job no **1420**

13~15 John's Mews

Structural calculations relating to a proposed basement extension.

prepared by

GT Scott. BSc(Hons); MSc, CEng, MIStructE

Trevor Scott Consulting Ltd
8 Herons Mead
Bromham
Bedford MK43 8LQ
tel: 01234 826110 M: 07595 022242
e-mail: gts@tscuk.com

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Proposed Basement Extension

The proposal is for the construction of a single level basement extension over the full plan area of both properties. The properties share a common party wall line and have separate party wall conditions on the remaining sides. The following calculations cover the design of the basement only as part of the specific requirements contained within the overall Basement Impact Assessment (BIA) for submission to Camden Planning.

The existing building is a two-storey office/workshop development. The current proposals convert this to residential adding mansard level accommodation in addition to the basement. The following loading is assumed.

1. Roof (proposed) - traditional @ 2D + 0.75L
2. 2nd floor (proposed) - timber @ 1.0D + 1.50L
3. 1st floor (existing) - 150 conc @ 5.0D + 1.50L
4. Ground (proposed) - 150 conc @ 5.0D + 1.50L
5. Party walls - 215 brick @ 4.5D/m²
6. Front/rear walls ditto @ 4.5D/m²

2.00 Ground Floor

Holorib or similar construction

Max span 3050 mm

loading as 5.0Δ + 1.50L

Single span construction, N.W. Conc

130 thk unpropped 0.9 gauge can span 2.80 –

1.2 " " " 3.20 –

130 thk propped 0.9 g. " " 4.30

1.2 " " " 4.58 –

Vol. of conc = 0.122 m^3/m^2 = 2.93 kN/m^2 (3.00)

Imposed load capacity = 5.00 "

Σ 7.93

Proposed: 5 + 1.50 = 6.50 < 7.93 O.K.

2.01 Typical ground floor beam

Span = 5266 (clear) + 2×150 = 5566 say 5600

w = (5.0Δ + 1.5L) × 3.00 = 15Δ + 4.5L

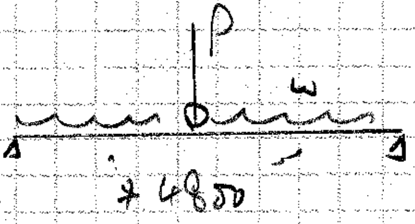
Ref page 2.02 – provide 203UC 60

Reactions = 42Δ + 12.6L (79 ult)

2.02 Stair trimmer

P = (5Δ + 1.5L) × 2.4 × 2.2

= 26Δ + 8L



w = 0.6Δ × 2.8 say 1.7Δ/m

Ref pg 2.03 – provide 203UC 46 $R_{A,B}$ = 17Δ, 4L (30 ult)

Trevor Scott Consulting Ltd
8 Herons Mead
Bromham
Bedford
MK43 8LQ
Tel:01234 826110

Proj: 13/15 John's Mews
Ref : Typical GF beam

Date: 11/05/14 2.07

UC bending design

Calculations in accordance with BS5950:Part1:2000 and the SCI 'Steelwork Design Guide to BS5950'

Span	= 5.6 m		
Section size	- 203x203x60 UC	Section class -	1 (Plastic)
Steel grade	- S 275	Design strength	= 275 N/mm^2
E	= 205000 N/mm^2	I_{xx}	= 6130 cm^4

Restraint type (table 13)
Left hand end - Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as detailed in 4.2.2. Both flanges partially restrained against rotation on plan.
Right hand end - Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as detailed in 4.2.2. Both flanges partially restrained against rotation on plan.

Section restrained over its length.

	Span	**Left support**	**Right support**
Design forces			
Maximum span moment (kNm)	110.5		
Support reactions (kN)		79	79
Shear capacity - (kN)			
Clause 4.2.3. - LOW SHEAR		325.7	325.7
Moment capacity - (kNm)			
Clause 4.2.5.2.	180.4		
Deflection			
for total load	19.8 mm (span / 283)		
for imposed load	4.6 mm (span / 1217)		

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UC bending design

Calculations in accordance with BS5950:Part1:2000 and the SCI 'Steelwork Design Guide to BS5950'

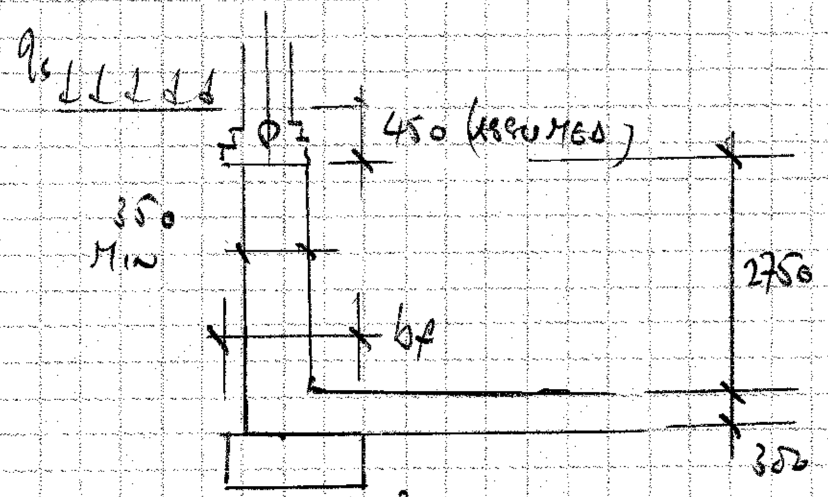
Span	= 4.8 m		
Section size	- 203x203x46 UC	Section class -	2 (Compact)
Steel grade	- S 275	Design strength	= 275 N/mm^2
E	= 205000 N/mm^2	I_{xx}	= 4570 cm^4

Restraint type (table 13)
Left hand end - Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as detailed in 4.2.2. Both flanges partially restrained against rotation on plan.
Right hand end - Compression flange laterally restrained. Nominal torsional restraint against rotation about longitudinal axis, as detailed in 4.2.2. Both flanges partially restrained against rotation on plan.

Section restrained over its length.

	Span	**Left support**	**Right support**
Design forces			
Maximum span moment (kNm)	65.9		
Support reactions (kN)		30.3	30.3
Shear capacity - (kN)			
Clause 4.2.3. - LOW SHEAR		241.2	241.2
Moment capacity - (kNm)			
Clause 4.2.5.2.	136.7		
Deflection			
for total load	9.6 mm (span / 500)		
for imposed load	2 mm (span / 2400)		

3.00 Basement retaining wall.
Design as simple cantilever. thus



q_s taken as $2.5\ kN/m^2$
Surcharge at top of wall = 0.45 × 18.5 + 2.5 = 10.8 (11) kN/m^2
Axial from party wall taken as follows

1. Self weight = 4.5D × 9.2h = 41.4D
2. Roof @ (2D + 0.75L) × 5.2/2 = 5.2D + 2.0L
3. Ditto (neighbours) = 5.2D + 2.0L
4. 2nd (new) @ (1.0D + 1.5L) × 5.2/2 = 2.6D + 3.9L
5. 1st (5D + 1.5L) × 5.2 (incl. neigh.) = 26.0D + 7.8L
6. Ground ditto = 26.0D + 7.8L

Σ 106.4D + 23.5L = 130. kN.

3.00.1 Assume safe ground bearing pressure of 15/19 ft
110 kN/m^2 (nett) at depth of 3.5 m to u/s.
of foundation.
Min width bf = $\frac{130}{1.1}$ = 1.18 say 1.2 metres

Actual ground capacity to be confirmed by site investigation

3.07.2 Retaining wall design

Refer to output on 3.02 3.03 for simple cantilever condition. Sliding and overturning are not critical by inspection

Mult = 96.6 kNm/m

$M/bd^2f_{cu} = 0.03$

Ast reqd = 802 mm²/m

d = 350 - 50 - 10 = 290 mm

$f_{cu} = 35\ N/mm^2$

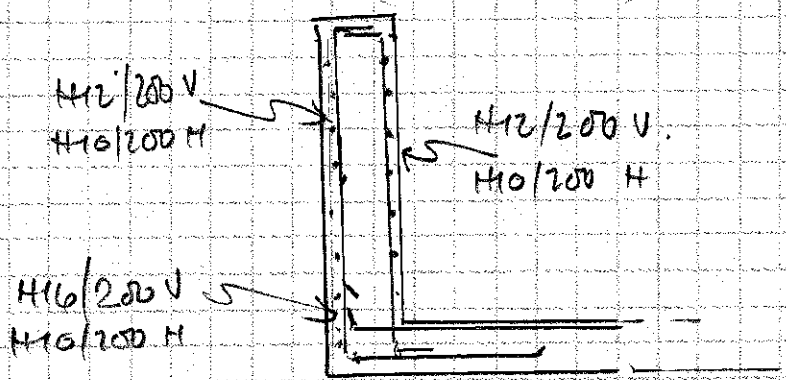
Vult = 96.91 kN/m

$v = \frac{96.91}{290} = 0.34\ N/mm^2$

$\frac{100 A_s}{bd}$ reqd $= \left(\frac{0.34 \times 1.25}{0.79 \times \left(\frac{400}{290}\right)^{\frac{1}{4}} \times \left(\frac{35}{25}\right)^{\frac{1}{3}}} \right)^3 = 0.09\%$

Provide 0.15% minimum ≡ 435 mm²/m

Provide H16 @ 200 (1010 mm²/m) on o.f.



Trevor Scott

Consulting Structural Engineer

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		calculations by	gts	
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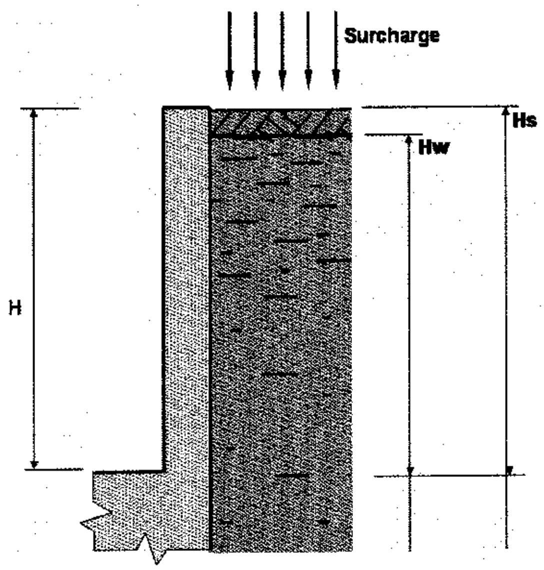
Design for Retaining Wall with Granular Retained Material

Typical restrained condition

Load Factors for this load case:

Soil	1.40
Water	1.40
Dead Load	1.40
Live Load	1.60



Wall Parameters:

Wall Type	Cantilever
Height, H	2.75 m

Soil and Loading Parameters:

Height of Soil, Hs	2.75 m
Height of Water, Hw	2.5 m
Soil Dry Density	1850 kg/m³
Submerged Soil Density	1110 kg/m³
k	0.5
Surcharge Dead Load	8.5 kN/m²
Surcharge Live Load	2.5 kN/m²

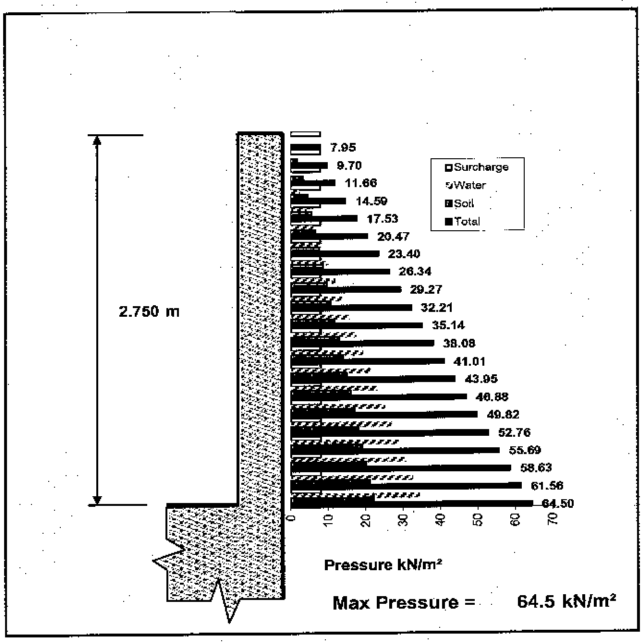
Analysis and Results:

Soil Pressures:
Pressure = LF x k x Depth x Density x g
where LF is the load factor
and gravity, g, is taken as 9.80665 m/s^2;
the submerged density is used below water

Water Pressure:
Pressure = LF x Depth x Density x g
where the density of water is 1000 kg/m³

Pressure due to surcharge:
Pressure = LF x k x Load

Max. Soil Pressure	22.22 kN/m²
Max. Water Pressure	34.32 kN/m²
Max. Surcharge Pressure	7.95 kN/m²
Max. Total Pressure	**64.50 kN/m²**



PRESSURES ON WALL

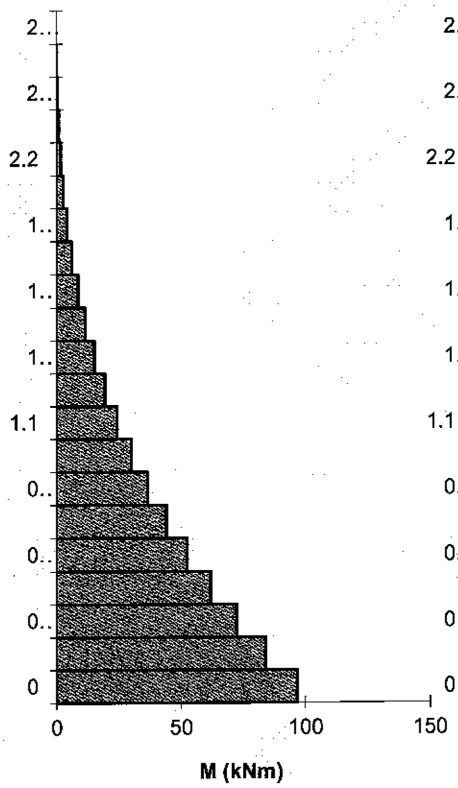
Design for Retaining Wall with Granular Retained Material

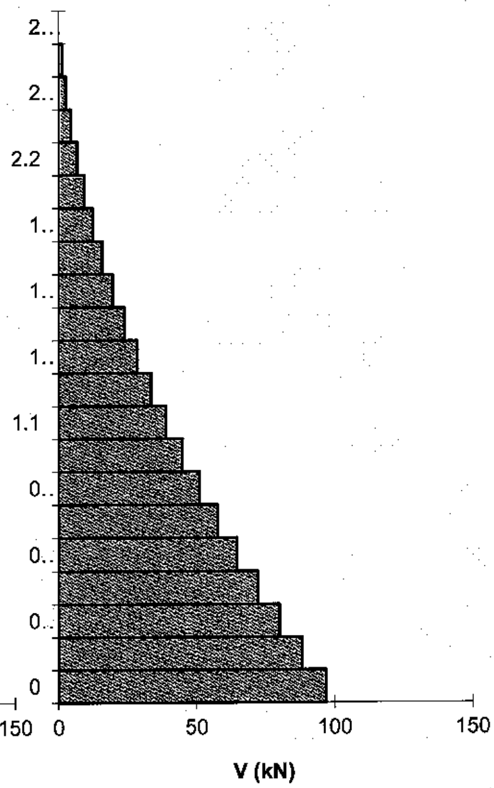
Typical restrained condition

Continued

Moments and Shears on Wall Stem:

Level (m)	Moment kNm	Shear kN
2.75	0.00	0.00
2.613	0.08	1.21
2.475	0.34	2.67
2.338	0.83	4.47
2.2	1.59	6.68
2.063	2.69	9.29
1.925	4.17	12.31
1.788	6.09	15.73
1.65	8.51	19.55
1.513	11.49	23.78
1.375	15.07	28.41
1.238	19.32	33.44
1.1	24.29	38.88
0.963	30.03	44.72
0.825	36.60	50.97
0.688	44.06	57.62
0.55	52.47	64.67
0.413	61.87	72.12
0.275	72.32	79.98
0.138	83.88	88.25
0	96.60	96.91





Max. Bottom Moment = 96.6 kNm

Max. Bottom Shear = 96.91 kN

3.00.3. Basement slab

Uplift pressure from water = 2.5×10 = 25 kN/m²

300 slab resisting = −0.3×25 = −7.5 〃

+100 finishes @ 0.1×20 = −2.0

Σ 15.5 kN/m² ↑

Max. M ≈ $\frac{15.5 \times 1.2 \times 5.7^2}{8}$

= 75.5 kNm/m

d = 300 − 50 − 8 = 242 (240) mm

Ast reqd = $\frac{75.5 \times 10^6}{0.95 \times 460 \times 0.95 \times 240}$ = 758 mm²/m

V = 15.5×1.2×5.7×0.6 = 63.6 kN/m

v = $\frac{63.6}{240}$ = 0.27 N/mm² − Min reinft only

Minimum reinft = 1.5×240 = 360 mm²/m

Provide H12 @ 150 (754) T+B in short span direction & H10 @ 150 T+B in long span direction (393 mm²/m)

3.10.4. Check for flotation

Check ignoring beneficial effect of adjoining properties (conservative but a realistic scenario if adjacent properties demolished for redevelopment)

Loading [ignoring imposed loads]

Party wall to No. 11 (from 3.10) = 41.6 kN × 12.8m	= 530 kN
ditto to no. 17	= 530 kN
Ditto between 13 & 15	= 530 kN
Roof at 2 kN × 12.8 × 11.4	= 292 kN
2nd at 1.0 × 12.8 × 11.4	= 146 kN
1st ditto	= 146 kN
Gnd at 5.0 × 12.8 × 11.4	= 730 kN
B'ment at 0.35 × 25 × 12.8 × 11.4	= 1277 kN
Perimeter wall at 0.35 × 25 × 2.5 × 47.70	= 1043 kN
Internal wall at 0.30 × 25 × 2.5 × 12.10	= 227 kN
	Σ 5451 kN

Spread over 12.8 × 11.4 m = 37.4 kN/m²

Max uplift pressure = 25.00 kN/m²

f.o.s against uplift = 1.49 > 1.10 OK

Conclusion – uplift under hydrostatic pressure is an unlikely event