NOTES

The structural drawing and calculations will need to be approved by a suitable checking Authority such as Building Control. This approval must be sought by the Client/Contractor prior to any works taking place or any materials being ordered.

DESIGN INFORMATION / PHILOSOPHY

It is proposed to construct a new dwelling at a site off Apethorn Lane, Gerrards Hollow, Gee-Cross, Hyde.

The proposed development will comprise of a 3 storey dwelling. The lower ground floor will include an area for parking which is independent to the new dwelling.

The lower ground floor itself will include a double garage, W.C , Utility, store and a staircase to access the upper storey.

The ground floor consists of a living room, dining/kitchen and a hall which has the staircases which facilitate the 1st floor and the lower ground floor. The ground floor has a terrace which is directly above the parking area.

The first floor includes 3 bedrooms , 1 with en-suite, a bathroom and storage area.

The dwelling will be built into a steep embankment and therefore will require an appropriate retaining structure in order for the dwelling to be constructed in a safe manner. This retaining structure will be installed independantly from the new dwelling.

We are not responsible for the quality of workmanship. This exercise is to justify the retaining structure only.

A slope stability assessment has been carried out by Ashton Bennett Consultancy (Engineering Geologists & Environmental Scientists). The report was conducted to clarify the ground stability for the proposed development.

These calculations cover a design check of the permanent contiguous bored piled retaining wall structure together with the capacity calculations for the support piles.

Contigous Bored Piled Retaining wall check (Wall A)

Details of the proposed retaining structure have been taken from the drawings supplied by Northern Design, project reference no. 2221, drawing no. 07.

One section of wall has been identified for this analysis, labelled Section A-A on C23.07.32 Sketch01

The section has been chosen to represent the general soil and structural criteria which is applicable to the requirements of the retaining wall.

SECTION A-A :
The retaining structure consists of a line of 450mm diameter reinforced concrete auger piles, 7.5m deep socketed into bedrock. These are installed in close proximity to the existing retaining wall to the footpath of Apethorn Lane. The piles are reinforced with T16 bars and connected into a continuous ring beam (capping beam) which is reinforced with T16 bars also. This section of wall has been installed at this level to provide enhanced retaining measures to the existing retaining wall to the footpath.

There has been a line of 600mm diameter reinforced concrete auger piles installed lower down the slope, these are also 7.5m deep and are socketed (1.5m approx.) into bedrock which return the lower section of ground.

A 500mm thick R.C slab has been installed which acts as a capping beam to the lower line of CFA piles and spans across to the higher line of CFA piles. The slab is reinforced with 2 layers of A393 mesh top and bottom. The slab provides support for a line of Stone faced lego blocks. The lego blocks have been connected back into the piles via rebar hoops.

The stone faced lego blocks are installed in an interconnected sequence and are (6 rows high) which is 3.6m retained height.

A 10kN/m² surcharge will be considered due to vehicular traffic (Apethorn Lane)

Ground conditions:
The borehole sample test results show that the clay is on medium to high strength from 1.0m – 5.0m depth and very high strength at 6.0m. (Details can be found on page 8 (Table 2 of the Slope Stability Assessment).

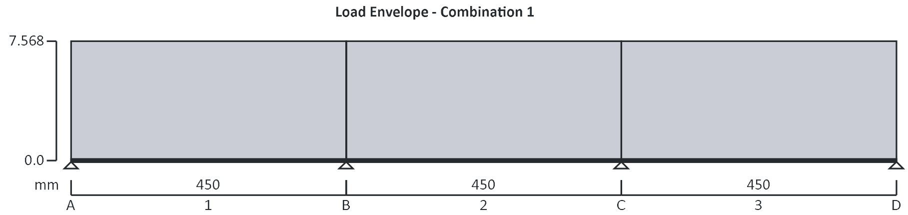
Groundwater was not encountered during the drilling of the borehole except as small seepages.

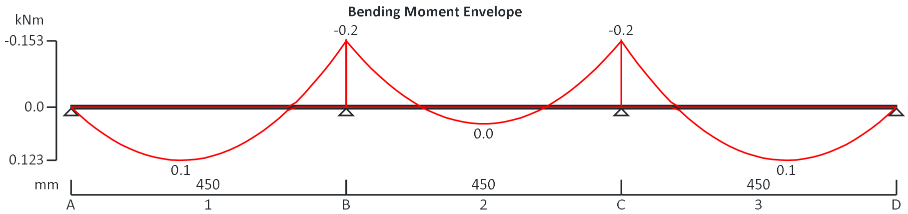
The wall will have a proprietary drain installed at the back of the wall to relieve any hydrostatic pressure.

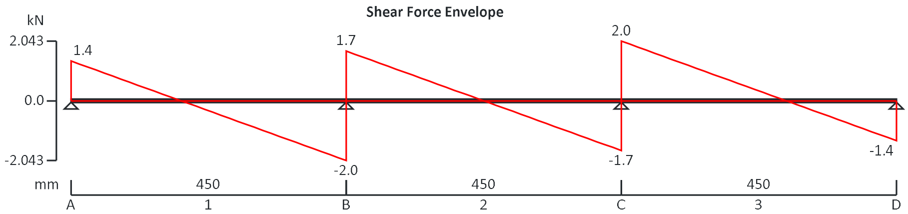
Design of capping beam on 450 dia. CFA pile

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.14







Support conditions

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

Applied loading

SWT of capping beam	Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.40

Ref: **Calculations** **Output**

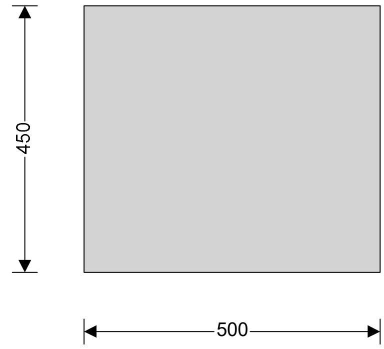
		Imposed × 1.60
	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
	Span 2	Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
	Span 3	Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment support A;	M_{A_max} = **0** kNm;	M_{A_red} = **0** kNm;
Maximum moment span 1 at 180 mm;	M_{s1_max} = **0** kNm;	M_{s1_red} = **0** kNm;
Maximum moment support B;	M_{B_max} = **-0** kNm;	M_{B_red} = **-0** kNm;
Maximum moment span 2 at 225 mm;	M_{s2_max} = **0** kNm;	M_{s2_red} = **0** kNm;
Maximum moment support C;	M_{C_max} = **-0** kNm;	M_{C_red} = **-0** kNm;
Maximum moment span 3 at 270 mm;	M_{s3_max} = **0** kNm;	M_{s3_red} = **0** kNm;
Maximum moment support D;	M_{D_max} = **0** kNm;	M_{D_red} = **0** kNm;
Maximum shear support A;	V_{A_max} = **1** kN;	V_{A_red} = **1** kN
Maximum shear support A span 1 at 384 mm;	$V_{A\_s1\_max}$ = **-2** kN;	$V_{A\_s1\_red}$ = **-2** kN
Maximum shear support B;	V_{B_max} = **-2** kN;	V_{B_red} = **-2** kN
Maximum shear support B span 1 at 66 mm;	$V_{B\_s1\_max}$ = **1** kN;	$V_{B\_s1\_red}$ = **1** kN
Maximum shear support B span 2 at 384 mm;	$V_{B\_s2\_max}$ = **-1** kN;	$V_{B\_s2\_red}$ = **-1** kN
Maximum shear support C;	V_{C_max} = **2** kN;	V_{C_red} = **2** kN
Maximum shear support C span 2 at 66 mm;	$V_{C\_s2\_max}$ = **1** kN;	$V_{C\_s2\_red}$ = **1** kN
Maximum shear support C span 3 at 384 mm;	$V_{C\_s3\_max}$ = **-1** kN;	$V_{C\_s3\_red}$ = **-1** kN
Maximum shear support D;	V_{D_max} = **-1** kN;	V_{D_red} = **-1** kN
Maximum shear support D span 3 at 66 mm;	$V_{D\_s3\_max}$ = **2** kN;	$V_{D\_s3\_red}$ = **2** kN
Maximum reaction at support A;	R_A = **1** kN	
Maximum reaction at support B;	R_B = **4** kN	
Maximum reaction at support C;	R_C = **4** kN	
Maximum reaction at support D;	R_D = **1** kN	

Rectangular section details

Section width; b = **500** mm; Section depth; h = **450** mm



Material details

Concrete strength class; **C30/37**; Char comp cube strength; f_{cu} = **37** N/mm^2

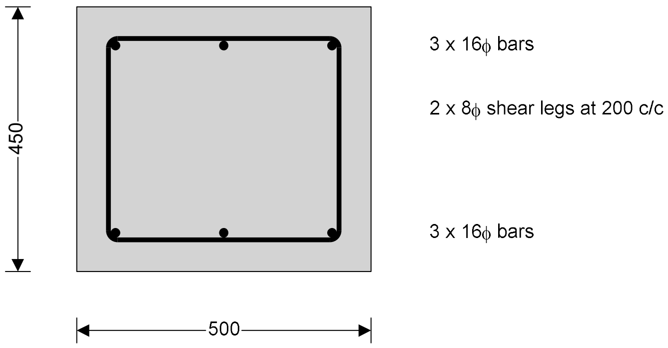
Modulus of elasticity of conc; E_c = **27400** N/mm^2; Maximum aggregate size; h_{agg} = **20** mm

Char yield strength of reinf; f_y = **500** N/mm^2; Char yield str of shear reinf; f_{yv} = **500** N/mm^2

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} = **50** mm

<u>Support A</u>



Rectangular section in shear

Shear - span 1 at 384 mm; V = **2** kN; Shear stress; v = **0.009** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

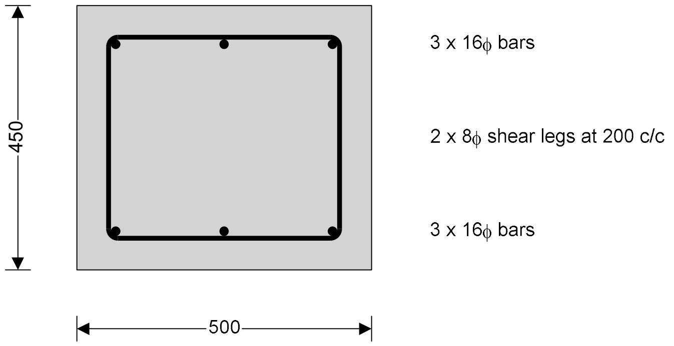
Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Gamble, King and Noone Limited Consulting Structural Engineers & Building Surveyors 5 Queen Street, Oldham. OL1 1RD. Tel – 0161 652 1183 www.gknltd.co.uk	Project New Dwelling at Gerrards Hollow, Gee Cross. SK14 5DT			Job Ref. C23.07.32	
	Section Structural Calculations			Sheet no./rev. 6	
	Calc. by S.King	Date Nov. 2023	Chk'd by	Date	App'd by	Date

Ref: **Calculations** **Output**

Mid span 1



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; M = **0** kNm; Depth to tension reinf.; d = **384** mm

K = **0.000**; K' = **0.156**

K' > K - No compression reinforcement is required

Lever arm; z = **365** mm; Depth of neutral axis; x = **43** mm

Area of tension reinf req'd; $A_{s,req}$ = **1** mm^2; Tension reinf provided; 3 × 16ϕ bars

Area of tension reinf prov; $A_{s,prov}$ = **603** mm^2; Minimum area of reinf; $A_{s,min}$ = **293** mm^2

Maximum area of reinf; $A_{s,max}$ = **9000** mm^2

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

Rectangular section in shear

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c

Area of shear reinf provided; $A_{sv,prov}$ = **503** mm^2/m; Minimum area of shear reinf; $A_{sv,min}$ = **460** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars; s = **168** mm; Min dist between bars; s_{min} = **25** mm

PASS - Satisfies the minimum spacing criteria

Design service stress; f_s = **0.4** N/mm^2; Max distance between bars; s_{max} = **300** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Span to depth ratio (T.3.9); $span\_to\_depth_{basic}$ = **26.0**; Service stress in tension rein; f_s = **0.4** N/mm^2

Modification for tension reinf; f_{tens} = **2.000**; Modification for comp reinf; f_{comp} = **1.095**

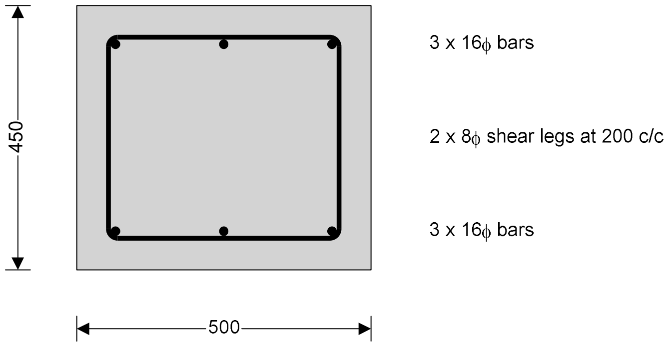
Modification for span > 10m; f_{long} = **1.000**; Allowable span to depth ratio; $span\_to\_depth_{allow}$ = **56.9**

Actual span to depth ratio; $span\_to\_depth_{actual}$ = **1.2**

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

Ref: **Calculations** **Output**

Support B



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; M = **0** kNm; Depth to tension reinf.; d = **384** mm

K = **0.000**; K' = **0.156**

K' > K - No compression reinforcement is required

Lever arm; z = **365** mm; Depth of neutral axis; x = **43** mm

Area of tension reinf req'd; $A_{s,req}$ = **1** mm^2; Tension reinf provided; 3 × 16ϕ bars

Area of tension reinf prov; $A_{s,prov}$ = **603** mm^2; Minimum area of reinf; $A_{s,min}$ = **293** mm^2

Maximum area of reinf; $A_{s,max}$ = **9000** mm^2

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

Rectangular section in shear

Shear - span 1 at 66 mm; V = **1** kN; Shear stress; v = **0.005** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Shear - span 2 at 384 mm; V = **1** kN; Shear stress; v = **0.007** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars; s = **168** mm; Min dist between bars; s_{min} = **25** mm

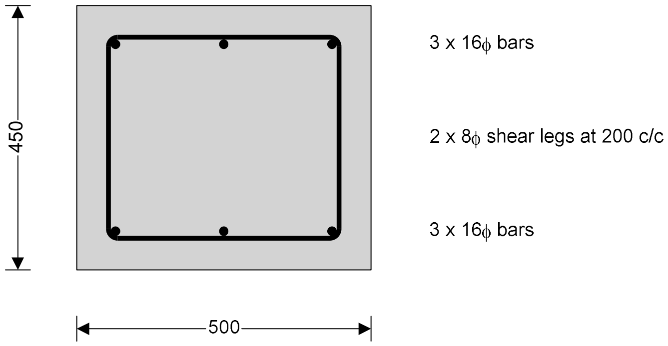
PASS - Satisfies the minimum spacing criteria

Design service stress; f_s = **0.5** N/mm^2; Max distance between bars; s_{max} = **300** mm

PASS - Satisfies the maximum spacing criteria

Ref:	Calculations	Output

Mid span 2



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; M = **0** kNm; Depth to tension reinf.; d = **384** mm

K = **0.000**; K' = **0.156**

K' > K - No compression reinforcement is required

Lever arm; z = **365** mm; Depth of neutral axis; x = **43** mm

Area of tension reinf req'd; $A_{s,req}$ = **0** mm^2; Tension reinf provided; 3 × 16ϕ bars

Area of tension reinf prov; $A_{s,prov}$ = **603** mm^2; Minimum area of reinf; $A_{s,min}$ = **293** mm^2

Maximum area of reinf; $A_{s,max}$ = **9000** mm^2

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

Rectangular section in shear

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c

Area of shear reinf provided; $A_{sv,prov}$ = **503** mm^2/m; Minimum area of shear reinf; $A_{sv,min}$ = **460** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars; s = **168** mm; Min dist between bars; s_{min} = **25** mm

PASS - Satisfies the minimum spacing criteria

Design service stress; f_s = **0.1** N/mm^2; Max distance between bars; s_{max} = **300** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Span to depth ratio (T.3.9); $span\_to\_depth_{basic}$ = **26.0**; Service stress in tension rein; f_s = **0.1** N/mm^2

Modification for tension reinf; f_{tens} = **2.000**; Modification for comp reinf; f_{comp} = **1.095**

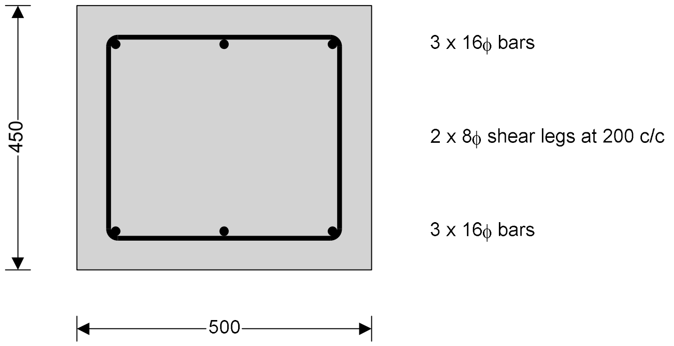
Modification for span > 10m; f_{long} = **1.000**; Allowable span to depth ratio; $span\_to\_depth_{allow}$ = **56.9**

Actual span to depth ratio; $span\_to\_depth_{actual}$ = **1.2**

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

Ref: **Calculations** **Output**

Support C



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; M = **0** kNm; Depth to tension reinf.; d = **384** mm

K = **0.000**; K' = **0.156**

K' > K - No compression reinforcement is required

Lever arm; z = **365** mm; Depth of neutral axis; x = **43** mm

Area of tension reinf req'd; $A_{s,req}$ = **1** mm^2; Tension reinf provided; 3 × 16ϕ bars

Area of tension reinf prov; $A_{s,prov}$ = **603** mm^2; Minimum area of reinf; $A_{s,min}$ = **293** mm^2

Maximum area of reinf; $A_{s,max}$ = **9000** mm^2

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

Rectangular section in shear

Shear - span 2 at 66 mm; V = **1** kN; Shear stress; v = **0.007** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Shear - span 3 at 384 mm; V = **1** kN; Shear stress; v = **0.005** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars; s = **168** mm; Min dist between bars; s_{min} = **25** mm

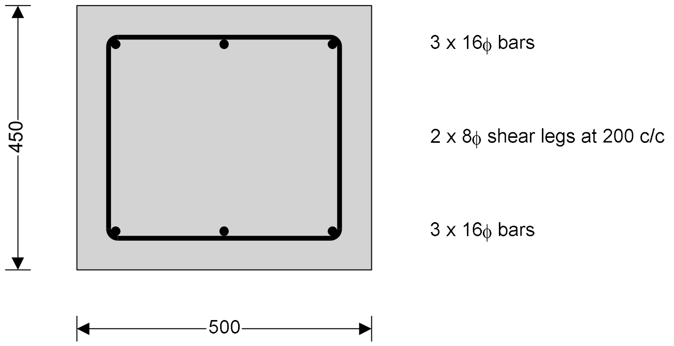
PASS - Satisfies the minimum spacing criteria

Design service stress; f_s = **0.5** N/mm^2; Max distance between bars; s_{max} = **300** mm

PASS - Satisfies the maximum spacing criteria

Ref: **Calculations** **Output**

Mid span 3



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; M = **0** kNm; Depth to tension reinf.; d = **384** mm

K = **0.000**; K' = **0.156**

K' > K - No compression reinforcement is required

Lever arm; z = **365** mm; Depth of neutral axis; x = **43** mm

Area of tension reinf req'd; $A_{s,req}$ = **1** mm^2; Tension reinf provided; 3 × 16ϕ bars

Area of tension reinf prov; $A_{s,prov}$ = **603** mm^2; Minimum area of reinf; $A_{s,min}$ = **293** mm^2

Maximum area of reinf; $A_{s,max}$ = **9000** mm^2

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

Rectangular section in shear

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c

Area of shear reinf provided; $A_{sv,prov}$ = **503** mm^2/m; Minimum area of shear reinf; $A_{sv,min}$ = **460** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual dist between bars; s = **168** mm; Min dist between bars; s_{min} = **25** mm

PASS - Satisfies the minimum spacing criteria

Design service stress; f_s = **0.4** N/mm^2; Max distance between bars; s_{max} = **300** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Span to depth ratio (T.3.9); $span\_to\_depth_{basic}$ = **26.0**; Service stress in tension rein; f_s = **0.4** N/mm^2

Modification for tension reinf; f_{tens} = **2.000**; Modification for comp reinf; f_{comp} = **1.095**

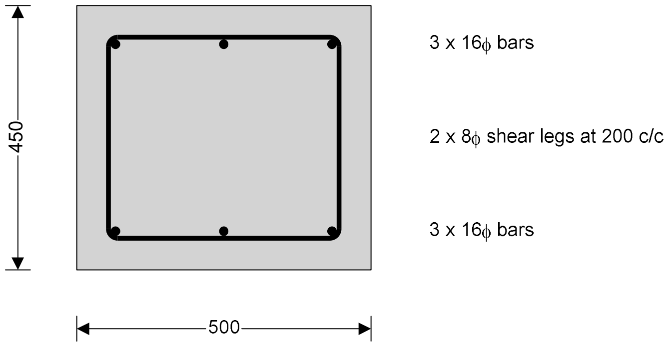
Modification for span > 10m; f_{long} = **1.000**; Allowable span to depth ratio; $span\_to\_depth_{allow}$ = **56.9**

Actual span to depth ratio; $span\_to\_depth_{actual}$ = **1.2**

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

Ref: **Calculations** **Output**

Support D



Rectangular section in shear

Shear - span 3 at 66 mm; V = **2** kN; Shear stress; v = **0.009** N/mm^2

Allowable design shear stress; v_{max} = **4.866** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $v < 0.5v_c$

Design shear resistance req'd; v_s = **0.400** N/mm^2; Area of shear reinf req'd; $A_{sv,req}$ = **460** mm^2/m

Shear reinforcement provided; 2 × 8ϕ legs at 200 c/c; Area of shear reinf. prov; $A_{sv,prov}$ = **503** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Max longitudinal spacing; $s_{vl,max}$ = **288** mm

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

;

500mm wide x 450mm deep R.C with Min. 3No.T16 bars top and bottom capping beam is adequate

Check 450mm dia. CHS unreinforced from lateral loads

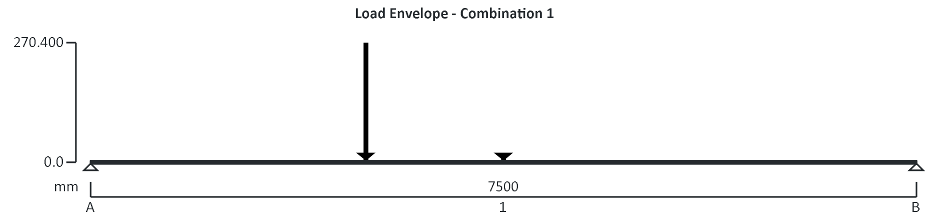
CHS section to be checked without reinforcement and concrete worst case.

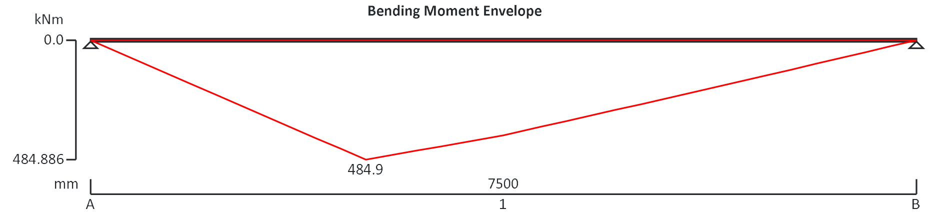
Pile will be checked as simply supported due to restraint at bottom via pile base being socketed into bedrock and continuous R.C capping beam installed at head of pile.

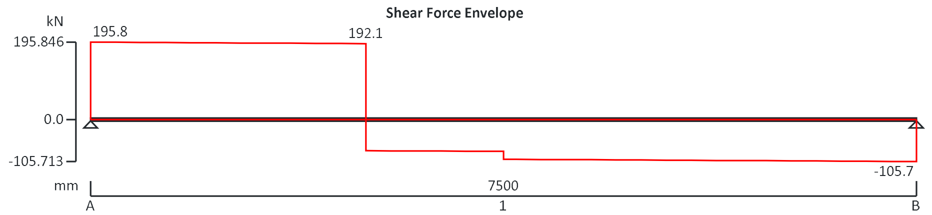
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
	Rotationally free

Ref:	Calculations	Output

Support B — Vertically restrained
Rotationally free

Applied loading

Beam loads — Dead self weight of beam × 1
PL from Surcharge - Imposed point load 12.38 kN at 3750 mm
PL from Soil - Imposed point load 169 kN at 2500 mm

Load combinations

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

Analysis results

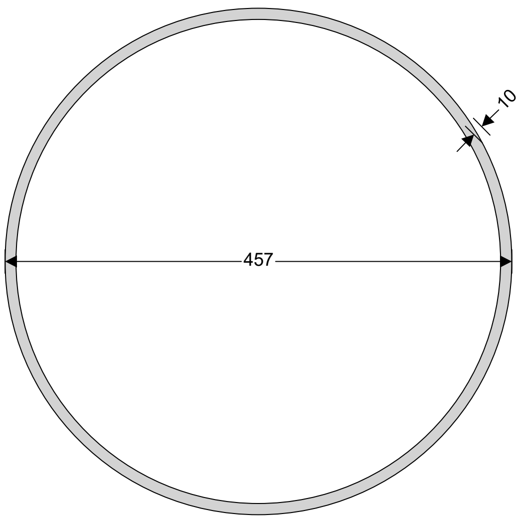
Maximum moment;	M_{max} = **484.9** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **195.8** kN;	V_{min} = **-105.7** kN
Deflection;	δ_{max} = **19.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **195.8** kN;	R_{A_min} = **195.8** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **4.1** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **118.9** kN	
Maximum reaction at support B;	R_{B_max} = **105.7** kN;	R_{B_min} = **105.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **4.1** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **62.5** kN	

Section details

Section type; **CHS 457.0x10.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))**
Steel grade; **S275**

From table 9: Design strength p_y

Thickness of element; t = **10.0** mm
Design strength; p_y = **275** N/mm^2
Modulus of elasticity; E = **205000** N/mm^2



Ref: **Calculations** **Output**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Tubular sections - Table 12

$D / t = 45.7 \times \varepsilon <= 50 \times \varepsilon^2$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = max(abs(V_{max}), abs(V_{min}))$ = **195.8** kN

Shear area; $A_v = 0.6 \times A$ = **8426** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **1390.2** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **484.9** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = min(p_y \times S, 1.2 \times p_y \times Z)$ = **506.8** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **20.833** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **19.877** mm

PASS - Maximum deflection does not exceed deflection limit

450mm diameter steel pile is suitable for lateral loads worst case.

Ref: **Calculations** **Output**

Check 450mm dia. Pile for compressive and tensile resistance.

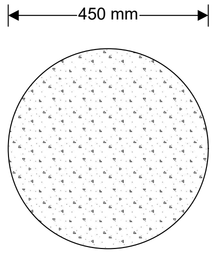
PILE ANALYSIS

In accordance with EN 1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

Tedds calculation version 1.0.08

Design summary

Description	Unit	Actual	Allowable	Utilisation	Result
Axial, compression	kN	4	479	0.008	PASS
Axial, tension	kN	185.1	378.7	0.489	PASS



Pile details

Installation method; Drilled; Shape; 450 mm diameter

Length; L = **7500** mm

Material details

Material; Concrete; Concrete strength class; C30/37

Partial safety factor, concrete; γ_C = **1.50**; Coefficient α_{cc} ; α_{cc} = **0.85**

Characteristic compression cylinder strength;

f_{ck} = **30** N/mm$^{2;}$ Design compressive strength; f_{cd} = **17.0** N/mm^2

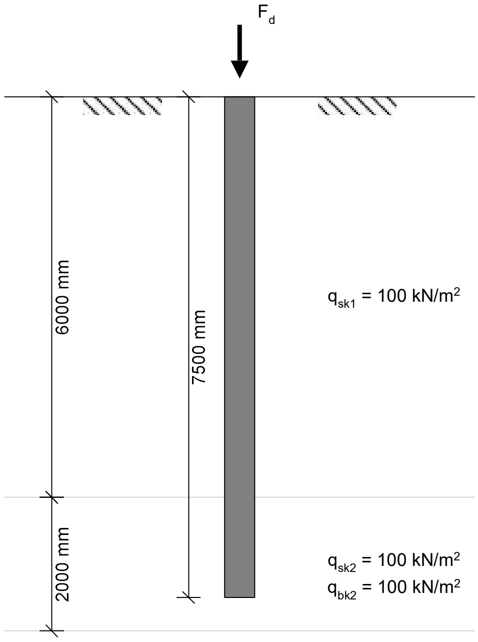
Mean value, cylinder strength; f_{cm} = **38.0** N/mm^2; Secant modulus of elasticity; E_{cm} = **32.8** kN/mm^2

Modulus of elasticity; E = E_{cm} = **32.8** kN/mm^2

Geometric properties

Bearing area; $A_{bearing}$ = **0.159** m^2; Pile perimeter; $Perim_{pile}$ = **1.414** m

Moment of inertia; I = **201289** cm^4



q_{ski} = Characteristic value, shaft resistance, q_{bki} = Characteristic value, base

Action details

Compression:

Char. perm. unfav. action; $G_{c,k,unfav}$ = **4** kN; Char. perm. fav. action; $G_{c,k,fav}$ = **0** kN;

Char. variable unfav action; $Q_{c,k}$ = **0** kN

Tension:

Char. perm. unfav. action; $G_{t,k,unfav}$ = **169** kN; Char. perm. fav. action; $G_{t,k,fav}$ = **0** kN

Char. variable unfav action; $Q_{t,k}$ = **12.4** kN

Geotechnical partial and model factors:

Design approach 1;

Model factor on comp. resist.; γ_{model} = **1.40**; Model factor on tens. resist.; $\gamma_{model,t}$ = **1.40**

Permanent unfavourable, A1; $\gamma_{G,unfav,A1}$ = **1.35**; Permanent favourable, A1; $\gamma_{G,fav,A1}$ = **1.00**

Variable unfavourable, A1; $\gamma_{Q,A1}$ = **1.50**

Permanent unfavourable, A2; $\gamma_{G,unfav,A2}$ = **1.00**; Permanent favourable, A2; $\gamma_{G,fav,A2}$ = **1.00**

Variable unfavourable, A2; $\gamma_{Q,A2}$ = **1.30**

Characteristic axial resistance

Charact. axial base resistance; R_{bk} = **15.9** kN; Charact. axial shaft resistance; R_{sk} = **1060.3** kN

Axial compressive resistance

Load combination 1: A1 + M1 + R1

Design compression action; $F_{c,d,C1}$ = **5.4** kN

Partial resist. factor, bearing; $\gamma_{b,R1}$ = **1.00**; Partial resist. factor, shaft; $\gamma_{s,R1}$ = **1.00**

Design compr. resistance; $R_{c,d,C1}$ = **768.7** kN; $F_{c,d,C1}$ / $R_{c,d,C1}$ = **0.007**

PASS - Design compressive resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design compression action; $F_{c,d,C2}$ = **4** kN

Partial resist. factor, bearing; $\gamma_{b,R4}$ = **2.00**; Partial resist. factor, shaft; $\gamma_{s,R4}$ = **1.60**

Design compr. resistance; $R_{c,d,C2}$ = **479** kN; $F_{c,d,C2}$ / $R_{c,d,C2}$ = **0.008**

PASS - Design compressive resistance exceeds design load

Axial tensile resistance

Load combination 1: A1 + M1 + R1

Design tension load; $F_{t,d,C1}$ = **246.8** kN; Part. resist. factor, shaft tens.; $\gamma_{s,t,R1}$ = **1**

Design tensile resistance; $R_{t,d,C1}$ = **757.3** kN; $F_{t,d,C1}$ / $R_{t,d,C1}$ = **0.326**

PASS - Design tensile resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design tension load; $F_{t,d,C2}$ = **185.1** kN; Part. resist. factor, shaft tens.; $\gamma_{s,t,R4}$ = **2**

Design tensile resistance; $R_{t,d,C2}$ = **378.7** kN; $F_{t,d,C2}$ / $R_{t,d,C2}$ = **0.489**

PASS - Design tensile resistance exceeds design load

;

450mm diameter CFA pile is suitable for purpose.

Check 3.6m High Lego Block Retaining Wall

Lego block dimensions = 1500mm long x 600mm wide x 600mm deep

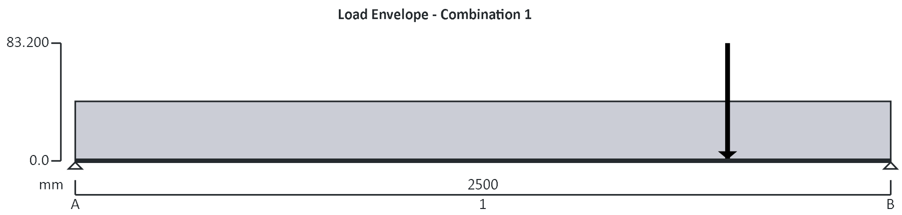
Lego blocks have been installed onto a 500mm R.C concrete slab.

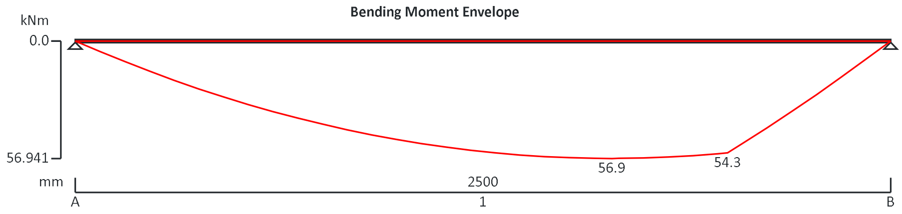
The lego blocks are not required to provide any retaining resistance due to the fact the CFA system has been installed directly behind. This has then been filled with no fines concrete with A393 fabric mesh. The lego block is acting as a shutter and has been installed upto a height of 3600mm. (6 rows of 600mm blocks) the sequence of installation should be verifyed by the Contractor who has installed them. By inspection the lego blocks are suitable for purpose. The 500mm thick slab will be checked for suitablity to support the self weight of the lego blocks.

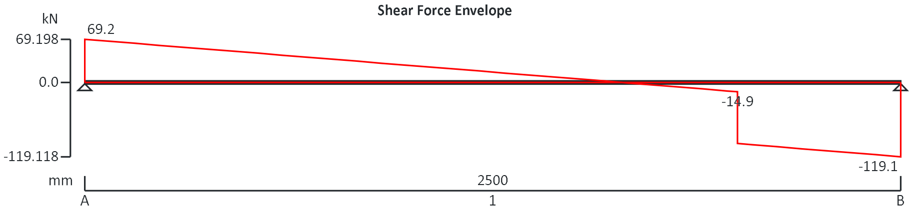
Check 500mm R.C slab – A393 mesh top and bottom

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.14







Support conditions

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

Applied loading

	Dead self weight of beam × 1
PL from Lego Block	Imposed point load 52 kN at 2000 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} = **0** kNm;	M_{A_red} = **0** kNm;
Maximum moment span 1 at 1646 mm;	M_{s1_max} = **57** kNm;	M_{s1_red} = **57** kNm;
Maximum moment support B;	M_{B_max} = **0** kNm;	M_{B_red} = **0** kNm;
Maximum shear support A;	V_{A_max} = **69** kN;	V_{A_red} = **69** kN
Maximum shear support A span 1 at 440 mm;	$V_{A\_s1\_max}$ = **50** kN;	$V_{A\_s1\_red}$ = **50** kN
Maximum shear support B;	V_{B_max} = **-119** kN;	V_{B_red} = **-119** kN
Maximum shear support B span 1 at 2050 mm;	$V_{B\_s1\_max}$ = **-100** kN;	$V_{B\_s1\_red}$ = **-100** kN
Maximum reaction at support A;	R_A = **69** kN	
Unfactored dead load reaction at support A;	R_{A_Dead} = **38** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **10** kN	
Maximum reaction at support B;	R_B = **119** kN	
Unfactored dead load reaction at support B;	R_{B_Dead} = **38** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **42** kN	

Rectangular section details

Section width;	b = **2500** mm
Section depth;	h = **500** mm

;

Ref: **Calculations** **Output**

Check as simply supported slab

RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

CONCRETE SLAB DESIGN (CL 3.5.3 & 4)

SIMPLE ONE WAY SPANNING SLAB DEFINITION

; Overall depth of slab; h = **500** mm

; Cover to tension reinforcement resisting sagging; c_b = **40** mm

; Trial bar diameter; D_{tryx} = **10** mm

Depth to tension steel (resisting sagging)

$d_x = h - c_b - D_{tryx}/2 =$ **455** mm

; Characteristic strength of reinforcement; f_y = **500** N/mm^2

; Characteristic strength of concrete; f_{cu} = **35** N/mm^2

ONE WAY SPANNING SLAB (CL 3.5.4)

MAXIMUM DESIGN MOMENTS IN SPAN

; Design sagging moment (per m width of slab); m_{sx} = **57.0** kNm/m

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

; Design sagging moment (per m width of slab); m_{sx} = **57.0** kNm/m

; Moment Redistribution Factor; β_{bx} = **1.0**

Area of reinforcement required

;; $K_x = abs(m_{sx}) / (d_x^2 \times f_{cu}) =$ **0.008**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2)) =$ **0.156**

Outer compression steel not required to resist sagging

One-way Spanning Slab requiring tension steel only (sagging) - mesh

;; $z_x = \min ((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)}))) =$ **432** mm

Neutral axis depth; $x_x = (d_x - z_x) / 0.45 =$ **51** mm

Area of tension steel required

;;; $A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x) =$ **303** mm^2/m

Tension steel

;;Use A393 Mesh;

$A_{sx\_prov} = A_{sl} =$ **393** mm^2/m; $A_{sy\_prov} = A_{st} =$ **393** mm^2/m

$D_x = d_{sl} =$ **10** mm; $D_y = d_{st} =$ **10** mm

Area of tension steel provided sufficient to resist sagging

Check min and max areas of steel resisting sagging

;Total area of concrete; $A_c = h =$ **500000** mm^2/m

; Minimum % reinforcement; k = **0.13** %

$A_{st\_min} = k \times A_c =$ **650** mm²/m

$A_{st\_max} = 4\ \% \times A_c =$ **20000** mm²/m

Steel defined:

; Outer steel resisting sagging; A_{sx_prov} = 786 mm²/m

Less than min area of outer steel (sagging) OKL

; Inner steel resisting sagging; A_{sy_prov} = 786 mm²/m

Less than min area of inner steel (sagging) OKL

Slab mesh has been nested top and bottom therefore ok.

SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)

Outer tension steel resisting sagging moments

; Depth to tension steel from compression face; d_x = **455** mm

; Area of tension reinforcement provided (per m width of slab); A_{sx_prov} = **393** mm²/m

; Design ultimate shear force (per m width of slab); V_x = **119** kN/m

; Characteristic strength of concrete; f_{cu} = **35** N/mm²

Applied shear stress

$v_x = V_x / d_x =$ **0.26** N/mm²

Check shear stress to clause 3.5.5.2

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **4.73** N/mm²

Shear stress - OK

Shear stresses to clause 3.5.5.3

Design shear stress

$f_{cu\_ratio} = \text{if}(f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.400**

$v_{cx} = 0.79\ N/mm^2 \times \min(3, 100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67, (400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

v_{cx} = **0.30** N/mm²

Applied shear stress

v_x = **0.26** N/mm²

No shear reinforcement required

SHEAR PERIMETERS FOR A RECTANGULAR CONCENTRATED LOAD (CL 3.7.7)

; Length of loaded rectangle; l = **600** mm

; Width of loaded rectangle; w = **1000** mm

; Depth to tension steel; d_x = **455** mm

; Dimension from edge of load to shear perimeter; $l_p = k_p \times d_x$ = **683** mm; where; k_p = **1.50**

For punching shear cases not affected by free edges or holes:

Total length of inner perimeter at edge of loaded area; $u_{0\_gen} = 2 \times (l + w)$ = **3200** mm

Total length of outer perimeter at l_p from loaded area; $u_{gen} = 2 \times (l + w) + 8 \times l_p$ = **8660** mm

PUNCHING SHEAR AT CONCENTRATED LOADS (CL 3.7.7)

Tension steel resisting sagging

; Total length of inner perimeter at edge of loaded area; u_0 = **3200** mm

Ref: **Calculations** **Output**

; Total length of outer perimeter at dimension l_p from loaded area; u = **8660** mm

; Depth to outer steel; d_x = **455** mm

; Depth to inner steel; d_y = **45** mm

Average depth to "tension" steel; $d_{av} = (d_x + d_y)/2$ = **250.0** mm

; Area of outer steel per m effective through the perimeter; A_{sx_prov} = **393** mm^2 /m

; Area of inner steel per m effective through the perimeter; A_{sy_prov} = **393** mm^2 /m

; Max shear effective across either perimeter under consideration; V_p = **119** kN

; Characteristic strength of concrete; f_{cu} = **35** N/mm^2

Applied shear stress

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av})$ = **0.149** N/mm^2

Stress around perimeter; $v = V_p / (u \times d_{av})$ = **0.055** N/mm^2

Check shear stress to clause 3.7.7.2

$v_{allowable}$ = min ((0.8 N$^{1/2}$/mm) × √(f_{cu}), 5 N/mm^2) = **4.733** N/mm^2

Shear stress - OK

Shear stresses to clause 3.7.7.4

Design shear stress

f_{cu_ratio} = if (f_{cu} > 40 N/mm^2 , 40/25 , f_{cu}/(25 N/mm^2)) = **1.400**

; Effective steel area for shear strength determination:; A_{s_eff} =**393** mm^2/m;

v_c = 0.79 N/mm^2 × min(3, 100×(A_{s_eff} / d_{av}))$^{1/3}$ × max(0.67, (400 mm / d_{av})$^{1/4}$) / 1.25 × $f_{cu\_ratio}^{1/3}$

v_c = **0.429** N/mm^2

No shear reinforcement required

<u>CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)</u>

; Slab span length; l_x = **2.500** m

; Design ultimate moment in shorter span per m width; m_{sx} = **57** kNm/m

; Depth to outer tension steel; d_x = **455** mm

Tension steel

; Area of outer tension reinforcement provided; A_{sx_prov} = **393** mm^2/m

; Area of tension reinforcement required; A_{sx_req} = **303** mm^2/m

; Moment Redistribution Factor; β_{bx} = **1.00**

Modification Factors

;Basic span / effective depth ratio (Table 3.9); $ratio_{span\_depth}$ = **20**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

;$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **257.2** N/mm^2

$factor_{tens}$ = min (2 , 0.55 + (477 N/mm^2 - f_s) / (120 × (0.9 N/mm^2 + m_{sx} / d_x^2))) = **2.000**

Calculate Maximum Span

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span; $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **18.20** m

Ref: **Calculations** **Output**

Check the actual beam span

Actual span/depth ratio; l_x / d_x = **5.49**

Span depth limit; $\text{ratio}_{span\_depth} \times \text{factor}_{tens}$ = **40.00**

Span/Depth ratio check satisfied

CHECK OF NOMINAL COVER (SAGGING) – (BS8110:PT 1, TABLE 3.4)

; Slab thickness; h = **500** mm

; Effective depth to bottom outer tension reinforcement; d_x = **455.0** mm

; Diameter of tension reinforcement; D_x = **10** mm

; Diameter of links; L_{diax} = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **40.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **40.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; c_{min} = **40** mm

Cover over steel resisting sagging OK

;

500mm Thick R.C slab with 2xA393 mesh top and bottom (40mm cover) is adequate to support stone faced lego blocks (3.6m high)

Check 600mm dia. CHS unreinforced from lateral loads

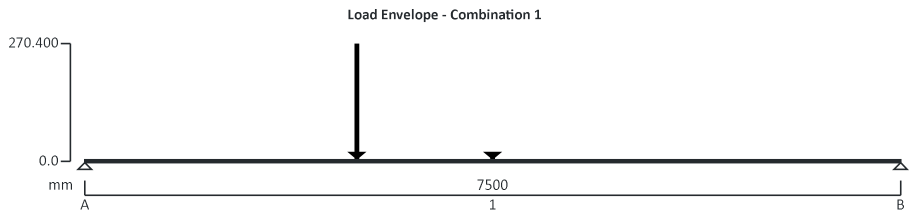
CHS section to be checked without reinforcement and concrete worst case. (508 x 10 CHS checked based on closest wall thickness)

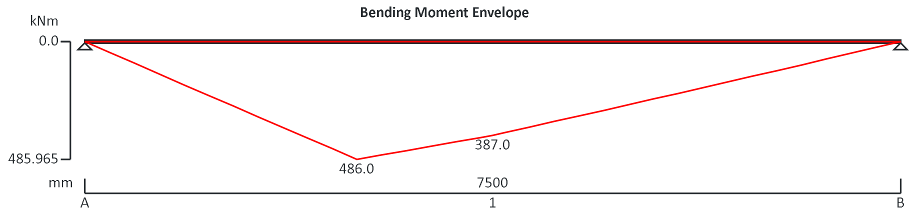
Pile will be checked as simply supported due to restraint at bottom via pile base being socketed into bedrock and continuous R.C capping beam installed at head of pile.

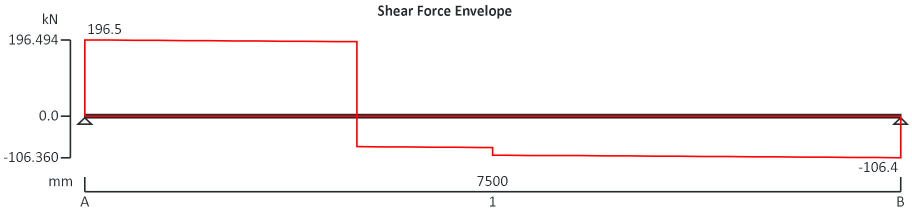
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
Rotationally free

Ref: **Calculations** **Output**

Support B — Vertically restrained; Rotationally free

Applied loading

Beam loads — Dead self weight of beam × 1

PL from Surcharge - Imposed point load 12.38 kN at 3750 mm

PL from Soil - Imposed point load 169 kN at 2500 mm

Load combinations

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

Analysis results

Maximum moment;	M_{max} = **486** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **196.5** kN;	V_{min} = **-106.4** kN
Deflection;	δ_{max} = **14.4** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **196.5** kN;	R_{A_min} = **196.5** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **4.5** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **118.9** kN	
Maximum reaction at support B;	R_{B_max} = **106.4** kN;	R_{B_min} = **106.4** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **4.5** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **62.5** kN	

Section details

Section type; **CHS 508.0x10.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))**

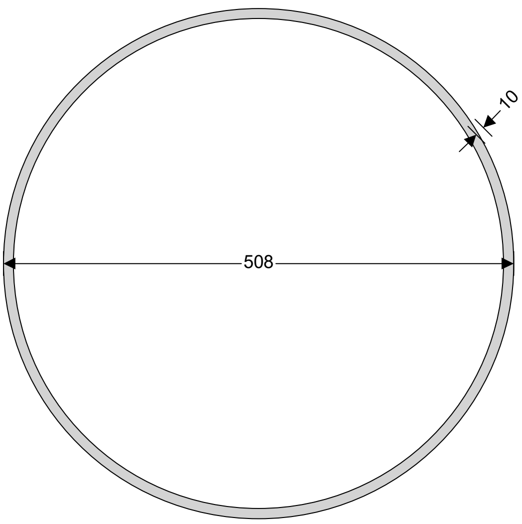
Steel grade; **S275**

From table 9: Design strength p_y

Thickness of element; t = **10.0** mm

Design strength; p_y = **275** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2



Ref: **Calculations** **Output**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Tubular sections - Table 12

$D / t = 50.8 \times \varepsilon <= 140 \times \varepsilon^2$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **196.5** kN

Shear area; $A_v = 0.6 \times A$ = **9387** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **1548.9** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **486** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = 10 \times \varepsilon$ = **10**

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 15 \times \varepsilon$ = **15**

Limiting value for class 2 compact web; $\beta_{2w} = 100 \times \varepsilon$ = **100**

Limiting value for class 3 semi-compact web; $\beta_{3w} = 120 \times \varepsilon$ = **120**

Effective plastic modulus - cl.3.5.6.4

$S_{eff} = \min(Z + 1.485 \times (S - Z) \times [\sqrt{[(140 / (D / t)) \times (275 \text{ N/mm}^2 / p_y)]} - 1], S)$ = **2469106** mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z)$ = **630.4** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **20.833** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **14.427** mm

PASS - Maximum deflection does not exceed deflection limit

600mm diameter steel pile is suitable for lateral loads worst case.

Check 600mm dia. Pile for compressive and tensile resistance.

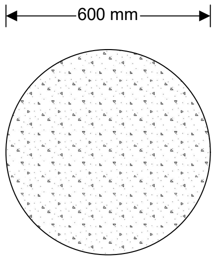
PILE ANALYSIS

In accordance with EN 1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

Tedds calculation version 1.0.08

Design summary

Description	Unit	Actual	Allowable	Utilisation	Result
Axial, compression	kN	119	641.2	0.186	PASS
Axial, tension	kN	185.1	504.9	0.367	PASS



Pile details

Installation method; Drilled; Shape; 600 mm diameter

Length; L = **7500** mm

Material details

Material; Concrete; Concrete strength class; C30/37

Partial safety factor, concrete; γ_C = **1.50**; Coefficient α_{cc} ; α_{cc} = **0.85**

Characteristic compression cylinder strength; f_{ck} = **30** N/mm^2; Design compressive strength; f_{cd} = **17.0** N/mm^2

Mean value, cylinder strength; f_{cm} = **38.0** N/mm^2; Secant modulus of elasticity; E_{cm} = **32.8** kN/mm^2

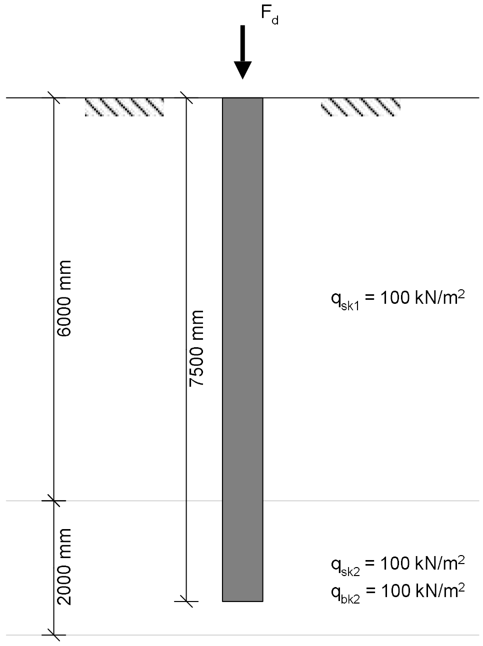
Modulus of elasticity; E = E_{cm} = **32.8** kN/mm^2

Geometric properties

Bearing area; $A_{bearing}$ = **0.283** m^2; Pile perimeter; $Perim_{pile}$ = **1.885** m

Moment of inertia; I = **636173** cm^4

Ref: **Calculations** **Output**



q_{ski} = Characteristic value, shaft resistance, q_{bki} = Characteristic value, base

Action details

Compression:

Char. perm. unfav. action; $G_{c,k,unfav}$ = **119** kN; Char. perm. fav. action; $G_{c,k,fav}$ = **0** kN;

Char. variable unfav action; $Q_{c,k}$ = **0** kN

Tension:

Char. perm. unfav. action; $G_{t,k,unfav}$ = **169** kN; Char. perm. fav. action; $G_{t,k,fav}$ = **0** kN

Char. variable unfav action; $Q_{t,k}$ = **12.4** kN

Geotechnical partial and model factors:

Design approach 1;

Model factor on comp. resist.; γ_{model} = **1.40**; Model factor on tens. resist.; $\gamma_{model,t}$ = **1.40**

Permanent unfavourable, A1; $\gamma_{G,unfav,A1}$ = **1.35**; Permanent favourable, A1; $\gamma_{G,fav,A1}$ = **1.00**

Variable unfavourable, A1; $\gamma_{Q,A1}$ = **1.50**

Permanent unfavourable, A2; $\gamma_{G,unfav,A2}$ = **1.00**; Permanent favourable, A2; $\gamma_{G,fav,A2}$ = **1.00**

Variable unfavourable, A2; $\gamma_{Q,A2}$ = **1.30**

Characteristic axial resistance

Charact. axial base resistance; R_{bk} = **28.3** kN; Charact. axial shaft resistance; R_{sk} = **1413.7** kN

Axial compressive resistance

Load combination 1: A1 + M1 + R1

Design compression action; $F_{c,d,C1}$ = **160.7** kN

Partial resist. factor, bearing; $\gamma_{b,R1}$ = **1.00**; Partial resist. factor, shaft; $\gamma_{s,R1}$ = **1.00**

Design compr. resistance; $R_{c,d,C1}$ = **1030** kN; $F_{c,d,C1}$ / $R_{c,d,C1}$ = **0.156**

PASS - Design compressive resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design compression action; $F_{c,d,C2}$ = **119** kN

Partial resist. factor, bearing; $\gamma_{b,R4}$ = **2.00**; Partial resist. factor, shaft; $\gamma_{s,R4}$ = **1.60**

Ref: **Calculations** **Output**

Design compr. resistance; $R_{c,d,C2}$ = **641.2** kN; $F_{c,d,C2}$ / $R_{c,d,C2}$ = **0.186**

PASS - Design compressive resistance exceeds design load

Axial tensile resistance

Load combination 1: A1 + M1 + R1

Design tension load; $F_{t,d,C1}$ = **246.8** kN; Part. resist. factor, shaft tens.; $\gamma_{s,t,R1}$ = **1**

Design tensile resistance; $R_{t,d,C1}$ = **1009.8** kN; $F_{t,d,C1}$ / $R_{t,d,C1}$ = **0.244**

PASS - Design tensile resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design tension load; $F_{t,d,C2}$ = **185.1** kN; Part. resist. factor, shaft tens.; $\gamma_{s,t,R4}$ = **2**

Design tensile resistance; $R_{t,d,C2}$ = **504.9** kN; $F_{t,d,C2}$ / $R_{t,d,C2}$ = **0.367**

PASS - Design tensile resistance exceeds design load

;

600mm diameter CFA pile is suitable for purpose.

Design of new reinforced concrete retaining wall (WALL A) (No vertical load scenario)

Max. height of retaining wall = 3.5m Max. retained height.

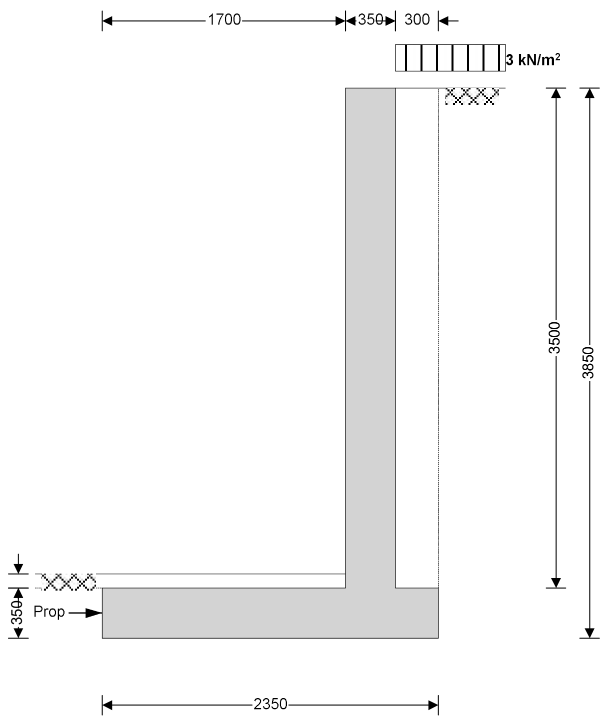
2.5kN/m² Surcharge (Garden) worst case

The wall will have a proprietary drain installed at the back of the wall to relieve any hydrostatic pressure.

The base has been designed as propped at the bottom due to the construction a new concrete floor slab.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type;	**Cantilever propped at base**
Height of retaining wall stem;	h_{stem} = **3500** mm
Thickness of wall stem;	t_{wall} = **350** mm
Length of toe;	l_{toe} = **1700** mm
Length of heel;	l_{heel} = **300** mm
Overall length of base;	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2350** mm
Thickness of base;	t_{base} = **350** mm
Depth of downstand;	d_{ds} = **0** mm
Position of downstand;	l_{ds} = **1600** mm

Thickness of downstand; t_{ds} = **350** mm

Height of retaining wall; $h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3850** mm

Depth of cover in front of wall; d_{cover} = **100** mm

Depth of unplanned excavation; d_{exc} = **100** mm

Height of ground water behind wall; h_{water} = **0** mm

Height of saturated fill above base; $h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **0** mm

Density of wall construction; γ_{wall} = **23.6** kN/m^3

Density of base construction; γ_{base} = **23.6** kN/m^3

Angle of rear face of wall; α = **90.0** deg

Angle of soil surface behind wall; β = **0.0** deg

Effective height at virtual back of wall; $h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3850** mm

Retained material details

Mobilisation factor; M = **1.5**

Moist density of retained material; γ_m = **18.0** kN/m^3

Saturated density of retained material; γ_s = **21.0** kN/m^3

Design shear strength; ϕ' = **25.0** deg

Angle of wall friction; δ = **19.3** deg

Base material details

Firm clay

Moist density; γ_{mb} = **18.0** kN/m^3

Design shear strength; ϕ'_b = **24.2** deg

Design base friction; δ_b = **18.6** deg

Allowable bearing pressure; $P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.358}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 - \sin(\phi')$ = **0.577**

Loading details

Surcharge load on plan; Surcharge = **2.5** kN/m^2

Applied vertical dead load on wall; W_{dead} = **0.0** kN/m

Applied vertical live load on wall; W_{live} = **0.0** kN/m

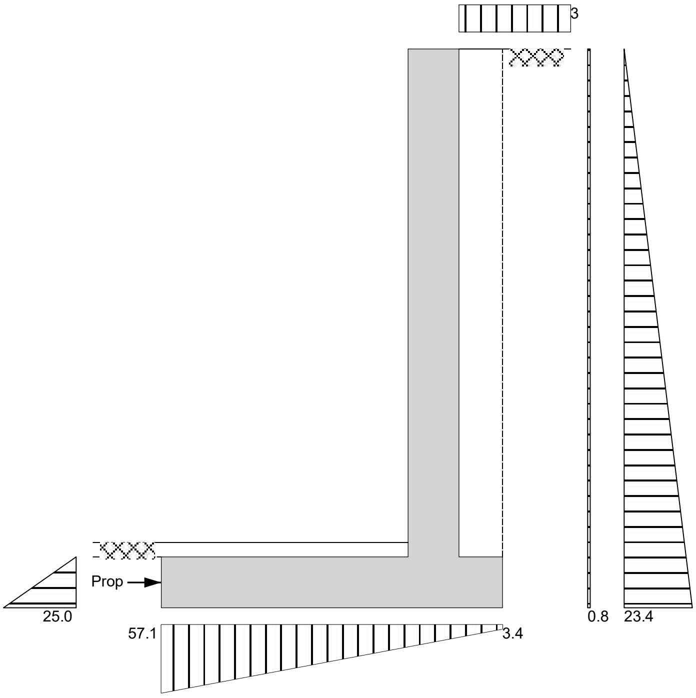
Position of applied vertical load on wall; l_{load} = **0** mm

Applied horizontal dead load on wall; F_{dead} = **0.0** kN/m

Applied horizontal live load on wall; F_{live} = **0.0** kN/m

Height of applied horizontal load on wall; h_{load} = **0** mm

Ref: **Calculations** **Output**



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

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.9** kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **19.4** kN/m

Surcharge; w_{sur} = Surcharge $\times\ l_{heel}$ = **0.8** kN/m

Moist backfill to top of wall; $w_{m\_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **18.9** kN/m

Soil in front of wall; $w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **3.1** kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + w_{sur} + w_{m\_w} + w_p$ = **71** kN/m

Horizontal forces on wall

Surcharge; $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times$ Surcharge $\times\ h_{eff}$ = **3.3** kN/m

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **45.1** kN/m

Total horizontal load; $F_{total} = F_{sur} + F_{m\_a}$ = **48.3** kN/m

Calculate propping force

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4** kN/m

Propping force; $F_{prop} = \max(F_{total} - F_p - (W_{total} - w_{sur} - w_p) \times \tan(\delta_b), 0\ \text{kN/m})$

F_{prop} = **21.3** kN/m

Overturning moments

Surcharge; $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **6.3** kNm/m

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **57.8** kNm/m

Total overturning moment; $M_{ot} = M_{sur} + M_{m\_a}$ = **64.1** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **54.2** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **22.8** kNm/m

Moist backfill; $M_{m\_r} = (w_{m\_w} \times (l_{base} - l_{heel} / 2) + w_{m\_s} \times (l_{base} - l_{heel} / 3))$ = **41.6** kNm/m

Ref: **Calculations** **Output**

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{m\_r} = \mathbf{118.6}$ kNm/m

Check bearing pressure

Surcharge; $M_{sur\_r} = w_{sur} \times (l_{base} - l_{heel} / 2) = \mathbf{1.7}$ kNm/m

Soil in front of wall; $M_{p\_r} = w_p \times l_{toe} / 2 = \mathbf{2.6}$ kNm/m

Total moment for bearing; $M_{total} = M_{rest} - M_{ot} + M_{sur\_r} + M_{p\_r} = \mathbf{58.7}$ kNm/m

Total vertical reaction; $R = W_{total} = \mathbf{71.0}$ kN/m

Distance to reaction; $x_{bar} = M_{total} / R = \mathbf{827}$ mm

Eccentricity of reaction; $e = abs((l_{base} / 2) - x_{bar}) = \mathbf{348}$ mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) = \mathbf{57.1}$ kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) = \mathbf{3.4}$ kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Ref: | **Calculations** | **Output**

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor; γ_{f_d} = **1.4**

Live load factor; γ_{f_l} = **1.6**

Earth and water pressure factor; γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **40.5** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **27.2** kN/m

Surcharge; $w_{sur\_f} = \gamma_{f\_l} \times \text{Surcharge} \times l_{heel}$ = **1.2** kN/m

Moist backfill to top of wall; $w_{m\_w\_f} = \gamma_{f\_d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **26.5** kN/m

Soil in front of wall; $w_{p\_f} = \gamma_{f\_d} \times l_{toe} \times d_{cover} \times \gamma_{mb}$ = **4.3** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{sur\_f} + w_{m\_w\_f} + w_{p\_f}$ = **99.6** kN/m

Factored horizontal at-rest forces on wall

Surcharge; $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **8.9** kN/m

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **107.8** kN/m

Total horizontal load; $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **116.7** kN/m

Calculate propping force

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **6.1** kN/m

Propping force; $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - w_{sur\_f} - w_{p\_f}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop_f} = **78.9** kN/m

Factored overturning moments

Surcharge; $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **17.1** kNm/m

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **138.4** kNm/m

Total overturning moment; $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **155.5** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **75.9** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **31.9** kNm/m

Surcharge; $M_{sur\_r\_f} = w_{sur\_f} \times (l_{base} - l_{heel} / 2)$ = **2.6** kNm/m

Moist backfill; $M_{m\_r\_f} = (w_{m\_w\_f} \times (l_{base} - l_{heel} / 2) + w_{m\_s\_f} \times (l_{base} - l_{heel} / 3))$ = **58.2** kNm/m

Soil in front of wall; $M_{p\_r\_f} = w_{p\_f} \times l_{toe} / 2$ = **3.6** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{sur\_r\_f} + M_{m\_r\_f} + M_{p\_r\_f}$ = **172.3** kNm/m

Factored bearing pressure

Total moment for bearing; $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **16.8** kNm/m

Total vertical reaction; $R_f = W_{total\_f}$ = **99.6** kN/m

Distance to reaction; $x_{bar\_f} = M_{total\_f} / R_f$ = **169** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **1006** mm

Reaction acts outside middle third of base

Bearing pressure at toe; $p_{toe\_f} = R_f / (1.5 \times x_{bar\_f})$ = **393.4** kN/m^2

Bearing pressure at heel; p_{heel_f} = 0 kN/m^2 = **0** kN/m^2

Rate of change of base reaction; $\text{rate} = p_{toe\_f} / (3 \times x_{bar\_f})$ = **776.86** kN/m^2/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **0** kN/m^2

Ref: | **Calculations** | **Output**

Bearing pressure at mid stem; $p_{stem\_mid\_f} = max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) =$ **0** kN/m^2

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) =$ **0** kN/m^2

<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in toe; c_{toe} = **30** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} / 2$ = **99.6** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **19.7** kN/m

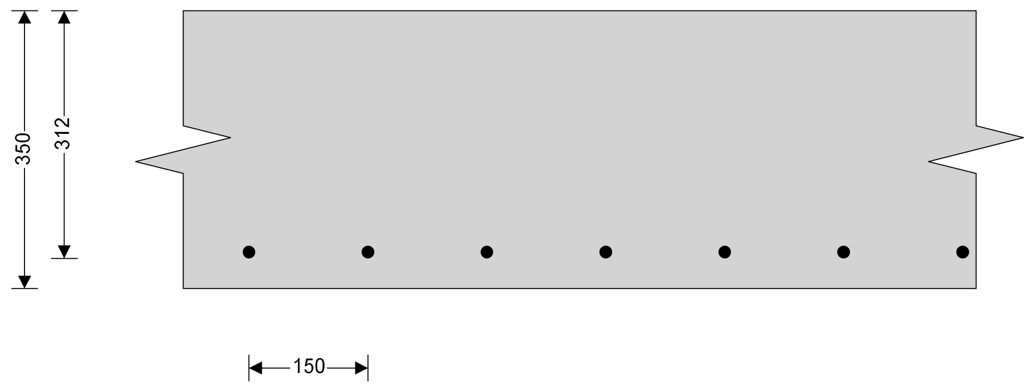
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base}$ = **79.9** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = 3 \times p_{toe\_f} \times x_{bar\_f} \times (l_{toe} - x_{bar\_f} + t_{wall} / 2) / 2$ = **169.9** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **20.3** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base}$ = **149.6** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **312.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.051**

Compression reinforcement is not required

Lever arm; $z_{toe} = min(0.5 + \sqrt{(0.25 - (min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

z_{toe} = **293** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **1173** mm^2/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required; $A_{s\_toe\_req} = Max(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **1173** mm^2/m

Reinforcement provided; **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.256** N/mm^2

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2$ = **4.382** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_toe} = **0.539** N/mm^2

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

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

Material properties

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in heel; c_{heel} = **30** mm

Calculate shear for heel design

Shear from weight of base; $V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base}$ = **3.5** kN/m

Shear from weight of moist backfill; $V_{heel\_wt\_m} = w_{m\_w\_f}$ = **26.5** kN/m

Shear from surcharge; $V_{heel\_sur} = w_{sur\_f}$ = **1.2** kN/m

Total shear for heel design; $V_{heel} = V_{heel\_wt\_base} + V_{heel\_wt\_m} + V_{heel\_sur}$ = **31.1** kN/m

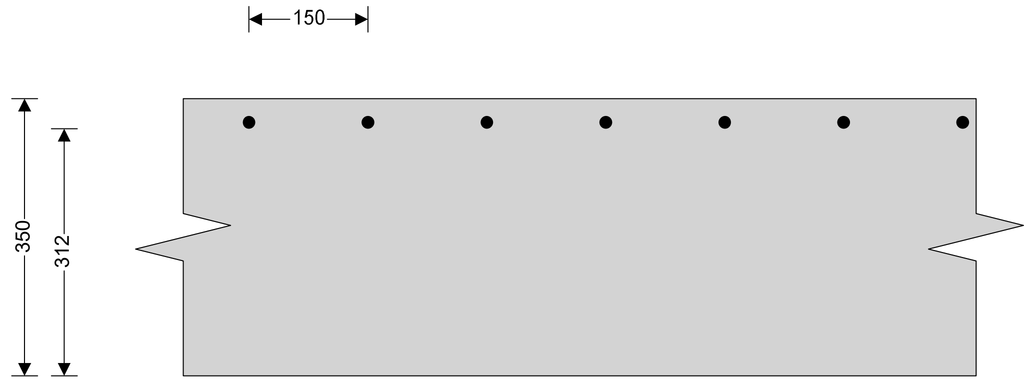
Calculate moment for heel design

Moment from weight of base; $M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2)$ = **1.3** kNm/m

Moment from weight of moist backfill; $M_{heel\_wt\_m} = w_{m\_w\_f} \times (l_{heel} + t_{wall}) / 2$ = **8.6** kNm/m

Moment from surcharge; $M_{heel\_sur} = w_{sur\_f} \times (l_{heel} + t_{wall}) / 2$ = **0.4** kNm/m

Total moment for heel design; $M_{heel} = M_{heel\_wt\_base} + M_{heel\_wt\_m} + M_{heel\_sur}$ = **10.3** kNm/m



Check heel in bending

Width of heel; b = **1000** mm/m

Depth of reinforcement; $d_{heel} = t_{base} - c_{heel} - (\phi_{heel} / 2)$ = **312.0** mm

Constant; $K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu})$ = **0.004**

Compression reinforcement is not required

Lever arm; $z_{heel} = \min(0.5 + \sqrt{(0.25 - (\min(K_{heel}, 0.225) / 0.9))}, 0.95) \times d_{heel}$

z_{heel} = **296** mm

Area of tension reinforcement required; $A_{s\_heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel})$ = **80** mm^2/m

Minimum area of tension reinforcement; $A_{s\_heel\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required; $A_{s\_heel\_req} = \mathrm{Max}(A_{s\_heel\_des}, A_{s\_heel\_min})$ = **455** mm^2/m

Reinforcement provided; **16 mm dia.bars @ 150 mm centres**

Ref: **Calculations** **Output**

Area of reinforcement provided; $A_{s\_heel\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress; $v_{heel} = V_{heel} / (b \times d_{heel})$ = **0.100** N/mm^2

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2$ = **4.382** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_heel} = **0.539** N/mm^2

$v_{heel} < v_{c\_heel}$ - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

Material properties

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Wall details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in stem; c_{stem} = **30** mm

Cover to reinforcement in wall; c_{wall} = **30** mm

Factored horizontal at-rest forces on stem

Surcharge; $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds})$ = **8.1** kN/m

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **89.1** kN/m

Calculate shear for stem design

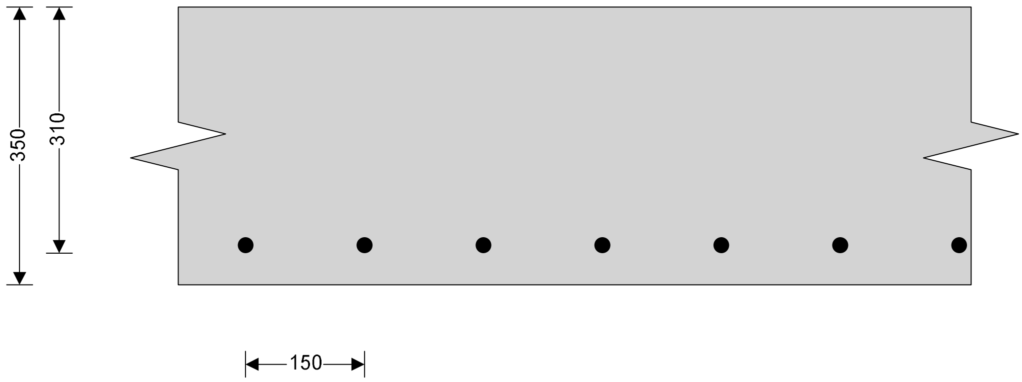
Shear at base of stem; $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} - F_{prop\_f}$ = **18.3** kN/m

Calculate moment for stem design

Surcharge; $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **15.6** kNm/m

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **119.6** kNm/m

Total moment for stem design; $M_{stem} = M_{s\_sur} + M_{s\_m\_a}$ = **135.1** kNm/m



Check wall stem in bending

Width of wall stem; b = **1000** mm/m

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **310.0** mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.047**

Compression reinforcement is not required

Lever arm; $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

z_{stem} = **293** mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **1061** mm²/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **455** mm²/m

Area of tension reinforcement required; $A_{s\_stem\_req} = Max(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **1061** mm²/m

Reinforcement provided; **20 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov}$ = **2094** mm²/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.059** N/mm²

Allowable shear stress; $v_{adm} = min(0.8 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5) \times 1 N/mm^2$ = **4.382** N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.628** N/mm²

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Basic span/effective depth ratio; $ratio_{bas}$ = **7**

Design service stress; $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **168.8** N/mm²

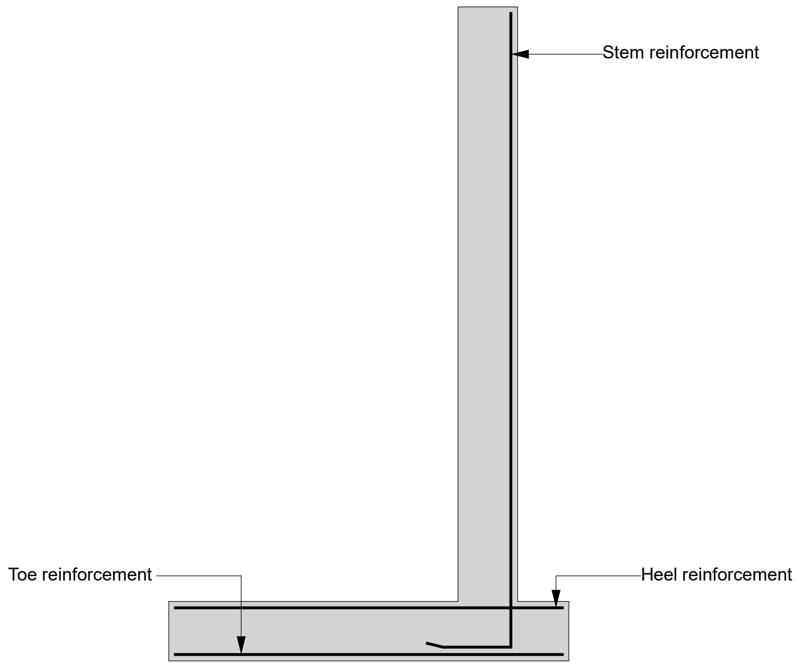
Modification factor; $factor_{tens} = min(0.55 + (477 N/mm^2 - f_s)/(120 \times (0.9 N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.66**

Maximum span/effective depth ratio; $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **11.65**

Actual span/effective depth ratio; $ratio_{act} = h_{stem} / d_{stem}$ = **11.29**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Heel bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Stem bars - 20 mm dia.@ 150 mm centres - (2094 mm^2/m)

RETAINING WALL CONSTRUCTION (WALL A) :

350mm Thick R.C Wall
Wall to be reinforced with T20 Bars to inner face @ 150mm c/c,
T16 Bars to outer face @ 150mm c/c
(Distribution steel to be T12 bars at 150mm c/c)
Starter bars as per wall reinforcement.

Design of new reinforced concrete retaining wall (WALL B) (vertical load scenario)

Max. height of retaining wall = 3.5m Max. retained height.

No surcharge but will have vertical load from the proposed dwelling.

Dead load

Main roof = 9.0m / 2 x 1.00kN/m² = 4.50kN/m
1st floor = 5.5m / 2 x 0.55kN/m² = 1.51kN/m
Ground floor = 5.5m / 2 x 0.55kN/m² = 1.51kN/m
Ext. Rear Wall = 4.0m high x 4.0kN/m² = 16.0kN/m
Total dead load = 23.52kN/m

Super load

Main roof = 9.0m / 2 x 1.00kN/m² = 4.50kN/m
1st floor = 5.5m / 2 x 1.50kN/m² = 4.13kN/m
Ground floor = 5.5m / 2 x 1.50kN/m² = 4.13kN/m
Total dead load = 12.8kN/m

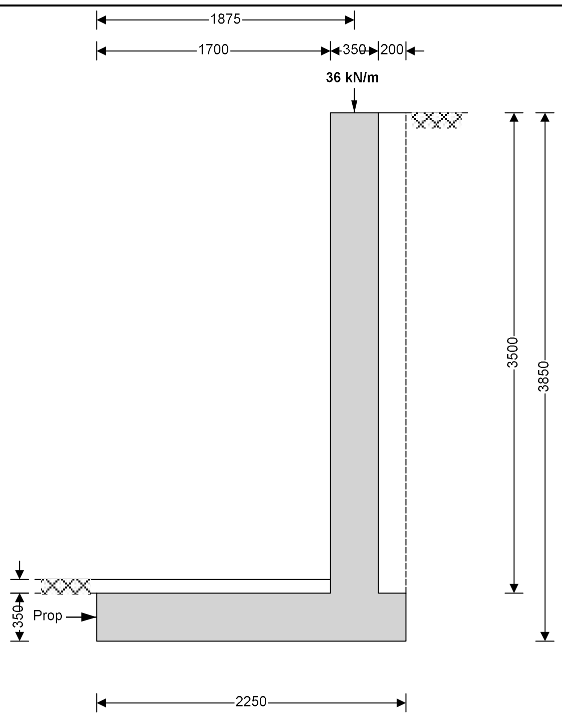
The wall will have a proprietary drain installed at the back of the wall to relieve any hydrostatic pressure.

The base has been designed as propped at the bottom due to the construction a new concrete floor slab.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ref: **Calculations** **Output**



Wall details

Retaining wall type; **Cantilever propped at base**

Height of retaining wall stem; h_{stem} = **3500** mm

Thickness of wall stem; t_{wall} = **350** mm

Length of toe; l_{toe} = **1700** mm

Length of heel; l_{heel} = **200** mm

Overall length of base; $l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2250** mm

Thickness of base; t_{base} = **350** mm

Depth of downstand; d_{ds} = **0** mm

Position of downstand; l_{ds} = **1600** mm

Thickness of downstand; t_{ds} = **350** mm

Height of retaining wall; $h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3850** mm

Depth of cover in front of wall; d_{cover} = **100** mm

Depth of unplanned excavation; d_{exc} = **100** mm

Height of ground water behind wall; h_{water} = **0** mm

Height of saturated fill above base; $h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **0** mm

Density of wall construction; γ_{wall} = **23.6** kN/m^3

Density of base construction; γ_{base} = **23.6** kN/m^3

Angle of rear face of wall; α = **90.0** deg

Angle of soil surface behind wall; β = **0.0** deg

Effective height at virtual back of wall; $h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3850** mm

Retained material details

Mobilisation factor; M = **1.5**

Moist density of retained material; γ_m = **18.0** kN/m^3

Saturated density of retained material; γ_s = **21.0** kN/m^3

Ref: **Calculations** **Output**

Design shear strength; $\phi' = \mathbf{25.0}$ deg

Angle of wall friction; $\delta = \mathbf{19.3}$ deg

Base material details

Firm clay

Moist density; $\gamma_{mb} = \mathbf{18.0}$ kN/m^3

Design shear strength; $\phi'_b = \mathbf{24.2}$ deg

Design base friction; $\delta_b = \mathbf{18.6}$ deg

Allowable bearing pressure; $P_{bearing} = \mathbf{100}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.358}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material; $K_0 = 1 – \sin(\phi') = \mathbf{0.577}$

Loading details

Surcharge load on plan; Surcharge = **0.0** kN/m^2

Applied vertical dead load on wall; $W_{dead} = \mathbf{23.5}$ kN/m

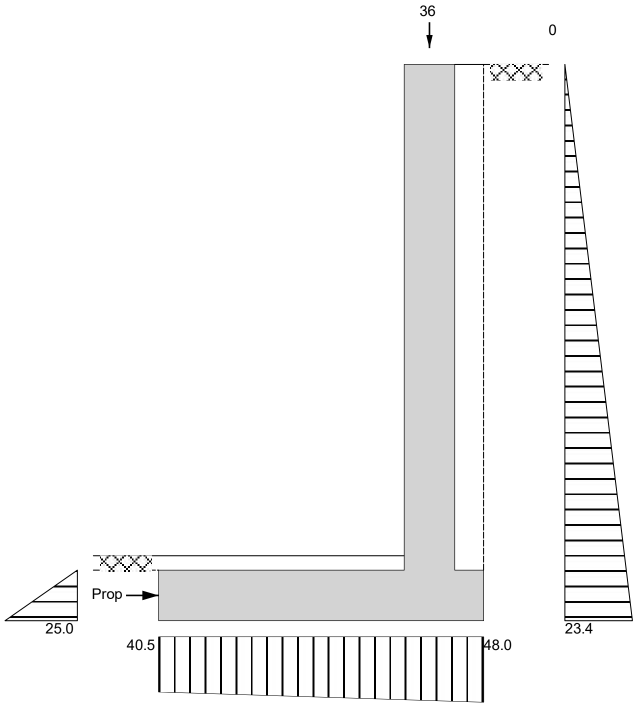
Applied vertical live load on wall; $W_{live} = \mathbf{12.8}$ kN/m

Position of applied vertical load on wall; $l_{load} = \mathbf{1875}$ mm

Applied horizontal dead load on wall; $F_{dead} = \mathbf{0.0}$ kN/m

Applied horizontal live load on wall; $F_{live} = \mathbf{0.0}$ kN/m

Height of applied horizontal load on wall; $h_{load} = \mathbf{0}$ mm



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

Vertical forces on wall

Wall stem; $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.9** kN/m

Wall base; $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **18.6** kN/m

Moist backfill to top of wall; $w_{m\_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **12.6** kN/m

Soil in front of wall; $w_p = l_{toe} \times d_{cover} \times \gamma_{mb}$ = **3.1** kN/m

Applied vertical load; $W_v = W_{dead} + W_{live}$ = **36.3** kN/m

Total vertical load; $W_{total} = w_{wall} + w_{base} + w_{m\_w} + w_p + W_v$ = **99.5** kN/m

Horizontal forces on wall

Moist backfill above water table; $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **45.1** kN/m

Total horizontal load; $F_{total} = F_{m\_a}$ = **45.1** kN/m

Calculate propping force

Passive resistance of soil in front of wall; $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4** kN/m

Propping force; $F_{prop} = \max(F_{total} - F_p - (W_{total} - w_p - W_{live}) \times \tan(\delta_b), 0\ kN/m)$

F_{prop} = **12.6** kN/m

Overturning moments

Moist backfill above water table; $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **57.8** kNm/m

Total overturning moment; $M_{ot} = M_{m\_a}$ = **57.8** kNm/m

Restoring moments

Wall stem; $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **54.2** kNm/m

Wall base; $M_{base} = w_{base} \times l_{base} / 2$ = **20.9** kNm/m

Moist backfill; $M_{m\_r} = (w_{m\_w} \times (l_{base} - l_{heel} / 2) + w_{m\_s} \times (l_{base} - l_{heel} / 3))$ = **27.1** kNm/m

Design vertical dead load; $M_{dead} = W_{dead} \times l_{load}$ = **44.1** kNm/m

Total restoring moment; $M_{rest} = M_{wall} + M_{base} + M_{m\_r} + M_{dead}$ = **146.3** kNm/m

Check bearing pressure

Soil in front of wall; $M_{p\_r} = w_p \times l_{toe} / 2$ = **2.6** kNm/m

Design vertical live load; $M_{live} = W_{live} \times l_{load}$ = **24** kNm/m

Total moment for bearing; $M_{total} = M_{rest} - M_{ot} + M_{p\_r} + M_{live}$ = **115.1** kNm/m

Total vertical reaction; $R = W_{total}$ = **99.5** kN/m

Distance to reaction; $x_{bar} = M_{total} / R$ = **1157** mm

Eccentricity of reaction; $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **32** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **40.5** kN/m^2

Bearing pressure at heel; $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **48** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Ref: **Calculations** **Output**

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor; γ_{f_d} = **1.4**

Live load factor; γ_{f_l} = **1.6**

Earth and water pressure factor; γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem; $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **40.5** kN/m

Wall base; $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **26** kN/m

Moist backfill to top of wall; $w_{m\_w\_f} = \gamma_{f\_d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m$ = **17.6** kN/m

Soil in front of wall; $w_{p\_f} = \gamma_{f\_d} \times l_{toe} \times d_{cover} \times \gamma_{mb}$ = **4.3** kN/m

Applied vertical load; $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **53.4** kN/m

Total vertical load; $W_{total\_f} = w_{wall\_f} + w_{base\_f} + w_{m\_w\_f} + w_{p\_f} + W_{v\_f}$ = **141.8** kN/m

Factored horizontal at-rest forces on wall

Moist backfill above water table; $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **107.8** kN/m

Total horizontal load; $F_{total\_f} = F_{m\_a\_f}$ = **107.8** kN/m

Calculate propping force

Passive resistance of soil in front of wall; $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **6.1** kN/m

Propping force; $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - w_{p\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0\text{ kN/m})$

F_{prop_f} = **62.3** kN/m

Factored overturning moments

Moist backfill above water table; $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **138.4** kNm/m

Total overturning moment; $M_{ot\_f} = M_{m\_a\_f}$ = **138.4** kNm/m

Restoring moments

Wall stem; $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **75.9** kNm/m

Wall base; $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **29.3** kNm/m

Moist backfill; $M_{m\_r\_f} = (w_{m\_w\_f} \times (l_{base} - l_{heel} / 2) + w_{m\_s\_f} \times (l_{base} - l_{heel} / 3))$ = **37.9** kNm/m

Soil in front of wall; $M_{p\_r\_f} = w_{p\_f} \times l_{toe} / 2$ = **3.6** kNm/m

Design vertical load; $M_{v\_f} = W_{v\_f} \times l_{load}$ = **100.1** kNm/m

Total restoring moment; $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{m\_r\_f} + M_{p\_r\_f} + M_{v\_f}$ = **246.9** kNm/m

Factored bearing pressure

Total moment for bearing; $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **108.5** kNm/m

Total vertical reaction; $R_f = W_{total\_f}$ = **141.8** kN/m

Distance to reaction; $x_{bar\_f} = M_{total\_f} / R_f$ = **765** mm

Eccentricity of reaction; $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **360** mm

Reaction acts within middle third of base

Bearing pressure at toe; $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **123.6** kN/m²

Bearing pressure at heel; $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **2.5** kN/m²

Rate of change of base reaction; $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **53.80** kN/m²/m

Bearing pressure at stem / toe; $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0\text{ kN/m}^2)$ = **32.1** kN/m²

Bearing pressure at mid stem; $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\text{ kN/m}^2)$ = **22.7** kN/m²

Bearing pressure at stem / heel; $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\text{ kN/m}^2)$ = **13.3** kN/m²

Ref: **Calculations** **Output**

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

Material properties

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in toe; c_{toe} = **40** mm

Calculate shear for toe design

Shear from bearing pressure; $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2$ = **132.3** kN/m

Shear from weight of base; $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base}$ = **19.7** kN/m

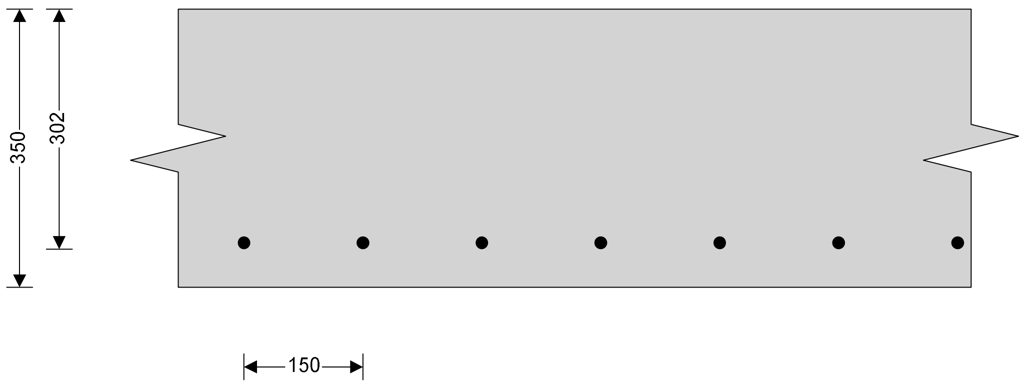
Total shear for toe design; $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base}$ = **112.7** kN/m

Calculate moment for toe design

Moment from bearing pressure; $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6$ = **158.1** kNm/m

Moment from weight of base; $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2)$ = **20.3** kNm/m

Total moment for toe design; $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base}$ = **137.8** kNm/m



Check toe in bending

Width of toe; b = **1000** mm/m

Depth of reinforcement; $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2)$ = **302.0** mm

Constant; $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu})$ = **0.050**

Compression reinforcement is not required

Lever arm; $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

z_{toe} = **284** mm

Area of tension reinforcement required; $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe})$ = **1115** mm^2/m

Minimum area of tension reinforcement; $A_{s\_toe\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required; $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min})$ = **1115** mm^2/m

Reinforcement provided; **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_toe\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress; $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.373** N/mm^2

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2$ = **4.382** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_toe} = **0.550** N/mm^2

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

<u>Design of reinforced concrete retaining wall heel (BS 8002:1994)</u>

Material properties

Characteristic strength of concrete; f_{cu} = **30** N/mm^2

Characteristic strength of reinforcement; f_y = **500** N/mm^2

Base details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in heel; c_{heel} = **40** mm

Calculate shear for heel design

Shear from bearing pressure; $V_{heel\_bear} = (p_{heel\_f} + p_{stem\_heel\_f}) \times l_{heel} / 2$ = **1.6** kN/m

Shear from weight of base; $V_{heel\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{heel} \times t_{base}$ = **2.3** kN/m

Shear from weight of moist backfill; $V_{heel\_wt\_m} = w_{m\_w\_f}$ = **17.6** kN/m

Total shear for heel design; $V_{heel} = - V_{heel\_bear} + V_{heel\_wt\_base} + V_{heel\_wt\_m}$ = **18.4** kN/m

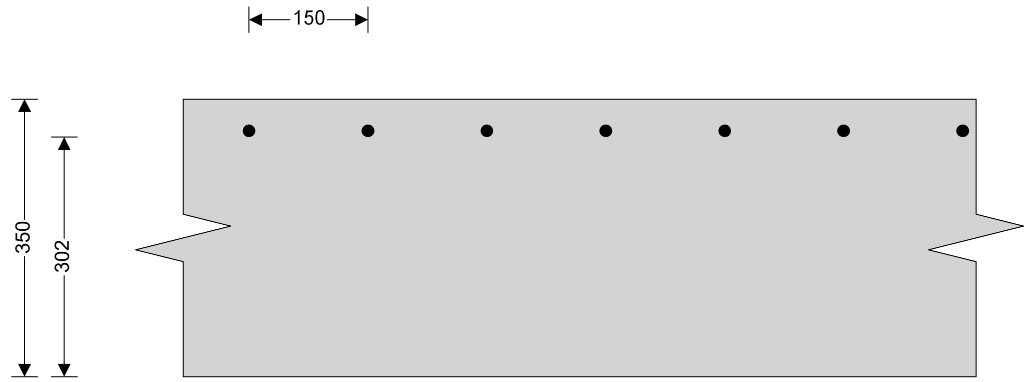
Calculate moment for heel design

Moment from bearing pressure; $M_{heel\_bear} = (2 \times p_{heel\_f} + p_{stem\_mid\_f}) \times (l_{heel} + t_{wall} / 2)^2 / 6$ = **0.6** kNm/m

Moment from weight of base; $M_{heel\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2)$ = **0.8** kNm/m

Moment from weight of moist backfill; $M_{heel\_wt\_m} = w_{m\_w\_f} \times (l_{heel} + t_{wall}) / 2$ = **4.9** kNm/m

Total moment for heel design; $M_{heel} = - M_{heel\_bear} + M_{heel\_wt\_base} + M_{heel\_wt\_m}$ = **5** kNm/m



Check heel in bending

Width of heel; b = **1000** mm/m

Depth of reinforcement; $d_{heel} = t_{base} – c_{heel} – (\phi_{heel} / 2)$ = **302.0** mm

Constant; $K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu})$ = **0.002**

Compression reinforcement is not required

Lever arm; $z_{heel} = \min(0.5 + \sqrt{(0.25 - (\min(K_{heel}, 0.225) / 0.9))},0.95) \times d_{heel}$

z_{heel} = **287** mm

Area of tension reinforcement required; $A_{s\_heel\_des} = M_{heel} / (0.87 \times f_y \times z_{heel})$ = **40** mm^2/m

Minimum area of tension reinforcement; $A_{s\_heel\_min} = k \times b \times t_{base}$ = **455** mm^2/m

Area of tension reinforcement required; $A_{s\_heel\_req} = \mathrm{Max}(A_{s\_heel\_des}, A_{s\_heel\_min})$ = **455** mm^2/m

Reinforcement provided; **16 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_heel\_prov}$ = **1340** mm^2/m

PASS - Reinforcement provided at the retaining wall heel is adequate

Ref: **Calculations** **Output**

Check shear resistance at heel

Design shear stress; $v_{heel} = V_{heel} / (b \times d_{heel}) = \mathbf{0.061}$ N/mm^2

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = \mathbf{4.382}$ N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; $v_{c\_heel} = \mathbf{0.550}$ N/mm^2

$v_{heel} < v_{c\_heel}$ - No shear reinforcement required

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

Material properties

Characteristic strength of concrete; $f_{cu} = \mathbf{30}$ N/mm^2

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

Wall details

Minimum area of reinforcement; k = **0.13** %

Cover to reinforcement in stem; $c_{stem} = \mathbf{40}$ mm

Cover to reinforcement in wall; $c_{wall} = \mathbf{40}$ mm

Factored horizontal at-rest forces on stem

Moist backfill above water table; $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{89.1}$ kN/m

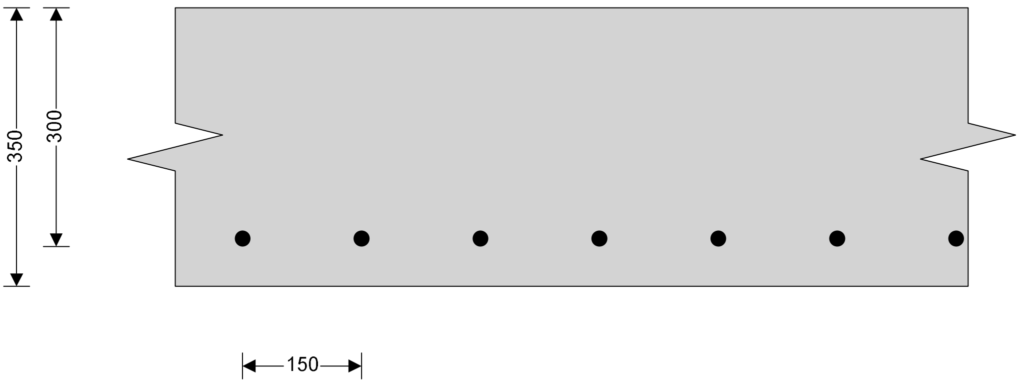
Calculate shear for stem design

Shear at base of stem; $V_{stem} = F_{s\_m\_a\_f} - F_{prop\_f} = \mathbf{26.8}$ kN/m

Calculate moment for stem design

Moist backfill above water table; $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 = \mathbf{119.6}$ kNm/m

Total moment for stem design; $M_{stem} = M_{s\_m\_a} = \mathbf{119.6}$ kNm/m



Check wall stem in bending

Width of wall stem; b = **1000** mm/m

Depth of reinforcement; $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{300.0}$ mm

Constant; $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.044}$

Compression reinforcement is not required

Lever arm; $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

$z_{stem} = \mathbf{284}$ mm

Area of tension reinforcement required; $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{966}$ mm^2/m

Minimum area of tension reinforcement; $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{455}$ mm^2/m

Area of tension reinforcement required; $A_{s\_stem\_req} = \text{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{966}$ mm^2/m

Reinforcement provided; **20 mm dia.bars @ 150 mm centres**

Area of reinforcement provided; $A_{s\_stem\_prov}$ = **2094** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress; $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.089** N/mm^2

Allowable shear stress; $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2$ = **4.382** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress; v_{c_stem} = **0.640** N/mm^2

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Basic span/effective depth ratio; $ratio_{bas}$ = **7**

Design service stress; $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **153.8** N/mm^2

Modification factor; $factor_{tens} = \min(0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.76**

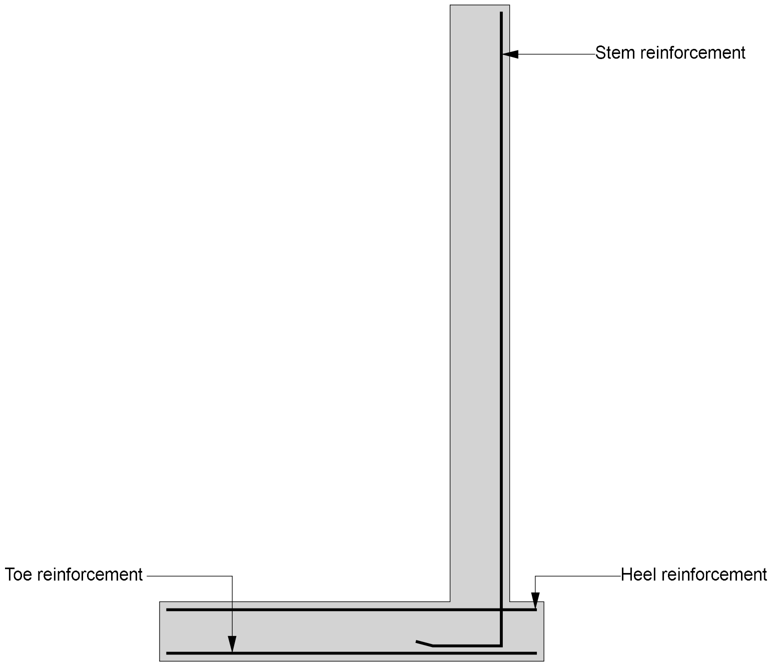
Maximum span/effective depth ratio; $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **12.31**

Actual span/effective depth ratio; $ratio_{act} = h_{stem} / d_{stem}$ = **11.67**

PASS - Span to depth ratio is acceptable

Ref: **Calculations** **Output**

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Heel bars - 16 mm dia.@ 150 mm centres - (1340 mm^2/m)
Stem bars - 20 mm dia.@ 150 mm centres - (2094 mm^2/m)

RETAINING WALL CONSTRUCTION (WALL B) :

350mm Thick R.C Wall
Wall to be reinforced with T20 Bars to inner face @ 150mm c/c,
T16 Bars to outer face @ 150mm c/c
(Distribution steel to be T12 bars at 150mm c/c)
Starter bars as per wall reinforcement.