kN

Total load on member $W_{tot} = 15.770$ kN

Reactions at support A $R_{A\_max} = 7.885$ kN $R_{A\_min} = 7.885$ kN

Unfactored permanent load reaction at support A $R_{A\_Permanent} = 1.616$ kN

Unfactored variable load reaction at support A $R_{A\_Variable} = 3.803$ kN

Reactions at support B $R_{B\_max} = 7.885$ kN $R_{B\_min} = 7.885$ kN

Unfactored permanent load reaction at support B $R_{B\_Permanent} = 1.616$ kN

Unfactored variable load reaction at support B $R_{B\_Variable} = 3.802$ kN

Timber section details

Breadth of section $b = 100$ mm Depth of section $h = 250$ mm

Number of sections $N = 1$ Breadth of member $b_b = 100$ mm

Timber strength class **C24**

Member details

Service class of timber 1 Load duration **Long-term**

Length of bearing $L_b = 100$ mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d} = 0.789$ N/mm² Design compressive strength $f_{c.90.d} = 1.346$ N/mm²

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d} = 2.460$ N/mm² Design bending strength $f_{m.d} = 12.923$ N/mm²

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 0.706$ N/mm² Permissible shear stress $f_{v.d} = 2.154$ N/mm²

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 5.200$ mm Total final deflection $\delta_{fin} = 0.442$ mm

PASS - Total final deflection is less than the deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG4				Start page no./Revision T 30	
	Calcs by RS	Calcs date 04/06/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

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

Analysis results

Maximum moment	M_{max} = 119.7 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 91.3 kN	V_{min} = -119.6 kN
Deflection	δ_{max} = 7.1 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 91.3 kN	R_{A_min} = 91.3 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 56.6 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 9.9 kN	
Maximum reaction at support B	R_{B_max} = 119.6 kN	R_{B_min} = 119.6 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 77.6 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 9.9 kN	

Section details

Section type	**UC 203x203x60 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = 120 kN	Design shear resistance	$V_{c,Rd}$ = 351.8 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = 119.7 kNm	Des.bending resist.moment	$M_{c,Rd}$ = 180.4 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\overline{\lambda}_{LT}$ = 0.493	Limiting slenderness ratio	$\overline{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 178.1 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	δ_{lim} = 9.3 mm	Maximum deflection	δ = 7.08 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG5				Start page no./Revision T 31	
	Calcs by RS	Calcs date 04/03/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 59.2 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 102.9 kN	V_{min} = -102.9 kN
Deflection	δ_{max} = 2.7 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 102.9 kN	R_{A_min} = 102.9 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 71 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 4.7 kN	
Maximum reaction at support B	R_{B_max} = 102.9 kN	R_{B_min} = 102.9 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 71 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 4.7 kN	

Section details

Section type **2 x UB 203x102x23 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 103 kN Design shear resistance $V_{c,Rd}$ = 393.1 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 59.2 kNm Des.bending resist.moment $M_{c,Rd}$ = 128.7 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = 0.794 Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 108.9 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 δ_{lim} = 6.4 mm Maximum deflection δ = 2.714 mm

PASS - Maximum deflection does not exceed deflection limit

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 36.3 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 41.5 kN	V_{min} = -41.5 kN
Deflection	δ_{max} = 6.7 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 41.5 kN	R_{A_min} = 41.5 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 17.9 kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = 11.6 kN	
Maximum reaction at support B	R_{B_max} = 41.5 kN	R_{B_min} = 41.5 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 17.9 kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = 11.6 kN	

Section details

Section type	**UB 203x133x25 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = 42 kN — Design shear resistance $V_{c,Rd}$ = 203.5 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 36.3 kNm — Des.bending resist.moment $M_{c,Rd}$ = 70.9 kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = 0.904 — Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = 0.400

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = 55.3 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 δ_{lim} = 9.7 mm — Maximum deflection δ = 6.698 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BLG10				Start page no./Revision T 33	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **36.3 kNm**	M_{min} = **0 kNm**
Maximum shear	V_{max} = **41.5 kN**	V_{min} = **-41.5 kN**
Deflection	δ_{max} = **6.7 mm**	δ_{min} = **0 mm**
Maximum reaction at support A	R_{A_max} = **41.5 kN**	R_{A_min} = **41.5 kN**
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **17.9 kN**	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **11.6 kN**	
Maximum reaction at support B	R_{B_max} = **41.5 kN**	R_{B_min} = **41.5 kN**
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **17.9 kN**	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **11.6 kN**	

Section details

Section type **UB 203x133x25 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **42 kN** Design shear resistance $V_{c,Rd}$ = **203.5 kN**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **36.3 kNm** Des.bending resist.moment $M_{c,Rd}$ = **70.9 kNm**

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.904** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **55.3 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

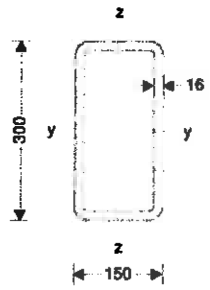
Limiting deflection δ_{lim} = **9.7 mm** Maximum deflection δ = **6.698 mm**

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **RHS 300x150x16.0**

System length y axis buckling L_y = **6500** mm — System length z axis buckling L_z = **6500** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **670** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **0.0** kNm — Moment about y axis at end 2 $M_{y,Ed2}$ = **0.0** kNm

Moment about z axis at end 1 $M_{z,Ed1}$ = **104.0** kNm — Moment about z axis at end 2 $M_{z,Ed2}$ = **0.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **0** kN — Shear force parallel to y axis $V_{y,Ed}$ = **0** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm² — Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm² — Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** — Buckling length L_{cr_y} = **6500** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** — Buckling length L_{cr_z} = **6500** mm

Section classification (Table 5.2)

Web classification **1** — Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **0.0** kN — Plastic shear resistance $V_{pl,z,Rd}$ = **1386.7** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= $0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = 0.0 kN — Plastic shear resistance $V_{pl,y,Rd}$ = 693.4 kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= $0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = 670 kN — Design resistance $N_{c,Rd}$ = 3603 kN

PASS - The compression design resistance exceeds the design force

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = 104.0 kNm — Design resistance $M_{c,z,Rd}$ = 201.4 kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = 104.0 kNm — Modified design resistance $M_{N,z,Rd}$ = 189.2 kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

<u>Buckling resistance (cl. 6.3)</u>

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = 3019.4 kN — Flexural buck resist about z $N_{b,z,Rd}$ = 1768.4 kN

Min. buckling resistance $N_{b,Rd}$ = 1768.4 kN

PASS - The axial load buckling resistance exceeds the design axial load

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = 0.464 — Section utilisation UR_{B_2} = 0.783

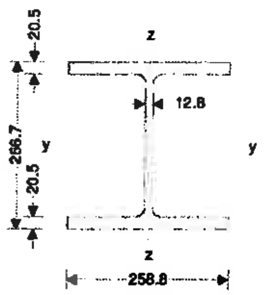
PASS - The buckling resistance is adequate

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Beam BG1a				Start page no./Revision T 36	
	Calcs by RS	Calcs date 09/06/2014	Checked by	Checked date	Approved by	Approved date

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **UC 254x254x107**

System length y axis buckling L_y = **6000** mm | System length z axis buckling L_z = **6000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **1445** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **19.4** kNm | Moment about y axis at end 2 $M_{y,Ed2}$ = **0.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **6.2** kNm | Moment about z axis at end 2 $M_{z,Ed2}$ = **0.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **0** kN | Shear force parallel to y axis $V_{y,Ed}$ = **0** kN

Material details

Steel grade **S275**

Yield strength f_y = **265** N/mm² | Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm² | Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** | Buckling length L_{cr_y} = **6000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** | Buckling length L_{cr_z} = **6000** mm

Section classification (Table 5.2)

Web classification **1** | Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **0.0** kN | Plastic shear resistance $V_{pl,z,Rd}$ = **583.0** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= $0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **0.0 kN** Plastic shear resistance $V_{pl,y,Rd}$ = **1503.6 kN**

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= $0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **1445 kN** Design resistance $N_{c,Rd}$ = **3614 kN**

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **19.4 kNm** Design resistance $M_{c,y,Rd}$ = **393.4 kNm**

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **6.2 kNm** Design resistance $M_{c,z,Rd}$ = **184.7 kNm**

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **19.4 kNm** Modified design resistance $M_{N,y,Rd}$ = **265.5 kNm**

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **6.2 kNm** Modified design resistance $M_{N,z,Rd}$ = **175.1 kNm**

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.007** Section utilisation at end 2 UR_{CS_2} = **0.000**

PASS - The cross-section resistance is adequate

__**Buckling resistance (cl. 6.3)**__

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **3027.5 kN** Flexural buck resist about z $N_{b,z,Rd}$ = **1891.1 kN**

Torsional buck. length factor K_T = **1.00** Torsional/tor flex buck resist $N_{b,T,Rd}$ = **2841.2 kN**

Min. buckling resistance $N_{b,Rd}$ = **1891.1 kN**

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **19.4 kNm**

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **393.4 kNm**

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

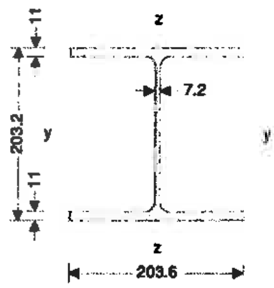
Section utilisation UR_{B_1} = **0.539** Section utilisation UR_{B_2} = **0.846**

PASS - The buckling resistance is adequate

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **UC 203x203x46**

System length y axis buckling L_y = **3250** mm — System length z axis buckling L_z = **3250** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **460** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **48.4** kNm — Moment about y axis at end 2 $M_{y,Ed2}$ = **0.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **0.0** kNm — Moment about z axis at end 2 $M_{z,Ed2}$ = **0.0** kNm

Shear force parallel to z axis $V_{z,Ed}$ = **0** kN — Shear force parallel to y axis $V_{y,Ed}$ = **0** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm^2 — Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2 — Shear modulus G = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** — Buckling length L_{cr_y} = **3250** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** — Buckling length L_{cr_z} = **3250** mm

Section classification (Table 5.2)

Web classification **1** — Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **0.0** kN — Plastic shear resistance $V_{pl,z,Rd}$ = **269.5** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project: 45 Maresfield Gardens NW3 London			Job no.: 14.073	
	Calcs for: Restraining Beam			Start page no./Revision: T 53	
	Calcs by: RS	Calcs date: 24/07/2014	Checked by:	Checked date:	Approved by:	Approved date:

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 3.0.06

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = 7.1 kNm	M_{min} = 0 kNm
Maximum shear	V_{max} = 10.9 kN	V_{min} = -10.9 kN
Deflection	δ_{max} = 0.8 mm	δ_{min} = 0 mm
Maximum reaction at support A	R_{A_max} = 10.9 kN	R_{A_min} = 10.9 kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = 8.1 kN	
Maximum reaction at support B	R_{B_max} = 10.9 kN	R_{B_min} = 10.9 kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = 8.1 kN	

Section details

Section type **UB 203x102x23 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = 11 kN Design shear resistance $V_{c,Rd}$ = 196.6 kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = 7.1 kNm Des.bending resist.moment $M_{c,Rd}$ = 64.4 kNm

PASS - Design bending resistance moment exceeds design bending moment

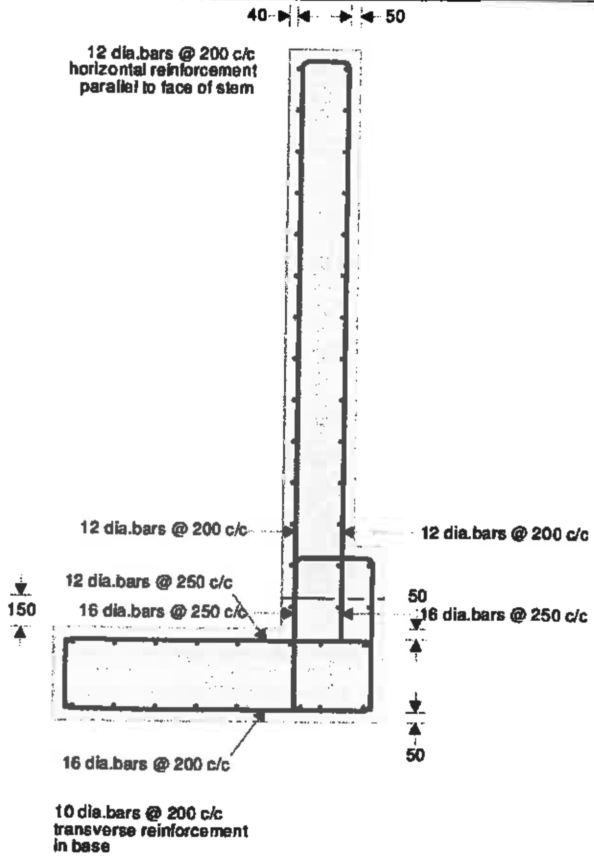
Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = 7.2 mm Maximum deflection δ = 0.838 mm

PASS - Maximum deflection does not exceed deflection limit

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Retaining Wall				Start page no./Revision T 52	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date



Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Retaining Wall				Start page no./Revision T 51	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

Serviceability bending moment	M_{sls} = **53.9** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **143.9** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **133667** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.008**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.395**
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	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **532** 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 = **0.219** mm
	w_k / w_{max} = **0.729**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **133.6** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.714**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.003**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.430** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **168.7** kN/m
	$V / V_{Rd.c}$ = **0.792**

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	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm²/m

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

Modular ratio	$\alpha_e = E_s / E_{cm} = $ **6.395**
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_{sr1} / \rho_{p.eff} = $ **547** 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 = $ **0.199** mm
	$w_k / w_{max} = $ **0.662**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = $ **39.1** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = $ **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = $ **1.825**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02) = $ **0.002**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = $ **0.473** N/mm²
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} = $ **138.9** kN/m
	$V / V_{Rd.c} = $ **0.281**

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_{s1}) = $ **500** mm²/m
Maximum spacing of reinforcement – cl.9.6.3(2)	$s_{sx_max} = $ **400** mm
Transverse reinforcement provided	12 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = $ **565** mm²/m

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

Check base design

Depth of section	$h = $ **450** mm

Rectangular section in flexure - Section 6.1

Design bending moment at toe	$M = $ **73.5** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2 = $ **392** mm
	$K = M / (d^2 \times f_{ck}) = $ **0.016**
	$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 = $ **372** mm
Depth of neutral axis	$x = 2.5 \times (d - z) = $ **49** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z) = $ **454** mm²/m
Tension reinforcement provided	16 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = $ **1005** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = $ **590** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h = $ **18000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = $ **0.587**

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	$\psi_2 = $ **0.6**

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	$s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sf} / \rho_{p.eff}$ = **638** 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 = **0.073** mm
	w_k / w_{max} = **0.242**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **177.4** kN/m
	$C_{Rd.c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.674**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sf.prov} / d, 0.02)$ = **0.002**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.415** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **182.7** kN/m
	$V / V_{Rd.c}$ = **0.971**

PASS - Design shear resistance exceeds design shear force

Check stem design at 400 mm

Depth of section	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment at 400 mm up stem	M = **32.7** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{srt} / 2$ = **294** mm
	$K = M / (d^2 \times f_{ck})$ = **0.013**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **279** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required	$A_{sr1.req} = M / (f_{yd} \times z)$ = **269** mm²/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{sr1.prov} = \pi \times \phi_{sr1}^2 / (4 \times s_{sr1})$ = **565** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sr1.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **443** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr1.max} = 0.04 \times h$ = **14000** mm²/m
	$\max(A_{sr1.req}, A_{sr1.min}) / A_{sr1.prov}$ = **0.783**

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} = **20.1** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr1.prov} \times z)$ = **127.2** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **104417** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sr1.prov} / A_{c.eff}$ = **0.005**

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{6.395}$

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 $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{660}$ mm

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

$w_k = \mathbf{0.06}$ mm

$w_k / w_{max} = \mathbf{0.2}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{177.4}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.673}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.002}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.415}$ N/mm²

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

$V_{Rd.c} = \mathbf{183.3}$ kN/m

$V / V_{Rd.c} = \mathbf{0.968}$

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure - Section 6.1

Design bending moment $M = \mathbf{21.5}$ kNm/m

Depth to tension reinforcement $d = h - c_{sl} - \phi_{sx} - \phi_{sl} / 2 = \mathbf{440}$ mm

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{418}$ mm

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{55}$ mm

Area of tension reinforcement required $A_{sl.req} = M / (f_{yd} \times z) = \mathbf{119}$ mm²/m

Tension reinforcement provided 16 dia.bars @ 250 c/c

Area of tension reinforcement provided $A_{sl.prov} = \pi \times \phi_{sl}^2 / (4 \times s_{sl}) = \mathbf{804}$ mm²/m

Minimum area of reinforcement - exp.9.1N $A_{sl.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{663}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sl.max} = 0.04 \times h = \mathbf{20000}$ mm²/m

$\max(A_{sl.req}, A_{sl.min}) / A_{sl.prov} = \mathbf{0.824}$

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

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 $M_{sls} = \mathbf{13.4}$ kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sl.prov} \times z) = \mathbf{39.9}$ N/mm²

Load duration Long term

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

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{148333}$ mm²/m

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

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

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{6.395}$

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	$f_{ctk,0.05} = 0.7 \times f_{ctm} = 2.0\ N/mm^2$
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	$\gamma_C = 1.50$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 17.0\ N/mm^2$
Maximum aggregate size	$h_{agg} = 20\ mm$

Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = 500\ N/mm^2$
Modulus of elasticity of reinforcement	$E_s = 210000\ N/mm^2$
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	$c_{sf} = 40\ mm$
Rear face of stem	$c_{sr} = 50\ mm$
Top face of base	$c_{bt} = 50\ mm$
Bottom face of base	$c_{bb} = 50\ mm$

Check stem design for maximum moment

Depth of section — $h = 500\ mm$

Rectangular section in flexure - Section 6.1

Design bending moment	$M = 17.2\ kNm/m$
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2 = 442\ mm$
	$K = M / (d^2 \times f_{ck}) = 0.003$
	$K' = 0.207$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = 420\ mm$
Depth of neutral axis	$x = 2.5 \times (d - z) = 55\ mm$
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z) = 94\ mm^2/m$
Tension reinforcement provided	16 dia.bars @ 250 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 804\ mm^2/m$
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 666\ mm^2/m$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h = 20000\ mm^2/m$
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = 0.828$

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	$\psi_2 = 0.6$
Serviceability bending moment	$M_{sls} = 10.7\ kNm/m$
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) = 31.7\ N/mm^2$
Load duration	Long term
Load duration factor	$k_t = 0.4$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = 145000\ mm^2/m$
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = 2.9\ N/mm^2$
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = 0.006$

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = A_{stem} \times \gamma_{stem} = 26$ kN/m

Wall base $F_{base} = A_{base} \times \gamma_{base} = 18$ kN/m

Surcharge load $F_{sur\_v} = Surcharge_Q \times (l_{heel} + t_{s1} - t_{s2}) = 1.1$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = 109.5$ kN/m

Moist retained soil $F_{moist\_v} = A_{moist} \times \gamma_{mr} = 5.9$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{moist\_v} + F_{sur\_v} + F_{P\_v} = 160.5$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} = 10.2$ kN/m

Moist retained soil $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr} \times h_{eff}^2 / 2 = 39.9$ kN/m

Total $F_{total\_h} = F_{moist\_h} + F_{sur\_h} = 50.1$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = 33.5$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = 14.4$ kNm/m

Surcharge load $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = -13.9$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = 139.6$ kNm/m

Moist retained soil $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = -31.6$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} + M_P = 142.1$ kNm/m

Check bearing pressure

Maximum friction force $F_{friction\_max} = F_{total\_v} \times \tan(\delta_{bb.d}) = 34.1$ kN/m

Maximum base soil resistance $F_{pass\_h\_max} = K_P \times \cos(\delta_{b.d}) \times \gamma_{mb} \times (d_{cover} + h_{base})^2 / 2 = 3.5$ kN/m

Base soil resistance $F_{pass\_h} = \min(\max((M_{total} + F_{total\_h} \times (h_{prop} + t_{base}) + F_{friction\_max} \times (h_{prop} + t_{base}) - F_{total\_v} \times l_{base} / 2) / (x_{pass\_h} - h_{prop} - t_{base}), 0 \text{ kN/m}), F_{pass\_h\_max}) = 0$ kN/m

Propping force $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h}) = -22.8$ kN/m

Friction force $F_{friction} = F_{total\_h} - F_{pass\_h} - F_{prop\_stem} = 72.9$ kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = -13.7$ kNm/m

Distance to reaction $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = 800$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = 0$ mm

Loaded length of base $l_{load} = l_{base} = 1600$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = 100.3$ kN/m²

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = 100.3$ kN/m²

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = 1.246$

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 2.2.01

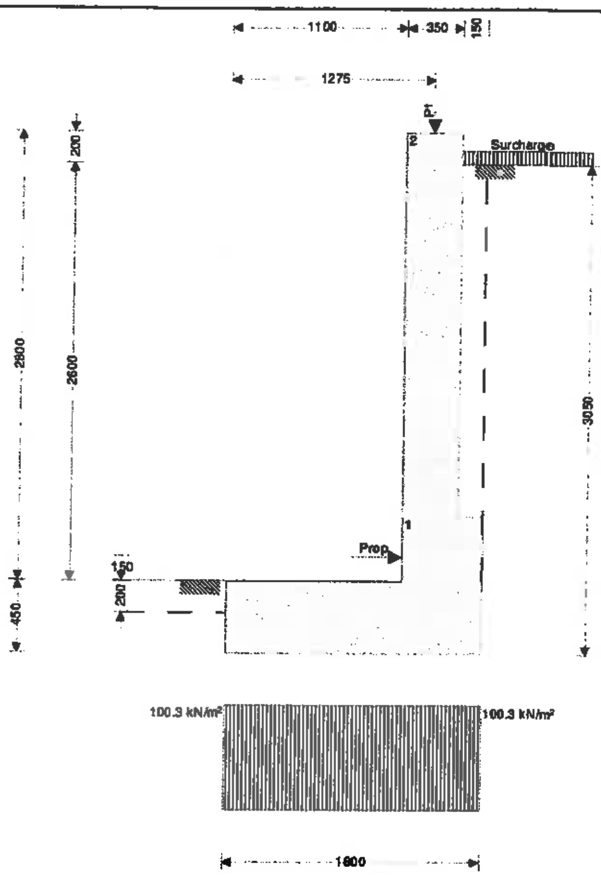
Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class C30/37

Characteristic compressive cylinder strength $f_{ck} = 30$ N/mm²

Characteristic compressive cube strength $f_{ck,cube} = 37$ N/mm²

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project: 45 Maresfield Gardens NW3 London			Job no.: 14.073	
	Calcs for: Retaining Wall			Start page no./Revision: T 45	
	Calcs by: RS	Calcs date: 23/07/2014	Checked by	Checked date	Approved by	Approved date



Calculate retaining wall geometry

Base length $l_{base} = l_{toe} + t_{s1} = \mathbf{1600}$ mm

Moist soil height $h_{moist} = h_{soil} = \mathbf{2600}$ mm

Length of surcharge load $l_{sur} = (l_{heel} + t_{s1} - t_{s2}) = \mathbf{150}$ mm

- Distance to vertical component $x_{sur\_v} = l_{base} - (l_{heel} + t_{s1} - t_{s2}) / 2 = \mathbf{1525}$ mm

Effective height of wall $h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{3050}$ mm

- Distance to horizontal component $x_{sur\_h} = h_{eff} / 2 = \mathbf{1525}$ mm

Area of wall stem $A_{stem} = h_{s1} \times t_{s1} + h_{s2} \times t_{s2} = \mathbf{1.04}$ $\mathbf{m^2}$

- Distance to vertical component $x_{stem} = l_{base} - (h_{s1} \times t_{s1} \times (l_{heel} + t_{s1} - t_{s1} / 2) + h_{s2} \times t_{s2} \times (l_{heel} + t_{s1} - t_{s2} / 2)) / A_{stem} = \mathbf{1289}$ mm

Area of wall base $A_{base} = l_{base} \times t_{base} = \mathbf{0.72}$ $\mathbf{m^2}$

- Distance to vertical component $x_{base} = l_{base} / 2 = \mathbf{800}$ mm

Area of moist soil $A_{moist} = l_{heel} \times h_{s1} + (l_{heel} + t_{s1} - t_{s2}) \times (h_{soil} - h_{s1}) = \mathbf{0.33}$ $\mathbf{m^2}$

- Distance to vertical component $x_{moist\_v} = l_{base} - (l_{heel} \times h_{s1} \times l_{heel} / 2 + (l_{heel} + t_{s1} - t_{s2}) \times (h_{soil} - h_{s1}) \times (l_{heel} + t_{s1} - t_{s2}) / 2) / A_{moist} = \mathbf{1525}$ mm

- Distance to horizontal component $x_{moist\_h} = h_{eff} / 3 = \mathbf{1017}$ mm

Using Coulomb theory

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

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for Retaining Wall				Start page no./Revision T 44	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

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 2.2.01

Retaining wall details

Stem type	Propped cantilever with stepped rear face
Stem height	h_{stem} = **2800 mm**
Prop height	h_{prop} = **150 mm**
Number of steps	N_{steps} = **2**
Height of step 1	h_{s1} = **400 mm**
Thickness of step 1	t_{s1} = **500 mm**
Height of step 2	h_{s2} = **2400 mm**
Thickness of step 2	t_{s2} = **350 mm**
Angle to rear face of stem	α = **90 deg**
Stem density	γ_{stem} = **25 kN/m^3**
Toe length	l_{toe} = **1100 mm**
Base thickness	t_{base} = **450 mm**
Base density	γ_{base} = **25 kN/m^3**
Height of retained soil	h_{ret} = **2600 mm**
Angle of soil surface	β = **0 deg**
Depth of cover	d_{cover} = **0 mm**
Depth of excavation	d_{exc} = **200 mm**

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	Organic clay
Moist density	γ_{mb} = **15 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	$P_{bearing}$ = **125 kN/m^2**

Loading details

Variable surcharge load	$Surcharge_Q$ = **7 kN/m^2**
Vertical line load at 1275 mm	P_{G1} = **97.5 kN/m**
	P_{Q1} = **12 kN/m**

Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London				Job no. 14.073	
	Calcs for New Enlarged foundation under Columns C3 and C4				Start page no./Revision T 43	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

Soil self weight — $F_{soil} = h_{soil} \times \rho_{soil} = \mathbf{4.000}$ **kN/m²**

Total foundation load — $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{335.6}$ **kN**

Calculate pad base reaction

Total base reaction — $T = F + P_A + P_B = \mathbf{1172.4}$ **kN**

Eccentricity of base reaction in x — $e_{Tx} = (P_A \times e_{PxA} + P_B \times e_{PxB} + M_{xA} + M_{xB} + (H_{xA} + H_{xB}) \times h) / T = \mathbf{61}$ **mm**

Eccentricity of base reaction in y — $e_{Ty} = (P_A \times e_{PyA} + P_B \times e_{PyB} + M_{yA} + M_{yB} + (H_{yA} + H_{yB}) \times h) / T = \mathbf{107}$ **mm**

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B = \mathbf{0.050}$

Base reaction acts within combined middle third of base

Calculate pad base pressures

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{67.713}$ **kN/m²**

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{106.422}$ **kN/m²**

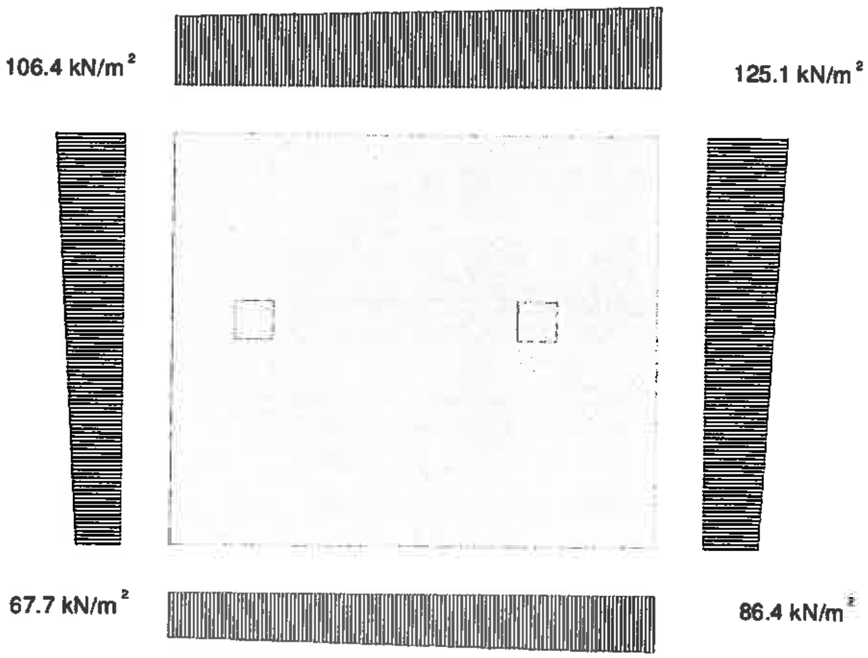
$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{86.409}$ **kN/m²**

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{125.118}$ **kN/m²**

Minimum base pressure — $q_{min} = min(q_1, q_2, q_3, q_4) = \mathbf{67.713}$ **kN/m²**

Maximum base pressure — $q_{max} = max(q_1, q_2, q_3, q_4) = \mathbf{125.118}$ **kN/m²**

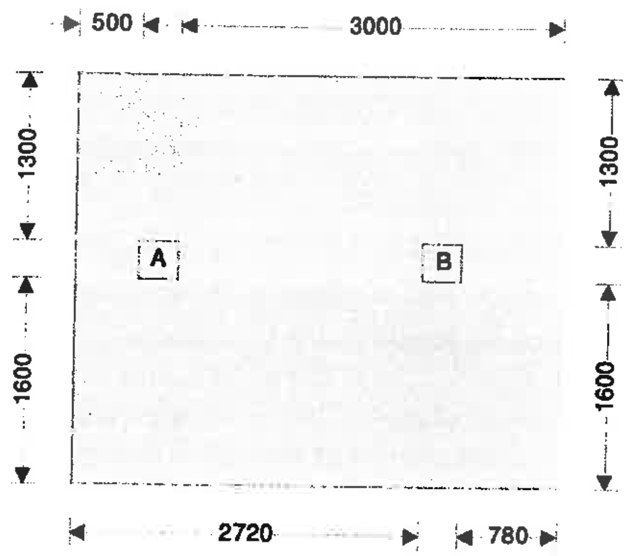
PASS - Maximum base pressure is less than allowable bearing pressure



Tedds Martin Redston Associates 6 Hale Lane London NW7 3NX	Project 45 Maresfield Gardens NW3 London			Job no. 14.073	
	Calcs for New Enlarged foundation under Columns C3 and C4			Start page no./Revision T 42	
	Calcs by RS	Calcs date 23/07/2014	Checked by	Checked date	Approved by	Approved date

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

TEDDS calculation version 2.0.03.00



Pad footing details

Length of pad footing	$L = 3800$ mm
Width of pad footing	$B = 3200$ mm
Area of pad footing	$A = L \times B = 12.160$ m^2
Depth of pad footing	$h = 1000$ mm
Depth of soil over pad footing	$h_{soil} = 200$ mm
Density of concrete	$\rho_{conc} = 23.6$ kN/m^3

Column details	**Column A**	**Column B**
Column base length	$l_A = 300$ mm	$l_B = 300$ mm
Column base width	$b_A = 300$ mm	$b_B = 300$ mm
Column eccentricity in x	$e_{PxA} = -1250$ mm	$e_{PxB} = 970$ mm
Column eccentricity in y	$e_{PyA} = 150$ mm	$e_{PyB} = 150$ mm

Soil details

Density of soil	$\rho_{soil} = 20.0$ kN/m^3
Design shear strength	$\phi' = 25.0$ deg
Design base friction	$\delta = 19.3$ deg
Allowable bearing pressure	$P_{bearing} = 135$ kN/m^2

Axial loading on columns	**Column A**	**Column B**
Dead axial load on column	$P_{GA} = 273.2$ kN	$P_{GB} = 416.6$ kN
Imposed axial load on column	$P_{QA} = 60.0$ kN	$P_{QB} = 87.0$ kN
Wind axial load on column	$P_{WA} = 0.0$ kN	$P_{WB} = 0.0$ kN
Total axial load on column	$P_A = 333.2$ kN	$P_B = 503.6$ kN

Foundation loads

Dead surcharge load	$F_{Gsur} = 0.000$ kN/m^2
Imposed surcharge load	$F_{Qsur} = 0.000$ kN/m^2
Pad footing self weight	$F_{swt} = h \times \rho_{conc} = 23.600$ kN/m^2

$V_{z,Ed}$ <= $0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **0.0 kN** Plastic shear resistance $V_{pl,y,Rd}$ = **754.8 kN**

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= $0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **710 kN** Design resistance $N_{c,Rd}$ = **1823 kN**

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **78.0 kNm** Design resistance $M_{c,y,Rd}$ = **156.0 kNm**

PASS - The bending design resistance about the y axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **78.0 kNm** Modified design resistance $M_{N,y,Rd}$ = **107.6 kNm**

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **1673.2 kN** Flexural buck resist about z $N_{b,z,Rd}$ = **1295.0 kN**

Torsional buck. length factor K_T = **1.00** Torsional/tor flex buck resist $N_{b,T,Rd}$ = **1457.6 kN**

Min. buckling resistance $N_{b,Rd}$ = **1295.0 kN**

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **78.0 kNm**

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **156.0 kNm**

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

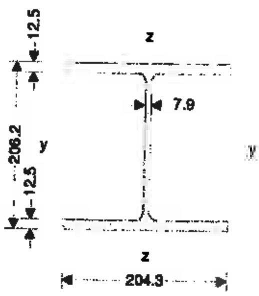
Section utilisation UR_{B_1} = **0.752** Section utilisation UR_{B_2} = **0.992**

PASS - The buckling resistance is adequate

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **UC 203x203x52**

System length y axis buckling L_y = **3250** mm

System length z axis buckling L_z = **3250** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **710** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **78.0** kNm

Moment about y axis at end 2 $M_{y,Ed2}$ = **0.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **0.0** kNm

Moment about z axis at end 2 $M_{z,Ed2}$ = **0.0** kNm

Shear force parallel to z axis $V_{z,Ed}$ = **0** kN

Shear force parallel to y axis $V_{y,Ed}$ = **0** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm²

Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm²

Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000**

Buckling length L_{cr_y} = **3250** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000**

Buckling length L_{cr_z} = **3250** mm

Section classification (Table 5.2)

Web classification **1**

Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **0.0** kN

Plastic shear resistance $V_{pl,z,Rd}$ = **297.6** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= $0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = 0.0 kN Plastic shear resistance $V_{pl,y,Rd}$ = 663.0 kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= $0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = 460 kN Design resistance $N_{c,Rd}$ = 1615 kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = 48.4 kNm Design resistance $M_{c,y,Rd}$ = 136.8 kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = 48.4 kNm Modified design resistance $M_{N,y,Rd}$ = 111.0 kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = 1479.8 kN Flexural buck resist about z $N_{b,z,Rd}$ = 1141.2 kN

Torsional buck. length factor K_T = 1.00 Torsional/tor flex buck resist $N_{b,T,Rd}$ = 1263.7 kN

Min. buckling resistance $N_{b,Rd}$ = 1141.2 kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = 48.4 kNm

Lat. torsional buck length fact K_{LT} = 1.00 Design buckling resistance mt $M_{b,Rd}$ = 136.8 kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

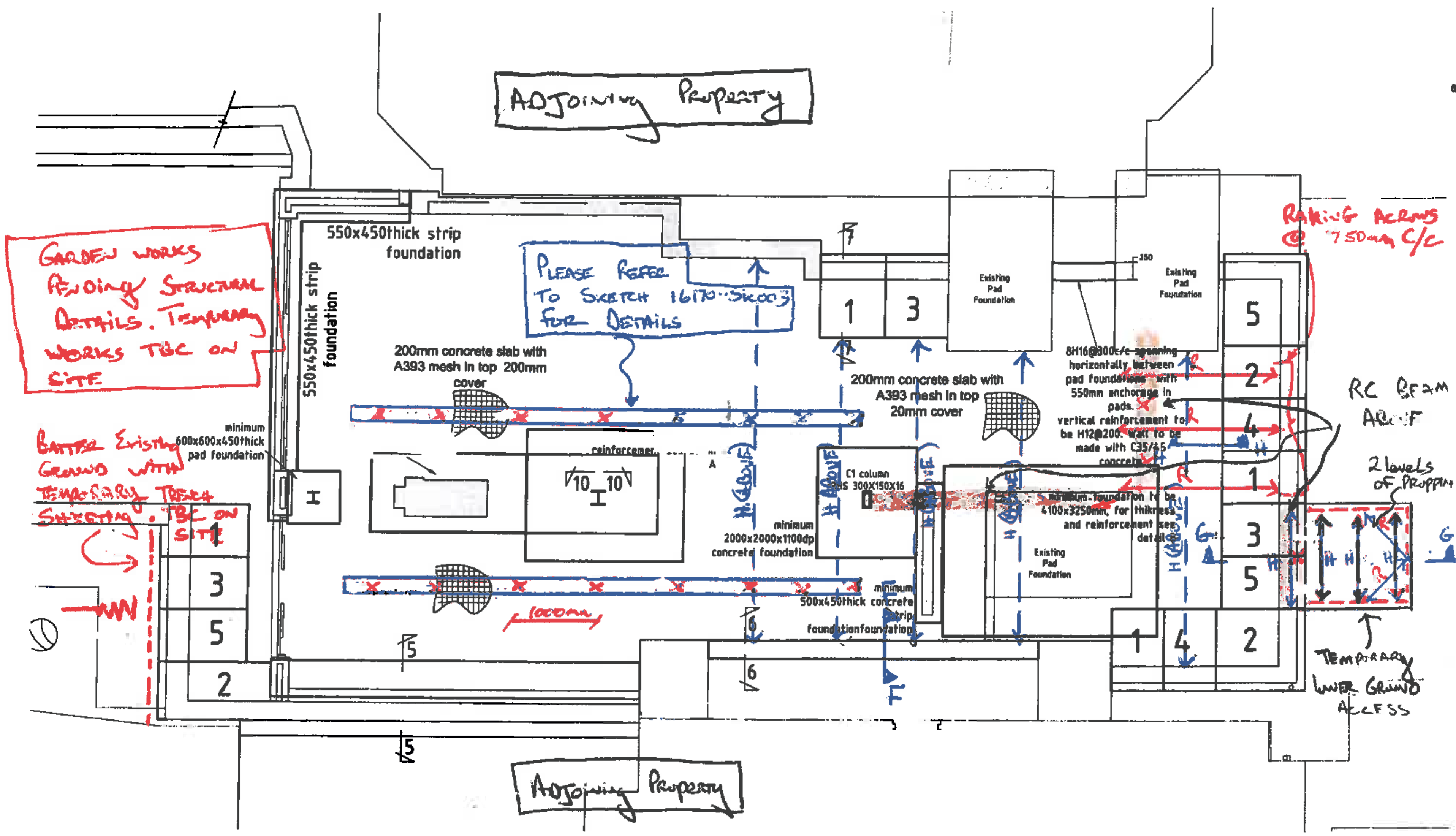
Combined bending and axial compression (cl. 6.3.3)

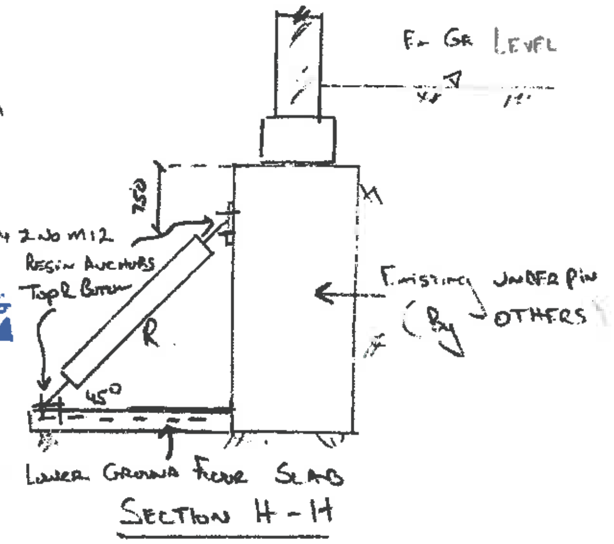
Section utilisation UR_{B_1} = 0.539 Section utilisation UR_{B_2} = 0.728

PASS - The buckling resistance is adequate

NOTE:

• HORIZONTAL PROPS AS PER SECTION F-F & PROPPING TO EXISTING RC BEAMS & FIRST FLOOR SLAB TO BE INSTALLED PRIOR TO FORMING NEW LOWER GROUND FLOOR SLAB.





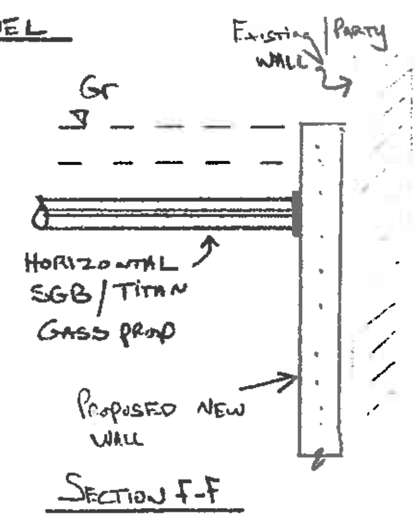
LEGEND

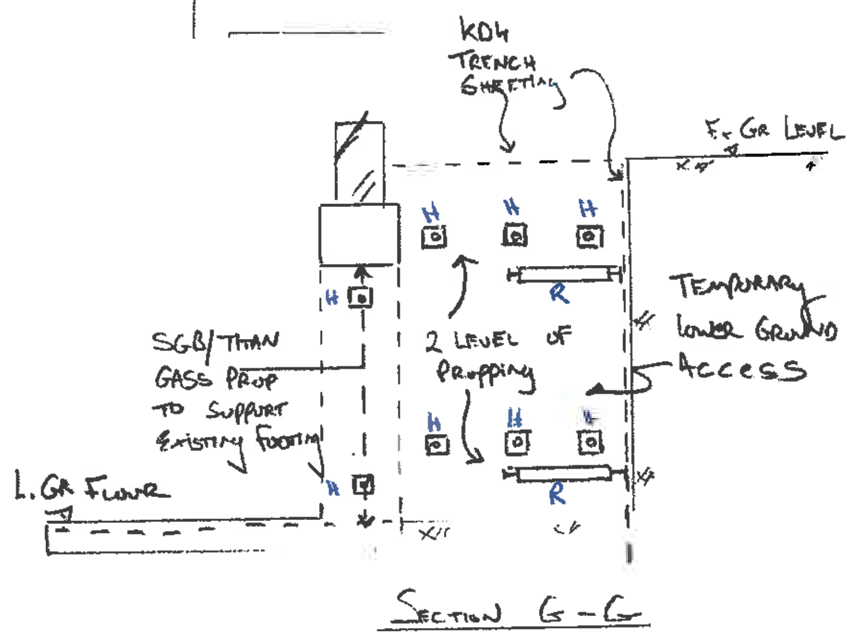
X : SGB/TITAN GASS PROPS

- - - : KD4 TRENCH SHEETS (GROUNDFORCE)

R : PUSH/PULL ACROW

H : SGB/TITAN GASS PROP (HORIZONTAL)





Notes :

1. All drawings are to be read in conjuction with all relevant Specifications, Bills of Quantities, Architectural, Services and Engineering drawings.
2. Any discrepancies between these documents shall be brought to the attention of the Temporary Works Engineer.
3. All dimensions are in millimetres, unless noted otherwise.
4. All levels are in metres related to Ordnance Datum.
5. Drawings are not to be scaled.
6. Existing support structure to be confirmed on site.
7. Permanent works details as per Martin Redston Associates drawings and specifications.
8. Contractor responsible for temporary works currently installed.

Rev.	Date	Description	by	ch'd	app
A	21.07.14	ISSUED FOR APPROVAL	SC	MR	MR

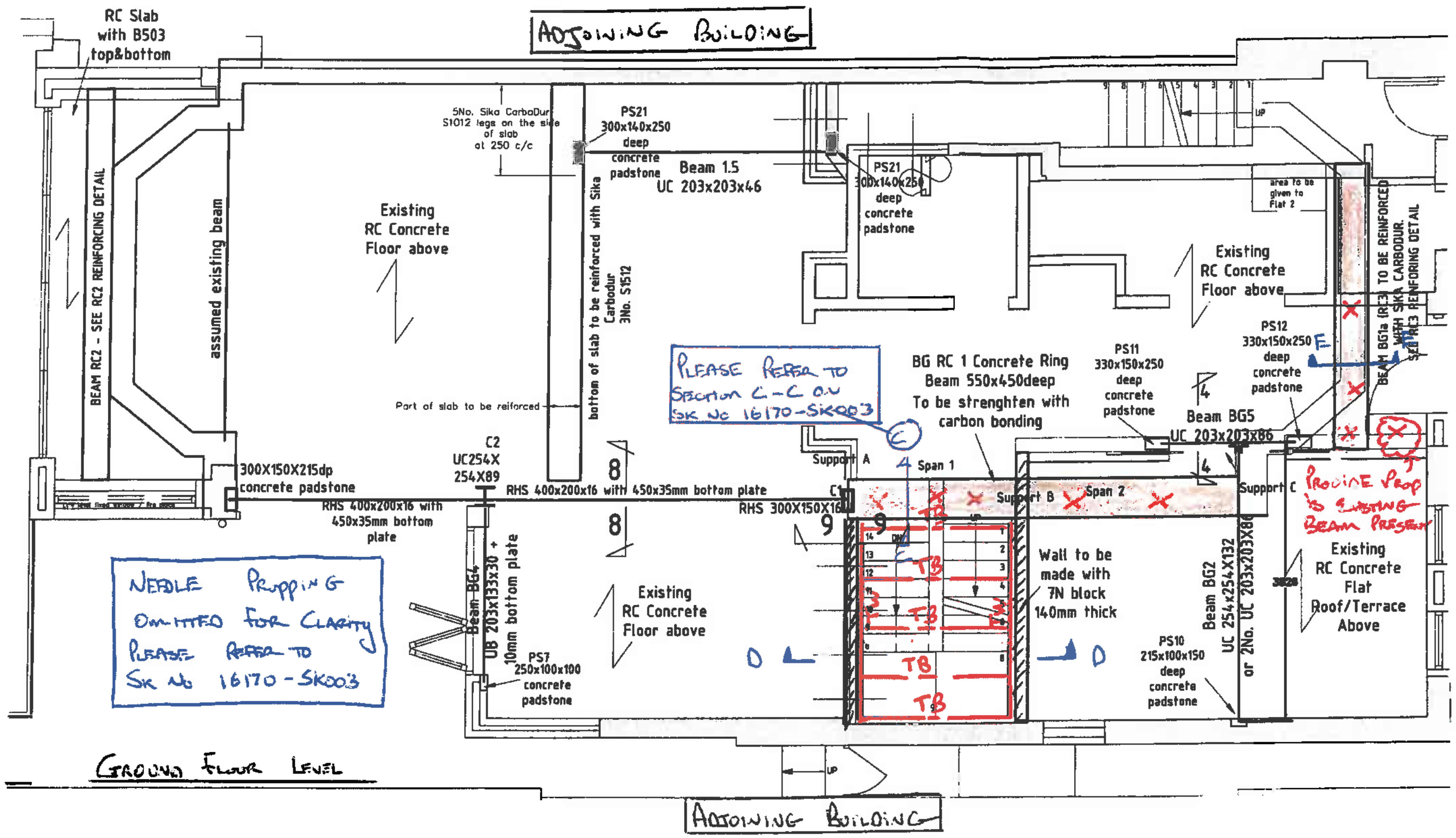
Client: HESTIA DEVELOPMENTS

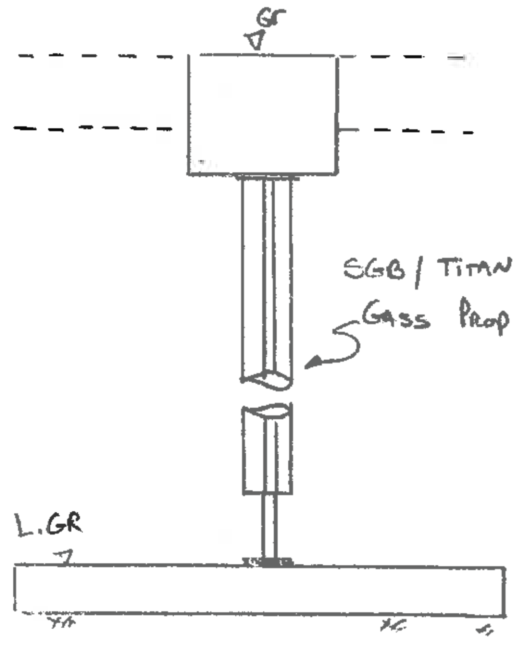
Project: 45 MARESFIELD GARDENS

Title: BASEMENT TEMPORARY WORKS

Malachy Walsh and Partners
Consulting Engineers
London | Cork | Tralee | Limerick

Scales (A3)	AS SHOWN
Drawn	SC
Checked	MR
Drg. No.	16170-SK001
Rev.	A





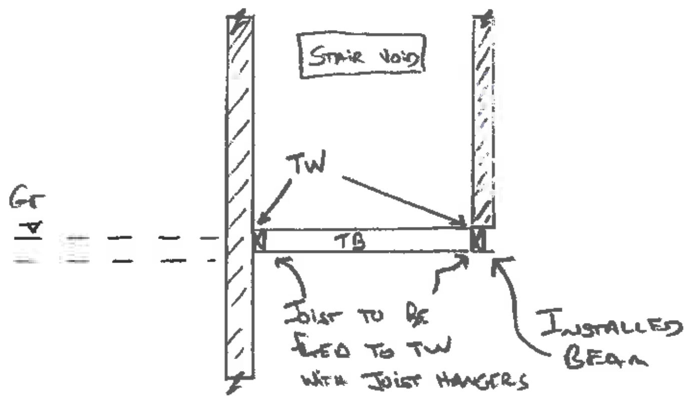
SECTION E–E

LEGEND

TW – TIMBER WALL PLATE 200x75 C24 TIMBER FIXED TO WALL @ 750mm c/c STAGGERED. WITH M10 RESIN ANCHORS

TB – TIMBER BEAMS/JOISTS 200x50 C24 TIMBERS

x – SGB/ TITAN GASS PROPS



SECTION D–D

Notes :

1. All drawings are to be read in conjuction with all relevant Specifications, Bills of Quantities, Architectural, Services and Engineering drawings.
2. Any discrepancies between these documents shall be brought to the attention of the Temporary Works Engineer.
3. All dimensions are in millimetres, unless noted otherwise.
4. All levels are in metres related to Ordnance Datum.
5. Drawings are not to be scaled.
6. Existing support structure to be confirmed on site.
7. Permanent works details as per Martin Redston Associates drawings and specifications.
8. Contractor responsible for temporary works currently installed.

Rev.	Date	Description	by	ch'd	app
A	21.07.14	ISSUED FOR APPROVAL	SC	MR	MR

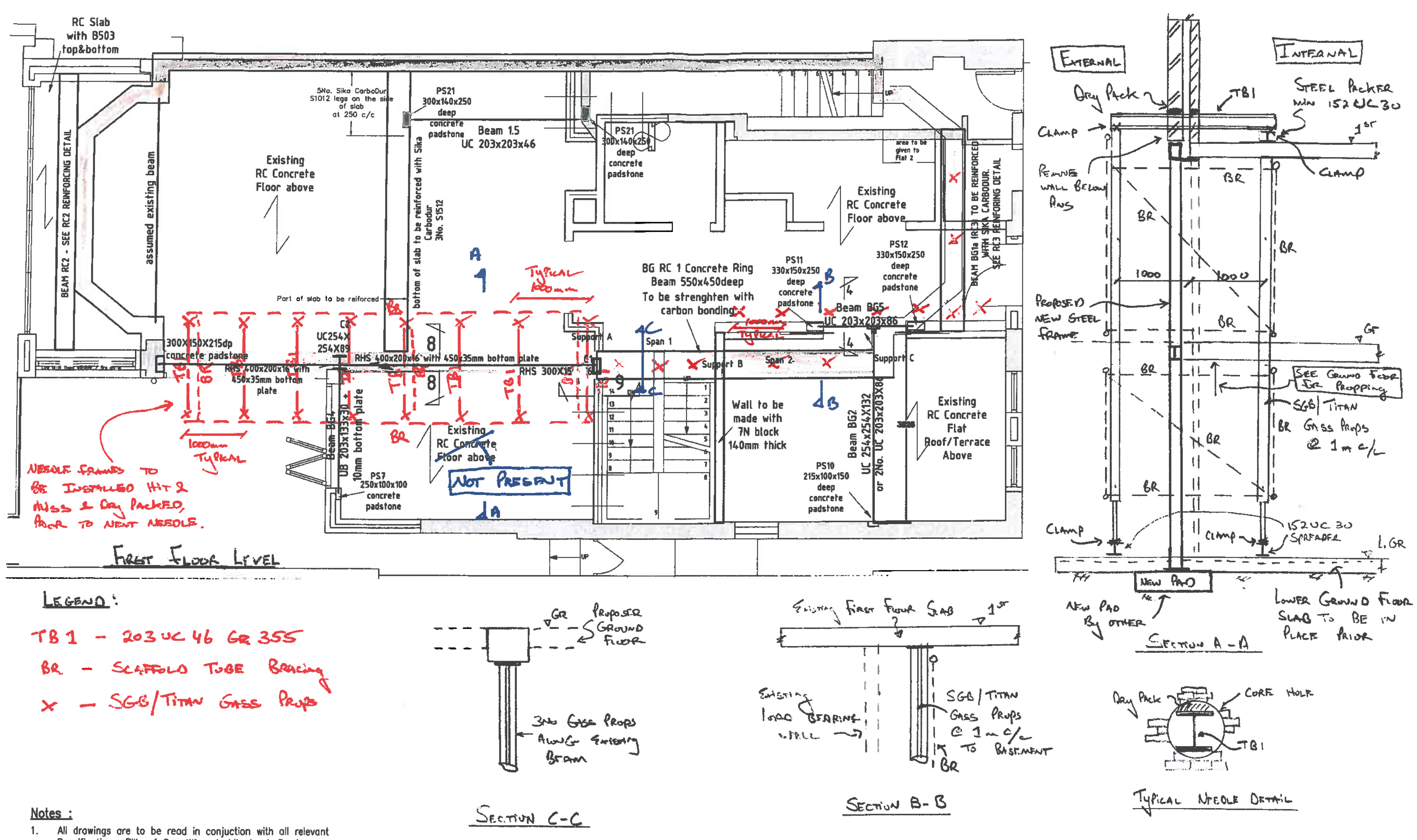
Client: HESTIA DEVELOPMENTS

Project: 45 MARESFIELD GARDENS

Title: GROUND FLOOR TEMPORARY WORKS

Malachy Walsh and Partners
Consulting Engineers
London | Cork | Tralee | Limerick

Scales (A3)	AS SHOWN
Drawn	SC
Checked	MR
Drg. No.	16170-SK002
Rev.	A



Notes :

1. All drawings are to be read in conjuction with all relevant Specifications, Bills of Quantities, Architectural, Services and Engineering drawings.
2. Any discrepancies between these documents shall be brought to the attention of the Temporary Works Engineer.
3. All dimensions are in millimetres, unless noted otherwise.
4. All levels are in metres related to Ordnance Datum.
5. Drawings are not to be scaled.
6. Existing support structure to be confirmed on site.
7. Permanent works details as per Martin Redston Associates drawings and specifications.
8. Contractor responsible for temporary works currently installed.

Rev.	Date	Description	by	ch'd	app
A	22.07.14	ISSUED FOR APPROVAL	SC	MR	MR

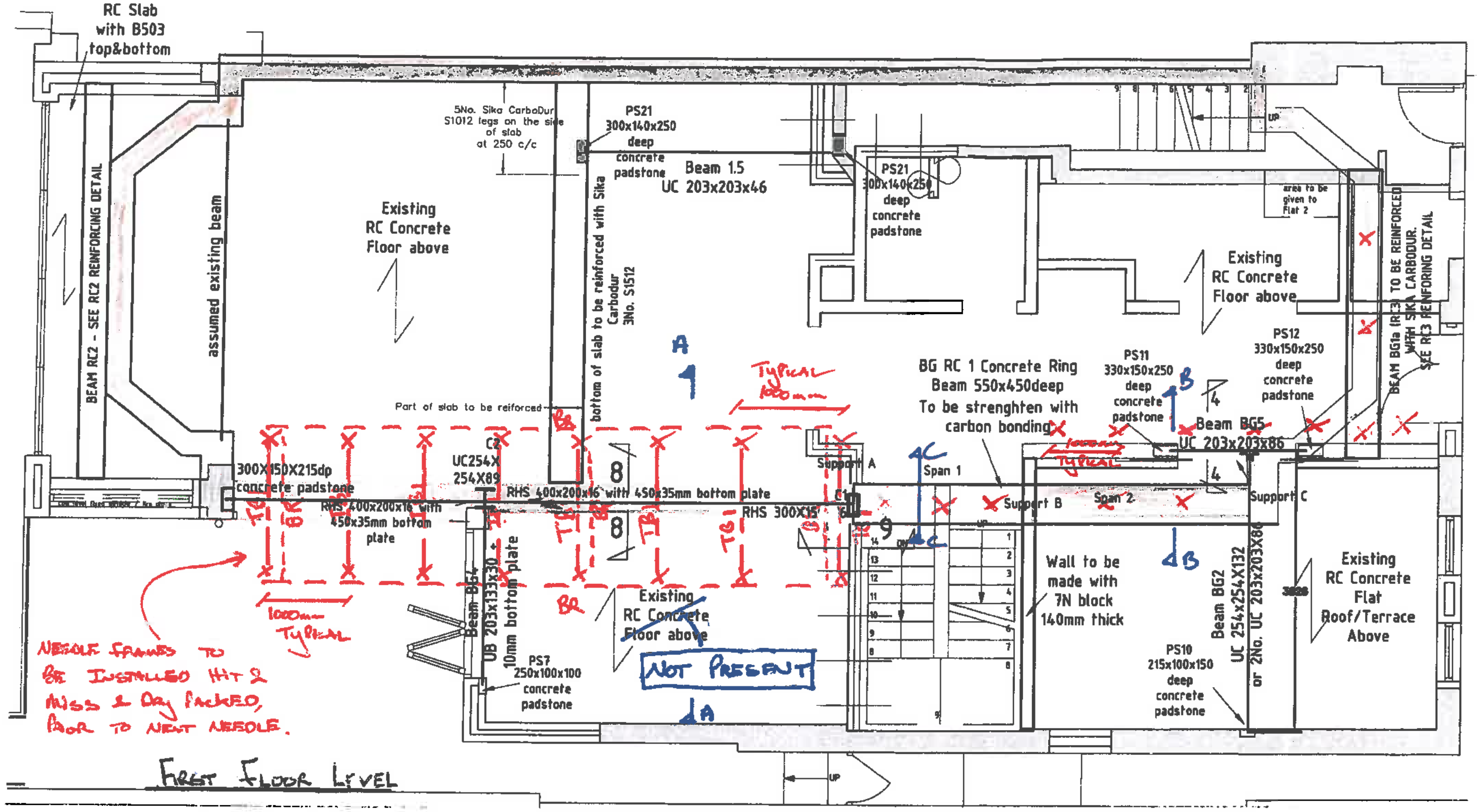
Client: HESTIA DEVELOPMENTS

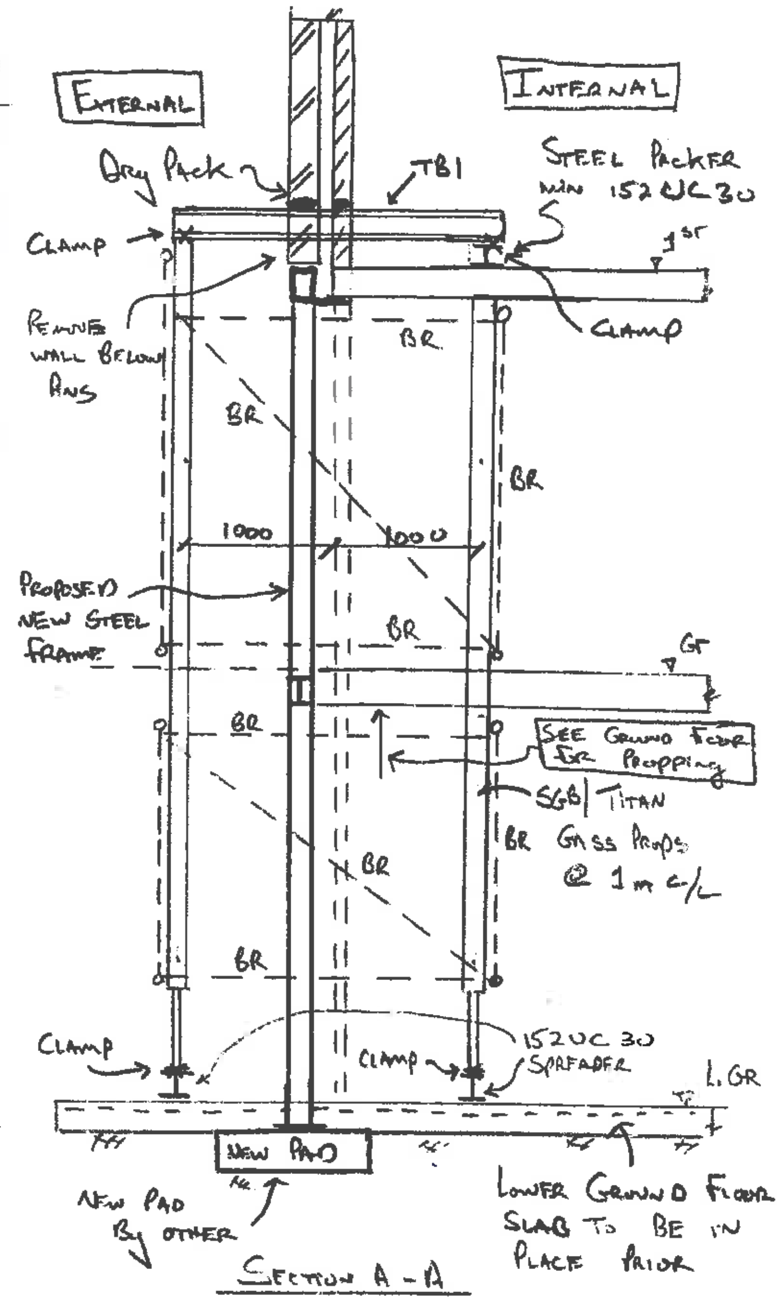
Project: 45 MARESFIELD GARDENS

Title: EXISTING FIRST FLOOR SLAB TEMPORARY PROPPING

Malachy Walsh and Partners
Consulting Engineers
London | Cork | Tralee | Limerick

Scales (A3)	AS SHOWN
Drawn	SC
Checked	MR
Drg. No.	16170-SK003
Rev.	A



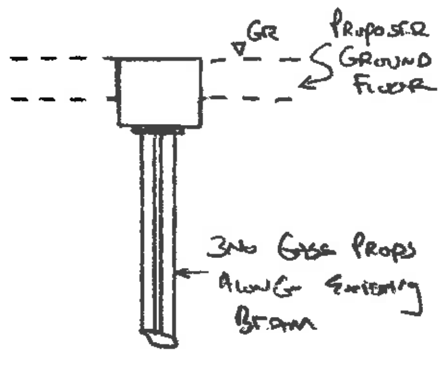


LEGEND:

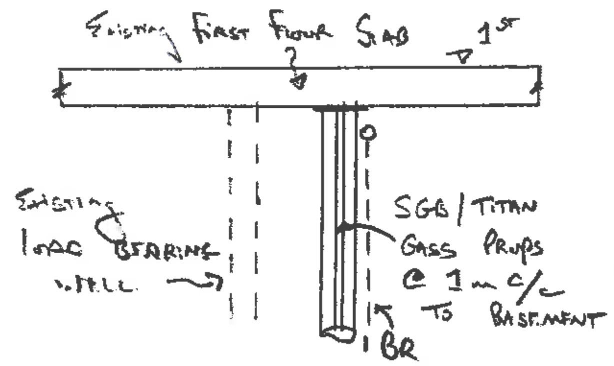
TB1 - 203UC46 GR355

BR - SCAFFOLD TUBE BRACING

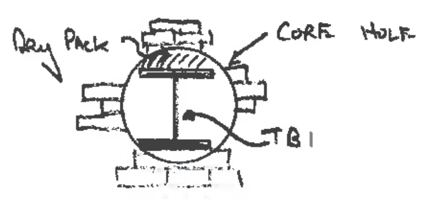
X - SGB/TITAN GASS PROPS



SECTION C-C



SECTION B-B



TYPICAL NEEDLE DETAIL

Notes :

1. All drawings are to be read in conjuction with all relevant Specifications, Bills of Quantities, Architectural, Services and Engineering drawings.
2. Any discrepancies between these documents shall be brought to the attention of the Temporary Works Engineer.
3. All dimensions are in millimetres, unless noted otherwise.
4. All levels are in metres related to Ordnance Datum.
5. Drawings are not to be scaled.
6. Existing support structure to be confirmed on site.
7. Permanent works details as per Martin Redston Associates drawings and specifications.
8. Contractor responsible for temporary works currently installed.

Rev.	Date	Description	by	ch'd	app
A	22.07.14	ISSUED FOR APPROVAL	SC	MR	MR

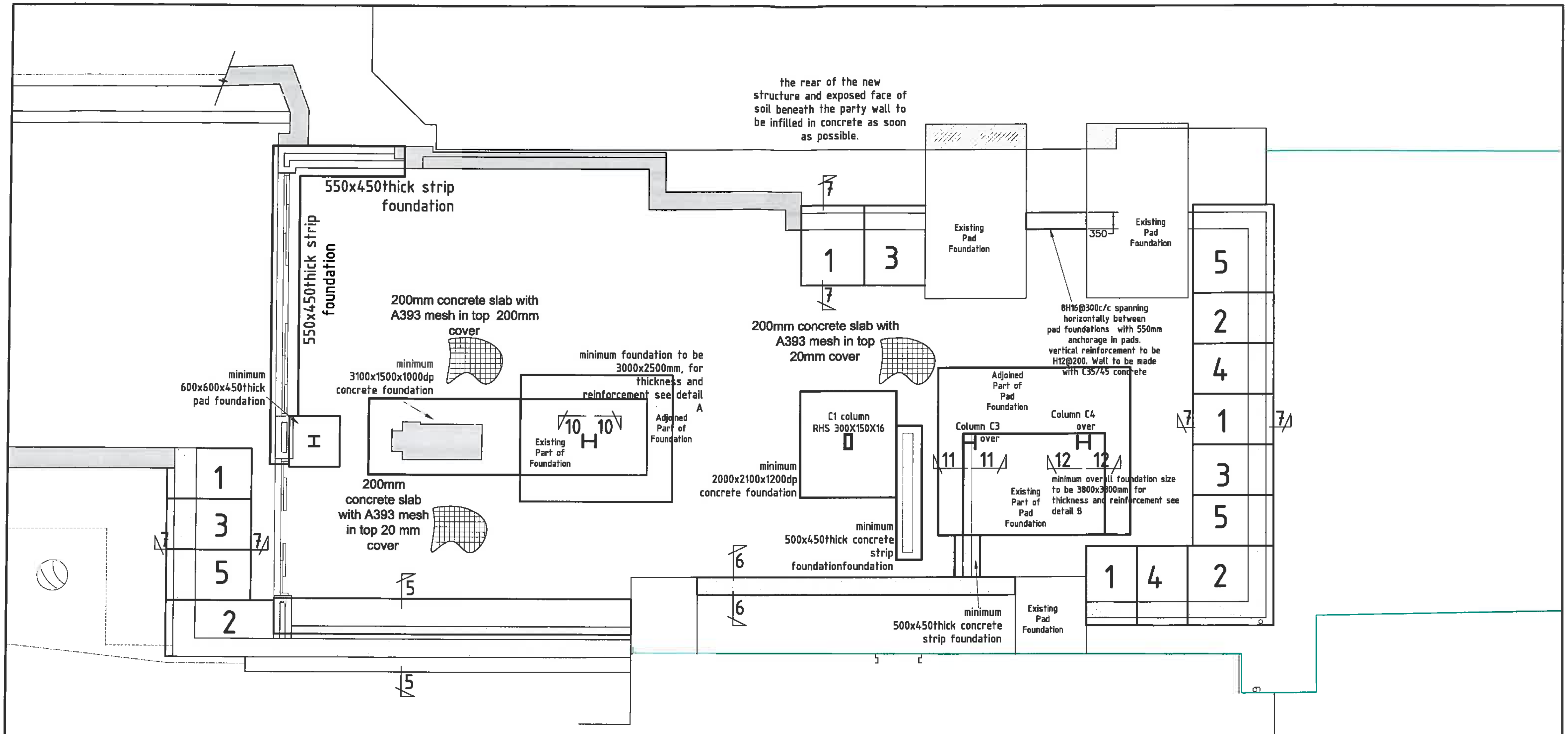
Client: HESTIA DEVELOPMENTS

Project: 45 MARESFIELD GARDENS

Title: EXISTING FIRST FLOOR SLAB TEMPORARY PROPPING

Malachy Walsh and Partners
Consulting Engineers
London | Cork | Tralee | Limerick

Scales (A3)	AS SHOWN
Drawn	SC
Checked	MR
Drg. No.	16170-SK003
Rev.	A



PROPOSED BASEMENT LAYOUT

SEQUENCE OF UNDERPINNING

1. WORKING IN STRIPS NOT EXCEEDING 1.2m LONG EXCAVATE TO REQUIRED DEPTH BENEATH EXISTING FOOTING.
2. CAREFULLY CUT AWAY TO PROVIDE LEVEL SOFFIT TO EXISTING BRICK FOOTING AND THOROUGHLY CLEAN BEFORE UNDERPINNING.
3. CAST NEW CONCRETE TO WITHIN 50mm OF SOFFIT OF EXISTING FOOTING AND ALLOW 24 HOURS TO CURE.
4. RAM IN DRY PACK MORTAR BETWEEN NEW AND EXISTING FOOTINGS.
5. NEVER EXCAVATE TWO ADJACENT STRIPS WITHOUT ALLOWING 3 DAYS FROM TIME OF DRY PACKING.
6. WHEN ADJACENT SECTIONS ARE OPENED UP THE EXPOSED CONCRETE SURFACES SHOULD BE THOROUGHLY CLEANED OF ALL LOOSE MATERIAL AND SCABBLED TO FORM A GOOD KEY.
7. UNDERPINNING WIDTH TO BE AS NOTED ON THE DRAWINGS
8. ALL NEW CONCRETE BELOW GROUND TO BE SULPHATE RESISTING CEMENT CONC. GRADE C 40.
9. DRY PACK - 1:3 CEMENT/SAND
10. ALL UNDERPINS ARE TO BE DOWELLED TOGETHER WITH H20 BARS 800mm LONG O/A AT 500mm CENTRES VERTICALLY. ALTERNATIVELY PROVIDE FULL WIDTH TOOTHED JOINTS ONE THIRD HIGH AND 250mm DEEP AT MID HEIGHT OF ALL UNDERPINS.

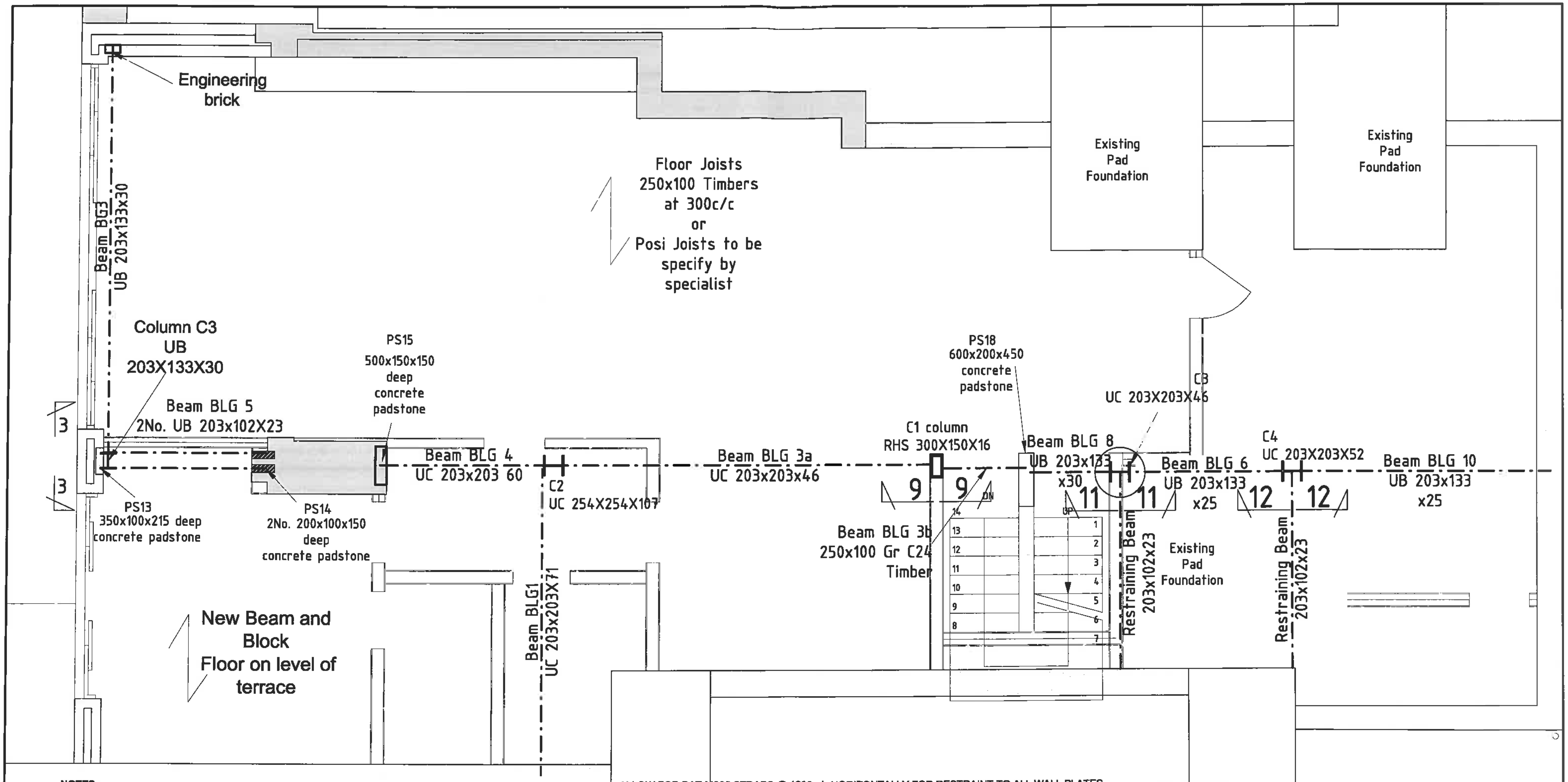
Martin Redston Associates

Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 1F
Eng.: RS	Scale 1/75@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed Underpinning / Fopundation	



NOTES

1. ALL DIMENSIONS TO BE VERIFIED ON SITE.
2. READ IN CONJUNCTION WITH ARCHITECT'S DRAWINGS.
3. ALL STEELWORK DESIGNED AND FABRICATED TO EN1993-1.
4. ALL STEEL MEMBERS TO BE GRADE S275 STEEL.
5. APPLY 2 COATS ZINC RICH PRIMER TO ALL STEEL PRIOR TO ERECTION.
6. ALL FIRE PROTECTION TO ARCHITECT'S SPECIFICATION.
7. ALL EXTERNAL STEEL (INCLUDING STEEL WITHIN CAVITY) AND STEEL BELOW GROUND LEVEL TO BE GALVANISED OR HAVE 2 COATS OF BITUMASTIC PAINT (RIW OR SIMILAR).
8. ALL WELDING TO BE 6mm FILLET WELDS CARRIED OUT IN WORKSHOP.
9. ALL BOLTS TO BE GALVANISED GRADE 8.8.
10. ALL TIMBERWORK DESIGNED TO EN1995-1 OR BS 5268.
11. CONNECTIONS:

TIMBER/BRICK: BAT STRAP HANGER WHEN THERE IS A MINIMUM OF 675mm OF BRICKWORK ABOVE, IF NOT USE MAXI SPEEDY HANGERS.

TIMBER/TIMBER: BAT MAXI SPEEDY HANGER OR BAT SPEEDY JOIST HANGER.

~~ALLOW FOR BAT M305 STRAPS @ 1200 c/c HORIZONTALLY FOR RESTRAINT~~ TO ALL WALL PLATES.

12. CONCRETE PADSTONES (PS) TO BE GRADE C25 (1:2:4).
13. FOUNDATION CONCRETE TO BE GRADE C35/45.
14. FOUNDATIONS TO BE 600mm BELOW ANY ROOT ACTIVITY (Min 1100mm DEPTH).
15. ALL TEMPORARY PROPPING BY THE CONTRACTOR.
16. NEW BRICKWORK TO BE 35N/mmsq, NEW BLOCKWORK TO BE 4N/mmsq SET IN 1:1: 6 MORTAR. U.N.O.
17. ALL WATERPROOFING AND DRAINAGE TO ARCHITECT'S SPECIFICATION.
18. ALL WORKS TO BE APPROVED BY THE BUILDING CONTROL OFFICER.
19. NO WORK TO COMMENCE ON SITE PRIOR TO BUILDING CONTROL APPROVAL OF STRUCTURAL DETAILS.
20. ANY EXCAVATIONS WORKS WITHIN 3m OF ANY ADJOINING PROPERTY OR PARTY STRUCTURE MAY BE SUBJECT TO PARTY WALL AGREEMENT.
21. ANY BEAMS, JOISTS HANGERS, OR OTHER STRUCTURAL WORKS ATTACHED TO PARTY WALL MAY BE SUBJECT TO PARTY WALL AGREEMENT.
22. ALL NEW TO EXISTING MASONRY TO HAVE FURFIX PROFILES.

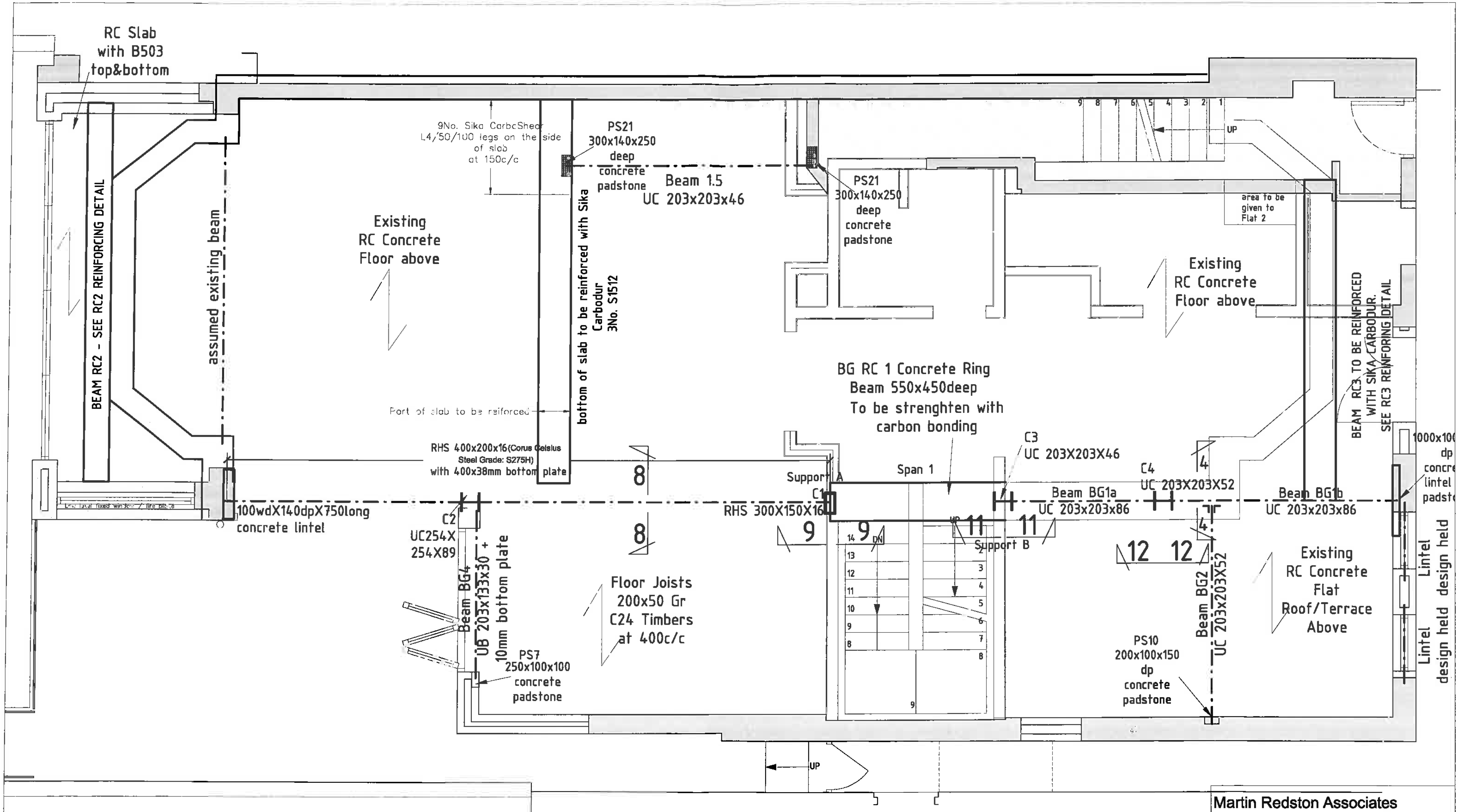
PROPOSED BASEMENT LAYOUT

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 2E
Eng.: RS	Scale 1/50@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed Lower Ground Fl. Level Showing Structure Over	



PROPOSED GROUND FLOOR LAYOUT

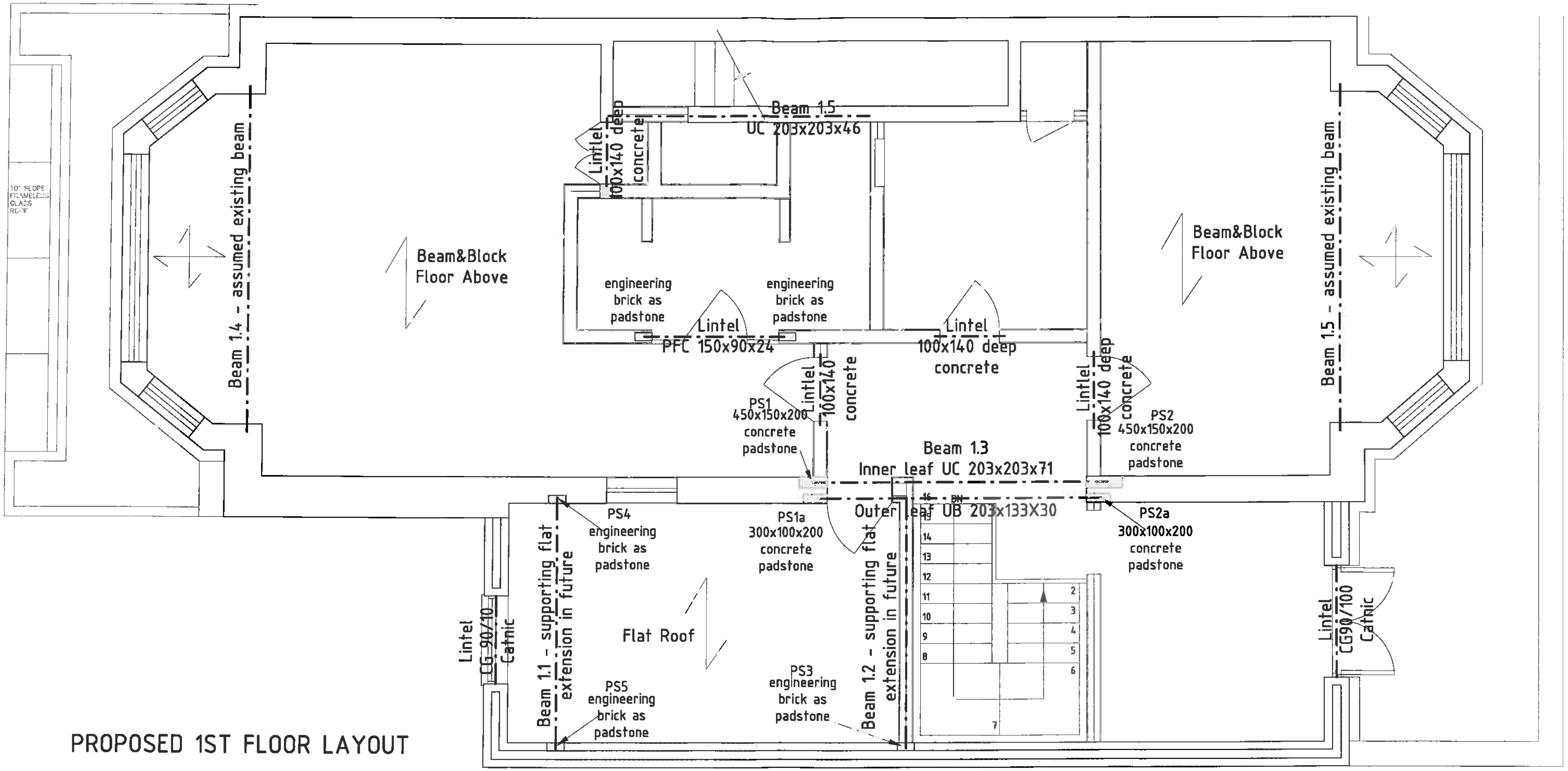
Martin Redston Associates

Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 3E
Eng.: RS	Scale 1/50@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed Ground Fl. Level Showing Structure Over	



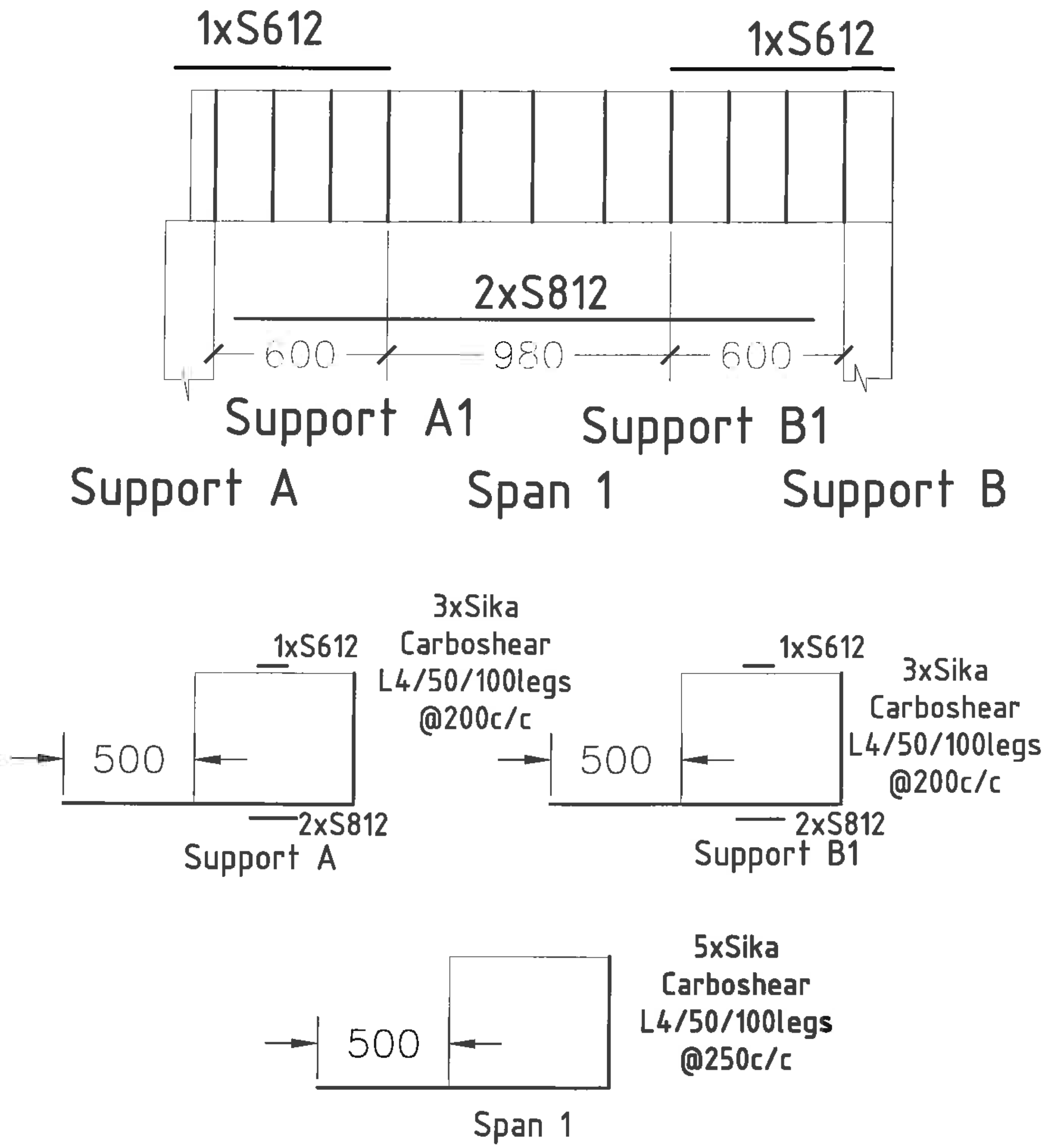
PROPOSED 1ST FLOOR LAYOUT

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 4
Eng.: RS	Scale 1/50@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed First Fl. Level Showing Structure Over	



All bottom and top reinforcing to be Sika Carbodur S Plates according to catalogue :

Identification no:
02 04 01 01 System
Sika® CarboDur® Plates

All links to be L-shaped Sika Carboshear L4/50 /100 (2mm thick)

Identification no:
02 04 01 01 004 0 000002
Sika® CarboShear® L

Martin Redston Associates
Consulting Civil & Structural Engineers

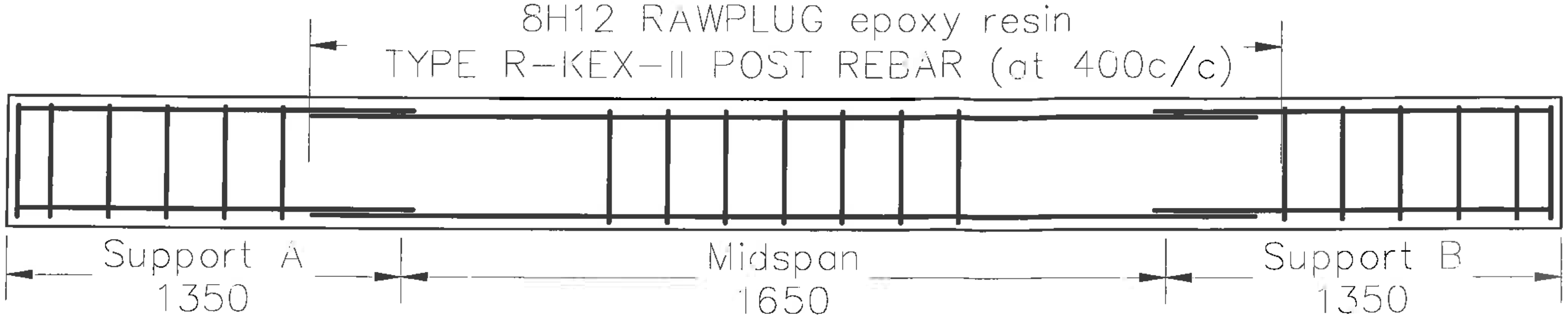
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 5A
Eng.: RS	Scale 1/50@A3
Job No.: 14.073	

Project: 45 Maresfield Gardens

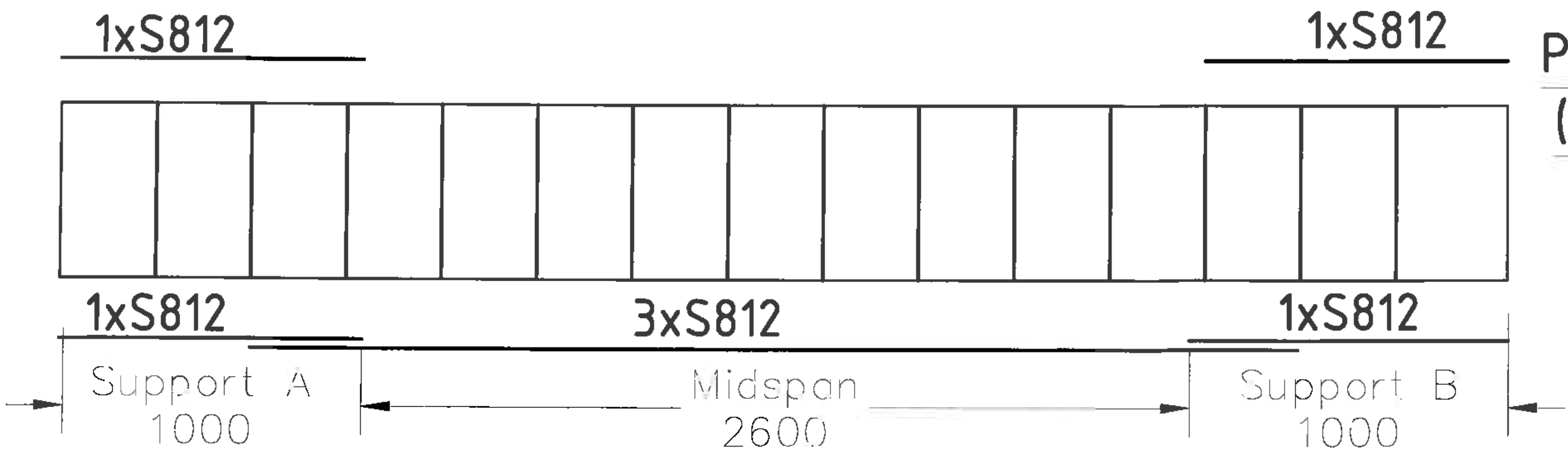
Drawing Title: Reinforcing of existing Beam BG RC1

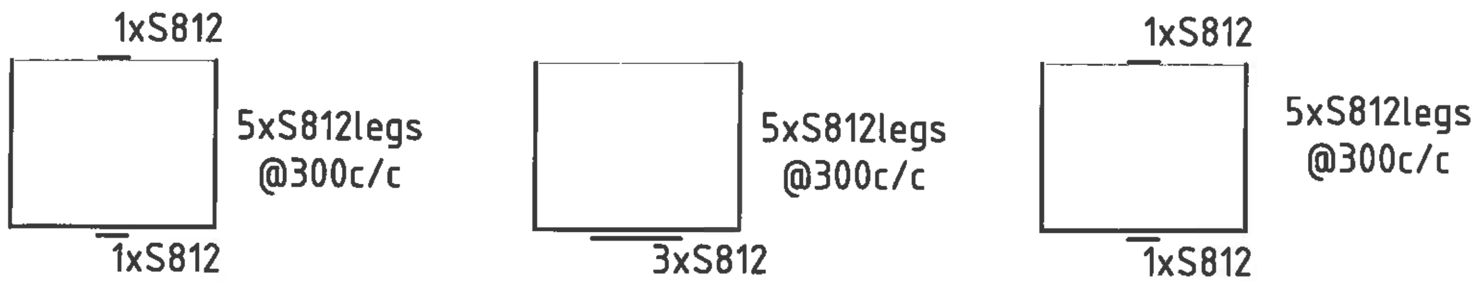


BEAM RC2 REINFORCING DETAIL

FIX 8H12 BARS 300mm long (150mm anchorage to each member) with RAWLPLUG EPOXY RESIN TYPE R-KEX II @400c/c HORIZONTALLY

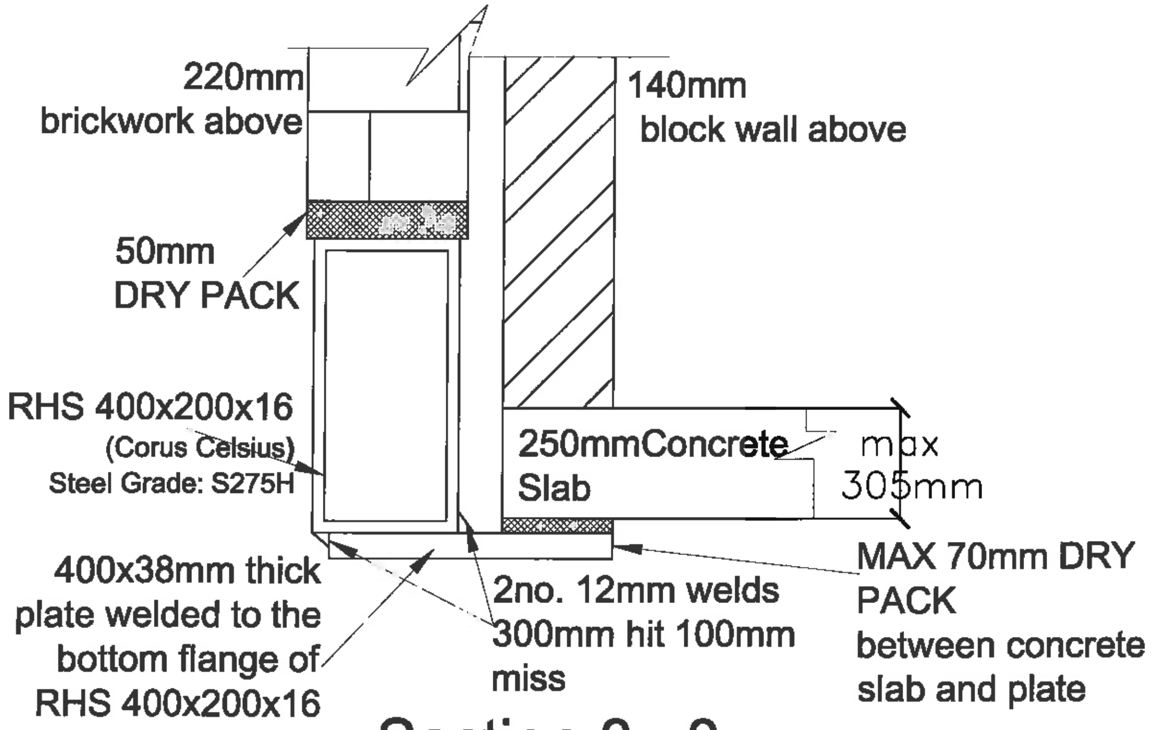
USE STRAIGHT SIKA CARBODUR PLATES FOR SHEARING REINFORCEMENT (INSTEAD OF L SHAPED LEGS) - BEAM IN THE LEVEL OF SLAB



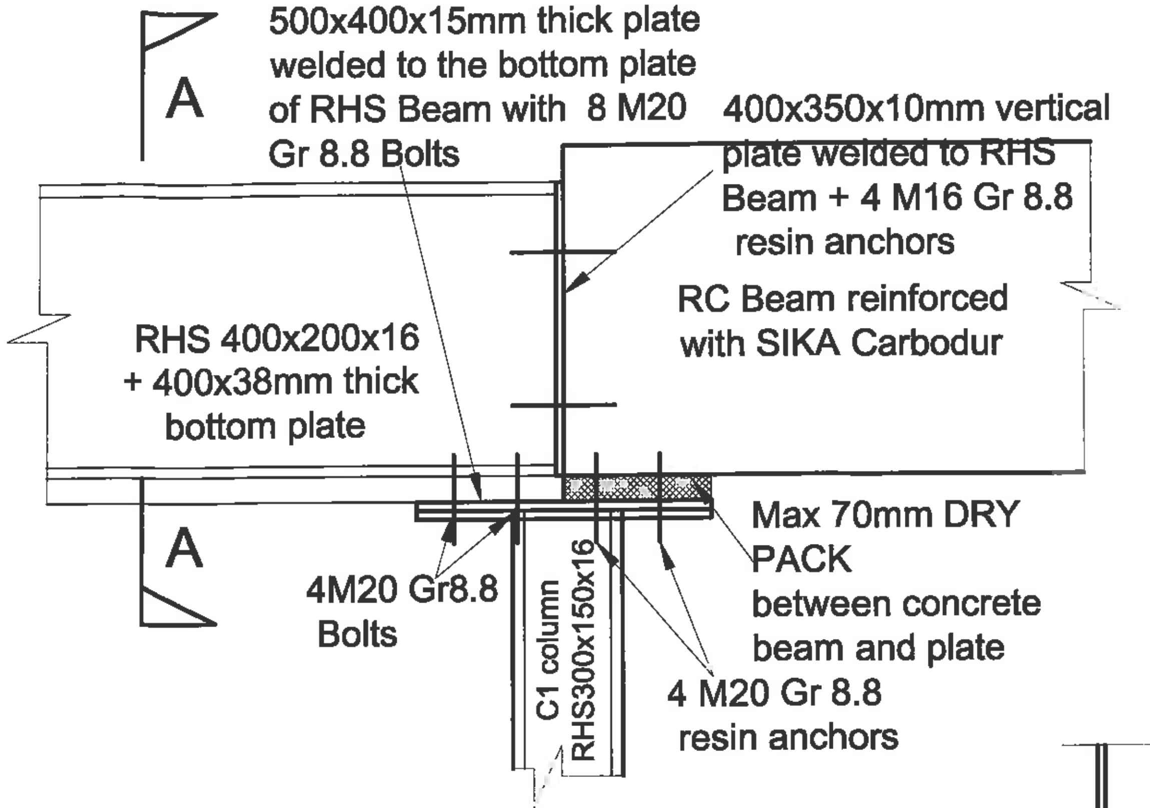


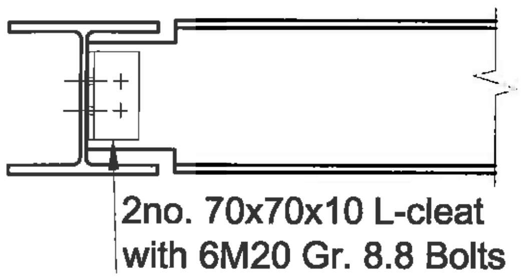
Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 6B
Eng.: RS	Scale 1/50@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Beam RC2 and RC3 reinforcing detail	

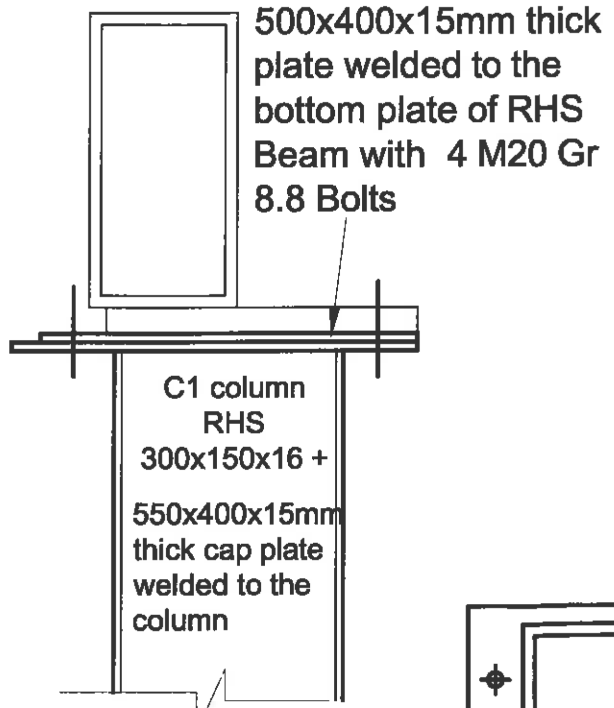


Section 8 - 8
Wall support detail 1/10 @A3

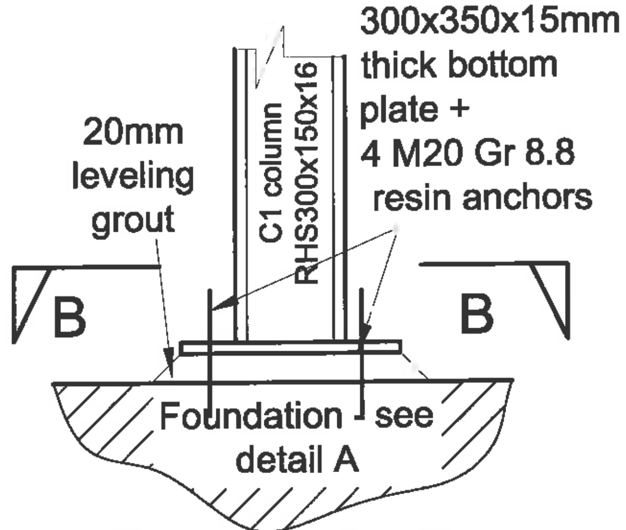




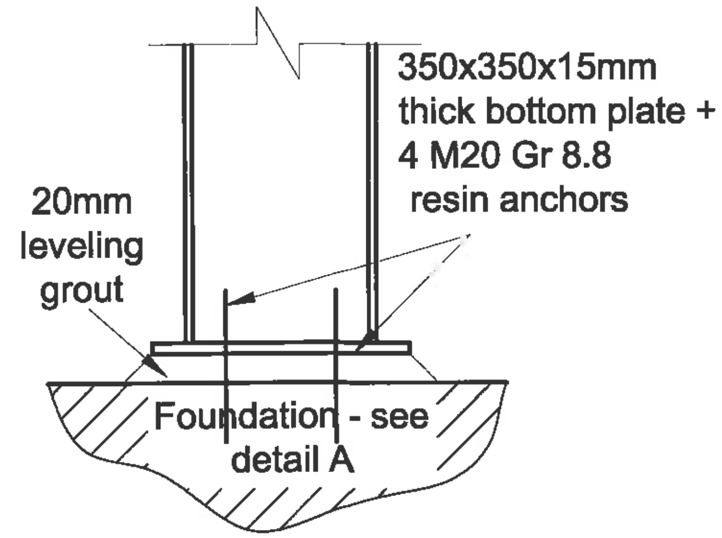
Typical Connection
detail 1/10 @A3



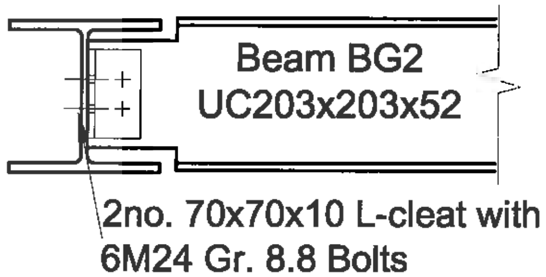
Section A - A
Wall support detail
1/10 @A3



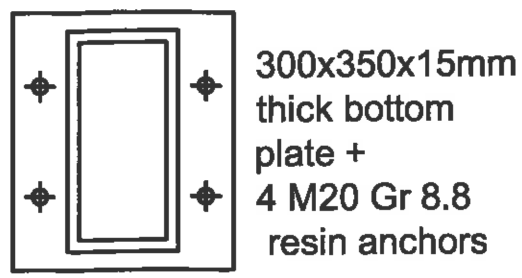
Section 9 - 9
Wall support detail
1/10 @A3



Section 10 - 10
Column C2 connection
to foundation detail
1/10 @A3



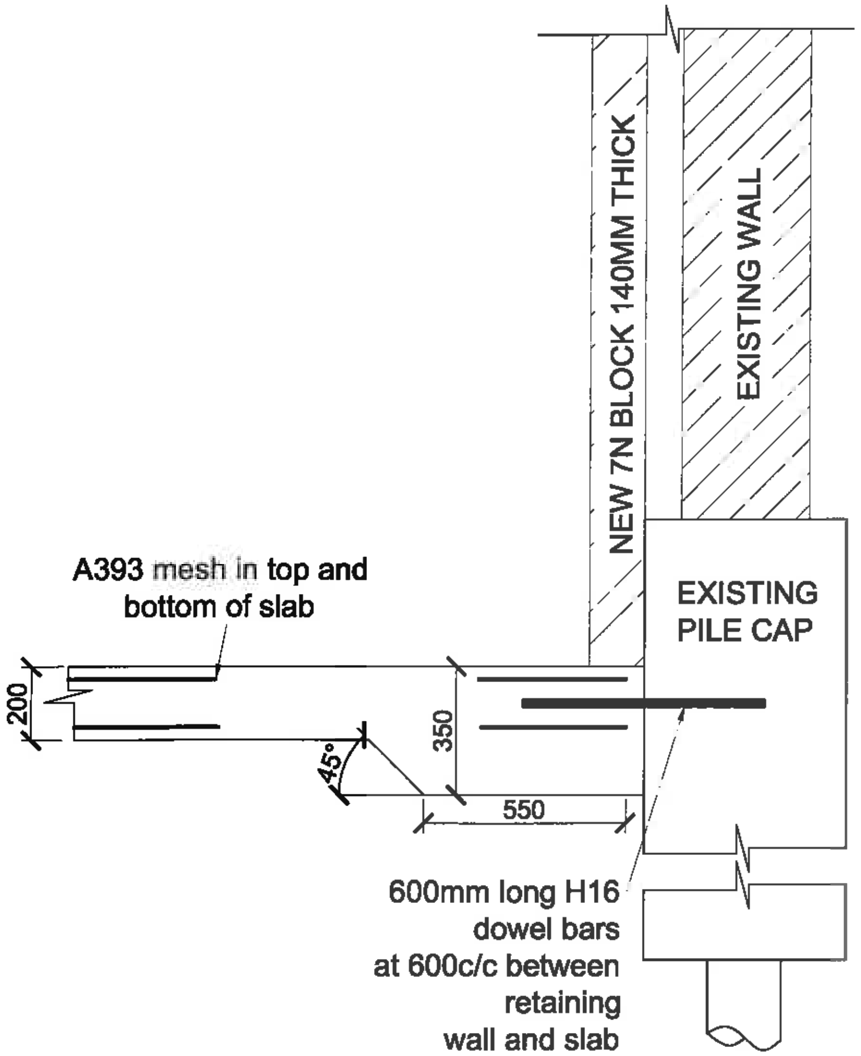
Section 4-4
1/10 @A3



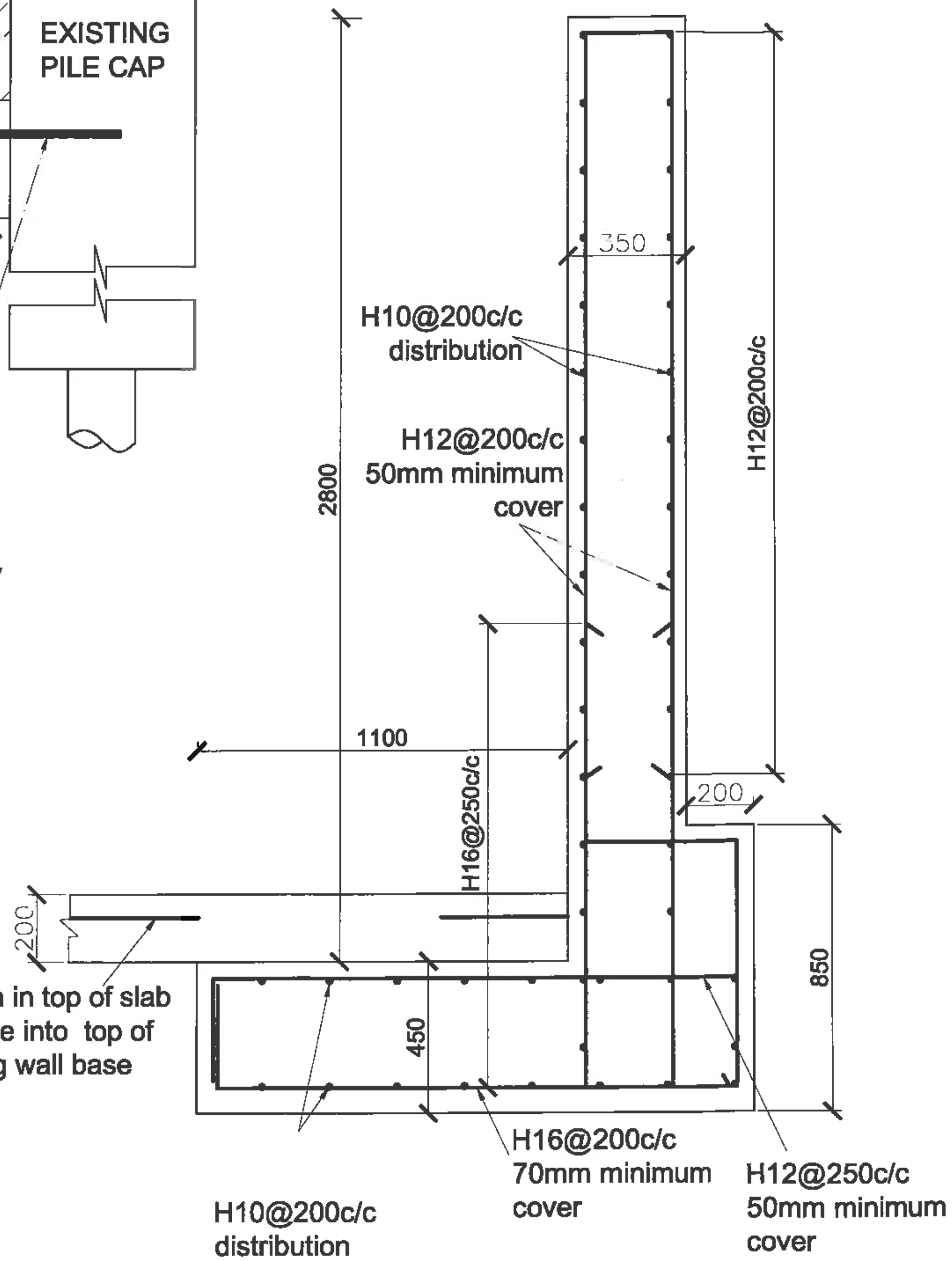
Section B-B

Martin Redston Associates
Consulting Civil & Structural Engineers
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211
6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

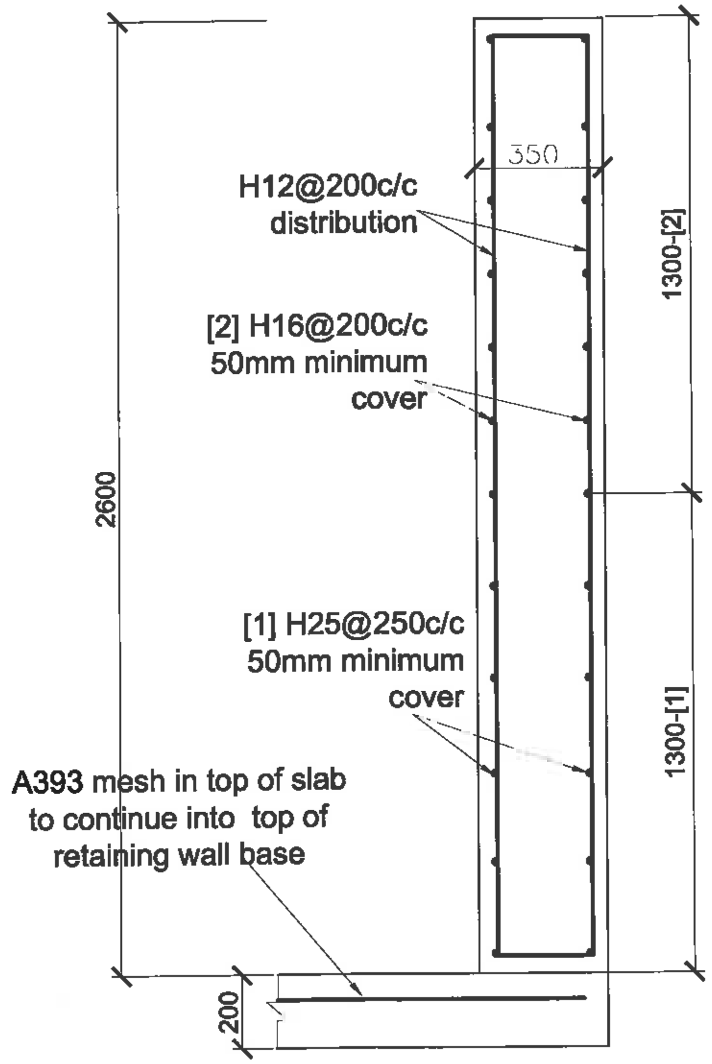
Date: 09/03/2014	Sheet No. 7A
Eng.: RS	Scale 1/10@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed Sections 1	



SECTION 5-5
New wall by party wall detail



SECTION 7-7
Retaining Wall Detail



SECTION 6-6
Wall between pad foundations

Martin Redston Associates
Consulting Civil & Structural Engineers

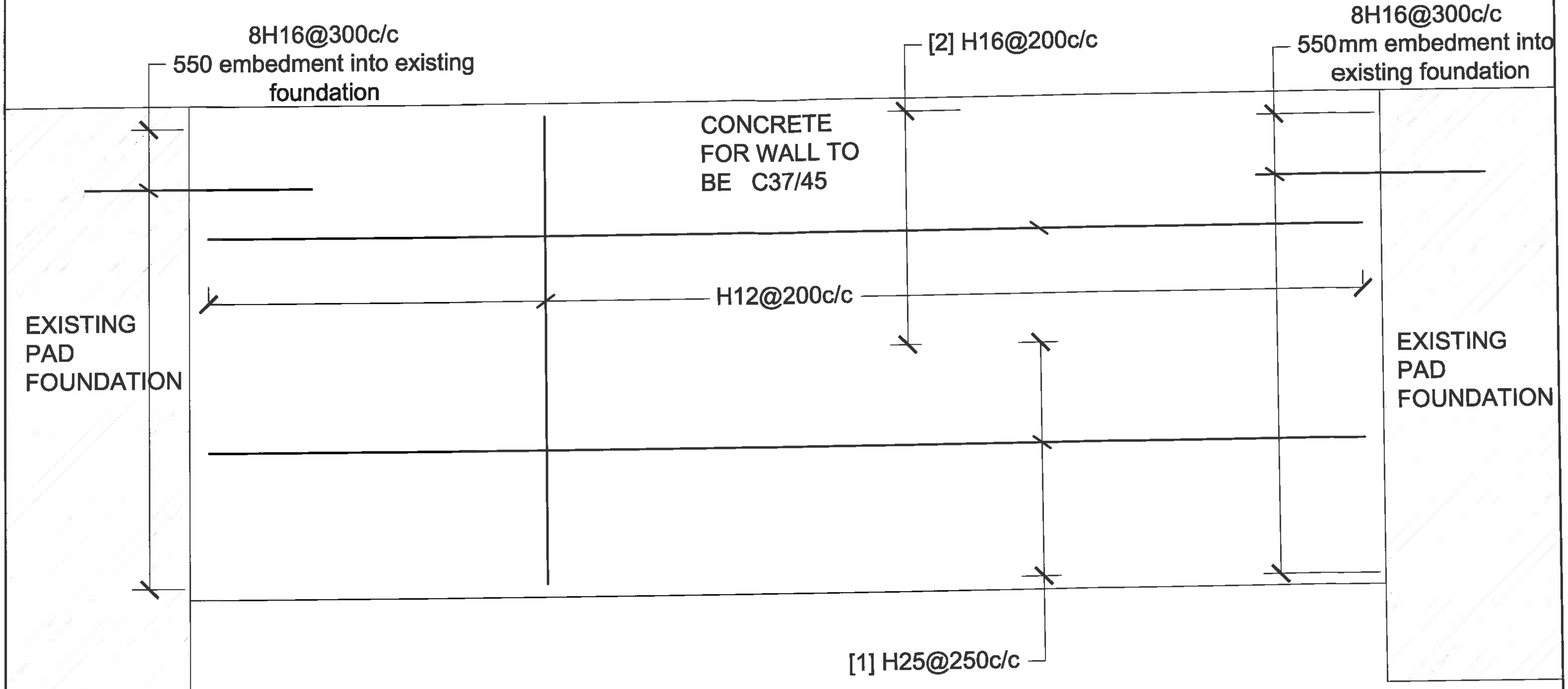
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/03/2014	Sheet No. 8
Eng.: RS	Scale 1/10@A3
Job No.: 14.073	

Project: 45 Maresfield Gardens

Drawing Title: Proposed Sections 2



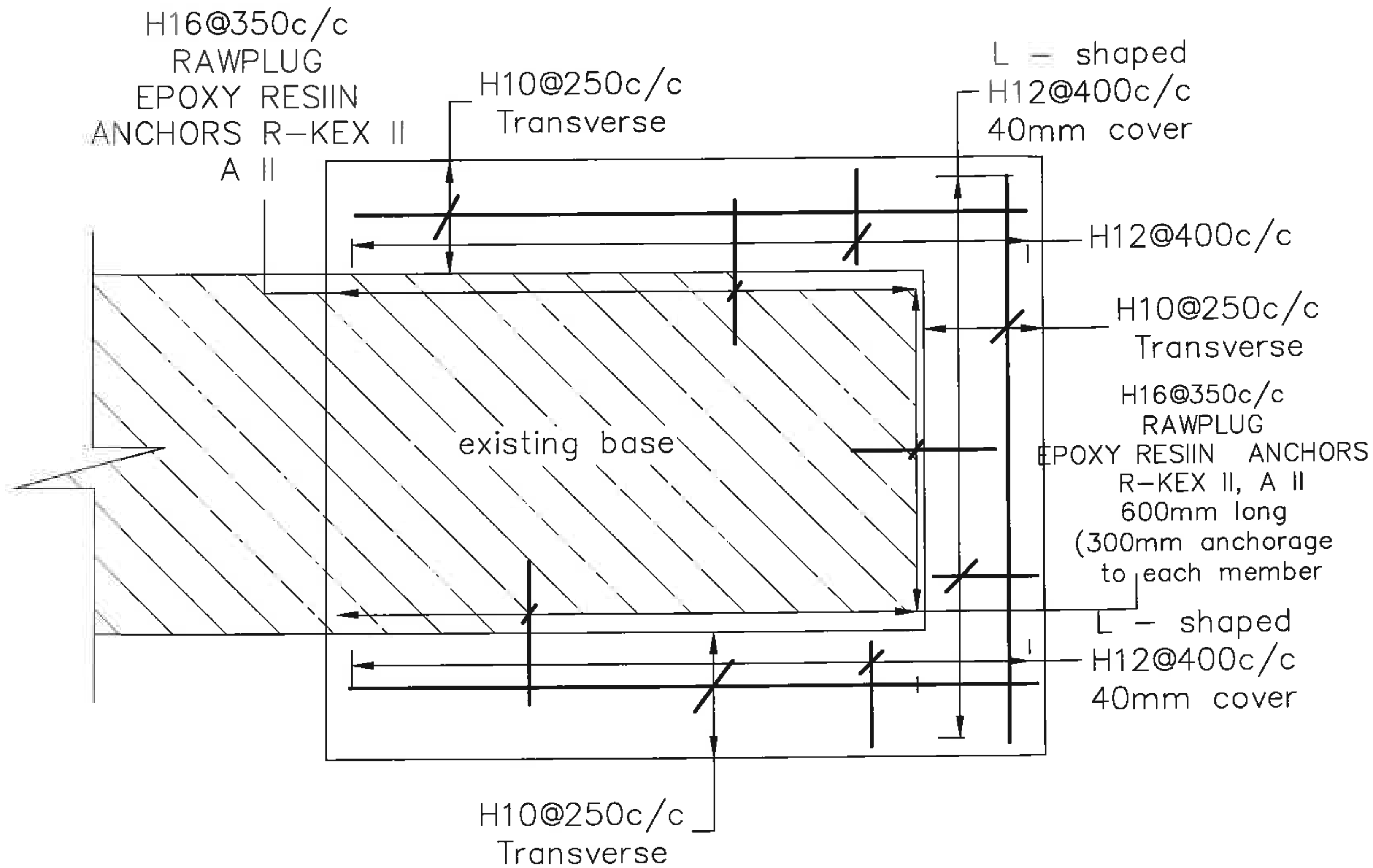
Section 6 - 6

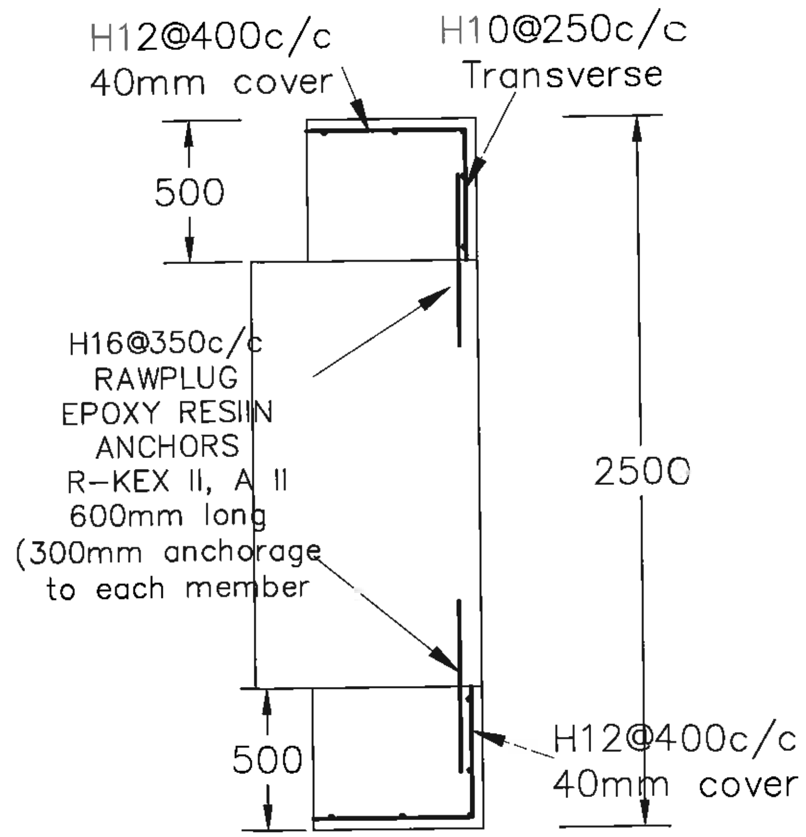
Martin Redston Associates
Consulting Civil & Structural Engineers

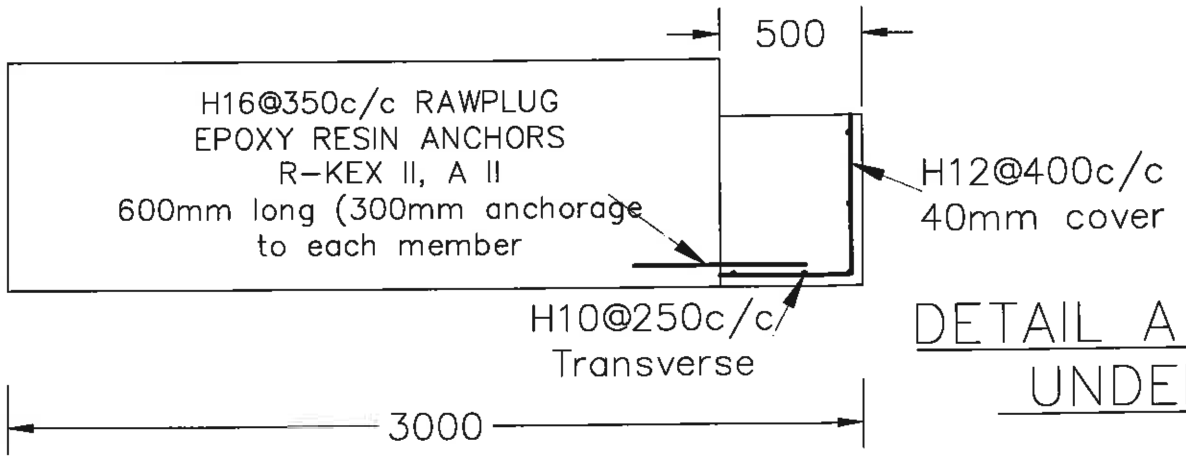
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 09/04/2014	Sheet No. 10
Eng.: RS	Scale 1/20@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Proposed Sections 3	







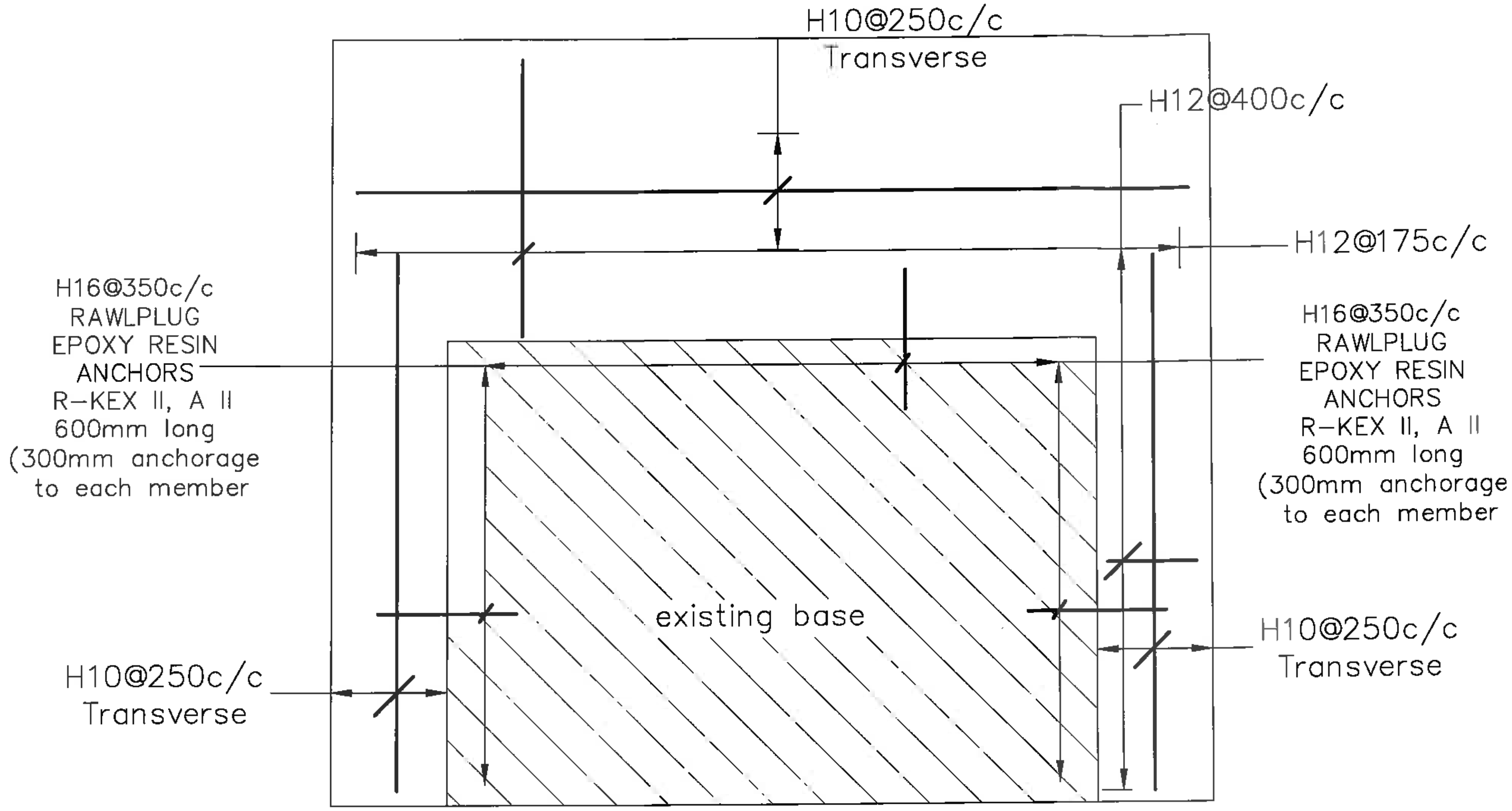
DETAIL A OF FOUNDATION
UNDER COLUMN C2

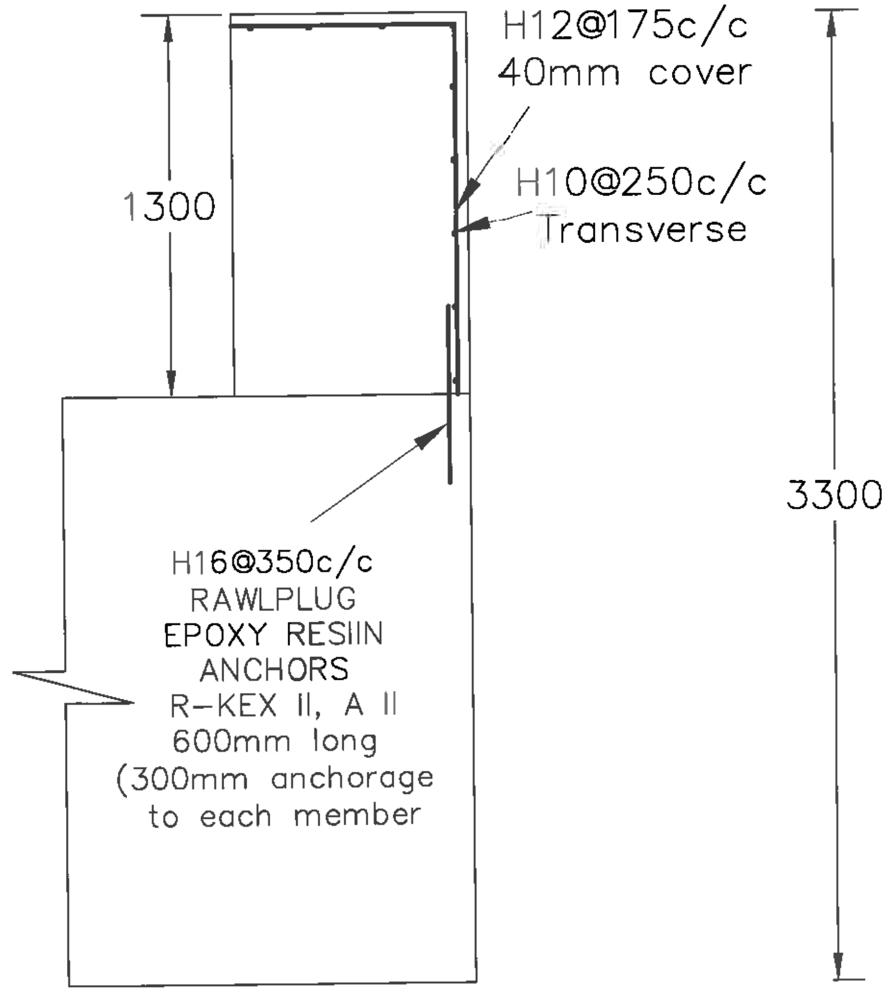
Martin Redston Associates
Consulting Civil & Structural Engineers

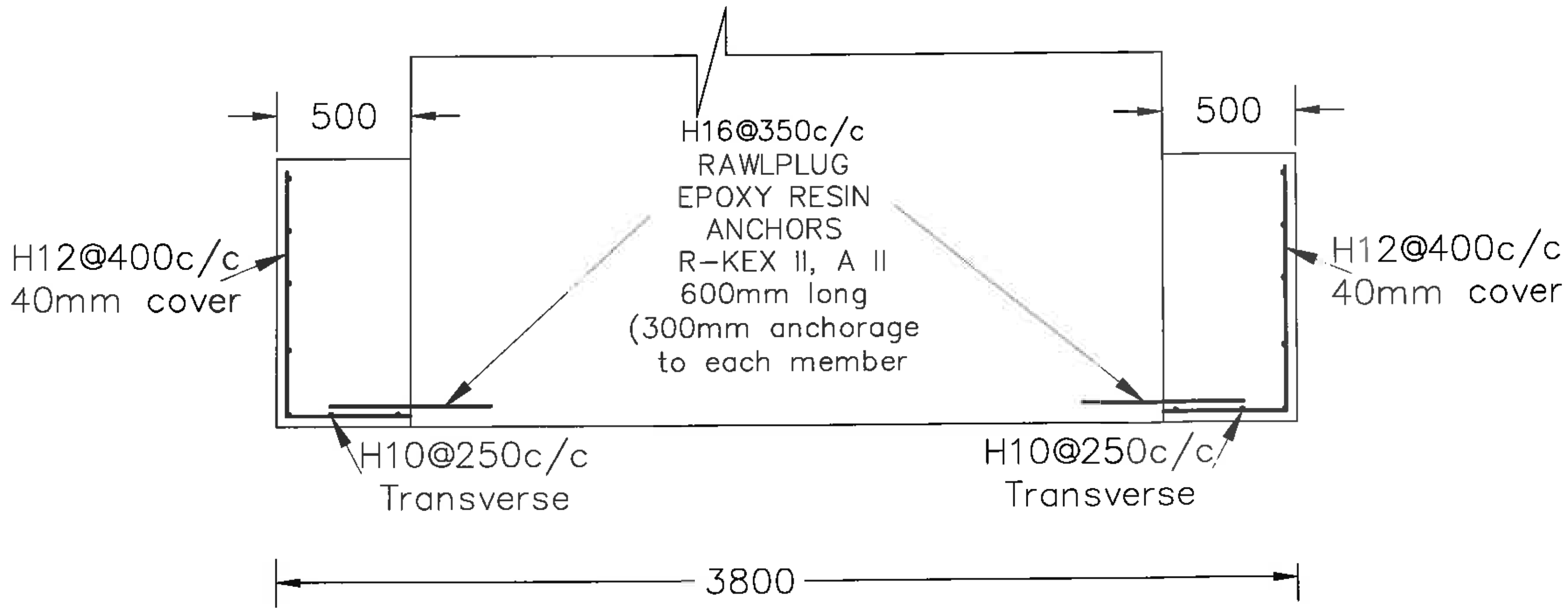
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 04/06/2014	Sheet No. 11b
Eng.: RS	Scale 1/25@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Section 4 - Detail A	







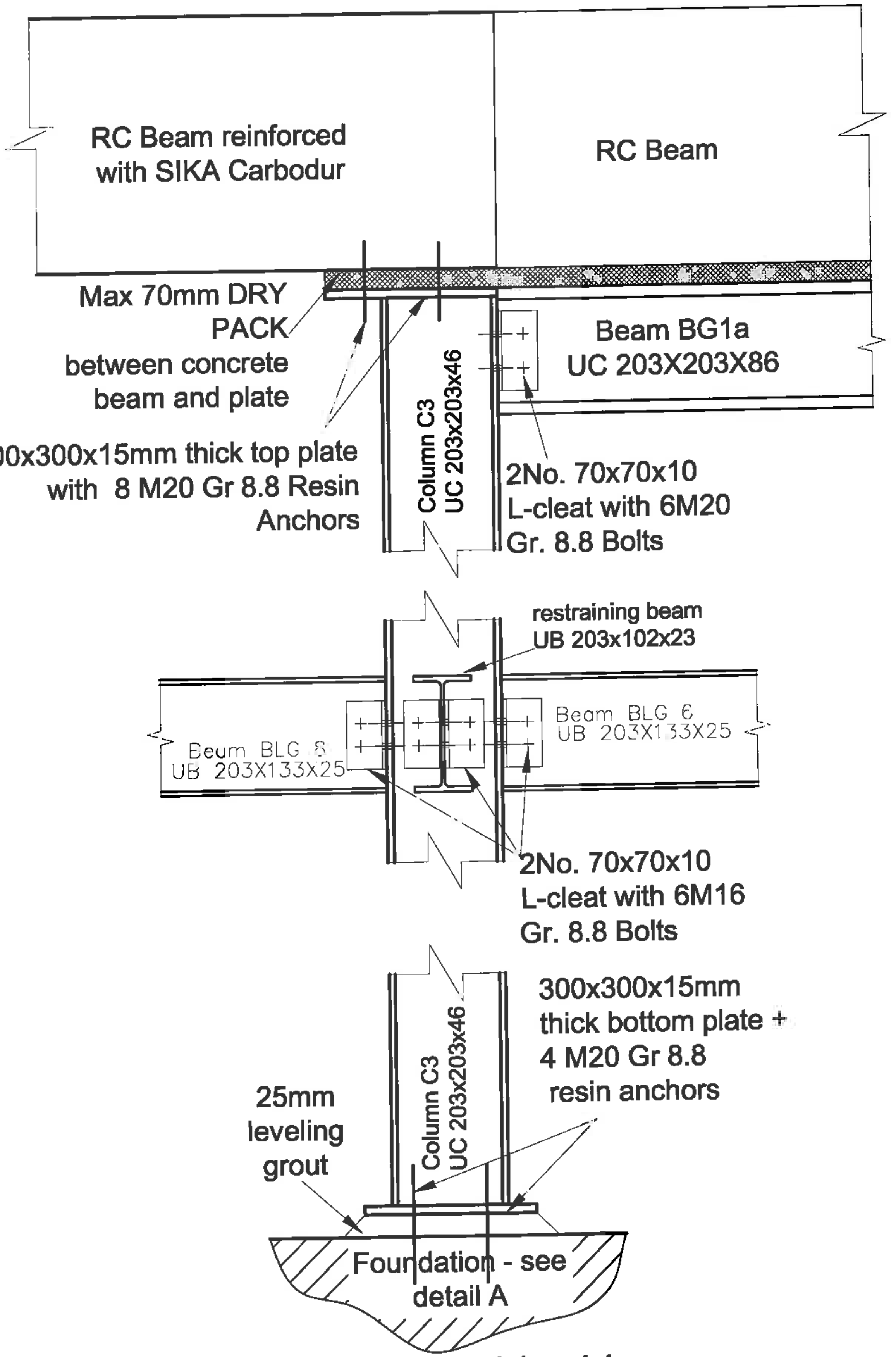
DETAIL B OF FOUNDATION UNDER 7N BLOCK WALL

Martin Redston Associates
Consulting Civil & Structural Engineers

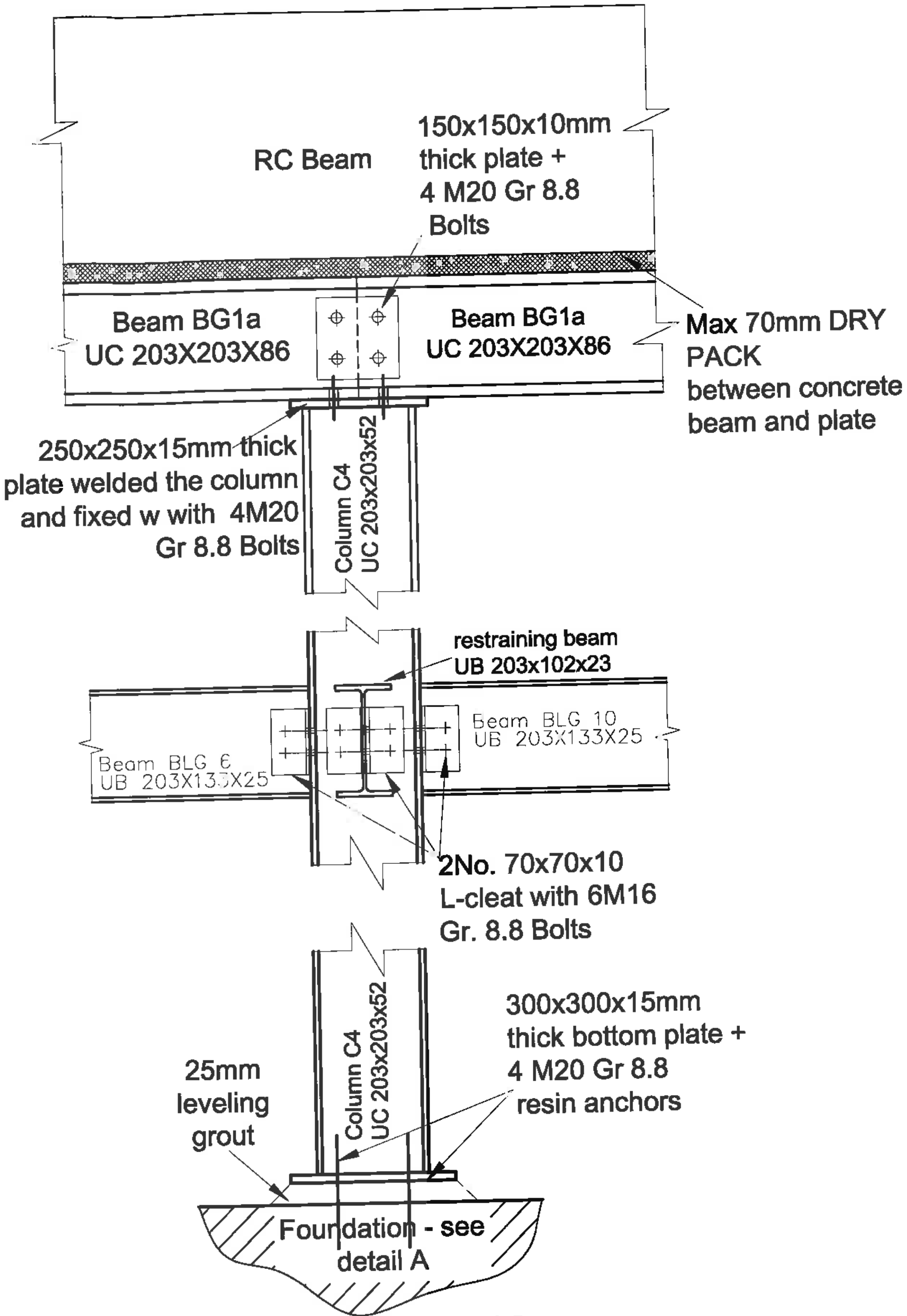
3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 13/06/2014	Sheet No. 12A
Eng.: RS	Scale 1/25@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Section 5 - Detail B	



Section 11 - 11
Column C2 connection
to foundation detail
1/10 @A3



Section 12 - 12
Column C2 connection
to foundation detail
1/10 @A3

Martin Redston Associates
Consulting Civil & Structural Engineers

3 Edward Square, London N1 0SP
Tel: 020 7837 5377; Fax: 020 7837 3211

6 Hale Lane, London NW7 3NX
Tel: 020 8959 1666; Fax: 020 8906 8503

Date: 30/07/2014	Sheet No. 13
Eng.: RS	Scale 1/25@A3
Job No.: 14.073	
Project: 45 Maresfield Gardens	
Drawing Title: Sections 11-11 and 12-12	