Appendix D

Scheme Structural Calculations

calculation sheet	page no. 01	project no. 2018-059
project 58A Robinson Road, NW3	by [initials]	checked
section BASEMENT SCHEME DESIGN	date OCT 2018	date

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The Detailed Design of the permanent works will be completed by Elite Designers Ltd.

The temporary works lateral ground support propping will be designed by the Principle Contractors temporary works designer, with their construction sequence and design reviewed by Elite Designers Ltd.

These scheme calculations have been carried out to validate the concept design, and to provide wall and slab thicknesses for the basement, and to assess the ground bearing pressures below new foundations and thus confirm that these can be supported by the natural soils at the level of the new basement slab.

- Loadings to BS6399 · RC to BS 8110
- Soil parameters as per S.I. Interpretative Report by GEA: P02
- Scheme Designs:–

Key Walls:
- Front basement wall — 02
- Rear flank wall, at boundary with N° 58 — 08
- Wall below Party Wall to N° 58 — 16

Basement Slab: With residual heave + water pressure — 23-26

calculation sheet	page no. 02	project no. 2018-059
project	by DM	checked.
section	date 4/10/2018	date

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SCHEME CALCULATIONS

Soil Parameters:- (Ref GEA report J18142, September 2018)

Claygate Member Bulk density = 1900kg/m³ = 19kN/m³

Effective cohesion = 0

Effective friction angle = 24°

∴ use $K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.43$

Consider r.c. basement walls supporting lateral loads

from · soil pressures

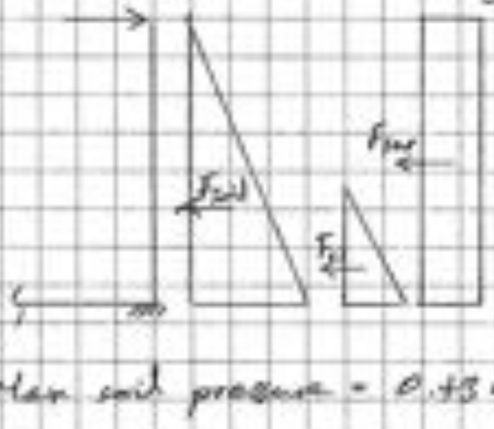
· water pressure

and · surcharge pressure.

Basement Walls:

At Front of Basement:- See Section A-A on drawing 2018-059-06.

Propped cantilevered retaining wall 3.8m high
with 1.5m water (1m higher than recorded)
and surcharge from 3m of soil



Max soil pressure = 0.43×19×3.8 = 31.0 kN/m²

Max water pressure = 1.5×10 = 15 kN/m²

Surcharge pressure = 0.43×3.0×19 = 24.5kN/m²

calculation sheet	page no. 03	project no. 2008-059
project	by DY	checked.
section	date 6/10/2008	date

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See Tekla analysis: for design to ZSB110

350 r.c. wall with 50 outer face cover, c40
with H12 @ 150 c/c vert, both faces
and 4 legs H10 shear legs at 300 horiz & 200 vert c/c okay.

secondary bars: $0.0013 A_c = 455 \text{ mm}^2/\text{m}$
$\approx 228 \text{ mm}^2/\text{m}$ each face

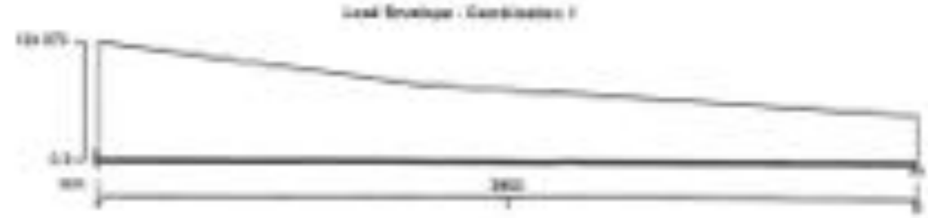
Provide H10 @ 200 c/c horiz, both faces = $393 \text{ mm}^2/\text{m}$.

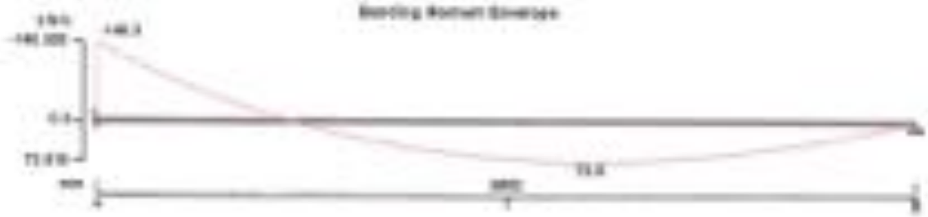
Deflection okay

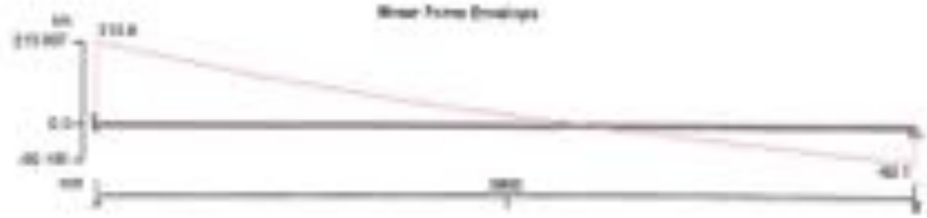
Check crack widths at Detailed Design stage
& limit to 0.2mm

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.12







Support conditions

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

Applied loading

	Dead self weight of beam × 1
Span 1 loads	Imposed UDL 24.500 kN/m from 0 mm to 3800 mm
	Imposed VDL 15.000 kN/m at 0 mm to 0.000 kN/m at 1500 mm
	Imposed VDL 31.000 kN/m at 0 mm to 0.000 kN/m at 3800 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment support A	M_{A_max} = -146 kNm	M_{A_red} = -146 kNm
Maximum moment span 1 at 2286 mm	M_{s1_max} = 74 kNm	M_{s1_red} = 74 kNm
Maximum moment support B	M_{B_max} = 0 kNm	M_{B_red} = 0 kNm
Maximum shear support A	V_{A_max} = 214 kN	V_{A_red} = 214 kN
Maximum shear support A span 1 at 300 mm	$V_{A\_s1\_max}$ = 178 kN	$V_{A\_s1\_red}$ = 178 kN
Maximum shear support B	V_{B_max} = -92 kN	V_{B_red} = -92 kN
Maximum shear support B span 1 at 3600 mm	$V_{B\_s1\_max}$ = -76 kN	$V_{B\_s1\_red}$ = -76 kN
Maximum reaction at support A	R_A = 214 kN	
Unfactored dead load reaction at support A	R_{A_Dead} = 30 kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = 116 kN	
Maximum reaction at support B	R_B = 92 kN	
Unfactored dead load reaction at support B	R_{B_Dead} = 12 kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = 47 kN	

Rectangular section details

Section width	b = 1000 mm
Section depth	h = 350 mm

Concrete details

Concrete strength class	C40/50
Characteristic compressive cube strength	f_{cu} = 50 N/mm²
Modulus of elasticity of concrete	E_c = 20kN/mm² + 200 × f_{cu} = 30000 N/mm²
Maximum aggregate size	h_{agg} = 20 mm

Reinforcement details

Characteristic yield strength of reinforcement	f_y = 500 N/mm²
Characteristic yield strength of shear reinforcement	f_{yv} = 500 N/mm²

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = 35 mm
Nominal cover to bottom reinforcement	c_{nom_b} = 50 mm
Nominal cover to side reinforcement	c_{nom_s} = 35 mm

Mid span 1

Design moment resistance of rectangular section (cl. 3.4.4) - Positive moment

Design bending moment: $M = abs(M_{m1\_red}) = \mathbf{74}$ **kNm**

Depth to tension reinforcement: $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2 = \mathbf{284}$ **mm**

Redistribution ratio: $\beta_b = \min(1 - m_{m1}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{cu}) = \mathbf{0.018}$

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm: $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{270}$ **mm**

Depth of neutral axis: $x = (d - z) / 0.45 = \mathbf{32}$ **mm**

Area of tension reinforcement required: $A_{s,bot,req} = M / (0.87 \times f_y \times z) = \mathbf{626}$ **mm²**

Tension reinforcement provided: $7 \times 12\phi$ bars

Area of tension reinforcement provided: $A_{s,bot,prov} = \mathbf{792}$ **mm²**

Minimum area of reinforcement: $A_{s,min} = 0.0013 \times b \times h = \mathbf{455}$ **mm²**

Maximum area of reinforcement: $A_{s,max} = 0.04 \times b \times h = \mathbf{14000}$ **mm²**

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

Rectangular section in shear

Shear reinforcement provided: $4 \times 10\phi$ legs at 200 c/c

Area of shear reinforcement provided: $A_{sv,prov} = \mathbf{1571}$ **mm²/m**

Minimum area of shear reinforcement (Table 3.7): $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv}) = \mathbf{920}$ **mm²/m**

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5): $s_{vl,max} = 0.75 \times d = \mathbf{213}$ **mm**

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

Design concrete shear stress: $v_c = 0.79N/mm^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400mm / d)^{1/4}) \times (\min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m = \mathbf{0.526}$ **N/mm²**

Design shear resistance provided: $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b = \mathbf{0.683}$ **N/mm²**

Design shear stress provided: $v_{prov} = v_{s,prov} + v_c = \mathbf{1.209}$ **N/mm²**

Design shear resistance: $V_{prov} = v_{prov} \times (b \times d) = \mathbf{343.5}$ **kN**

Shear links provided valid between 0 mm and 3600 mm with tension reinforcement of 792 mm²

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension: $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) / (N_{bot} - 1) - \phi_{bot} = \mathbf{138}$ **mm**

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension: $s_{min} = h_{agg} + 5$ mm $= \mathbf{25}$ **mm**

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress — $f_s = (2 \times f_y \times A_{s\_req}) / (3 \times A_{s\_prov} \times \beta_b)$ = **263.7 N/mm²**

Maximum distance between bars in tension — s_{max} = min(47000 N/mm / f_s, 300 mm) = **178 mm**

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) — $span\_to\_depth_{basic}$ = **20.0**

Design service stress in tension reinforcement — $f_s = (2 \times f_y \times A_{s\_req}) / (3 \times A_{s\_prov} \times \beta_b)$ = **263.7 N/mm²**

Modification for tension reinforcement

f_{tens} = min(2.0, 0.55 + (477N/mm² - f_s) / (120 × (0.9N/mm² + (M / (b × d²))))) = **1.531**

Modification for compression reinforcement

f_{comp} = min(1.5, 1 + (100 × A_{s2_prov} / (b × d)) / (3 + (100 × A_{s2_prov} / (b × d)))) = **1.086**

Modification for span length — f_{long} = 1.000

Allowable span to depth ratio — $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **33.2**

Actual span to depth ratio — $span\_to\_depth_{actual} = L_{eff} / d$ = **13.4**

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

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project	by [illegible]	checked
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RC Retaining Wall Scheme Design.

Rear Flank Wall, at boundary with no 58.

Un-propped Cantilever. (as Section P on Dwg 12)

10kN/m² Surcharge

--> Temp? Lateral Supports

$\gamma = 19 kN/m^3$

$K_a = 0.43$

Water level > 2m below slab

150kN/m² safe bearing capacity

#A: Temporary condition – propped top & bottom.

#B: Permanent condition – un-propped

See Tedds analyses:-

#A: Bearing pressure = 27kN/m²

Propping loads = 14kN/m at top & 19kN at base of wall

Reinforcement: H12@200c/c EW EF

#B:

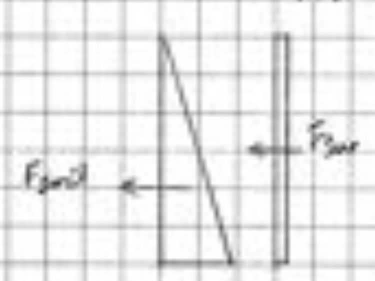
TBD

2:-

Max soil pressure = 0.43 × 3.05 × 19 = 42.3 kN/m² (SLS)

Surcharge pressure = 0.43 × 10 = 4.3 kN/m² (SLS)

Water level > 2m below base ∴ no contribution



$F_{soil} = \frac{42.3}{2} \times 3.05 = 64.5$ kN/m (SLS)

$F_{sur} = 4.3 \times 3.05 = 13.1$ kN/m (SLS)

$\therefore \overline{\Sigma M} = \left(64.5 \times \frac{3.05}{3}\right) + \left(13.1 \times \frac{3.05}{2}\right)$

$= 65.6 + 20.0 = 85.6$ kNm/m of wall (SLS)

$\times \underline{1.6}$

136.9 kNm/m (ULS)

BS8110:-

h = 350 mm
cover = 50
d = 350 − 50 − 10 − 10
= 280 mm
b = 1000
f_{cu} = 40
f_y = 460

Bending:- $K = M/bd^2f_{cu} = 0.044$

$\Rightarrow z = 0.949d = 266$ mm

$\therefore As_{req'd} = M/0.95 f_y z$

$= 1178$ mm²/m Vert, Outer Face

H16 @ 150 % ⇒ As prov = 1340 mm²/m

Shear:-

$\overline{\Sigma V} = 64.5 + 13.1 = 77.2$ kN/m of wall (SLS)

$\times \underline{1.6}$

123.5 kN/m (ULS)

$v = V/bd = 0.44$ N/mm²

$\left.\begin{array}{l} \frac{100 A_s}{bd} = 0.48 \\ d = 280 \end{array}\right\}$ T3.9 ⇒ v_c = 1.17 × 0.54 = 0.63 N/mm²

$v_c > v$ ∴ No shear links req'd

	25?	200	300
0.25	0.45	0.44	0.42
0.48		0.54	
0.50	0.54	0.55	0.54

Deflection: $\frac{span}{depth} = \frac{2700}{280} = 9.6 > 7$

PTO

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section	date 8/10/2018	date

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Tension Re-bar Mod Factor:-

$M/bd^2 = 1.75$

Service stress $= \frac{1198}{1360} \times 288 = 253 N/mm^2$ } T3.8 ⇒ Mod Factor = 1.27

∴ Permissible Span/Depth = 7 × 1.27 = 8.9

Actual Span/Depth = 9.7 > 8.9

Compression Re-bar Mod Factor:-

For H12 @ 200%, $A_s' = 566 mm^2/m$

⇒ $\frac{100 A_s'}{bd} = 0.20$ T3.12 ⇒ Mod Factor = 1.065

∴ Permissible Span/Depth = 8.9 × 1.065 = 9.5 < 9.7

∴ Increase Re-bar

For H12 @ 150% $A_s' = 754 mm^2/m$

⇒ $\frac{100 A_s'}{bd} = 0.30$ ⇒ Mod Factor = 1.09

⇒ Permissible Span/Depth = 8.9 × 1.09 = 9.7

Actual Span/Depth = 9.7 ∴ OKAY

Summary:-

Vert Outer face H16 @ 150%

Vert Inner face H12 @ 150%

Horiz both faces: $\Sigma A_s = 0.0013 A_c$

$= 455 mm^2/m$

i.e. $228 mm^2/m$ / face

H10 @ 200% ⇒ $393 mm^2/m$.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **3000** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1350** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **3350** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1000** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = **9.81** kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **3350** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **19.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.0** deg	Angle of wall friction	δ = **18.5** deg
Design shear strength	ϕ'_b = **24.0** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **19.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **150** kN/m^2

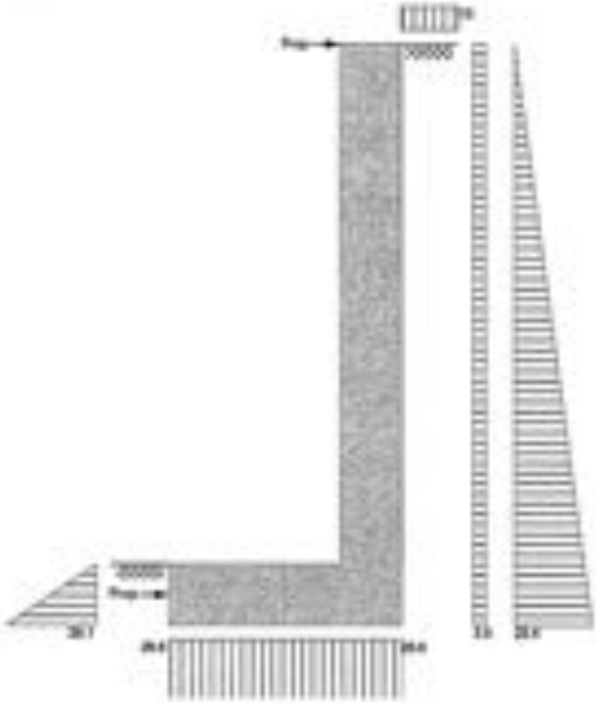
Using Coulomb theory

Active pressure	K_a =**0.372**	Passive pressure	K_p = **4.143**
At-rest pressure	K_0 = **0.593**		

Loading details

Surcharge load	Surcharge = **10.0** kN/m^2		
Vertical dead load	W_{dead} = **0.0** kN/m	Vertical live load	W_{live} = **0.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m

Position of vertical load l_{load} = 0 mm Height of horizontal load h_{load} = 0 mm



Loads shown in kN/m, pressures shown in kN/m²

Calculate propping force

Propping force F_{prop} = **32.8** kN/m

Check bearing pressure

Total vertical reaction R = **35.9** kN/m Distance to reaction x_{bar} = **675** mm

Eccentricity of reaction e = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = **26.6** kN/m² Bearing pressure at heel p_{heel} = **26.6** kN/m²

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall F_{prop_top} = **13.750** kN/m Propping force to base of wall F_{prop_base} = **19.004** kN/m

<table>
<tr><td rowspan="3">Tekla
Tedds
Elite Designers Ltd
3 Princeton Court
53-55 Felsham Road
London SW15 1AZ</td><td colspan="4">Project
58A Redington Road</td><td colspan="2">Job no.
2018-059</td></tr>
<tr><td colspan="4">Calcs for
Cantilevered basement Wall - Temporary Condition</td><td colspan="2">Start page no./Revision
3 /3</td></tr>
<tr><td>Calcs by
BH</td><td>Calcs date
08/10/2018</td><td>Checked by</td><td>Checked date</td><td>Approved by</td><td>Approved date</td></tr>
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RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor $\gamma_{f\_d} = \mathbf{1.4}$ Live load factor $\gamma_{f\_l} = \mathbf{1.6}$

Earth pressure factor $\gamma_{f\_e} = \mathbf{1.4}$

Calculate propping force

Propping force $F_{prop} = \mathbf{32.8}$ **kN/m**

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f} = \mathbf{20.366}$ **kN/m** Propping force to base of wall $F_{prop\_base\_f} = \mathbf{27.853}$ **kN/m**

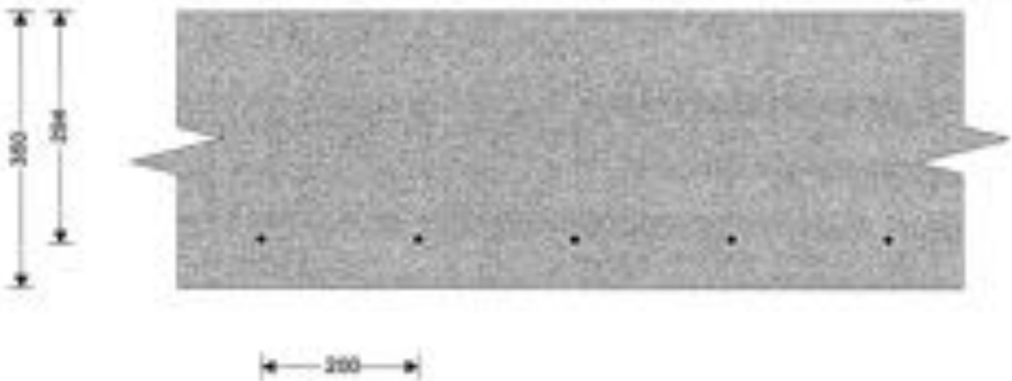
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete $f_{cu} = \mathbf{40}$ **N/mm²** Strength of reinforcement $f_y = \mathbf{500}$ **N/mm²**

Base details

Minimum reinforcement $k = \mathbf{0.13}$ **%** Cover in toe $c_{toe} = \mathbf{50}$ **mm**



Design of retaining wall toe

Shear at heel $V_{toe} = \mathbf{25.7}$ **kN/m** Moment at heel $M_{toe} = \mathbf{17.7}$ **kNm/m**

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_toe\_req} = \mathbf{455.0}$ **mm²/m** Area provided $A_{s\_toe\_prov} = \mathbf{565}$ **mm²/m**

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = \mathbf{0.087}$ **N/mm²** Allowable shear stress $v_{adm} = \mathbf{5.000}$ **N/mm²**

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress $v_{c\_toe} = \mathbf{0.461}$ **N/mm²**

$v_{toe} < v_{c\_toe}$ ***- No shear reinforcement required***

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Strength of concrete $f_{cu} = \mathbf{40}$ **N/mm²** Strength of reinforcement $f_y = \mathbf{500}$ **N/mm²**

Wall details

Minimum reinforcement $k = \mathbf{0.13}$ **%**

Cover in stem $c_{stem} = \mathbf{50}$ **mm** Cover in wall $c_{wall} = \mathbf{50}$ **mm**

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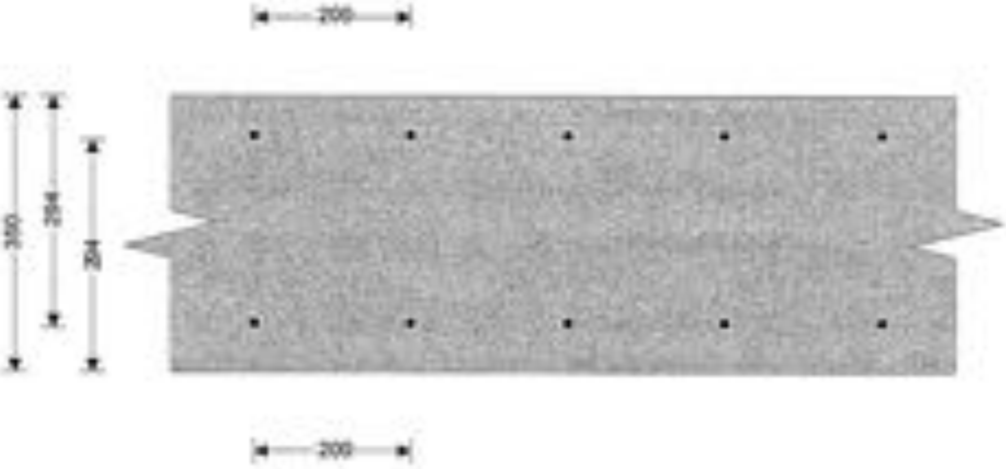
	Calcs for Cantilevered basement Wall - Temporary Condition				Start page no./Revision 4	
	Calcs by BH	Calcs date 08/10/2018	Checked by	Checked date	Approved by	Approved date



Design of retaining wall stem

Shear at base of stem V_{stem} = **43.3** kN/m Moment at base of stem M_{stem} = **26.3** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_stem\_req}$ = **455.0** mm²/m Area provided $A_{s\_stem\_prov}$ = **565** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.147** N/mm² Allowable shear stress v_{adm} = **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.461** N/mm²

v_{stem} < v_{c_stem} - No shear reinforcement required

Design of retaining wall at mid height

Moment at mid height M_{wall} = **12.0** kNm/m

Compression reinforcement is not required

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_wall\_req}$ = **455.0** mm²/m Area provided $A_{s\_wall\_prov}$ = **565** mm²/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **39.89** Actual span/depth ratio $ratio_{act}$ = **10.20**

PASS - Span to depth ratio is acceptable

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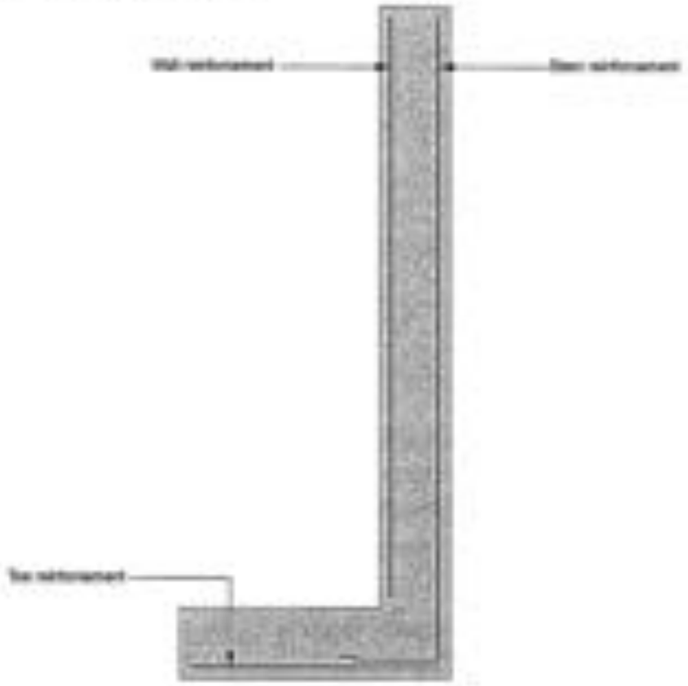
	Calcs for Cantilevered basement Wall - Temporary Condition			Start page no./Revision 5 15	
	Calcs by BH	Calcs date 08/10/2018	Checked by	Checked date	Approved by	Approved date

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 200 mm centres - (565 mm²/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm²/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm²/m)

calculation sheet	page no. 16	project no. 2018-05?3
project	by BH	checked.
section	date 8/10/2018	date

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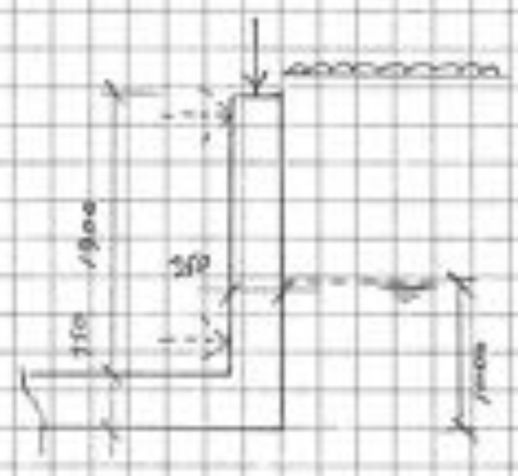
Retaining Wall below Party Wall to Nº58:-

As sections c-c etc.

Design as vertical cantilever
with vertical load from Party Wall.



Allow for 1.2m of soil @ 19 kN/m³
⇒ 22.8 kN/m²
Allow for 5 kN/m² surcharge

∴ Σ Surcharge = 28 kN/m² (sls)

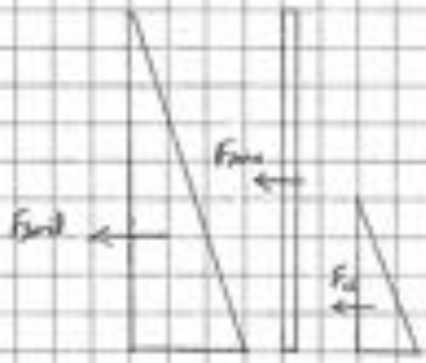
Allow for accidental water level
1.0m above base.

$\gamma_{soil} = 19$ kN/m³

$k_a = 0.43$

Party Wall proposed load = 66 kN/m (sls)

Permissible bearing pressure = 150 kN/m²



\# A: Temporary condition - See Tedds analysis

Max bearing pressure = 69 kN/m² < 150 ∴ ok

Propping loads = -2 kN/m at top & 5 kN/m at base
ie top props not req'd.

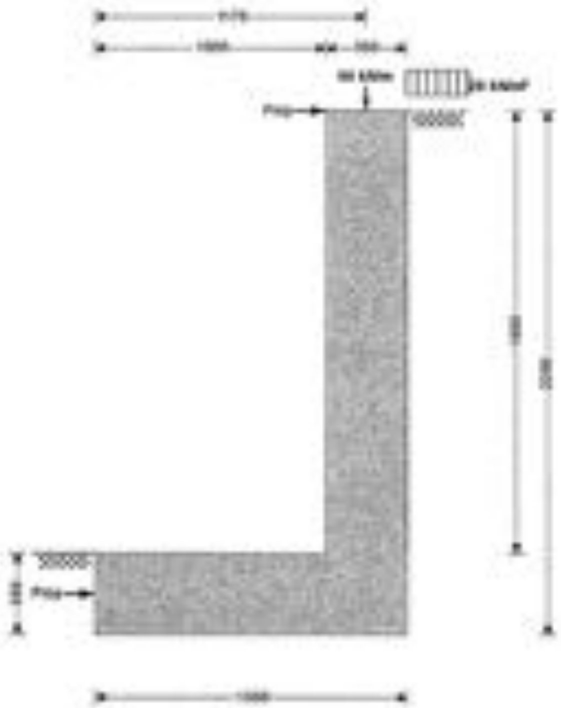
H12 @ 200% ok

\# B:

PTO

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever**		
Height of wall stem	h_{stem} = **1900** mm	Wall stem thickness	t_{wall} = **350** mm
Length of toe	l_{toe} = **1000** mm	Length of heel	l_{heel} = **0** mm
Overall length of base	l_{base} = **1350** mm	Base thickness	t_{base} = **350** mm
Height of retaining wall	h_{wall} = **2250** mm		
Depth of downstand	d_{ds} = **0** mm	Thickness of downstand	t_{ds} = **350** mm
Position of downstand	l_{ds} = **1000** mm		
Depth of cover in front of wall	d_{cover} = **0** mm	Unplanned excavation depth	d_{exc} = **0** mm
Height of ground water	h_{water} = **0** mm	Density of water	γ_{water} = 9.81 kN/m^3
Density of wall construction	γ_{wall} = **23.6** kN/m^3	Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of soil surface	β = **0.0** deg	Effective height at back of wall	h_{eff} = **2250** mm
Mobilisation factor	M = **1.5**		
Moist density	γ_m = **19.0** kN/m^3	Saturated density	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.0** deg	Angle of wall friction	δ = **18.5** deg
Design shear strength	ϕ'_b = **24.0** deg	Design base friction	δ_b = **18.6** deg
Moist density	γ_{mb} = **19.0** kN/m^3	Allowable bearing	$P_{bearing}$ = **150** kN/m^2

Using Coulomb theory

Active pressure	K_a =**0.372**	Passive pressure	K_p = **4.143**
At-rest pressure	K_0 = **0.593**		

Loading details

Surcharge load	Surcharge = **28.0** kN/m^2		
Vertical dead load	W_{dead} = **66.0** kN/m	Vertical live load	W_{live} = **0.0** kN/m
Horizontal dead load	F_{dead} = **0.0** kN/m	Horizontal live load	F_{live} = **0.0** kN/m

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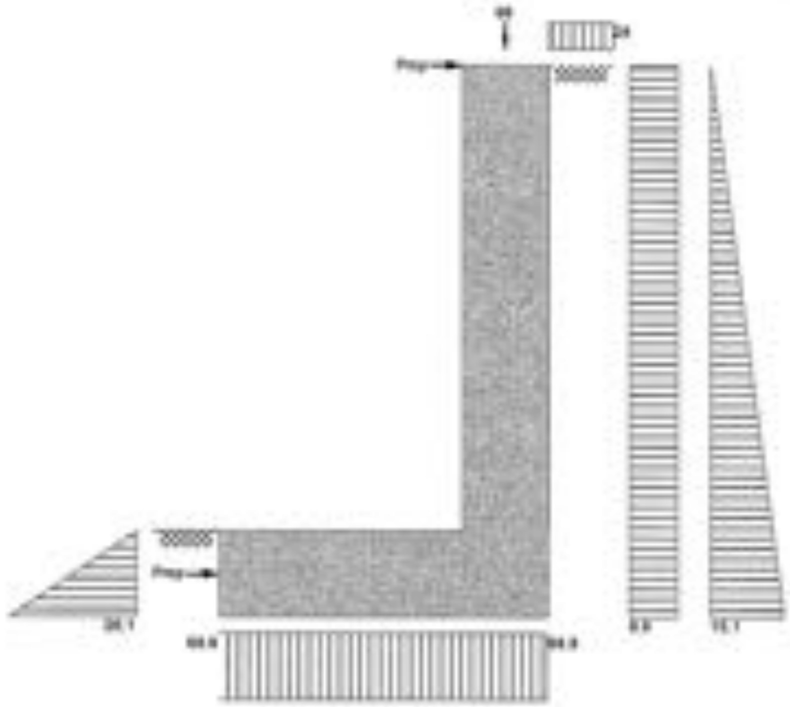
Position of vertical load l_{load} = 1175 mm Height of horizontal load h_{load} = 0 mm



Loads shown in kN/m, pressures shown in kN/m^2

Calculate propping force

Propping force F_{prop} = 3.4 kN/m

Check bearing pressure

Total vertical reaction R = 92.8 kN/m Distance to reaction x_{bar} = 675 mm

Eccentricity of reaction e = 0 mm

Reaction acts within middle third of base

Bearing pressure at toe p_{toe} = 68.8 kN/m^2 Bearing pressure at heel p_{heel} = 68.8 kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall F_{prop_top} = -1.792 kN/m Propping force to base of wall F_{prop_base} = 5.157 kN/m

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RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = 1.4 Live load factor γ_{f_l} = 1.6

Earth pressure factor γ_{f_e} = 1.4

Calculate propping force

Propping force F_{prop} = 3.4 kN/m

Calculate propping forces to top and base of wall

Propping force to top of wall $F_{prop\_top\_f}$ = -0.474 kN/m Propping force to base of wall $F_{prop\_base\_f}$ = 9.630 kN/m

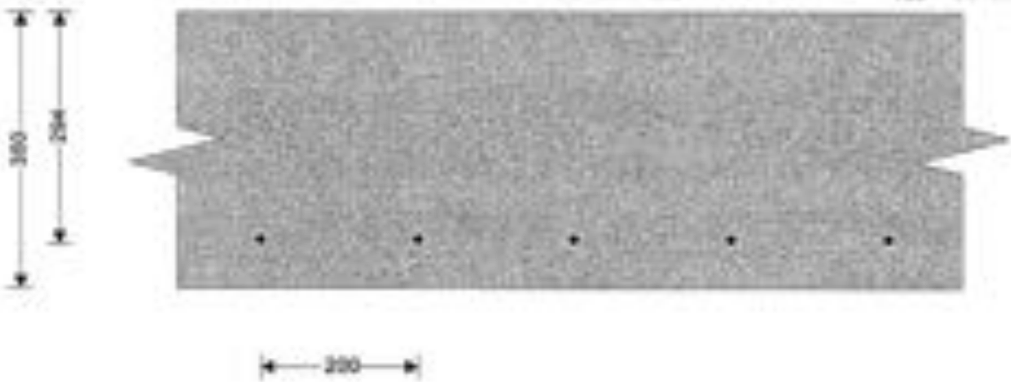
Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = 40 N/mm^2 Strength of reinforcement f_y = 500 N/mm^2

Base details

Minimum reinforcement k = 0.13 % Cover in toe c_{toe} = 50 mm



Design of retaining wall toe

Shear at heel V_{toe} = 84.7 kN/m Moment at heel M_{toe} = 58.5 kNm/m

Compression reinforcement is not required

Check toe in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_toe\_req}$ = 481.4 mm^2/m Area provided $A_{s\_toe\_prov}$ = 565 mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress v_{toe} = 0.288 N/mm^2 Allowable shear stress v_{adm} = 5.000 N/mm^2

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_toe} = 0.461 N/mm^2

v_{toe} < v_{c_toe} - ***No shear reinforcement required***

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Strength of concrete f_{cu} = 40 N/mm^2 Strength of reinforcement f_y = 500 N/mm^2

Wall details

Minimum reinforcement k = 0.13 %

Cover in stem c_{stem} = 50 mm Cover in wall c_{wall} = 50 mm

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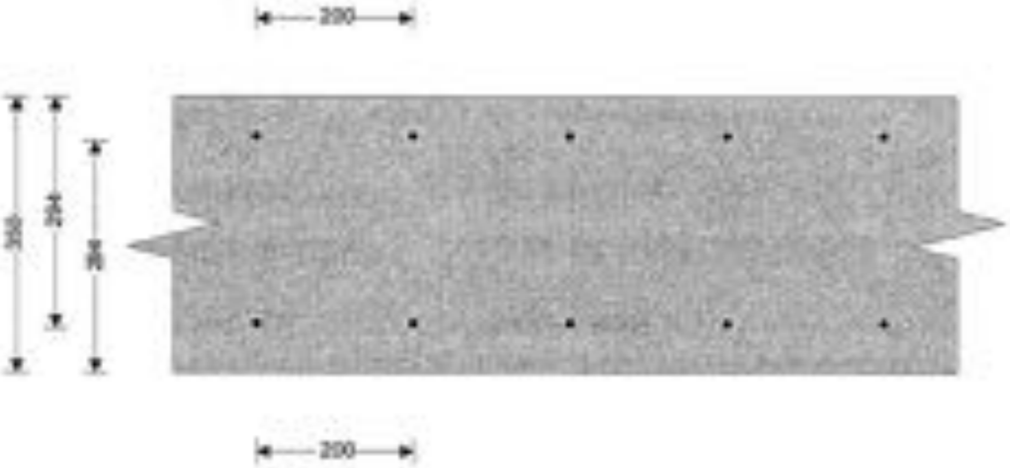

Design of retaining wall stem

Shear at base of stem V_{stem} = **31.7** kN/m Moment at base of stem M_{stem} = **13.1** kNm/m

Compression reinforcement is not required

Check wall stem in bending

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_stem\_req}$ = **455.0** mm²/m Area provided $A_{s\_stem\_prov}$ = **565** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress v_{stem} = **0.108** N/mm² Allowable shear stress v_{adm} = **5.000** N/mm²

PASS - Design shear stress is less than maximum shear stress

Concrete shear stress v_{c_stem} = **0.461** N/mm²

v_{stem} < v_{c_stem} - No shear reinforcement required

Design of retaining wall at mid height

Moment at mid height M_{wall} = **6.7** kNm/m

Compression reinforcement is not required

Reinforcement provided **12 mm dia.bars @ 200 mm centres**

Area required $A_{s\_wall\_req}$ = **455.0** mm²/m Area provided $A_{s\_wall\_prov}$ = **565** mm²/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Max span/depth ratio $ratio_{max}$ = **40.00** Actual span/depth ratio $ratio_{act}$ = **6.46**

PASS - Span to depth ratio is acceptable

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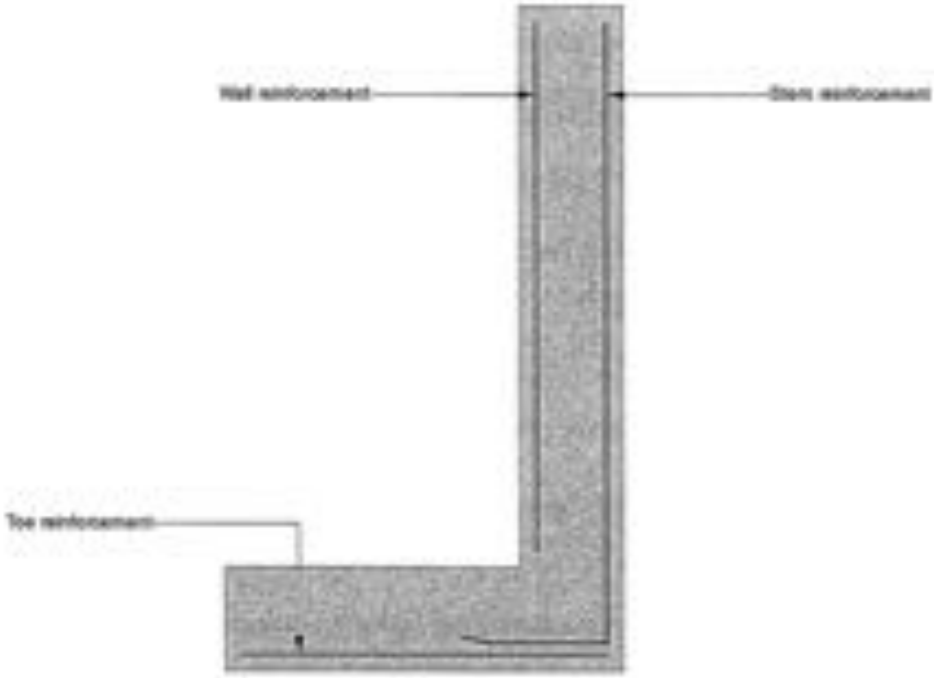
Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Wall bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)
Stem bars - 12 mm dia.@ 200 mm centres - (565 mm^2/m)

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B:- <u>Permanent condition</u>

Max soil pressure = $0.43 \times 2.25 \times 19 = 18.4 \text{ kN/m}^2$ (sls)

Surcharge pressure = $0.43 \times 28 = 12.0 \text{ kN/m}^2$

Max water pressure = $1.0 \times 10 = 10.0 \text{ kN/m}^2$

$\therefore F_{soil} = \frac{18.4}{2} \times 2.25 = 20.7 \text{ kN/m}$ of wall (sls)

$F_{sur} = 2.25 \times 12.0 = 27.0 \text{ kN/m}$

$F_{water} = \frac{10.0}{2} \times 1.0 = 5 \text{ kN/m}$

$\therefore \Sigma M = \left(20.7 \times \frac{2.25}{3}\right) + \left(27.0 \times \frac{2.25}{2}\right) + \left(5.0 \times \frac{1.0}{3}\right)$

$= 15.5 + 30.4 + 1.7 = 47.6 \text{ kNm/m}$ of wall (sls)

$\times 1.6$

76.1 kNm/m (uls)

b = 1000
h = 250
cover = 50
d = 200
$f_{cu} = 40$
$f_y = 460$

$K = M/bd^2 f_{cu} = 0.024$

$\Rightarrow Z = 0.95d = 266\text{mm}$

$\therefore A_s \text{ req'd} = \frac{M}{0.95 f_y Z} = 656 \text{ mm}^2/\text{m}$

<u>H12 @ 150 c/c Vert, Outer face</u> $\Rightarrow A_{s,prov} = 754 \text{ mm}^2/\text{m}$

$\Sigma F = V = 20.7 + 27.0 + 5.0 = 52.7 \text{ kN/m}$ (sls)

$\times 1.6$

84.3 kN/m (uls)

b = 1000
d = 280

$v = V/bd = 0.30 \text{ N/mm}^2$

$\left.\begin{array}{r} \frac{100 A_s}{bd} = \frac{754 \times 100}{1000 \times 280} = 0.27 \\ d > 280 \end{array}\right\}$ T3.9 $\Rightarrow v_c = 1.17 \times 0.45$

$= 0.53 > v$

$\therefore$ <u>No Shear Links req'd.</u>

Deflection: $\text{span}/\text{depth} = \frac{1900}{280} = 6.8 < 7 \therefore$ okay.

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Basement Slab

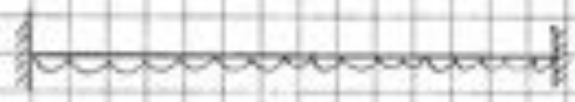
Consider as encastre, between the basement walls.

Max width of basement ≈ 6.6m between walls
∴ Design span ≈ 7.0m

From GEA report, residual upwards pressure due to unloading of the ground could result in a load of 60% of 50 kN/m²

ie design for 30 kN/m² upwards load

Allow also 1.5m head of water at end nearest the front but no water pressure in the rear half of the site.
ie 15 kN/m²



Σ UDL = 45 kN/m² (SLS)
×1.6
72 kN/m² (ULS)

See Tedds analysis: for design to BS 8110

350 r.c. slab with 50 cover top & bottom

H16 @ 100 c/c Top & Bottom

+ Shear legs/m at 200 c/c across the basement

Secondary steel H10 @ 200 c/c Top & Bottom

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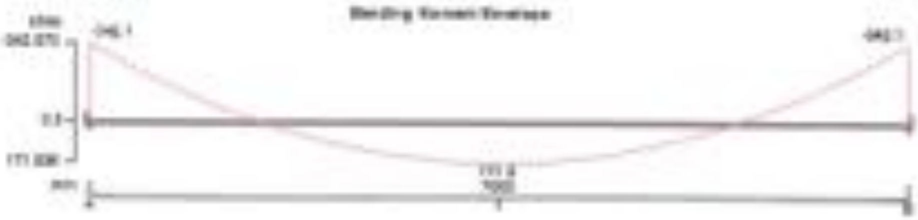
RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.12



Support conditions

Support A — Vertically restrained; Rotationally restrained

Support B — Vertically restrained; Rotationally restrained

Applied loading

Dead self weight of beam × 1

Span 1 loads — Imposed UDL 15.000 kN/m from 0 mm to 7000 mm; Imposed UDL 30.000 kN/m from 0 mm to 7000 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment support A — M_{s_max} = -342 kNm — M_{s_red} = -342 kNm

Maximum moment span 1 at 3500 mm M_{s1_max} = 171 kNm M_{s1_red} = 171 kNm
Maximum moment support B M_{B_max} = -342 kNm M_{B_red} = -342 kNm
Maximum shear support A V_{A_max} = 293 kN V_{A_red} = 293 kN
Maximum shear support A span 1 at 300 mm $V_{A\_s1\_max}$ = 268 kN $V_{A\_s1\_red}$ = 268 kN
Maximum shear support B V_{B_max} = -293 kN V_{B_red} = -293 kN
Maximum shear support B span 1 at 6700 mm $V_{B\_s1\_max}$ = -268 kN $V_{B\_s1\_red}$ = -268 kN
Maximum reaction at support A R_A = 293 kN
Maximum reaction at support B R_B = 293 kN

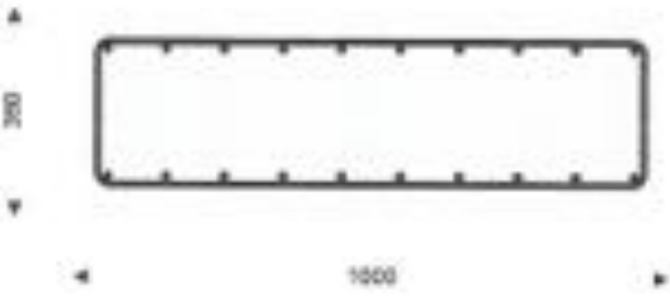
Rectangular section details

Section width b = 1000 mm Section depth h = 350 mm

Material details

Concrete strength class **C40/50** Char comp cube strength f_{cu} = 50 N/mm²
Modulus of elasticity of conc E_c = 30000 N/mm² Maximum aggregate size h_{agg} = 20 mm
Char yield strength of reinf f_y = 500 N/mm² Char yield str of shear reinf f_{yv} = 500 N/mm²
Nominal cover to top reinf c_{nom_t} = 50 mm Nominal cover to bottom reinf c_{nom_b} = 50 mm
Nominal cover to side reinf c_{nom_s} = 35 mm

Mid span 1



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment M = 171 kNm Depth to tension reinf d = 282 mm
K = 0.043 K' = 0.156
K' > K - No compression reinforcement is required

Lever arm z = 268 mm Depth of neutral axis x = 32 mm
Area of tension reinf req'd $A_{s,req}$ = 1468 mm² Tension reinf provided 10 × 16φ bars
Area of tension reinf prov $A_{s,prov}$ = 2011 mm² Minimum area of reinf $A_{s,min}$ = 455 mm²
Maximum area of reinf $A_{s,max}$ = 14000 mm²
PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear

Shear reinforcement provided 4 × 10ϕ legs at 200 c/c

Area of shear reinf provided $A_{sv,prov}$ = **1571** mm²/m Minimum area of shear reinf $A_{sv,min}$ = **920** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing $s_{vl,max}$ = **212** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars s = **83** mm Min dist between bars s_{min} = **25** mm

PASS - Satisfies the minimum spacing criteria

Design service stress f_s = **243.4** N/mm² Max distance between bars s_{max} = **193** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Span to depth ratio (T.3.9) $span\_to\_depth_{basic}$ = **20.0** Service stress in tension rein f_s = **243.4** N/mm²

Modification for tension reinf f_{tens} = **1.188** Modification for comp reinf f_{comp} = **1.192**

Modification for span > 10m f_{long} = **1.000** Allowable span to depth ratio $span\_to\_depth_{allow}$ = **28.3**

Actual span to depth ratio $span\_to\_depth_{actual}$ = **24.8**

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

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PARTY WALL LOADINGS

A EXISTING.

Roof, attic/ceiling joists, 1st floor & ground floor all span parallel to the Party Wall

∴ nominal loads from each

Roof : DL = 1.2 kN/m² on plan

ceiling DL = 0.75

1st DL 0.6
Partitions 1.0

Gd DL 0.6
Partitions 1.0

Σ = 4.75 kN/m²

Allow for, say, 0.3 m of contribution

⇒ UDL = 1.4 kN/m (SLS)

Which will be removed

Contributions from adjoining property will be similar.

Wall self wt = ?

Allow for 9" solid from Upper Gd to 1st to Roof @ 5 kN/m²

& 13½" solid from Lower Gd to Upper Gd @ 7.5 kN/m²

Ave ht from Upper Gd to Roof = 7.8m ⇒ 39.0 kN/m

ht from Lwr to Upper G = 3m, but 4m to foundations

⇒ 22.5 kN/m

⇒ Foundation DL = 61.5 kN/m

⇒ Σ foundation load = 61.5 + (2 × 1.4) = 64.3 kN/m (SLS)

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As Existing

Assessed Weight of House :-

See Demolitions Report.

Total weight of materials = ?

Recycled	27.3
Re-used	5.1
Waste	6.1
	38.5 tonnes
External Walls	60.1
	Σ = 98.6 tonnes

Area of House ≈ 9.7 × 7.0 = 67.9 m²

⇒ Ave = 98.6 / 67.9 = 1.5 tonnes/m² average

≈ 14.2 kN/m² Dead load only

Check:-

Roof DL ≈	1.2 kN/m²
Ceiling	0.15
1st	0.6
Upper Gd	0.6
	2.55 kN/m²
Partitions at 1st	1.0
Partitions at Gd	1.0
	4.55 kN/m²

External Walls:

Front Facade: 5 kN/m² × 9 m × 65% solid × 7 m long
⇒ 159 kN

Rear Facade: Similar ⇒ 159 kN

Flank Wall: 6.3 kN/m² ave × 8 m long × 11 m ave high
⇒ 554 kN

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∴ Total wt of external walls
≑ 554 + 159 + 159
≑ 872 kN

Area of footprint ≑ 68 m²

⇒ Ave = 12.8 kN/m²

∴ Σ Ave DL pressure = 12.8 + 4.35
= <u>17.2 kN/m²</u> To be Removed. (conservative, as rear facade already almost fully removed).

Assessed Weight of Rear Extension:

Roof	23.2 tonnes
Upper Gd	20.6
Lower Gd	20.6
Base Slab	15.5
Walls	73.6

Σ = 153.5 tonnes

Area ≑ 42 m² ⇒ 3.7 tonnes/m² average

≡ 35.9 kN/m² ave

Note: Rear extension carried to two Ground Beams approx 6m × 1m

$$\Rightarrow \frac{153.5}{2} \times \frac{1}{(6 \times 1)} = 12.8 \text{ tonnes/m}^2$$

≡ <u>126 kN/m²</u>

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As PROPOSED.

New roof & floors will span parallel to Party Wall, onto load-bearing walls. These walls will be carried on r.c. transfer beams at lower Ground floor level, spanning across the width of the Basement, onto the basement walls.

Therefore allow for half of the house weight to be applied onto each basement wall.

Roof, DL ≈ 1.4 kN/m² on plan

Total area ≈ 14 × 7 = 98m² ⇒ 137 kN

2nd, DL & Partitions
0.6
1.0
1.6 kN/m²

Area ≈ 9×7 = 63m² ⇒ 101 kN

1st, DL & Partitions
0.6
1.0
1.6 kN/m²

Area ≈ 10×7 = 70m² ⇒ 112 kN

Ground, DL & Partitions
0.6
1.5
2.1 kN/m²

Area ≈ 14×7 = 98m² ⇒ 206 kN

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Lower Ground, DL $\doteq$ 0.4 + (24 × 0.225) = 5.8 kN/m²

Area $\doteq$ 15.5 × 7 = 108m² ⇒ 629 kN

Block walls: 3m × 2kN/m² × 20m ⇒ <u>120 kN</u>

Σ = 749 kN

Garden Floor; DL $\doteq$ 0.35 + (24 × 0.3) = 7.5 kN/m²

Area $\doteq$ 108m² to main house ⇒ 805 kN

Block walls: 3m × 36m × 2kN/m² ⇒ <u>216</u>

Σ = 1021 kN

∴ Σ house loads (excluding front & rear facades & flank walls)

= 1021 + 749 + 206 + 112 + 101 + 137

= 2326 kN (DL) carried on opposite side walls

i.e. 1163 kN on 15.5m of wall

≈ 75 kN/m onto each side wall.

Flank Walls DL:

Allow for 4.5 kN/m²

Ave height $\doteq$ 11m ⇒ 50 kN/m

<u>Flank underpinning</u> DL:

Allow 0.5m × 24 kN/m² × 3m ⇒ 36 kN/m

⇒ Σ on Flank wall foundation

= 36 + 50 + 75 = <u>161 kN/m (DL)</u>

<u>Party Wall underpin</u> DL $\doteq$ 36 kN/m

⇒ Σ = (69.3 - 2 + 4) + 36 + 75 = <u>177 kN/m (DL)</u>

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<u>Front Facade, DL:</u>

Allow $0.5\,kN/m^2 \times 7m \times 9m \times 0.85$ solid

$\Rightarrow$ 184 kN on transfer beam

$\Rightarrow$ 92 kN onto Flank & PW Ugirs with 45° spread onto foundations

$\Rightarrow \frac{92}{3.8m} \Rightarrow 24\,kN/m$ ave

$\therefore$ allow for $177 + 24 = \underline{201\,kN/m}$ ave onto <u>Party Wall founds</u>

& $161 + 24 = \underline{185\,kN/m}$ ave onto <u>Flank Wall founds</u>