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Material Properties

Characteristic strength of concrete, f_{cu} (≤ 60N/mm^2; HSC N/A)	35	N/mm^2	**OK**
Yield strength of longitudinal steel, f_y	460	N/mm^2	
Yield strength of shear link steel, f_{yv}	460	N/mm^2	
Type of concrete and density, ρ_c	Normal Weight / 24	kN/m^3	

Factor of Safety

Factor of safety (overall net (effective) bearing), FOS_1 (usually 2.5 to 3.0)	3.0		
Factor of safety (overall sliding resistance), FOS_2 (usually 1.6)	1.6		
Factor of safety (overall uplift resistance), FOS_3 (usually 1.0)	1.0		
Factor of safety (overall overturning resistance), FOS_4 (usually 1.6)	1.6		
Loading factor, K (between 1.40 and 1.60 depending on DL to LL ratio)	1.50		*BS8110*

Note loading factor K multiplies SLS loads for ULS loads for section (reinforcement) design; *cl. 2.4.3.1.1*

Soil Description

Water unit weight, γ_w = 9.81kN/m^3	9.8	kN/m^3	
Soil name	Loose Sand		
Dry bulk unit weight, γ_{dry}	18.0	kN/m^3	
Saturated bulk unit weight, γ_{sat}	20.5	kN/m^3	
Undrained shear strength limit to adopt ?	Lower Limit		
Undrained shear strength (lower limit), $S_{u,ll}$	N/A	kPa	
Undrained shear strength (upper limit), $S_{u,ul}$	N/A	kPa	
Undrained shear strength limit adopted, S_u = {$S_{u,ll}$, $(S_{u,ll}+S_{u,ul})/2$, $S_{u,ul}$}	**N/A**	kPa	
Note that S_u can be obtained from SPT (Stroud) values;			*Tomlinson*
Effective cohesion, c'	Exclude / **0.0**	kPa	
Effective angle of shear resistance, ϕ'	**35.0**	degrees	
Note that ϕ' can be obtained from SPT (Peck) or CPT (Durgunoglu and Mitchell) values;			*Tomlinson*
Effective angle of friction or 0.66ø' (Insitu Concrete Active Zone - Soil Interface)	**23.1**	degrees	
Bearing capacity limit to adopt ?	Upper Limit		
Bearing capacity values from allowable bearing capacity, $BC_{ll,a/ul,a}$ value N s ?			
Factor for SPT, N value, K_{SPT}	Drained Soil: 30.0 / **30.0**		
Bearing capacity (lower limit), $FOS_1.BC_{ll,a}$	N/A	kPa	
SPT (lower limit), N_{ll}	4		
Bearing capacity (upper limit), $FOS_1.BC_{ul,a}$	N/A	kPa	
SPT (upper limit), N_{ul}	10		

Note that FOS_1 is multipled onto the allowable bearing capacity, $BC_{ll,a/ul,a}$ at this stage because it will be refactored when the empirical overall net effective bearing capacity is calculated;

Ground water level modification for beari GWL >= B	Non Cohesive Soil / **1.00**		*BS5975*

Table 18 — Ground water level modification factor

Condition	Modification factor for: Cohesive soils	Non-cohesive soils	Rocks
Ground water level at B, or less, below level of foundation (where B is the width of foundation)	1.0	0.5	1.0
Site liable to flooding	0.67	0.5	1.0

Bearing capacity adopted, BC	**300**	kPa

Note BC = MOD_{BC} . [FOS_1.{$BC_{ll,a}$, $(BC_{ll,a}+BC_{ul,a})/2$, $BC_{ul,a}$} or {$K_{SPT}.N_{ll}$, $K_{SPT}.(N_{ll}+N_{ul})/2$, $K_{SPT}.N_{ul}$}];

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Analysis Method

Undrained, drained or empirical analysis ? — Empirical Analysis

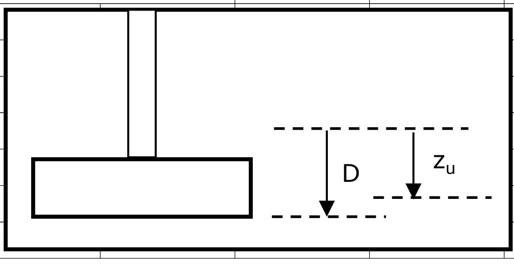
For clays, perform undrained, drained and empirical analyses;
For sands / gravels, perform drained and empirical analyses;
For rocks, perform drained and empirical analyses;

Evaluate overall uplift resistance ? — No

Note that overall uplift resistance (mid third) is conservative to overturning, thus may in certain instances be deemed to be overconservative and subsequently ignored;

Foundation Dimensions

Foundation type — Pad Footing



Depth of foundation founding level from ground level, D (>= 0.000m)	0.650	m	OK
Depth of water table from ground level, z_u	0.650	m	

Note that the soil beneath the water table has an effective submerged unit weight of about half of the soil above the water table, thus reducing the drained overall net effective bearing capacity;
Hence use the highest water table forseeable;
Enter a negative z_u value for water table above ground level, this representing a flood event or a bridge pier within a sea or river with the ground level being the sea or river bed;
However, a water table above ground level may unconservatively decrease the overall (effective) bearing capacity utilisation, thus consider also the case when the water table is at ground level;

Foundation Reinforcement

Cover to all (bottom and side) reinforcement, $cover_1$ (usually 75)	50	mm
Cover to all (top) reinforcement, $cover_2$ (usually 45)	25	mm

Foundation SLS Loading

Surcharge at surface, $p_{surface}$	0	kPa

Note that (unlike retaining walls) surface surcharging increases overall (effective) bearing capacity, thus consider the case when there is no surcharge unless it can be guaranteed;

Consider reduction of working pressure due to surcharge above founding level, p_0 or p_0' in net (effective) working pressure, q_{wnet} or q_{wnet}' ? — Yes

Note that for the case where an excavation and backfill (embedded footing) takes place prior to application of working pressure at the founding level: -
i. include additional soil (above footing) weight, $F_{above,soil}$
ii. do consider reduction of working pressure due to p_0 or p_0' in q_{wnet} or q_{wnet}'

Note that for the case where an excavation without backfill takes place prior to application of working pressure at the founding level: -
i. do not include additional soil (above footing) weight, $F_{above,soil}$
ii. do consider reduction of working pressure due to p_0 or p_0' in q_{wnet} or q_{wnet}'

Note that for the case where an excavation had already taken place in the past prior to application of working pressure at the founding level: -
i. do not include additional soil (above footing) weight, $F_{above,soil}$
ii. do not consider reduction of working pressure due to p_0 or p_0' in q_{wnet} or q_{wnet}'

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Executive Summary

Undrained overall net bearing capacity	**N/A**	**N/A**
Drained overall net effective bearing capacity	**N/A**	**N/A**
Empirical overall net effective bearing capacity	**91%**	**OK**
Overall sliding resistance capacity	**0%**	**OK**
Overall uplift resistance capacity	**N/A**	**N/A**
Overall overturning resistance capacity	**0%**	**OK**

Pad Footing

Sagging bending moment in plane of width	**5%**	**OK**
Sagging bending moment in plane of length	**10%**	**OK**
% Min sag reinforcement in plane of width	**34%**	**OK**
% Min sag reinforcement in plane of length	**34%**	**OK**
Punching shear at column base face	**12%**	**OK**
Punching shear at first shear perimeter	**14%**	**OK**
Punching shear at second shear perimeter	**0%**	**OK**
Ultimate shear stress for bending in plane of width	**4%**	**OK**
Shear design capacity for bending in plane of width	**7%**	**OK**
Ultimate shear stress for bending in plane of length	**5%**	**OK**
Shear design capacity for bending in plane of length	**14%**	**OK**
Detailing requirements	**NOT OK**	

Strip Footing

Sagging bending moment	**N/A**	**N/A**
% Min sag reinforcement	**N/A**	**N/A**
Ultimate shear stress	**N/A**	**N/A**
Shear design capacity	**N/A**	**N/A**
Detailing requirements	**N/A**	

Multi Column Footing

Sagging bending moment in plane of width	**N/A**	**N/A**
Sagging bending moment in plane of length	**N/A**	**N/A**
Hogging bending moment in plane of length	**N/A**	**N/A**
% Min sag reinforcement in plane of width	**N/A**	**N/A**
% Min sag reinforcement in plane of length	**N/A**	**N/A**
% Min hog reinforcement in plane of length	**N/A**	**N/A**
Punching shear at column base face	**N/A**	**N/A**
Punching shear at first shear perimeter	**N/A**	**N/A**
Punching shear at second shear perimeter	**N/A**	**N/A**
Ultimate shear stress for bending in plane of width	**N/A**	**N/A**
Shear design capacity for bending in plane of width	**N/A**	**N/A**
Ultimate shear stress for bending in plane of length	**N/A**	**N/A**
Shear design capacity for bending in plane of length	**N/A**	**N/A**
Detailing requirements	**N/A**	

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Combined Footing			
Sagging bending moment in plane of width	N/A	N/A	
Sagging bending moment in plane of length	N/A	N/A	
Hogging bending moment in plane of length	N/A	N/A	
% Min sag reinforcement in plane of width	N/A	N/A	
% Min sag reinforcement in plane of length	N/A	N/A	
% Min hog reinforcement in plane of length	N/A	N/A	
Punching shear at column base face	N/A	N/A	
Punching shear at first shear perimeter	N/A	N/A	
Punching shear at second shear perimeter	N/A	N/A	
Ultimate shear stress for bending in plane of width	N/A	N/A	
Shear design capacity for bending in plane of width	N/A	N/A	
Ultimate shear stress for bending in plane of length	N/A	N/A	
Shear design capacity for bending in plane of length	N/A	N/A	
Detailing requirements	N/A		
Strap Footing			
Sagging bending moment in plane of width of outer foo	N/A	N/A	
Hogging bending moment in beam	N/A	N/A	
% Min sag reinforcement in plane of width of outer foo	N/A	N/A	
% Min hog reinforcement in beam	N/A	N/A	
Punching shear at column base face	N/A	N/A	
Punching shear at first shear perimeter	N/A	N/A	
Punching shear at second shear perimeter	N/A	N/A	
Ultimate shear stress for bending in plane of width of c	N/A	N/A	
Shear design capacity for bending in plane of width of	N/A	N/A	
Ultimate shear stress in beam	N/A	N/A	
Shear design capacity in beam	N/A	N/A	
Detailing requirements	N/A		
Raft			
Design reinforcement based on the combination of multi column footings, combined			
Overall utilisation summary		91%	
% Sagging reinforcement in plane of width		0.38	%
% Sagging reinforcement in plane of length		0.38	%
% Hogging reinforcement in plane of length (or in beam for strap footing)		N/A	%
Estimated steel reinforcement quantity (60 – 70kg/m^3)		59	kg/m^3
[Note that steel quantity in kg/m^3 can be obtained from 78.5 x % rebar];			
Material cost: concrete, c 260 units/m^3	steel, s	3200	units/tonne
Reinforced concrete material cost = c+(est. rebar quant).s		449	units/m^3

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Relevant Foundation Parameters

Relevant foundation type: **Pad Footing**

	Overall (Effective) Bearing Capacity and Overall Sliding Resistance Capacity							
	B	(m)	L	(m)	B'	(m)	L'	(m)
Pad Footing	B_{pad}	0.600	L_{pad}	0.750	B_{pad}'	0.557	L_{pad}'	0.750
Strip Footing	B_{strip}	N/A	infinity	N/A	B_{strip}'	N/A	infinity	N/A
Multi Column Footi	B_{multi}	N/A	L_{multi}	N/A	B_{multi}	N/A	L_{multi}	N/A
Combined Footing	B_{com}	N/A	L_{com}	N/A	B_{com}	N/A	L_{com}	N/A
Strap Footing	$B_{strap,1}$	N/A	$L_{strap,1}$	N/A	$B_{strap,1}$	N/A	$L_{strap,1}$	N/A
Raft	B_{raft}	N/A	L_{raft}	N/A	B_{raft}	N/A	L_{raft}	N/A
	B	**0.600**	**L**	**0.750**	**B'**	**0.557**	**L'**	**0.750**

Gross working pressure, q_w **102** kPa

Note q_w above is $q_{w,1}$ for strap footing;

	Overall Sliding Resistance Capacity				Overall Uplift Resistance Capacity			
	Vertical Load	(kN or kN/m)	Horizontal Load	(kN or kN/m)	e_B (m)	$e_{B,limit}$ (m)	e_L (m)	$e_{L,limit}$ (m)
Pad Footing	$F_{pad,v}'$	43	$F_{pad,h}$	0	0.021	0.100	0.000	0.125
Strip Footing	$F_{strip,v}'$	N/A	$F_{strip,h}$	N/A	N/A	N/A	N/A	N/A
Multi Column Footi	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Combined Footing	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Strap Footing	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Raft	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	F_v'	**43**	**F_h**	**0**	**0.021**	**0.100**	**0.000**	**0.125**

	Overall Overturning Resistance Capacity			
	$M_{ot,B}$ (kNm or kNm/m)	$M_{rt,B}$ (kNm or kNm/m)	$M_{ot,L}$ (kNm or kNm/m)	$M_{rt,L}$ (kNm or kNm/m)
Pad Footing	0	7	0	10
Strip Footing	N/A	N/A	N/A	N/A
Multi Column Footi	N/A	N/A	N/A	N/A
Combined Footing	N/A	N/A	N/A	N/A
Strap Footing	N/A	N/A	N/A	N/A
Raft	N/A	N/A	N/A	N/A
	0	**7**	**0**	**10**

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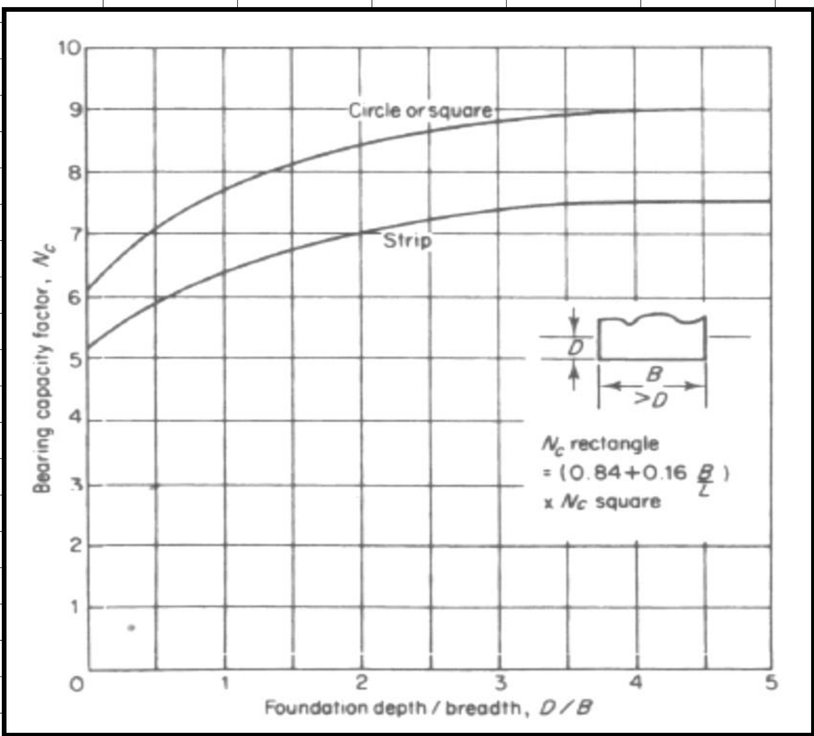
Undrained Overall Net Bearing Capacity

Total surcharge above founding level, p_0	**N/A**	kPa	
Case when (z_u –D) >= MAX (B, L)	***N/A***		
$p_0 = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
Case when 0 < (z_u –D) < MAX (B, L)	***N/A***		
$p_0 = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
Case when (z_u –D) = 0	***N/A***		
$p_0 = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
Case when (z_u –D) < 0 and z_u >= 0	***N/A***		
$p_0 = p_{surface} + \gamma_{sat}.(D - z_u) + \gamma_{dry}.z_u$	N/A	kPa	
Case when z_u < 0	***N/A***		
$p_0 = p_{surface} + \gamma_{sat}.D + \gamma_w.(-z_u)$	N/A	kPa	
Net bearing capacity, $q_{fnet} = q_f - p_0$	**N/A**	kPa	
Gross bearing capacity, $q_f = s_c.d_c.N_{c,strip}.S_u + p_0$	**N/A**	kPa	*Terzaghi*



Shape factor, $s_c = 1 + 0,2\,(B'/L')$	N/A		*EC7*
Depth factor, $d_c = 1+(0.053D/B')^{0.5}$ for D/B' <= 4.0	N/A		**N/A**
Bearing capacity factor, $N_{c,strip}$ $N_c = (2 + \pi) = 5.14$	N/A		*Skempton*
Net working pressure, $q_{wnet} = q_w$ - (p_0 or 0)	**N/A**	kPa	
Gross working pressure, q_w	**N/A**	kPa	

Note a negative q_{wnet} indicates an excavation, the following analysis ascertains the susceptibility of the system to base heave instability, conservatively however ignoring the contribution of the shearing resistance of the soil interface above the founding level and any wall embedment below the founding level;

Undrained overall net bearing capacity (factored), q_{fnet} / FOS_1	**N/A**	kPa	
Undrained overall net bearing capacity utilisation = ABS (q_{wnet}) / (q_{fnet} / FOS_1	**N/A**		**N/A**

Note an absolute function is applied to the above to present the susceptibility to base heave instability as well as the overall net bearing capacity;

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Drained Overall Net Effective Bearing Capacity

Effective surcharge above founding level, p_0'	**N/A**	kPa	
Unit weight, γ'	N/A	kN/m^3	
Case when (z_u – D) >= MAX (B, L)	***N/A***		
$p_0' = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
$\gamma' = \gamma_{dry}$	N/A	kN/m^3	
Case when 0 < (z_u – D) < MAX (B, L)	***N/A***		
$p_0' = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
$\gamma' = z_u/MAX(B,L) . [\gamma_{dry} - (\gamma_{sat} - \gamma_w)] + (\gamma_{sat} - \gamma_w)$	N/A	kN/m^3	
Case when (z_u – D) = 0	***N/A***		
$p_0' = p_{surface} + \gamma_{dry}.D$	N/A	kPa	
$\gamma' = \gamma_{sat} - \gamma_w$	N/A	kN/m^3	
Case when (z_u – D) < 0 and z_u >= 0	***N/A***		
$p_0' = p_{surface} + (\gamma_{sat} - \gamma_w).(D - z_u) + \gamma_{dry}.z_u$	N/A	kPa	
$\gamma' = \gamma_{sat} - \gamma_w$	N/A	kN/m^3	
Case when z_u < 0	***N/A***		
$p_0' = p_{surface} + \gamma_{sat}.D + \gamma_w.(-z_u) - \gamma_w.(D + (-z_u))$	N/A	kPa	
Note that the above equation reduces to $p_0' = p_{surface} + (\gamma_{sat} - \gamma_w).D$;			
$\gamma' = \gamma_{sat} - \gamma_w$	N/A	kN/m^3	
Net effective bearing capacity, $q_{fnet}' = q_f' - p_0'$	**N/A**	kPa	
Gross effective bearing capacity, q_f'	**N/A**	kPa	*Terzaghi*
= $s_c.d_c.N_{c,strip}.c'$	N/A	kPa	
+ $s_q.d_q.N_{q,strip}.p_0'$	N/A	kPa	
+ $s_\gamma.d_\gamma.N_{\gamma,strip}.B'/2.\gamma'$	N/A	kPa	

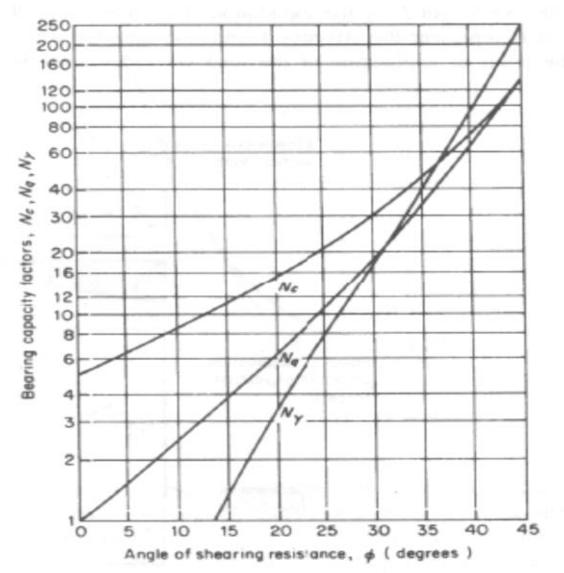

Fig. 2.8 Bearing capacity factors N_c, N_q, and N_γ (after Brinch Hansen[2.1]).

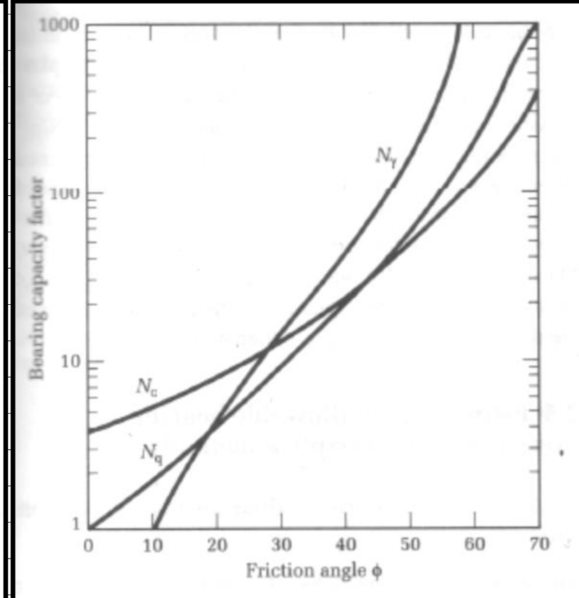

Fig. 2.17 Wedge bearing capacity factor for foundations on rock.

Equations for bearing capacity	Prandtl, Reissner and Hansen Equations for Soils		
Cohesion Factors			
Shape factor, $s_c = (s_q.N_q - 1)/(N_q - 1)$	N/A		*EC7*
Depth factor, $d_c = 1 + 0.4 \arctan(D/B)$	N/A		
Note B in the above equation is B';			
Bearing capacity factor, $N_{c,strip}$	N/A		
Soils $N_{c,strip} = (N_{q,strip} - 1).\cot\phi'$ $\quad N_c = (N_q - 1) \cot \varphi'$	N/A		*EC7 (Prandt*
Rocks $N_{c,strip}$ $N_c = 2N_\phi^{1/2}(N_\phi + 1)$ $\quad N_\phi = Tan^2(45° + \phi/2)$	N/A		*Kulhawy and Goo*

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Surcharge Factors

Shape factor,	$s_q = 1 + (B' / L') \sin \varphi'$,	N/A	*EC7*
Depth factor, d_q	$1 + 2 \tan\varphi'(1-\sin\varphi')^2 \arctan\frac{D}{B'}$	N/A	
Bearing capacity factor, $N_{q,strip}$		N/A	
Soils	$N_q = e^{\pi \tan\varphi'} \tan^2(45 + \varphi'/2)$	N/A	*EC7 (Reissne*
Rocks	$N_{q,strip}$ $N_q : N_\phi^2$ $N_\phi : Tan^2(45° + \phi/2)$	N/A	*Kulhawy and Goo*

Self Weight Factors

Shape factor,	$s_\gamma = 1 - 0,3\ (B'/L')$,	N/A	*EC7*
Depth factor,	$d_\gamma = 1.0$	N/A	
Bearing capacity factor, $N_{q,strip}$		N/A	
Soils	$N_{\gamma,strip}$ = 2.0($N_{q,strip}$-1).ta $N_\gamma = 2\ (N_q - 1) \tan \varphi'$,	N/A	*EC7 (Hanser*
Rocks	$N_{\gamma,strip}$ $N_\gamma : N_\phi^{1/2}(N_\phi^2 - 1)$ $N_\phi : Tan^2(45° + \phi/2)$	N/A	*Kulhawy and Goo*

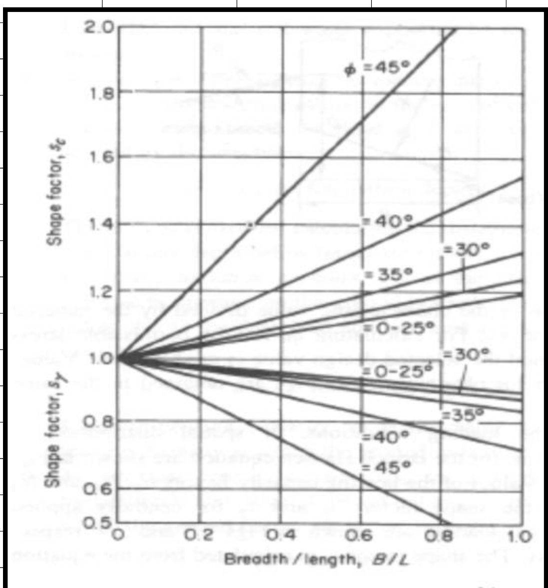

Fig. 2.9 Shape factors s_c and s_γ (after Brinch Hansen[2.1]).

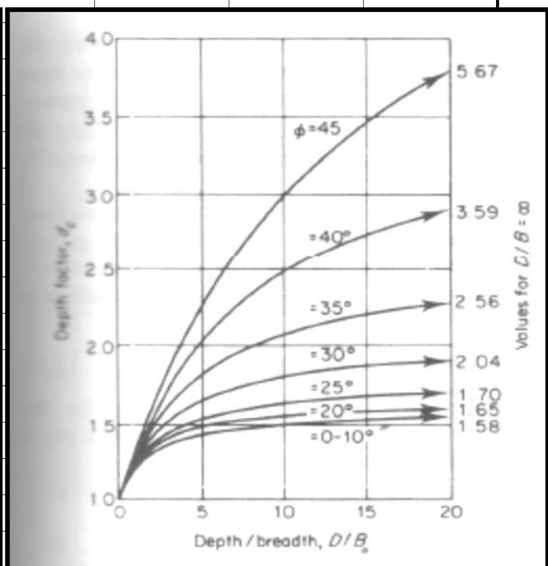

Fig. 2.11 Depth factor d_c (after Brinch Hansen[2.1]).

Shape of base	s_c	s_q	s_γ
Continuous strip	1.0	1.0	1.0
Rectangle	1 + 0.2*B/L*	1 + 0.2*B/L*	1 − 0.4*B/L*
Square	1.3	1.2	0.8
Circle (*B* = diameter)	1.3	1.2	0.6

Net effective working pressure, $q_{wnet}' = q_w' - (p_0'$ or 0)	**N/A**	kPa
Gross working pressure, q_w	**N/A**	kPa
Water pressure at founding level, u = γ_w . MAX (D − z_u, 0)	**N/A**	kPa
Gross effective working pressure, $q_w' = q_w$ - u	**N/A**	kPa

Note a negative q_{wnet}' *indicates an excavation, the following analysis ascertains the susceptibility of the system to base heave instability, conservatively however ignoring the contribution of the shearing resistance of the soil interface above the founding level and any wall embedment below the founding level;*

Drained overall net effective bearing capacity (factored), q_{fnet}' / FOS_1	**N/A**	kPa	
Drained overall net effective bearing capacity utilisation = ABS (q_{wnet}') / (q_{fne}	**N/A**		**N/A**

Note an absolute function is applied to the above to present the susceptibility to base heave instability as well as the overall net effective bearing capacity;

dman

Empirical Overall Net Effective Bearing Capacity

Effective surcharge above founding level, p_0'		12	kPa
Case when $(z_u - D) >= MAX(B, L)$		*Invalid*	
	$p_0' = p_{surface} + \gamma_{dry}.D$	N/A	kPa
Case when $0 < (z_u - D) < MAX(B, L)$		*Invalid*	
	$p_0' = p_{surface} + \gamma_{dry}.D$	N/A	kPa
r) *Case when $(z_u - D) = 0$*		*Valid*	
dman	$p_0' = p_{surface} + \gamma_{dry}.D$	12	kPa
Case when $(z_u - D) < 0$ and $z_u >= 0$		*Invalid*	
	$p_0' = p_{surface} + (\gamma_{sat} - \gamma_w).(D - z_u) + \gamma_{dry}.z_u$	N/A	kPa
Case when $z_u < 0$		*Invalid*	
	$p_0' = p_{surface} + \gamma_{sat}.D + \gamma_w.(-z_u) - \gamma_w.(D + (-z_u))$	N/A	kPa
)	*Note that the above equation reduces to $p_0' = p_{surface} + (\gamma_{sat} - \gamma_w).D$;*		
dman			
Net effective bearing capacity, $q_{fnet}' = q_f' - p_0'$		300	kPa
Gross effective bearing capacity, $q_f' = BC + p_0'$		312	kPa
Net effective working pressure, $q_{wnet}' = q_w' - (p_0'$ or $0)$		91	kPa
Gross working pressure, q_w		102	kPa
Water pressure at founding level, $u = \gamma_w . MAX(D - z_u, 0)$		0	kPa
Gross effective working pressure, $q_w' = q_w - u$		102	kPa

Note a negative q_{wnet}' indicates an excavation, the following analysis ascertains the susceptibility of the system to base heave instability, conservatively however ignoring the contribution of the shearing resistance of the soil interface above the founding level and any wall embedment below the founding level;

Empirical overall net effective bearing capacity (factored), q_{fnet}' / FOS_1	100	kPa	
Empirical overall net effective bearing capacity utilisation = ABS $(q_{wnet}') / (q_{fr}$	91%		OK

Note an absolute function is applied to the above to present the susceptibility to base heave instability as well as the overall net effective bearing capacity;

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Overall Sliding Resistance Capacity

Note for simplicity, only frictional resistance capacity considered, passive resistance of soil in an embedded foundation not considered, arguably rightly so, however reference should be made to retaining wall analysis should this additional capacity be required and can be guaranteed;

Dead load component of SLS vertical (downward) load, $k_{SLStoDL}$		0.60		

Note that the dead load component is applied to reduce the SLS vertical (downward) load to the dead vertical (downward) load, this required as the live load component cannot be guaranteed;

Total foundation dead vertical load, $k_{SLStoDL}.F_v'$		**26**	kN or kN/m	
Total foundation SLS horizontal load, F_h		**0**	kN or kN/m	
Overall sliding resistance capacity (factored), $F_{s,cap}$		**7**	kN or kN/m	
Undrained Analysis	$F_{s,cap}$ = (B'.L').S_u / FOS_2	N/A	kN or kN/r	*Tomlinson*
Drained Analysis	$F_{s,cap}$ = $k_{SLStoDL}.F_v'.\tan\delta'$ / FOS_2	N/A	kN or kN/r	*Tomlinson*
Empirical Analysis	$F_{s,cap}$ = $k_{SLStoDL}.F_v'.\tan\delta'$ / FOS_2	7	kN or kN/r	*Tomlinson*
Overall sliding resistance capacity utilisation = F_h / $F_{s,cap}$		**0%**		**OK**

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		Member/Location		
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Overall Uplift Resistance Capacity

Overall uplift in width resistance capacity utilisation = e_B / $e_{B,limit}$	**N/A**	**N/A**
Overall uplift in length resistance capacity utilisation = e_L / $e_{L,limit}$	**N/A**	**N/A**
Overall uplift resistance capacity utilisation = MAX (e_B / $e_{B,limit}$, e_L / $e_{L,limit}$)	**N/A**	**N/A**

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		Member/Location		
Job Title	Structure, Member Design - Geotechnics Pad, Strip and	Drg.		
Structure, Member Design - Geotechnics Pad, Strip and Raft		Made by XX	Date 21/11/2021	Chd.

Overall Overturning Resistance Capacity

Overall overturning in width resistance capacity utilisation = $M_{ot,B}$ / $M_{rt,B}$	**0%**	**OK**
Overall overturning in length resistance capacity utilisation = $M_{ot,L}$ / $M_{rt,L}$	**0%**	**OK**
Overall overturning resistance capacity utilisation = MAX ($M_{ot,B}$ / $M_{rt,B}$, $M_{ot,L}$ /	**0%**	**OK**

Cell References									
concrete grade								**3**	
25									
30									
35									
40									
45									
50									
55									
60									
65									
70									
75									
80									
85									
90									
95									
100									
105									
110									
115									
120									
longitudinal reinforcement steel grade								**2**	
250									
460									
shear link reinforcement steel grade								**2**	
250									
460									
type of concrete								**1**	
Normal Weight									
Light Weight									
soil name								**36**	
undrained shear strength limit to adopt								**1**	
Lower Limit									
Middle Limit									
Upper Limit									
ignore effective cohesion								**2**	
Include									
Exclude									
effective angle of friction								**5**	
1.00ø' (Cast in Place Concrete - Soil Interface)								1.00	
0.90ø' (Precast Concrete - Soil Interface)								0.90	
0.85ø' (Timber - Soil Interface)								0.85	
0.80ø' (Rough Corrugated Steel - Soil Interface)								0.80	
0.66ø' (Insitu Concrete Active Zone - Soil Interface)								0.66	
0.60ø' (Smooth Coated Steel - Soil Interface)								0.60	
0.50ø' (Insitu Concrete Passive Zone - Soil Interface)								0.50	

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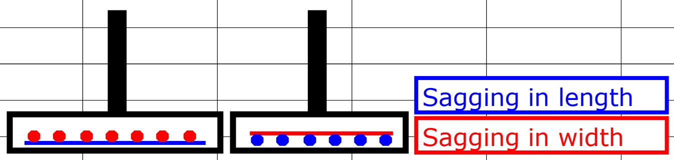
Pad Footing Foundation Dimensions

Width, B_{pad} (<=L_{pad})	0.600	m	OK
Length, L_{pad} (>=B_{pad})	0.750	m	OK
Thickness beneath base slab, $t_{1,pad}$	0.200	m	
Thickness of base slab, $t_{2,pad}$ (if no base slab, then enter 0.000m)	0.000	m	
Thickness of foundation, T_{pad} = $t_{1,pad}$ + $t_{2,pad}$	0.200	m	
Column base section type *(for punching shear only)*	Rectangular		
Column base location *(for punching shear only)*	Edge for Span in Width Direction		
Column base depth, h (rectangular) or diameter, D (Perpendicular to Edge	230	mm	
Column base width, b (rectangular) or N/A (circular) Parallel to Edge	230	mm	

Note where applicable, it is assumed that h is in same plane as L_{pad} and that the column base is always interior and located in the centre of the pad footing B_{pad} and L_{pad};

Pad Footing Foundation Reinforcement



Sagging steel reinforcement diameter in width, ϕ_{sx}	12	mm
Sagging steel reinforcement pitch for resistance in width, p_{sx}	150	mm
Sagging steel area provided in width, $A_{s,prov,x,s}$ = $(\pi.\phi_{sx}^2/4)/p_{sx}$	754	mm^2/m
Sagging steel reinforcement diameter in length, ϕ_{sy}	12	mm
Sagging steel reinforcement pitch for resistance in length, p_{sy}	150	mm
Sagging steel area provided in length, $A_{s,prov,y,s}$ = $(\pi.\phi_{sy}^2/4)/p_{sy}$	754	mm^2/m
Shear link diameter for first shear perimeter, $\phi_{link,2}$	None	mm
Number of link legs for first shear perimeter, $n_{l,2}$	30	
Area provided by all links for first shear perimeter, $A_{sv,prov,2}$ = $n_{l,2}.\pi.\phi_{link,2}^2/4$	0	mm^2
Shear link diameter for second shear perimeter, $\phi_{link,3}$	None	mm
Number of link legs for second shear perimeter, $n_{l,3}$	30	
Area provided by all links for second shear perimeter, $A_{sv,prov,3}$ = $n_{l,3}.\pi.\phi_{link,3}^2/$	0	mm^2
Shear link diameter for bending in width, $\phi_{link,x}$ = $\phi_{link,2}$	0	mm
Number of link legs per metre for bending in width, $n_{link,x}$	4	/m
Area provided by all links per metre for bending in width, $A_{sv,prov,x}$ = $n_{link,x}.\pi.\phi$	0	mm^2/m
Pitch of links for bending in width, S_x	150	mm
Shear link diameter for bending in length, $\phi_{link,y}$ = $\phi_{link,2}$	0	mm
Number of link legs per metre for bending in length, $n_{link,y}$	4	/m
Area provided by all links per metre for bending in length, $A_{sv,prov,y}$ = $n_{link,y}.\pi.$	0	mm^2/m
Pitch of links for bending in length, S_y	150	mm
Effective depth to sagging steel in width, $d_{x,s}$ = T_{pad} - $cover_1$ - MAX ($\phi_{link,2}$, ϕ_{lin}	132	mm
Effective depth to sagging steel in length, $d_{y,s}$ = T_{pad} - $cover_1$ - MAX ($\phi_{link,2}$, ϕ_{li}	144	mm

It is assumed that sagging steel in length is exterior to sagging steel in width;

Estimated steel reinforcement quantity	59	kg/m^3

[7.850 . ($A_{s,prov,x,s}$ +$A_{s,prov,y,s}$) / T_{pad}]; No curtailment; No laps; Links ignored;

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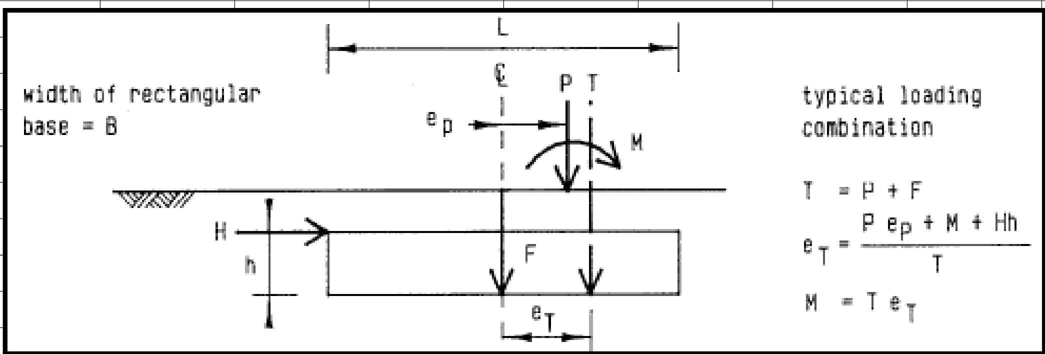
Pad Footing Foundation SLS Loading

SLS vertical (downward) load from column and base slab (if suspended), $F_{col,}$	36	kN	OK
Eccentricity of $F_{col,v}$ from centroid in width, e_1	0.025	m	
Eccentricity of $F_{col,v}$ from centroid in length, e_2	0.000	m	
SLS horizontal load from column in width, $F_{col,h1}$ (defined to add to e_1 eccentr	0	kN	
SLS horizontal load from column in length, $F_{col,h2}$ (defined to add to e_2 eccent	0	kN	
SLS moment from column in plane of width, $M_{col,1}$ (defined to add to e_1 eccer	0	kNm	
SLS moment from column in plane of length, $M_{col,2}$ (defined to add to e_2 ecce	0	kNm	
Note $F_{col,h1/h2}$ and $M_{col,1/2}$ are defined to add to the corresponding eccentricities, thus enter positive values;			
Pad footing (projection beneath base slab) weight, $F_{under,pad} = B_{pad}.L_{pad}.t_{1,pad}.\rho$	2	kN	
Additional soil (above footing) weight, $F_{above,soil} = B_{pad}.L_{pad}.MAX(0$ Include	4	kN	
Note additional soil above the footing is included for embedded footings whereby the top of the footing			
is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;			
Note that this has a stabilizing effect on footings subject to destabilizing moments, thus both			
inclusive and exclusive cases should be considered;			
Water pressure at founding level, $u = \gamma_w$. MAX (D – z_u, 0)	0	kPa	
Water uplift force at founding level, $F_{water} = u.B_{pad}.L_{pad}$	0	kN	
Total foundation SLS vertical (downward) load, $F_{pad,v} = F_{col,v} + F_{under,pad} + F_{ab}$	43	kN	
Total foundation SLS effective vertical (downward) load, $F_{pad,v}' = F_{pad,v} - F_{wate}$	43	kN	
Total foundation SLS horizontal load, $F_{pad,h} = (F_{col,h1}{}^2 + F_{col,h2}{}^2)^{0.5}$	0	kN	



Equivalent eccentricity in width, $e_B = ABS(F_{col,v}.e_1 + M_{col,1} + F_{col,h1}.T_{pad}) / F_{pad}$	0.021	m
Limiting eccentricity for no overall uplift (factored), $e_{B,limit} = (B_{pad} / 6) / FOS_3$	0.100	m
Equivalent eccentricity in length, $e_L = ABS(F_{col,v}.e_2 + M_{col,2} + F_{col,h2}.T_{pad}) / F_{pa}$	0.000	m
Limiting eccentricity for no overall uplift (factored), $e_{L,limit} = (L_{pad} / 6) / FOS_3$	0.125	m
Overturning moment in width, $M_{ot,B} = M_{col,1} + F_{col,h1}.T_{pad}$	0	kNm
Restoring moment in width, $M_{rt,B} = [F_{col,v}.(B_{pad}/2-e_1) + (F_{under,pad}+F_{above,soil}-F_{w}$	7	kNm
Overturning moment in length, $M_{ot,L} = M_{col,2} + F_{col,h2}.T_{pad}$	0	kNm
Restoring moment in length, $M_{rt,L} = [F_{col,v}.(L_{pad}/2-e_2) + (F_{under,pad}+F_{above,soil}-F_{w}$	10	kNm

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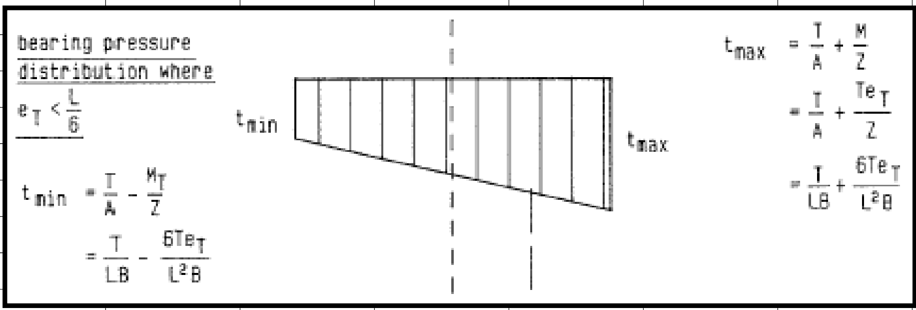
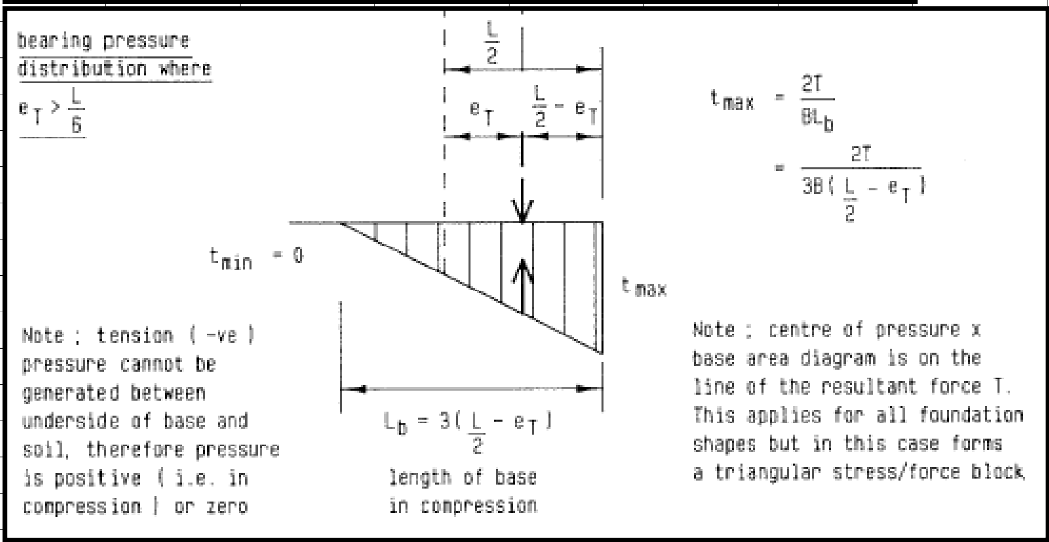
Structure, Member Design - Geotechnics Pad, Strip and Raft		Made by XX	Date 21/11/2021	Chd.





Maximum gross working pressure in width, $q_{w1,B} = F_{pad,v}/(B_{pad}.L_{pad}) + 6.(F_{col,v}$	115	kPa
Minimum gross working pressure in width, $q_{w2,B} = F_{pad,v}/(B_{pad}.L_{pad}) - 6.(F_{col,v}.$	75	kPa
Maximum gross working pressure in length, $q_{w1,L} = F_{pad,v}/(B_{pad}.L_{pad}) + 6.(F_{col,}$	95	kPa
Minimum gross working pressure in length, $q_{w2,L} = F_{pad,v}/(B_{pad}.L_{pad}) - 6.(F_{col,v}$	95	kPa
Maximum gross working pressure in width, $q_{w1,B} = 2F_{pad,v}/[3L_{pad}.(B_{pad}/2\text{-}e_B)]$	N/A	kPa
Minimum gross working pressure in width, $q_{w2,B} = 0.0$	N/A	kPa
Maximum gross working pressure in length, $q_{w1,L} = 2F_{pad,v}/[3B_{pad}.(L_{pad}/2\text{-}e_L)]$	N/A	kPa
Minimum gross working pressure in length, $q_{w2,L} = 0.0$	N/A	kPa
Equivalent width, $B_{pad}' = B_{pad} - 2e_B$	0.557	m
Equivalent length, $L_{pad}' = L_{pad} - 2e_L$	0.750	m
Gross working pressure, $q_w = F_{pad,v} / (B_{pad}' . L_{pad}')$	102	kPa

Pad Footing Foundation ULS Loading

ULS vertical (downward) load from column and base slab (if suspended), F_{col}	59	kN

Note it is assumed that the ULS load acts at the same eccentricity as the SLS load;

Note that this enhancement is required to cater for the moment as an enhanced load in the ULS design;

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		jXXX	16	
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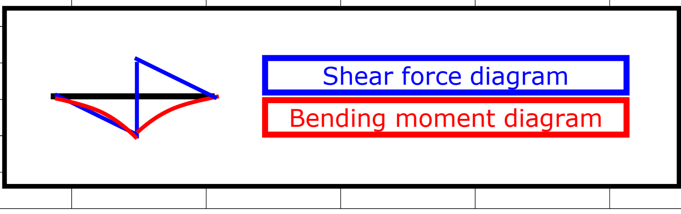
Structure, Member Design - Geotechnics Pad, Strip and Raft		Made by XX	Date 21/11/2021	Chd.

Pad Footing Foundation Reinforcement Design

Gross ULS Pressure

Gross ULS pressure, $q_{w,ULS} = F_{col,v,uls} / (B_{pad} . L_{pad})$		**131**	kPa	



Sagging Bending Moment Design in Plane of Width

Moment at column base face, $M_x = q_{w,ULS} . L_{pad} . [(B_{pad}-(b \text{ or } D))/2]^2 / 2$			**2**	kNm	
Moment at column base face per metre, M_x/L_{pad}			**2**	kNm/m	
Concrete moment capacity per metre, $M_{u,x} = 0.156f_{cu}.1000.d_{x,s}^2$			**95**	kNm/m	
Bending stress, $[M/bd^2]_x = (M_x/L_{pad}) / [(1000).d_{x,s}^2]$			0.13	N/mm^2	
Bending stress ratio, $K_x = [M/bd^2]_x / f_{cu} <= 0.156$			0.004		**OK**
Lever arm, $z_x = d_{x,s} . [0.5 + (0.25-K_x/0.9)^{0.5}] <= 0.95d_{x,s}$			125	mm	
Area of tension steel required, $A_{s,x} = (M_x/L_{pad}) / [(0.95f_y).z_x]$			**41**	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$			754	mm^2/m	
Sagging bending moment in plane of width utilisation = $A_{s,x} / A_{s,prov,x,s}$			**5%**		**OK**
Requirement to concentrate 2/3 rebar within $1.5d_{x,s}$ frc	*375*	< *470*	**No**		*3.11.3.2*
[Yes if $L_{pad}/2>3/4(h \text{ or } D)+9/4d_{x,s}$; No if not;]	*mm*	*mm*			*BS8110*
Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;					
% Min sag reinforcement in plane of width (>= $0.0024.1000.T_{pad}$ G250; >=			0.38	%	
% Min sag reinforcement in plane of width utilisation			**34%**		**OK**

Sagging Bending Moment Design in Plane of Length

Moment at column base face, $M_y = q_{w,ULS} . B_{pad} . [(L_{pad}-(h \text{ or } D))/2]^2 / 2$			**3**	kNm	
Moment at column base face per metre, M_y/B_{pad}			**4**	kNm/m	
Concrete moment capacity per metre, $M_{u,y} = 0.156f_{cu}.1000.d_{y,s}^2$			**113**	kNm/m	
Bending stress, $[M/bd^2]_y = (M_y/B_{pad}) / [(1000).d_{y,s}^2]$			0.21	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$			0.006		**OK**
Lever arm, $z_y = d_{y,s} . [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,s}$			137	mm	
Area of tension steel required, $A_{s,y} = (M_y/B_{pad}) / [(0.95f_y).z_y]$			**74**	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$			754	mm^2/m	
Sagging bending moment in plane of length utilisation = $A_{s,y} / A_{s,prov,y,s}$			**10%**		**OK**
Requirement to concentrate 2/3 rebar within $1.5d_{y,s}$ frc	*300*	< *497*	**No**		*3.11.3.2*
[Yes if $B_{pad}/2>3/4(b \text{ or } D)+9/4d_{y,s}$; No if not;]	*mm*	*mm*			*BS8110*
Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;					
% Min sag reinforcement in plane of length (>= $0.0024.1000.T_{pad}$ G250; >=			0.38	%	
% Min sag reinforcement in plane of length utilisation			**34%**		**OK**

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Punching Shear Design

ULS vertical (downward) load from column and base slab (if suspended), F_{col}	**59**	kN
Area of column base section, A_{c1} = b.h (rectangular) or $\pi D^2/4$ (circular)	52900	mm^2
Average effective depth of both rebar layers, $d = (d_{x,s} + d_{y,s})/2$	138	mm
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	754	mm^2/m
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	754	mm^2/m
Average area of tensile steel reinforcement provided, $A_{s,prov,s}$	754	mm^2/m
$\rho_w = 100A_{s,prov,s}/(1000.d)$	0.55	%
$\nu_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d)^{1/4}>0.67$	**0.75**	N/mm^2

Column Base Face Perimeter

Shear force at column base face, $V_1 = F_{col,v,uls} - q_{w,ULS}.A_{c1}$	52	kN
Effective shear force, $V_{eff,1} = 1.00 . V_1$	52	kN

Note $V_{eff,1} = 1.00 . V_1$ because moment effects have been accounted for in the derivation of $F_{col,v,uls}$;

Column base face perimeter, u_1 — 690 mm

	Rectangular		Circular		
Internal column:	*2.(b+h)*	*920*	*π.D*	*N/A*	mm
Edge column:	*2b+h or 2h+b*	*690*	*3/4(π.D)*	*N/A*	mm
Corner column:	*(b+h)*	*460*	*π.D/2*	*N/A*	mm

Shear stress at column base face perimeter, $\nu_1 = V_{eff,1} / u_1 d$ (< $0.8f_{cu}^{0.5}$ & 5N	**0.55**	N/mm^2	
Ultimate shear stress utilisation	**12%**		**OK**

First Shear Perimeter

Shear force 1.5d from column base face, $V_2 = F_{col,v,uls} - q_{w,ULS}.A_{c2}$ — 22 kN

	Rectangular		Circular		
Internal column:	*(b+3d).(h+3d)*	*0.41*	*$(D+3d)^2$*	*N/A*	m^2
Edge column:	*(b+1.5d).(h+3d) or (h+1.5d).(b+3d)*	*0.28*	*d).(D+3d)*	*N/A*	m^2
Corner column:	*(b+1.5d).(h+1.5d)*	*0.19*	*$(D+1.5d)^2$*	*N/A*	m^2

Effective shear force, $V_{eff,2} = 1.00 . V_2$ — 22 kN

Note $V_{eff,2} = 1.00 . V_2$ because moment effects have been accounted for in the derivation of $F_{col,v,uls}$;

Column base first perimeter, u_2 — 1518 mm

	Rectangular		Circular		
Internal column:	*2.(b+h)+12d*	*2576*	*4D+12d*	*N/A*	mm
Edge column:	*2b+h+6d or 2h+b+6d*	*1518*	*3D+6d*	*N/A*	mm
Corner column:	*(b+h)+3d*	*874*	*2D+3d*	*N/A*	mm

Shear stress at column base first perimeter, $\nu_2 = V_{eff,2} / u_2 d$ — **0.11** N/mm^2

*(Shear capacity enhancement by calculating v_d at 1.5d from "support" and comparing against unenhanced v_c as clause 3.7.7.6 BS8110 employed **instead of** calculating v_d at "support" and comparing against enhanced v_c within 1.5d of the "support" as clause 3.7.7.4 BS8110;)*

Case $\nu_2 < \nu_c$ — **VALID**

No links required.

Case $\nu_c < \nu_2 < 1.6\nu_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

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Case $1.6v_c < v_2 < 2.0v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $v_2 > 2.0v_c$ — **N/A**

First shear perimeter shear utilisation — **14%** — **OK**

Second Shear Perimeter

Shear force 2.25d from column base face, $V_3 = F_{col,v,uls} - q_{w,ULS}.A_{c3}$ — -1 kN

	Rectangular		Circular		
Internal column:	*(b+4.5d).(h+4.5d)*	*0.72*	*(D+4.5d)²*	*N/A*	m^2
Edge column:	*(b+2.25d).(h+4.5d) or (h+2.25d).(b+4.5d)*	*0.46*	*.(D+4.5d)*	*N/A*	m^2
Corner column:	*(b+2.25d).(h+2.25d)*	*0.29*	*(D+2.25d)²*	*N/A*	m^2

Effective shear force, $V_{eff,3} = 1.00 . V_3$ — -1 kN

Note $V_{eff,3} = 1.00 . V_3$ because moment effects have been accounted for in the derivation of $F_{col,v,uls}$;

Column base second perimeter, u_3 — 1932 mm

	Rectangular		Circular		
Internal column:	*2.(b+h)+18d*	*3404*	*4D+18d*	*N/A*	mm
Edge column:	*2b+h+9d or 2h+b+9d*	*1932*	*3D+9d*	*N/A*	mm
Corner column:	*(b+h)+4.5d*	*1081*	*2D+4.5d*	*N/A*	mm

Shear stress at column base second perimeter, $v_3 = V_{eff,3} / u_3d$ — **0.00** N/mm^2

Case $v_3 < v_c$ — **VALID**

No links required.

Case $v_c < v_3 < 1.6v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{(v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $1.6v_c < v_3 < 2.0v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $v_3 > 2.0v_c$ — **N/A**

Second shear perimeter shear utilisation — **0%** — **OK**

Note a negative shear stress v_2 and/or v_3 on a correctly specified column (wrt internal, edge or corner) indicates that the shear perimeter is beyond the physical extremes of the foundation and as such punching shear failure is not critical;

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Shear Design for Bending in Plane of Width

Shear force at column base face, $V_{x,ult} = q_{w,ULS} . L_{pad} . [(B_{pad}-(b \text{ or } D))/2]$	**18**	kN	
Shear force at column base face per metre, $V_{x,ult}/L_{pad}$	**24**	kN/m	
Shear force at $1.0d_{x,s}$ from column base face, $V_x = q_{w,ULS} . L_{pad} . [(B_{pad}-(b$ or	**5**	kN	
Shear force at $1.0d_{x,s}$ from column base face per metre, V_x/L_{pad}	**7**	kN/m	
Note the above shear forces are for bending in plane of width;			
Ultimate shear stress for bending in plane of width, $v_{ult,x}=(V_{x,ult}/L_{pad})/(1000.d$	**0.18**	N/mm^2	
Ultimate shear stress for bending in plane of width utilisation	**4%**		**OK**
Design shear stress for bending in plane of width, $v_{d,x}=(V_x/L_{pad})/(1000.d_{x,s})$	**0.05**	N/mm^2	

(Shear capacity enhancement by calculating v_d at d from "support" and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)*

Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	754	mm^2/m	
$\rho_w = 100A_{s,prov,x,s}/(1000.d_{x,s})$	0.57	%	
$v_{c,x} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{x,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{x,s})^{1/4}>0.67$	**0.77**	N/mm^2	
Check $v_{d,x} < v_{c,x}$ for no links	**VALID**		
Concrete shear capacity $v_{c,x}.(1000.d_{x,s})$	**102**	kN/m	
Check $v_{c,x} < v_{d,x} < 0.4 + v_{c,x}$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**0.92**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_{c,x}).(1000.d_{x,s})$	**155**	kN/m	
Check $v_{d,x} > 0.4 + v_{c,x}$ for design links	**N/A**		
Provide shear links $A_{sv} / S > 1000.(v_{d,x}-v_{c,x})/(0.95f_{yv})$ i.e. A_{sv} / S >	**0.92**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov,x}/S_x).(0.95f_{yv}).d_x$	**102**	kN/m	
Area provided by all links per metre, $A_{sv,prov,x}$	0	mm^2/m	
Tried $A_{sv,prov,x} / S_x$ value	0.00	$mm^2/mm/m$	
Design shear resistance for bending in plane of width utilisation	**7%**		**OK**

Shear Design for Bending in Plane of Length

Shear force at column base face, $V_{y,ult} = q_{w,ULS} . B_{pad} . [(L_{pad}-(h \text{ or } D))/2]$	20	kN	
Shear force at column base face per metre, $V_{y,ult}/B_{pad}$	34	kN/m	
Shear force at 1.0$d_{y,s}$ from column base face, $V_y = q_{w,ULS} . B_{pad} . [(L_{pad}-(h \text{ or }$	9	kN	
Shear force at 1.0$d_{y,s}$ from column base face per metre, V_y/B_{pad}	15	kN/m	
Note the above shear forces are for bending in plane of length;			
Ultimate shear stress for bending in plane of length, $v_{ult,y}=(V_{y,ult}/B_{pad})/(1000.$	0.24	N/mm^2	
Ultimate shear stress for bending in plane of length utilisation	5%		OK
Design shear stress for bending in plane of length, $v_{d,y}=(V_y/B_{pad})/(1000.d_{y,s})$	0.11	N/mm^2	

*(Shear capacity enhancement by calculating v_d at d from "support" and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 employed **instead of** calculating v_d at "support" and comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)*

Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	754	mm^2/m	
$\rho_w = 100A_{s,prov,y,s}/(1000.d_{y,s})$	0.52	%	
$v_{c,y} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{y,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{y,s})^{1/4}>0.67$	0.74	N/mm^2	
Check $v_{d,y} < v_{c,y}$ for no links	VALID		
Concrete shear capacity $v_{c,y}.(1000.d_{y,s})$	106	kN/m	
Check $v_{c,y} < v_{d,y} < 0.4 + v_{c,y}$ for nominal links	N/A		
Provide nominal links such that A_{sv} / S > 0.4.(1000)/(0.95f_{yv}) i.e.	0.92	mm^2/mm/m	
Concrete and nominal links shear capacity (0.4 + $v_{c,y}$).(1000.$d_{y,s}$)	164	kN/m	
Check $v_{d,y} > 0.4 + v_{c,y}$ for design links	N/A		
Provide shear links A_{sv} / S > 1000.($v_{d,y}$-$v_{c,y}$)/(0.95f_{yv}) i.e. A_{sv} / S >	0.92	mm^2/mm/m	
Concrete and design links shear capacity ($A_{sv,prov,y}/S_y$).(0.95f_{yv}).d_y	106	kN/m	
Area provided by all links per metre, $A_{sv,prov,y}$	0	mm^2/m	
Tried $A_{sv,prov,y}$ / S_y value	0.00	mm^2/mm/m	
Design shear resistance for bending in plane of length utilisation	14%		OK

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Detailing Requirements

All detailing requirements met ?	NOT OK		
Max sagging steel reinforcement pitch in plane of width (<3$d_{x,s}$, <750mm)	150	mm	OK
Max sagging steel reinforcement pitch in plane of length (<3$d_{y,s}$, <750mm)	150	mm	OK

Maximum spacing: 0.5% Ast or less - 300mm
Between 0.5% and 1.0% - 225mm
1.0% Ast or greater - 175mm

Max sagging steel reinforcement pitch in plane of width	150	mm	OK
Max sagging steel reinforcement pitch in plane of length	150	mm	OK
Min sagging steel reinforcement pitch in plane of width (>100mm)	150	mm	OK
Min sagging steel reinforcement pitch in plane of length (>100mm)	150	mm	OK

Note no allowance has been made for laps in the min pitch as not deemed to be required;

% Max sagging reinforcement in plane of width (<= 0.04.1000.T_{pad})	0.38	%	OK
% Max sagging reinforcement in plane of length (<= 0.04.1000.T_{pad})	0.38	%	OK
Sagging steel reinforcement diameter in plane of width, ϕ_{sx} (>=16mm)	12	mm	NOT OK
Sagging steel reinforcement diameter in plane of length, ϕ_{sy} (>=16mm)	12	mm	NOT OK

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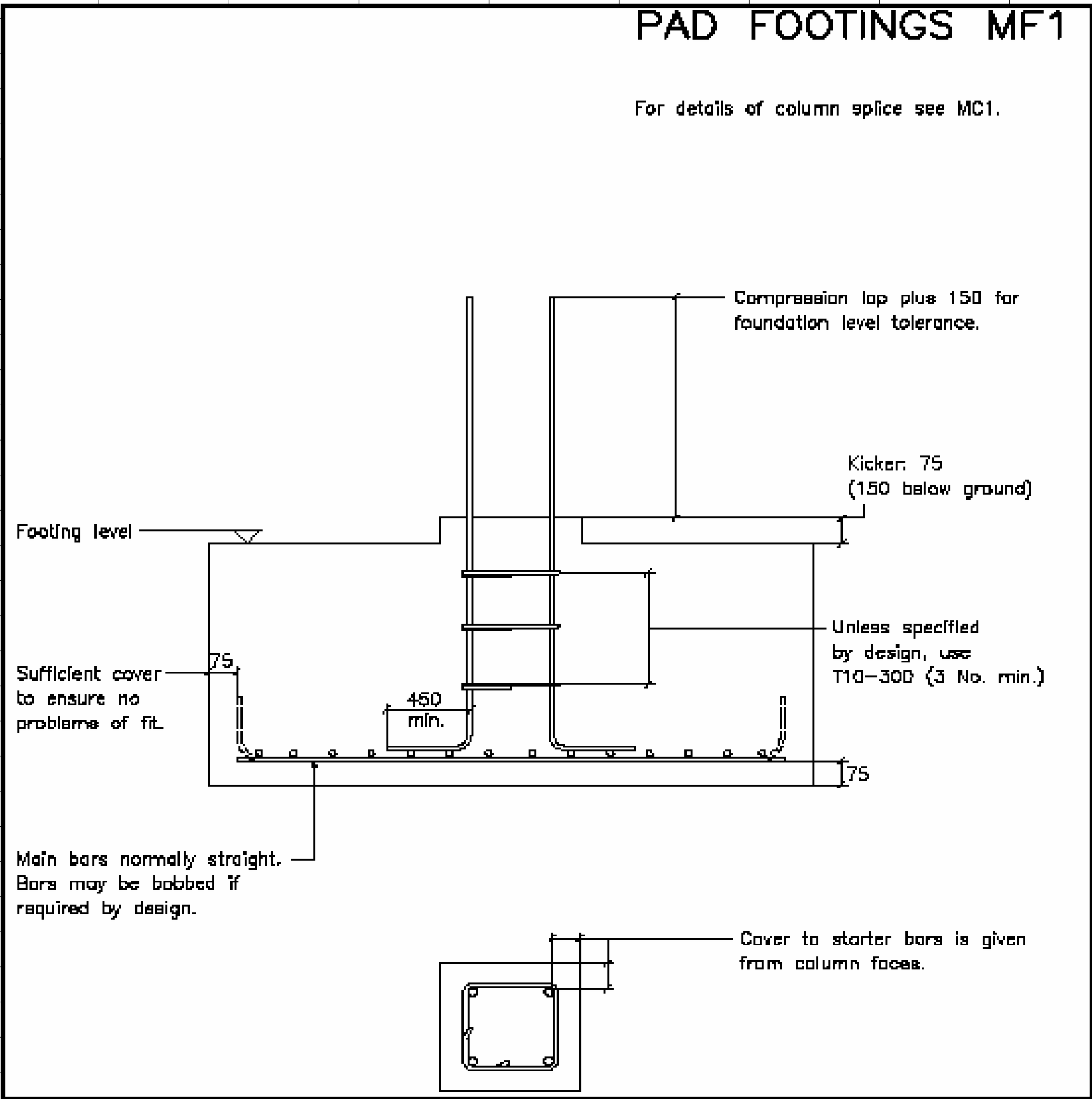
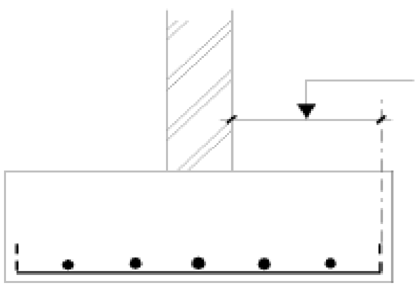
Standard Pad Footing Foundation Reinforcement Details



2.2.5.1 Pad Footings and Column Strips

Straight bars are normally used without curtailment, and should be detailed if nothing else is specified. However, an anchorage length should be provided from the face of the wall or column to the end of the bars. This may require bobs to be bent at the ends of bars.



Lap lengths provided (for nominal bars, etc.) should not be less than 15 times the bar size or 300mm, whichever is greater.

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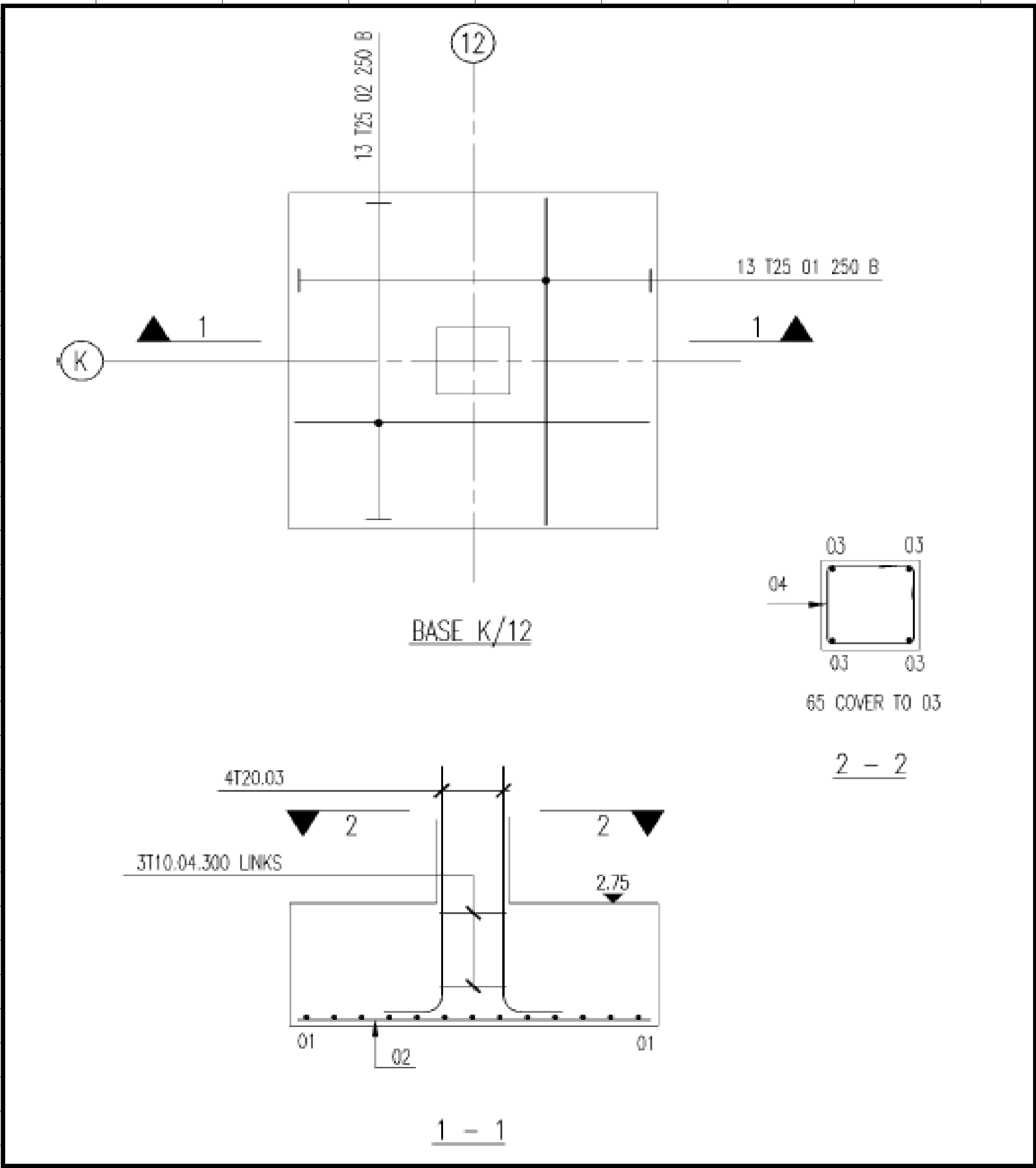

0.40ø'								0.40	
0.30ø'								0.30	
0.20ø'								0.20	
0.10ø'								0.10	
0.00ø' (No Friction Interface)								0.00	
bearing capacity limit to adopt								**3**	
Lower Limit									
Middle Limit									
Upper Limit									
bearing capacity values from allowable bearing capacity, $BC_{ll,a/ul,a}$ values or SP								**2**	
$BC_{ll,a/ul,a}$									
N									
factor for SPT, N value								**2**	
Undrained Soil: 42.6								42.6	
Drained Soil: 30.0								30.0	
ground water level modification for bearing capacity							**1**	**2**	
						GWL >= B	GWL < B	h Flooding	
Cohesive Soil		GWL >= B				1.00	1.00	0.67	
Non Cohesive Soil		GWL < B				1.00	0.50	0.50	
Rock		With Flooding				1.00	1.00	1.00	
method of analysis								**3**	
Undrained Analysis									
Drained Analysis									
Empirical Analysis									
evaluate overall uplift resistance								**2**	
Yes									
No									
foundation type								**1**	
Pad Footing									
Strip Footing									
Multi Column Footing									
Combined Footing									
Strap Footing									
Raft									
consider surcharge above founding level in net (effective) working pressure								**1**	
Yes									
No									
column base section type					**1**	**1**	**1**	**1**	
Rectangular									
Circular									
column base location					**2**	**1**	**1**	**2**	
Interior									
Edge for Span in Width Direction									
Edge for Span in Length Direction									

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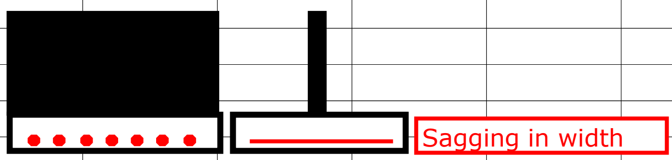
Strip Footing Foundation Dimensions

Width, B_{strip}	2.500	m
Thickness beneath base slab, $t_{1,strip}$	1.000	m
Thickness of base slab, $t_{2,strip}$ (if no base slab, then enter 0.000m)	0.000	m
Thickness of foundation, $T_{strip} = t_{1,strip} + t_{2,strip}$	N/A	m
Wall width, b	400	mm

Note where applicable, it is assumed that the wall is always interior and located in the centre of the strip footing B_{strip};

Strip Footing Foundation Reinforcement



Sagging steel reinforcement diameter, ϕ_s	20	mm
Sagging steel reinforcement pitch, p_s	200	mm
Sagging steel area provided, $A_{s,prov,s} = (\pi.\phi_s^2/4)/p_s$	N/A	mm^2/m
Shear link diameter, ϕ_{link}	10	mm
Number of link legs per metre, n_{link}	4	/m
Area provided by all links per metre, $A_{sv,prov} = n_{link}.\pi.\phi_{link}^2/4$	N/A	mm^2/m
Pitch of links, S	150	mm
Effective depth to sagging steel, $d_s = T_{strip}$ - $cover_1$ - ϕ_{link} - $\phi_s/2$	N/A	mm
Estimated steel reinforcement quantity	N/A	kg/m^3

[7.850 . ($A_{s,prov,s}$) / T_{strip}]; No curtailment; No laps; Links ignored; Distribution steel ignored;

Strip Footing Foundation SLS Loading

SLS vertical (downward) load from wall and base slab (if suspended), $F_{wall,v}$	1000	kN/m	N/A
Eccentricity of $F_{wall,v}$ from centroid, e	0.100	m	
SLS horizontal load from wall, $F_{wall,h}$ (defined to add to e eccentricity)	0	kN/m	
SLS moment from wall, M_{wall} (defined to add to e eccentricity)	0	kNm/m	

Note $F_{wall,h}$ and M_{wall} are defined to add to the corresponding eccentricity, thus enter positive values;

Strip footing (projection beneath base slab) weight, $F_{under,strip} = B_{strip}.t_{1,strip}.\rho_c$	N/A	kN/m
Additional soil (above footing) weight, $F_{above,soil} = B_{strip}$.MAX(0, D- [Exclude]	N/A	kN/m

Note additional soil above the footing is included for embedded footings whereby the top of the footing is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;

Note that this has a stabilizing effect on footings subject to destabilizing moments, thus both inclusive and exclusive cases should be considered;

Water pressure at founding level, $u = \gamma_w$. MAX (D – z_u, 0)	N/A	kPa
Water uplift force at founding level, $F_{water} = u.B_{strip}$	N/A	kN/m
Total foundation SLS vertical (downward) load, $F_{strip,v} = F_{wall,v} + F_{under,strip}$ + F	N/A	kN/m
Total foundation SLS effective vertical (downward) load, $F_{strip,v}' = F_{strip,v}$ - F_{wat}	N/A	kN/m
Total foundation SLS horizontal load, $F_{strip,h} = F_{wall,h}$	N/A	kN/m

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Equivalent eccentricity, e_B = ABS($F_{wall,v}$.e + M_{wall} + $F_{wall,h}$.T_{strip}) / $F_{strip,v}$'	N/A	m
Limiting eccentricity for no overall uplift (factored), $e_{B,limit}$ = (B_{strip} / 6) / FOS_3	N/A	m
Overturning moment, $M_{ot,B}$ = M_{wall} + $F_{wall,h}$.T_{strip}	N/A	kNm/m
Restoring moment, $M_{rt,B}$ = [$F_{wall,v}$.(B_{strip}/2-e) + ($F_{under,strip}$+$F_{above,soil}$-F_{water}).B_{strip}	N/A	kNm/m
Maximum gross working pressure, q_{w1} = $F_{strip,v}$/B_{strip} + 6.($F_{wall,v}$.e + M_{wall} + F_{w}	N/A	kPa
Minimum gross working pressure, q_{w2} = $F_{strip,v}$/B_{strip} – 6.($F_{wall,v}$.e + M_{wall} + F_{wa}	N/A	kPa
Maximum gross working pressure, q_{w1} = 2$F_{strip,v}$/[3.(B_{strip}/2-e_B)]	N/A	kPa
Minimum gross working pressure, q_{w2} = 0.0	N/A	kPa
Equivalent width, B_{strip}' = B_{strip} – 2e_B	**N/A**	m
Gross working pressure, q_w = $F_{strip,v}$ / B_{strip}'	**N/A**	kPa

Strip Footing Foundation ULS Loading

ULS vertical (downward) load from wall and base slab (if suspended), $F_{wall,v,ul}$	**N/A**	kN/m

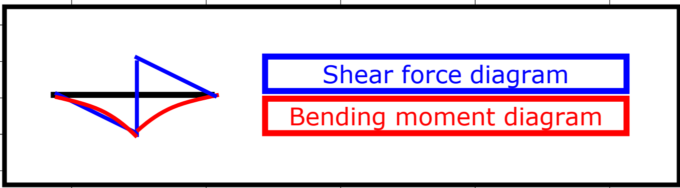
Note it is assumed that the ULS load acts at the same eccentricity as the SLS load;

Note that this enhancement is required to cater for the moment as an enhanced load in the ULS design;

Strip Footing Foundation Reinforcement Design

Gross ULS Pressure

Gross ULS pressure, $q_{w,ULS} = F_{wall,v,uls} / B_{strip}$	N/A	kPa	



Sagging Bending Moment Design

Moment at wall face per metre, $M = q_{w,ULS} . [(B_{strip}-b)/2]^2 / 2$	N/A	kNm/m	
Concrete moment capacity per metre, $M_u = 0.156f_{cu}.1000.d_s^2$	N/A	kNm/m	
Bending stress, $[M/bd^2] = M / [(1000).d_s^2]$	N/A	N/mm^2	
Bending stress ratio, $K = [M/bd^2] / f_{cu} <= 0.156$	N/A		N/A
Lever arm, $z = d_s . [0.5 + (0.25-K/0.9)^{0.5}] <= 0.95d_s$	N/A	mm	
Area of tension steel required, $A_s = M / [(0.95f_y).z]$	N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,s}$	N/A	mm^2/m	
Sagging bending moment utilisation = $A_s / A_{s,prov,s}$	N/A		N/A
% Min sag reinforcement (>= $0.0024.1000.T_{strip}$ G250; >= $0.0013.1000.T_{stri}$	N/A	%	
% Min sag reinforcement utilisation	N/A		N/A

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Shear Design

Shear force at wall face per metre, $V_{ult} = q_{w,ULS} . [(B_{strip}-b)/2]$	**N/A**	kN/m	
Shear force at $1.0d_s$ from wall face per metre, $V = q_{w,ULS} . [(B_{strip}-b)/2-d_s]$	**N/A**	kN/m	
Ultimate shear stress, $v_{ult}=V_{ult}/(1000.d_s)$ (< $0.8f_{cu}^{0.5}$ & $5N/mm^2$)	**N/A**	N/mm^2	
Ultimate shear stress utilisation	**N/A**		**N/A**
Design shear stress, $v_d=V/(1000.d_s)$	**N/A**	N/mm^2	

(Shear capacity enhancement by calculating v_d at d from "support" and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)*

Area of tensile steel reinforcement provided, $A_{s,prov,s}$	N/A	mm^2/m	
$\rho_w = 100A_{s,prov,s}/(1000.d_s)$	N/A	%	
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_s)^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_s)^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_d < v_c$ for no links	**N/A**		
Concrete shear capacity $v_c.(1000.d_s)$	**N/A**	kN/m	
Check $v_c < v_d < 0.4 + v_c$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_c).(1000.d_s)$	**N/A**	kN/m	
Check $v_d > 0.4 + v_c$ for design links	**N/A**		
Provide shear links $A_{sv} / S > 1000.(v_d-v_c)/(0.95f_{yv})$ i.e. $A_{sv} / S >$	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov}/S).(0.95f_{yv}).d_s +$	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov}$	N/A	mm^2/m	
Tried $A_{sv,prov}$ / S value	N/A	$mm^2/mm/m$	
Design shear resistance utilisation	**N/A**		**N/A**

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Detailing Requirements

All detailing requirements met ?	N/A		
Max sagging steel reinforcement pitch ($<3d_s$, <750mm)	N/A	mm	N/A

Maximum spacing:	0.5% Ast or less - 300mm Between 0.5% and 1.0% - 225mm 1.0% Ast or greater - 175mm

Max sagging steel reinforcement pitch	N/A	mm	N/A
Min sagging steel reinforcement pitch (>100mm)	N/A	mm	N/A

Note no allowance has been made for laps in the min pitch as not deemed to be required;

% Max sagging reinforcement ($<= 0.04.1000.T_{strip}$)	N/A	%	N/A
Sagging steel reinforcement diameter, ϕ_s (>=16mm)	N/A	mm	N/A

Standard Strip Footing Foundation Reinforcement Details

As per standard pad footing reinforcement details, but in width direction only;

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Corner									
parallel or perpendicular to edge					**2**	**1**	**1**	**2**	
Parallel to Edge			Perpendicular to Edge						
Perpendicular to Edge			Parallel to Edge						
longitudinal sagging rebar diameter				**6**	**6**	**6**	**4**	**4**	
						6	**6**	**6**	
6									
8									
10									
12									
16									
20									
25									
32									
40									
longitudinal hogging rebar diameter						**5**	**5**	**5**	
6									
8									
10									
12									
16									
20									
25									
32									
40									
shear link diameter				**1**	**1**	**4**	**1**	**1**	
				5	**1**	**1**	**1**	**1**	
None									
6									
8									
10									
12									
16									
20									
25									
soil above embedded footing				**2**	**2**	**2**	**2**	**1**	
							2	**2**	
Include									
Exclude									
equations for bearing capacity factors								**1**	
Prandtl, Reissner and Hansen Equations for Soils									
Kulhawy and Goodman Equations for Rocks									
shear case for direction x					**3**	**3**	**3**	**1**	
shear case for direction y					**3**	**3**	**3**	**1**	
shear case								**3**	

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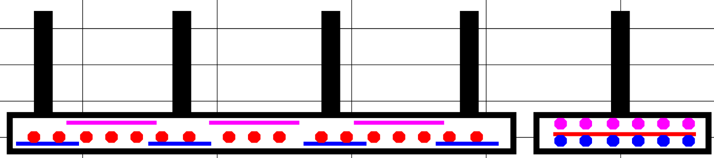
Multi Column Footing Foundation Dimensions

Note that the multi column footing is used when a more efficient use of a footing is required than a conventional pad footing. A pad footing is subject to sagging moments akin to a cantilever beam or slab whilst a multi column footing is subject to both sagging and hogging moments akin to a continuous beam or slab;

Width, B_{multi} (<=L_{multi})	2.200	m	N/A
Length (internal span), L_{multi} (>=B_{multi})	3.500	m	N/A
Thickness beneath base slab, $t_{1,multi}$	0.800	m	
Thickness of base slab, $t_{2,multi}$ (if no base slab, then enter 0.000m)	0.000	m	
Thickness of foundation, T_{multi} = $t_{1,multi}$ + $t_{2,multi}$	N/A	m	
Column base section type *(for punching shear only)*	Rectangular		
Column base location *(for punching shear only)*	Interior		
Column base depth, h (rectangular) or diameter, D (Parallel to Edge	400	mm	
Column base width, b (rectangular) or N/A (circular) Perpendicular to Edge	400	mm	

Note where applicable, it is assumed that h is in same plane as L_{multi} and that the column base is always interior and located in the centre of the multi column footing B_{multi};

Multi Column Footing Foundation Reinforcement



Sagging steel reinforcement diameter in width, ϕ_{sx}	20	mm
Sagging steel reinforcement pitch for resistance in width, p_{sx}	200	mm
Sagging steel area provided in width, $A_{s,prov,x,s}$ = $(\pi.\phi_{sx}^2/4)/p_{sx}$	N/A	mm^2/m
Sagging steel reinforcement diameter in length, ϕ_{sy}	20	mm
Sagging steel reinforcement pitch for resistance in length, p_{sy}	200	mm
Sagging steel area provided in length, $A_{s,prov,y,s}$ = $(\pi.\phi_{sy}^2/4)/p_{sy}$	N/A	mm^2/m
Hogging steel reinforcement diameter in length, ϕ_{hy}	16	mm
Hogging steel reinforcement pitch for resistance in length, p_{hy}	200	mm
Hogging steel area provided in length, $A_{s,prov,y,h}$ = $(\pi.\phi_{hy}^2/4)/p_{hy}$	N/A	mm^2/m
Shear link diameter for first shear perimeter, $\phi_{link,2}$	None	mm
Number of link legs for first shear perimeter, $n_{l,2}$	30	
Area provided by all links for first shear perimeter, $A_{sv,prov,2}$ = $n_{l,2}.\pi.\phi_{link,2}^2/4$	N/A	mm^2
Shear link diameter for second shear perimeter, $\phi_{link,3}$	None	mm
Number of link legs for second shear perimeter, $n_{l,3}$	30	
Area provided by all links for second shear perimeter, $A_{sv,prov,3}$ = $n_{l,3}.\pi.\phi_{link,3}^2/$	N/A	mm^2
Shear link diameter for bending in width, $\phi_{link,x}$ = $\phi_{link,2}$	0	mm
Number of link legs per metre for bending in width, $n_{link,x}$	4	/m
Area provided by all links per metre for bending in width, $A_{sv,prov,x}$ = $n_{link,x}.\pi.\phi$	N/A	mm^2/m
Pitch of links for bending in width, S_x	150	mm
Shear link diameter for bending in length, $\phi_{link,y}$ = $\phi_{link,2}$	0	mm
Number of link legs per metre for bending in length, $n_{link,y}$	4	/m
Area provided by all links per metre for bending in length, $A_{sv,prov,y}$ = $n_{link,y}.\pi.$	N/A	mm^2/m
Pitch of links for bending in length, S_y	150	mm
Effective depth to sagging steel in width, $d_{x,s}$ = T_{multi} - $cover_1$ - MAX ($\phi_{link,2}$, ϕ_{li}	N/A	mm
Effective depth to sagging steel in length, $d_{y,s}$ = T_{multi} - $cover_1$ - MAX ($\phi_{link,2}$, ϕ	N/A	mm
Effective depth to hogging steel in length, $d_{y,h}$ = T_{multi} - $cover_1$ - $\phi_{hy}/2$	N/A	mm

It is assumed that sagging steel in length is exterior to sagging steel in width;

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Estimated steel reinforcement quantity N/A kg/m^3

[7.850 . ($A_{s,prov,x,s}$ +$A_{s,prov,y,s}$ +$A_{s,prov,y,h}$) / T_{multi}]; No curtailment; No laps; Links ignored; Distribution steel i

Multi Column Footing Foundation SLS Loading

SLS vertical (downward) load from column and base slab (if suspended), $F_{col,}$	1350	kN	N/A
Multi column footing (projection beneath base slab) weight, $F_{under,multi}$ = B_{multi}	N/A	kN	
Additional soil (above footing) weight, $F_{above,soil}$ = B_{multi}.L_{multi}.MAX Exclude)	N/A	kN	

Note additional soil above the footing is included for embedded footings whereby the top of the footing is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;

Total foundation SLS vertical (downward) load, $F_{multi,v}$ = $F_{col,v}$ + $F_{under,multi}$ + F	N/A	kN
Gross working pressure, q_w = $F_{multi,v}$ / (B_{multi} . L_{multi})	N/A	kPa

Multi Column Footing Foundation ULS Loading

ULS vertical (downward) load from column and base slab (if suspended), F_{col}	N/A	kN

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Multi Column Footing Foundation Reinforcement Design

Gross ULS Pressure

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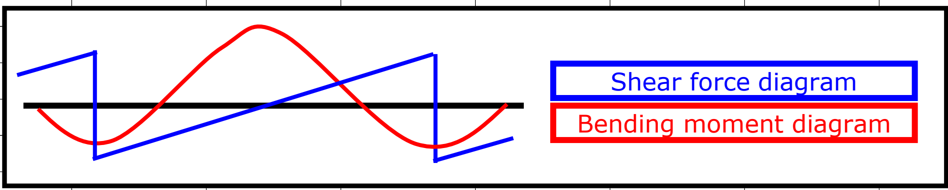
Gross ULS pressure, $q_{w,ULS} = F_{col,v,uls} / (B_{multi} . L_{multi})$	**N/A**	kPa	



Sagging Bending Moment Design in Plane of Width

Moment at column base centreline, $M_x = q_{w,ULS} . L_{multi} . (B_{multi}/2)^2 / 2$	**N/A**	kNm	
Moment at column base centreline per metre, M_x/L_{multi}	**N/A**	kNm/m	
Concrete moment capacity per metre, $M_{u,x} = 0.156f_{cu}.1000.d_{x,s}^2$	**N/A**	kNm/m	
Bending stress, $[M/bd^2]_x = (M_x/L_{multi}) / [(1000).d_{x,s}^2]$	N/A	N/mm^2	
Bending stress ratio, $K_x = [M/bd^2]_x / f_{cu} <= 0.156$	N/A		**N/A**
Lever arm, $z_x = d_{x,s} . [0.5 + (0.25-K_x/0.9)^{0.5}] <= 0.95d_{x,s}$	N/A	mm	
Area of tension steel required, $A_{s,x} = (M_x/L_{multi}) / [(0.95f_y).z_x]$	**N/A**	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
Sagging bending moment in plane of width utilisation = $A_{s,x} / A_{s,prov,x,s}$	**N/A**		**N/A**

Requirement to concentrate 2/3 rebar within $1.5d_{x,s}$ fro	*N/A*	<	*N/A*	**N/A**		*3.11.3.2*
[Yes if $L_{multi}/2>3/4(h \text{ or } D)+9/4d_{x,s}$; No if not;]	*mm*		*mm*			*BS8110*

Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;

% Min sag reinforcement in plane of width (>= $0.0024.1000.T_{multi}$ G250; >=	N/A	%	
% Min sag reinforcement in plane of width utilisation	**N/A**		**N/A**

Sagging Bending Moment Design in Plane of Length

Moment at column base centreline, $M_y = 0.08 . q_{w,ULS} . B_{multi} . L_{multi}^2$	**N/A**	kNm	*T.3.5*
Note moment coefficient based on internal span 0.08 instead of end span 0.11;			*BS8110*
Moment at column base centreline per metre, M_y/B_{multi}	**N/A**	kNm/m	
Concrete moment capacity per metre, $M_{u,y} = 0.156f_{cu}.1000.d_{y,s}^2$	**N/A**	kNm/m	
Bending stress, $[M/bd^2]_y = (M_y/B_{multi}) / [(1000).d_{y,s}^2]$	N/A	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$	N/A		**N/A**
Lever arm, $z_y = d_{y,s} . [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,s}$	N/A	mm	
Area of tension steel required, $A_{s,y} = (M_y/B_{multi}) / [(0.95f_y).z_y]$	**N/A**	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m	
Sagging bending moment in plane of length utilisation = $A_{s,y} / A_{s,prov,y,s}$	**N/A**		**N/A**

Requirement to concentrate 2/3 rebar within $1.5d_{y,s}$ fro	*N/A*	<	*N/A*	**N/A**		*3.11.3.2*
[Yes if $B_{multi}/2>3/4(b \text{ or } D)+9/4d_{y,s}$; No if not;]	*mm*		*mm*			*BS8110*

Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;

% Min sag reinforcement in plane of length (>= $0.0024.1000.T_{multi}$ G250; >=	N/A	%	
% Min sag reinforcement in plane of length utilisation	**N/A**		**N/A**

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Hogging Bending Moment Design in Plane of Length

Moment, $M_y = 0.07 \, . \, q_{w,ULS} \, . \, B_{multi} \, . \, L_{multi}^2$			N/A	kNm	*T.3.5*
Note moment coefficient based on internal span 0.07 instead of end span 0.09;					*BS8110*
Moment per metre, M_y/B_{multi}			N/A	kNm/m	
Concrete moment capacity per metre, $M_{u,y} = 0.156f_{cu}.1000.d_{y,h}^2$			N/A	kNm/m	
Bending stress, $[M/bd^2]_y = (M_y/B_{multi}) / [(1000).d_{y,h}^2]$			N/A	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$			N/A		N/A
Lever arm, $z_y = d_{y,h} \, . \, [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,h}$			N/A	mm	
Area of tension steel required, $A_{s,y} = (M_y/B_{multi}) / [(0.95f_y).z_y]$			N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,h}$			N/A	mm^2/m	
Hogging bending moment in plane of length utilisation = $A_{s,y} / A_{s,prov,y,h}$			N/A		N/A
Requirement to concentrate 2/3 rebar within $1.5d_{y,h}$ fro	*N/A*	< *N/A*	N/A		*3.11.3.2*
[Yes if $B_{multi}/2>3/4(b \text{ or } D)+9/4d_{y,h}$; No if not;]	*mm*	*mm*			*BS8110*
Note that should the above requirement be applicable, it is not automatically reflected in the					
detailing considerations and as such should be specifically reconsidered;					
% Min hog reinforcement in plane of length (>= $0.0024.1000.T_{multi}$ G250; >=			N/A	%	
% Min hog reinforcement in plane of length utilisation			N/A		N/A

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Punching Shear Design

ULS vertical (downward) load from column and base slab (if suspended), F_{col}	**N/A**	kN
Area of column base section, A_{c1} = b.h (rectangular) or $\pi D^2/4$ (circular)	N/A	mm^2
Average effective depth of both rebar layers, $d = (d_{x,s} + d_{y,s})/2$	N/A	mm
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m
Average area of tensile steel reinforcement provided, $A_{s,prov,s}$	N/A	mm^2/m
$\rho_w = 100A_{s,prov,s}/(1000.d)$	N/A	%
$\nu_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d)^{1/4}>0.67$	**N/A**	N/mm^2

Column Base Face Perimeter

Shear force at column base face, $V_1 = F_{col,v,uls} - q_{w,ULS}.A_{c1}$				N/A	kN
Effective shear force, $V_{eff,1} = 1.00 . V_1$				N/A	kN
Note $V_{eff,1}$ = 1.00 . V_1 because no moment effects assumed ;					
Column base face perimeter, u_1				N/A	mm
	Rectangular		*Circular*		
Internal column:	*2.(b+h)*	*N/A*	*π.D*	*N/A*	mm
Edge column:	*2b+h or 2h+b*	*N/A*	*3/4(π.D)*	*N/A*	mm
Corner column:	*(b+h)*	*N/A*	*π.D/2*	*N/A*	mm
Shear stress at column base face perimeter, $\nu_1 = V_{eff,1} / u_1 d$ (< $0.8f_{cu}^{0.5}$ & 5N				**N/A**	N/mm^2
Ultimate shear stress utilisation				**N/A**	**N/A**

First Shear Perimeter

Shear force 1.5d from column base face, $V_2 = F_{col,v,uls} - q_{w,ULS}.A_{c2}$				N/A	kN
	Rectangular		*Circular*		
Internal column:	*(b+3d).(h+3d)*	*N/A*	*$(D+3d)^2$*	*N/A*	m^2
Edge column:	*(b+1.5d).(h+3d) or (h+1.5d).(b+3d)*	*N/A*	*d).(D+3d)*	*N/A*	m^2
Corner column:	*(b+1.5d).(h+1.5d)*	*N/A*	*$(D+1.5d)^2$*	*N/A*	m^2
Effective shear force, $V_{eff,2} = 1.00 . V_2$				N/A	kN
Note $V_{eff,2}$ = 1.00 . V_2 because no moment effects assumed;					
Column base first perimeter, u_2				N/A	mm
	Rectangular		*Circular*		
Internal column:	*2.(b+h)+12d*	*N/A*	*4D+12d*	*N/A*	mm
Edge column:	*2b+h+6d or 2h+b+6d*	*N/A*	*3D+6d*	*N/A*	mm
Corner column:	*(b+h)+3d*	*N/A*	*2D+3d*	*N/A*	mm
Shear stress at column base first perimeter, $\nu_2 = V_{eff,2} / u_2 d$				**N/A**	N/mm^2

(Shear capacity enhancement by calculating v_d at 1.5d from "support" and comparing against unenhanced v_c as clause 3.7.7.6 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 1.5d of the "support" as clause 3.7.7.4 BS8110;)*

Case $\nu_2 < \nu_c$ — **N/A**

No links required.

Case $\nu_c < \nu_2 < 1.6\nu_c$ — **N/A**

$$\Sigma A_{sv} \sin\alpha \geq \frac{(v - v_c)ud}{0.95 f_{yv}}$$

N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case 1.6v_c < v_2 < 2.0v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case v_2 > 2.0v_c **N/A**

First shear perimeter shear utilisation **N/A** **N/A**

Second Shear Perimeter

Shear force 2.25d from column base face, $V_3 = F_{col,v,uls} - q_{w,ULS}.A_{c3}$ N/A kN

	Rectangular		Circular		
Internal column:	*(b+4.5d).(h+4.5d)*	*N/A*	*(D+4.5d)^2*	*N/A*	m^2
Edge column:	*(b+2.25d).(h+4.5d) or (h+2.25d).(b+4.5d)*	*N/A*	*.(D+4.5d)*	*N/A*	m^2
Corner column:	*(b+2.25d).(h+2.25d)*	*N/A*	*(D+2.25d)^2*	*N/A*	m^2

Effective shear force, $V_{eff,3}$ = 1.00 . V_3 N/A kN

Note $V_{eff,3}$ = 1.00 . V_3 because no moment effects assumed;

Column base second perimeter, u_3 N/A mm

	Rectangular		Circular		
Internal column:	*2.(b+h)+18d*	*N/A*	*4D+18d*	*N/A*	mm
Edge column:	*2b+h+9d or 2h+b+9d*	*N/A*	*3D+9d*	*N/A*	mm
Corner column:	*(b+h)+4.5d*	*N/A*	*2D+4.5d*	*N/A*	mm

Shear stress at column base second perimeter, $v_3 = V_{eff,3} / u_3d$ **N/A** N/mm^2

Case v_3 < v_c **N/A**

No links required.

Case v_c < v_3 < 1.6v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{(v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case 1.6v_c < v_3 < 2.0v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case v_3 > 2.0v_c **N/A**

Second shear perimeter shear utilisation **N/A** **N/A**

Note a negative shear stress v_2 and/or v_3 on a correctly specified column (wrt internal, edge or corner) indicates that the shear perimeter is beyond the physical extremes of the foundation and as such punching shear failure is not critical;

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Shear Design for Bending in Plane of Width

Shear force at column base centreline, $V_{x,ult} = q_{w,ULS} \cdot L_{multi} \cdot B_{multi}/2$	**N/A**	kN	
Shear force at column base centreline per metre, $V_{x,ult}/L_{multi}$	**N/A**	kN/m	
Shear force at $1.0d_{x,s}$ from column base centreline, $V_x = q_{w,ULS} \cdot L_{multi} \cdot (B_{multi}$	**N/A**	kN	
Shear force at $1.0d_{x,s}$ from column base centreline per metre, V_x/L_{multi}	**N/A**	kN/m	
Note the above shear forces are for bending in plane of width;			
Ultimate shear stress for bending in plane of width, $v_{ult,x}=(V_{x,ult}/L_{multi})/(1000.$	**N/A**	N/mm^2	
Ultimate shear stress for bending in plane of width utilisation	**N/A**		**N/A**
Design shear stress for bending in plane of width, $v_{d,x}=(V_x/L_{multi})/(1000.d_{x,s})$	**N/A**	N/mm^2	

(Shear capacity enhancement by calculating v_d at d from "support" and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)*

Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
$\rho_w = 100A_{s,prov,x,s}/(1000.d_{x,s})$	N/A	%	
$v_{c,x} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{x,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{x,s})^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_{d,x} < v_{c,x}$ for no links	**N/A**		
Concrete shear capacity $v_{c,x}.(1000.d_{x,s})$	**N/A**	kN/m	
Check $v_{c,x} < v_{d,x} < 0.4 + v_{c,x}$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_{c,x}).(1000.d_{x,s})$	**N/A**	kN/m	
Check $v_{d,x} > 0.4 + v_{c,x}$ for design links	**N/A**		
Provide shear links $A_{sv} / S > 1000.(v_{d,x}-v_{c,x})/(0.95f_{yv})$ i.e. A_{sv} / S >	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov,x}/S_x).(0.95f_{yv}).d_x$	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov,x}$	N/A	mm^2/m	
Tried $A_{sv,prov,x} / S_x$ value	N/A	$mm^2/mm/m$	
Design shear resistance for bending in plane of width utilisation	**N/A**		**N/A**

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Shear Design for Bending in Plane of Length

Shear force at column base centreline, $V_{y,ult} = q_{w,ULS} . B_{multi} . (0.55.L_{multi})$	**N/A**	kN	*T.3.5*
Note shear coefficient based on internal span 0.55 instead of end span 0.6;			*BS8110*
Shear force at column base centreline per metre, $V_{y,ult}/B_{multi}$	**N/A**	kN/m	
Shear force at $1.0d_{y,s}$ from column base centreline, $V_y = V_{y,ult} - q_{w,ULS} . B_{multi}$	**N/A**	kN	
Shear force at $1.0d_{y,s}$ from column base centreline per metre, V_y/B_{multi}	**N/A**	kN/m	
Note the above shear forces are for bending in plane of length;			
Ultimate shear stress for bending in plane of length, $v_{ult,y}=(V_{y,ult}/B_{multi})/(1000$	**N/A**	N/mm^2	
Ultimate shear stress for bending in plane of length utilisation	**N/A**		**N/A**
Design shear stress for bending in plane of length, $v_{d,y}=(V_y/B_{multi})/(1000.d_{y,s}$	**N/A**	N/mm^2	
(Shear capacity enhancement by calculating v_d at d from "support" and comparing against			
unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and*			
comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)			
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m	
$\rho_w = 100A_{s,prov,y,s}/(1000.d_{y,s})$	N/A	%	
$v_{c,y} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{y,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{y,s})^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_{d,y} < v_{c,y}$ for no links	**N/A**		
Concrete shear capacity $v_{c,y}.(1000.d_{y,s})$	**N/A**	kN/m	
Check $v_{c,y} < v_{d,y} < 0.4 + v_{c,y}$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_{c,y}).(1000.d_{y,s})$	**N/A**	kN/m	
Check $v_{d,y} > 0.4 + v_{c,y}$ for design links	**N/A**		
Provide shear links $A_{sv} / S > 1000.(v_{d,y}-v_{c,y})/(0.95f_{yv})$ i.e. A_{sv} / S >	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov,y}/S_y).(0.95f_{yv}).d_y$	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov,y}$	N/A	mm^2/m	
Tried $A_{sv,prov,y} / S_y$ value	N/A	$mm^2/mm/m$	
Design shear resistance for bending in plane of length utilisation	**N/A**		**N/A**

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Detailing Requirements

All detailing requirements met ?	**N/A**		
Max sagging steel reinforcement pitch in plane of width ($<3d_{x,s}$, <750mm)	N/A	mm	**N/A**
Max sagging steel reinforcement pitch in plane of length ($<3d_{y,s}$, <750mm)	N/A	mm	**N/A**
Max hogging steel reinforcement pitch in plane of length ($<3d_{y,h}$, <750mm)	N/A	mm	**N/A**

Maximum spacing: 0.5% Ast or less - 300mm
Between 0.5% and 1.0% - 225mm
1.0% Ast or greater - 175mm

Max sagging steel reinforcement pitch in plane of width	N/A	mm	**N/A**
Max sagging steel reinforcement pitch in plane of length	N/A	mm	**N/A**
Max hogging steel reinforcement pitch in plane of length	N/A	mm	**N/A**
Min sagging steel reinforcement pitch in plane of width ($>100mm+\phi_{sx}$)	N/A	mm	**N/A**
Min sagging steel reinforcement pitch in plane of length ($>100mm+\phi_{sy}$)	N/A	mm	**N/A**
Min hogging steel reinforcement pitch in plane of length ($>100mm+\phi_{hy}$)	N/A	mm	**N/A**

Note an allowance has been made for laps in the min pitch by increasing the criteria by the bar diameter.

% Max sagging reinforcement in plane of width ($<= 0.04.1000.T_{multi}$)	N/A	%	**N/A**
% Max sagging reinforcement in plane of length ($<= 0.04.1000.T_{multi}$)	N/A	%	**N/A**
% Max hogging reinforcement in plane of length ($<= 0.04.1000.T_{multi}$)	N/A	%	**N/A**
Sagging steel reinforcement diameter in plane of width, ϕ_{sx} (>=16mm)	N/A	mm	**N/A**
Sagging steel reinforcement diameter in plane of length, ϕ_{sy} (>=16mm)	N/A	mm	**N/A**
Hogging steel reinforcement diameter in plane of length, ϕ_{hy} (>=16mm)	N/A	mm	**N/A**

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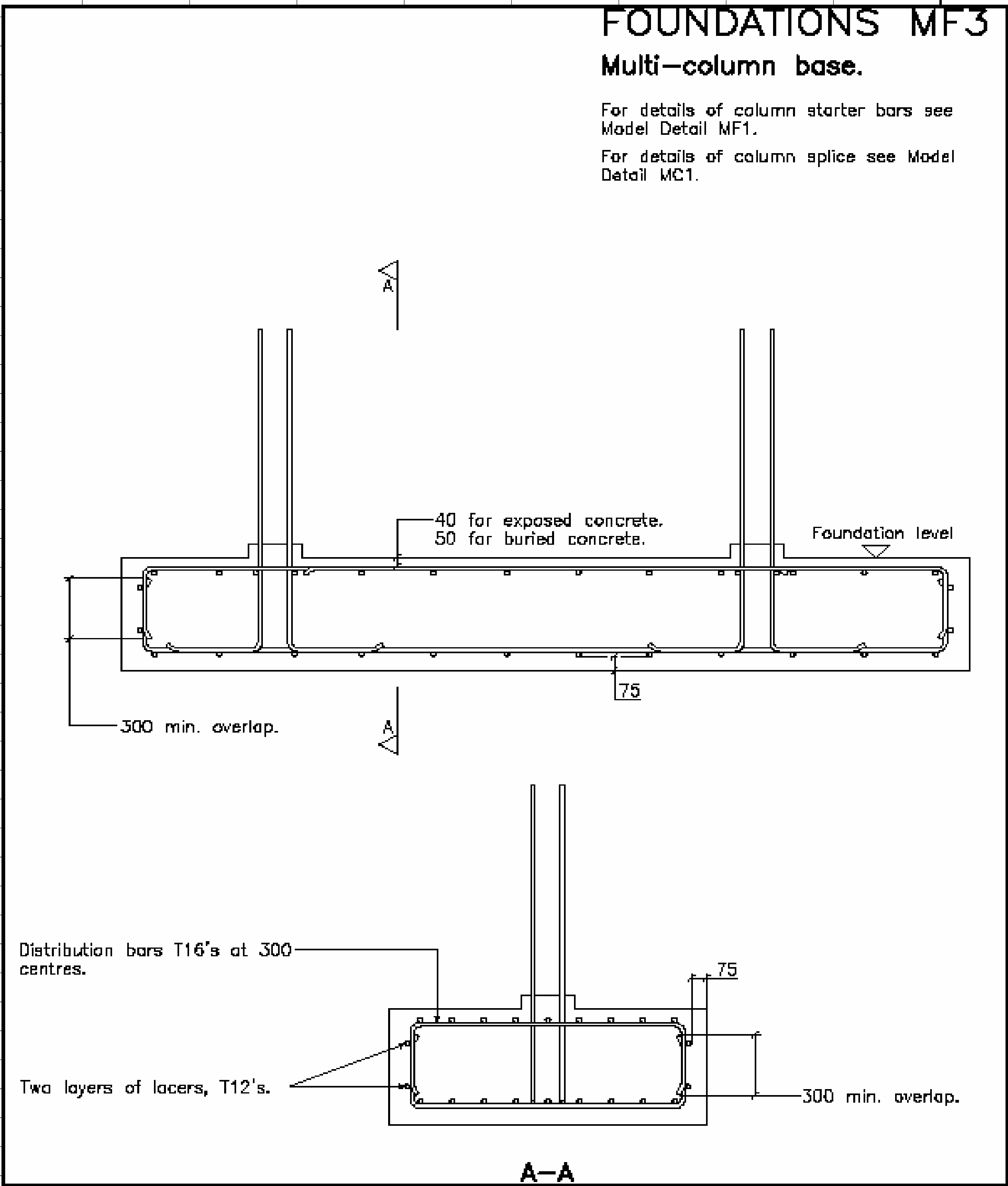
Standard Multi Column Footing Foundation Reinforcement Details



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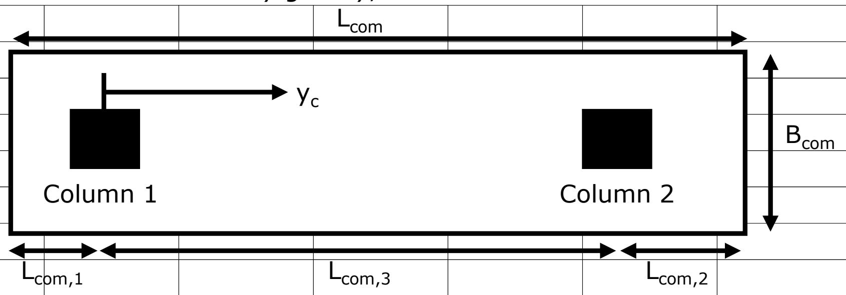
Combined Footing Foundation Dimensions

Note that the combined footing is used when a more efficient use of a footing is required than a conventional pad footing. It is also used when a column is too close to the site boundary to allow the employment of a conventional pad footing with no eccentricity. A pad footing is subject to sagging moments akin to a cantilever beam or slab whilst a combined footing is subject to both sagging and hogging moments akin to a continuous beam or slab. A combined footing differs from a multi column footing in the fact that the column loads are not uniform, and thus require that the dimensions of the combined footing be such that the resultant load passes through the centroid of the base area;

Width, B_{com} (<=L_{com})	2.300	m	N/A
Length, L_{com} (>=B_{com})	4.600	m	N/A
Length internal span, $L_{com,3}$ (<L_{com})	3.000	m	N/A
Centroid, $y_c = (F_{col,v,2}.L_{com,3})/(F_{col,v,1}+F_{col,v,2})$	N/A	m	
Length external span, $L_{com,1} = L_{com}/2-y_c$ (>0.000)	N/A	m	N/A
Length external span, $L_{com,2} = L_{com}/2-(L_{com,3}-y_c)$ (>0.000)	N/A	m	N/A

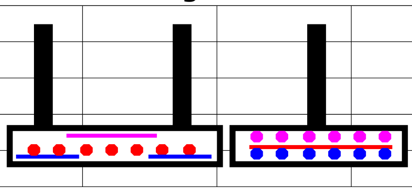
Note the combined footing is centred on the centroid, which is a function of the loads;
Should the relative loads vary greatly, the effectiveness of the combined footing reduces;



Thickness beneath base slab, $t_{1,com}$		0.800	m
Thickness of base slab, $t_{2,com}$ (if no base slab, then enter 0.000m)		0.000	m
Thickness of foundation, $T_{com} = t_{1,com} + t_{2,com}$		N/A	m
Column base section type *(for punching shear only)*	Rectangular		
Column base location *(for punching shear only)*	Interior		
Column 1 base depth, h_1 (rectangular) or diameter,	Parallel to Edge	300	mm
Column 1 base width, b_1 (rectangular) or N/A (circul	Perpendicular to Edge	300	mm
Column 2 base depth, h_2 (rectangular) or diameter,	Parallel to Edge	400	mm
Column 2 base width, b_2 (rectangular) or N/A (circul	Perpendicular to Edge	400	mm

Note where applicable, it is assumed that h_1 and h_2 are in same plane as L_{com} and that the column base is always interior and located in the centre of the combined footing B_{com}, although not L_{com};

Combined Footing Foundation Reinforcement



Sagging steel reinforcement diameter in width, ϕ_{sx}	20	mm
Sagging steel reinforcement pitch for resistance in width, p_{sx}	200	mm
Sagging steel area provided in width, $A_{s,prov,x,s} = (\pi.\phi_{sx}^2/4)/p_{sx}$	N/A	mm^2/m
Sagging steel reinforcement diameter in length, ϕ_{sy}	20	mm
Sagging steel reinforcement pitch for resistance in length, p_{sy}	200	mm
Sagging steel area provided in length, $A_{s,prov,y,s} = (\pi.\phi_{sy}^2/4)/p_{sy}$	N/A	mm^2/m
Hogging steel reinforcement diameter in length, ϕ_{hy}	16	mm
Hogging steel reinforcement pitch for resistance in length, p_{hy}	200	mm
Hogging steel area provided in length, $A_{s,prov,y,h} = (\pi.\phi_{hy}^2/4)/p_{hy}$	N/A	mm^2/m

Shear link diameter for first shear perimeter, $\phi_{link,2}$	None	mm
Number of link legs for first shear perimeter, $n_{l,2}$	30	
Area provided by all links for first shear perimeter, $A_{sv,prov,2} = n_{l,2}.\pi.\phi_{link,2}^2/4$	N/A	mm^2
Shear link diameter for second shear perimeter, $\phi_{link,3}$	None	mm
Number of link legs for second shear perimeter, $n_{l,3}$	30	
Area provided by all links for second shear perimeter, $A_{sv,prov,3} = n_{l,3}.\pi.\phi_{link,3}^2/$	N/A	mm^2
Shear link diameter for bending in width, $\phi_{link,x} = \phi_{link,2}$	0	mm
Number of link legs per metre for bending in width, $n_{link,x}$	4	/m
Area provided by all links per metre for bending in width, $A_{sv,prov,x} = n_{link,x}.\pi.\phi$	N/A	mm^2/m
Pitch of links for bending in width, S_x	150	mm
Shear link diameter for bending in length, $\phi_{link,y} = \phi_{link,2}$	0	mm
Number of link legs per metre for bending in length, $n_{link,y}$	4	/m
Area provided by all links per metre for bending in length, $A_{sv,prov,y} = n_{link,y}.\pi.$	N/A	mm^2/m
Pitch of links for bending in length, S_y	150	mm
Effective depth to sagging steel in width, $d_{x,s} = T_{com}$ - cover$_1$ - MAX ($\phi_{link,2}$, ϕ_{lir}	N/A	mm
Effective depth to sagging steel in length, $d_{y,s} = T_{com}$ - cover$_1$ - MAX ($\phi_{link,2}$, ϕ_{l}	N/A	mm
Effective depth to hogging steel in length, $d_{y,h} = T_{com}$ - cover$_1$ - $\phi_{hy}/2$	N/A	mm

It is assumed that sagging steel in length is exterior to sagging steel in width;

Estimated steel reinforcement quantity	N/A	kg/m^3

[7.850 . ($A_{s,prov,x,s}$ +$A_{s,prov,y,s}$ +$A_{s,prov,y,h}$) / T_{com}]; No curtailment; No laps; Links ignored; Distribution steel ig

Combined Footing Foundation SLS Loading

SLS vertical (downward) load from column 1 and base slab (if suspended), F	1200	kN	N/A
SLS vertical (downward) load from column 2 and base slab (if suspended), F	1650	kN	N/A
Combined footing (projection beneath base slab) weight, $F_{under,com} = B_{com}.L_{cor}$	N/A	kN	
Additional soil (above footing) weight, $F_{above,soil} = B_{com}.L_{com}$.MAX(Exclude γ_s	N/A	kN	

Note additional soil above the footing is included for embedded footings whereby the top of the footing is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;

Total foundation SLS vertical (downward) load, $F_{com,v} = F_{col,v,1} + F_{col,v,2} + F_{und}$	N/A	kN
Gross working pressure, $q_w = F_{com,v}$ / (B_{com} . L_{com})	N/A	kPa

Combined Footing Foundation ULS Loading

ULS vertical (downward) load from column 1 and base slab (if suspended), F	N/A	kN
ULS vertical (downward) load from column 2 and base slab (if suspended), F	N/A	kN

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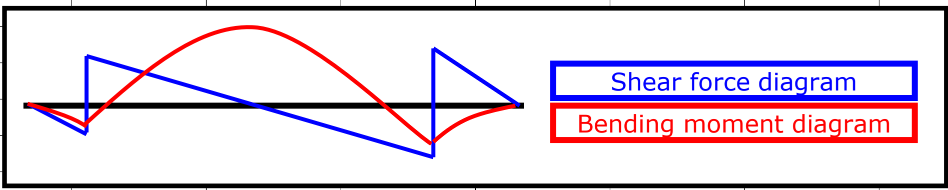
Combined Footing Foundation Reinforcement Design

Gross ULS Pressure

Gross ULS pressure, $q_{w,ULS} = (F_{col,v,1,uls} + F_{col,v,2,uls}) / (B_{com} . L_{com})$ | N/A | kPa



Sagging Bending Moment Design in Plane of Width

Moment at column base centreline, $M_x = q_{w,ULS} . L_{com} . (B_{com}/2)^2 / 2$	N/A	kNm	
Moment at column base centreline per metre, M_x/L_{com}	N/A	kNm/m	
Concrete moment capacity per metre, $M_{u,x} = 0.156f_{cu}.1000.d_{x,s}^2$	N/A	kNm/m	
Bending stress, $[M/bd^2]_x = (M_x/L_{com}) / [(1000).d_{x,s}^2]$	N/A	N/mm^2	
Bending stress ratio, $K_x = [M/bd^2]_x / f_{cu} <= 0.156$	N/A		N/A
Lever arm, $z_x = d_{x,s} . [0.5 + (0.25-K_x/0.9)^{0.5}] <= 0.95d_{x,s}$	N/A	mm	
Area of tension steel required, $A_{s,x} = (M_x/L_{com}) / [(0.95f_y).z_x]$	N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
Sagging bending moment in plane of width utilisation = $A_{s,x} / A_{s,prov,x,s}$	N/A		N/A

Requirement to concentrate 2/3 rebar within $1.5d_{x,s}$ fro	*N/A*	<	*N/A*	N/A	*3.11.3.2*
[Yes if max($L_{com,1}$, $L_{com,2}$, $L_{com,3}$/2)>3/4min(h_1 or D_1	*mm*		*mm*		*BS8110*

Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;

% Min sag reinforcement in plane of width (>= $0.0024.1000.T_{com}$ G250; >=	N/A	%	
% Min sag reinforcement in plane of width utilisation	N/A		N/A

Sagging Bending Moment Design in Plane of Length

Moment at column base face, $M_y = q_{w,ULS} . B_{com} . [max(L_{com,1}-h_1/2, L_{com,2}-h_2/$	N/A	kNm	
Moment at column base face per metre, M_y/B_{com}	N/A	kNm/m	
Concrete moment capacity per metre, $M_{u,y} = 0.156f_{cu}.1000.d_{y,s}^2$	N/A	kNm/m	
Bending stress, $[M/bd^2]_y = (M_y/B_{com}) / [(1000).d_{y,s}^2]$	N/A	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$	N/A		N/A
Lever arm, $z_y = d_{y,s} . [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,s}$	N/A	mm	
Area of tension steel required, $A_{s,y} = (M_y/B_{com}) / [(0.95f_y).z_y]$	N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m	
Sagging bending moment in plane of length utilisation = $A_{s,y} / A_{s,prov,y,s}$	N/A		N/A

Requirement to concentrate 2/3 rebar within $1.5d_{y,s}$ fro	*N/A*	<	*N/A*	N/A	*3.11.3.2*
[Yes if B_{com}/2>3/4min(b_1 or D_1, b_2 or D_2)+9/4$d_{y,s}$;	*mm*		*mm*		*BS8110*

Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;

% Min sag reinforcement in plane of length (>= $0.0024.1000.T_{com}$ G250; >=	N/A	%	
% Min sag reinforcement in plane of length utilisation	N/A		N/A

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Hogging Bending Moment Design in Plane of Length

Distance to zero shear force from column 1, $L_{com,4}$ = ($F_{col,v,1,uls}$-$q_{w,ULS}$.B_{com}.L_{cor}		N/A	m	
Moment, M_y = $F_{col,v,1,uls}$. $L_{com,4}$ - $q_{w,ULS}$. B_{com} . $(L_{com,1}+L_{com,4})^2$ / 2		N/A	kNm	
Moment per metre, M_y/B_{com}		N/A	kNm/m	
Concrete moment capacity per metre, $M_{u,y} = 0.156f_{cu}.1000.d_{y,h}^2$		N/A	kNm/m	
Bending stress, $[M/bd^2]_y = (M_y/B_{com}) / [(1000).d_{y,h}^2]$		N/A	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$		N/A		N/A
Lever arm, $z_y = d_{y,h} . [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,h}$		N/A	mm	
Area of tension steel required, $A_{s,y} = (M_y/B_{com}) / [(0.95f_y).z_y]$		N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,h}$		N/A	mm^2/m	
Hogging bending moment in plane of length utilisation = $A_{s,y} / A_{s,prov,y,h}$		N/A		N/A
Requirement to concentrate 2/3 rebar within $1.5d_{y,h}$ fro	*N/A* < *N/A*	N/A		*3.11.3.2*
[Yes if $B_{com}/2>3/4min(b_1$ or D_1, b_2 or $D_2)+9/4d_{y,h}$;	*mm* *mm*			*BS8110*
Note that should the above requirement be applicable, it is not automatically reflected in the				
detailing considerations and as such should be specifically reconsidered;				
% Min hog reinforcement in plane of length (>= 0.0024.1000.T_{com} G250; >=		N/A	%	
% Min hog reinforcement in plane of length utilisation		N/A		N/A

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Punching Shear Design

Critical column 1 or 2 i.e. MAX($F_{col,v,1,uls}$/(b_1.h_1), $F_{col,v,2,uls}$/(b_2.h_2)) or MAX($F_{col,}$	**N/A**	
ULS vertical (downward) load from relevant column and base slab (if suspend	**N/A**	kN
Relevant column base depth, h (rectangular) or diameter, D (circular)	N/A	mm
Relevant column base width, b (rectangular) or N/A (circular)	N/A	mm
Area of column base section, A_{c1} = b.h (rectangular) or $\pi D^2/4$ (circular)	N/A	mm^2
Average effective depth of both rebar layers, d = ($d_{x,s}$ + $d_{y,s}$)/2	N/A	mm
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m
Average area of tensile steel reinforcement provided, $A_{s,prov,s}$	N/A	mm^2/m
ρ_w = 100$A_{s,prov,s}$/(1000.d)	N/A	%
v_c = (0.79/1.25)($\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$; ρ_w<3; f_{cu}<40; $(400/d)^{1/4}$>0.67	**N/A**	N/mm^2

Column Base Face Perimeter

Shear force at column base face, V_1 = $F_{col,v,uls}$ - $q_{w,ULS}$.A_{c1}	N/A	kN
Effective shear force, $V_{eff,1}$ = 1.00 . V_1	N/A	kN
Note $V_{eff,1}$ = 1.00 . V_1 because no moment effects assumed ;		
Column base face perimeter, u_1	N/A	mm

	Rectangular		Circular		
Internal column:	*2.(b+h)*	*N/A*	*π.D*	*N/A*	mm
Edge column:	*2b+h or 2h+b*	*N/A*	*3/4(π.D)*	*N/A*	mm
Corner column:	*(b+h)*	*N/A*	*π.D/2*	*N/A*	mm

Shear stress at column base face perimeter, v_1 = $V_{eff,1}$ / u_1d (< 0.8$f_{cu}^{0.5}$ & 5N	**N/A**	N/mm^2	
Ultimate shear stress utilisation	**N/A**		**N/A**

First Shear Perimeter

Shear force 1.5d from column base face, V_2 = $F_{col,v,uls}$ - $q_{w,ULS}$.A_{c2}	N/A	kN

	Rectangular		Circular		
Internal column:	*(b+3d).(h+3d)*	*N/A*	*$(D+3d)^2$*	*N/A*	m^2
Edge column:	*(b+1.5d).(h+3d) or (h+1.5d).(b+3d)*	*N/A*	*d).(D+3d)*	*N/A*	m^2
Corner column:	*(b+1.5d).(h+1.5d)*	*N/A*	*$(D+1.5d)^2$*	*N/A*	m^2

Effective shear force, $V_{eff,2}$ = 1.00 . V_2	N/A	kN
Note $V_{eff,2}$ = 1.00 . V_2 because no moment effects assumed;		
Column base first perimeter, u_2	N/A	mm

	Rectangular		Circular		
Internal column:	*2.(b+h)+12d*	*N/A*	*4D+12d*	*N/A*	mm
Edge column:	*2b+h+6d or 2h+b+6d*	*N/A*	*3D+6d*	*N/A*	mm
Corner column:	*(b+h)+3d*	*N/A*	*2D+3d*	*N/A*	mm

Shear stress at column base first perimeter, v_2 = $V_{eff,2}$ / u_2d	**N/A**	N/mm^2

(Shear capacity enhancement by calculating v_d at 1.5d from "support" and comparing against unenhanced v_c as clause 3.7.7.6 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 1.5d of the "support" as clause 3.7.7.4 BS8110;)*

Case v_2 < v_c **N/A**

No links required.

Case v_c < v_2 < 1.6v_c **N/A**

$\Sigma A_{sv}\sin\alpha \geq \dfrac{(v - v_c)ud}{0.95 f_{yv}}$	N/A	>=	N/A	mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

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Case $1.6v_c < v_2 < 2.0v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95 f_{yv}}$$ N/A >= N/A mm²

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $v_2 > 2.0v_c$ — **N/A**

First shear perimeter shear utilisation — **N/A** — **N/A**

Second Shear Perimeter

Shear force 2.25d from column base face, $V_3 = F_{col,v,uls} - q_{w,ULS}.A_{c3}$				N/A	kN
	Rectangular		Circular		
Internal column:	*(b+4.5d).(h+4.5d)*	*N/A*	*(D+4.5d)²*	*N/A*	m²
Edge column:	*(b+2.25d).(h+4.5d) or (h+2.25d).(b+4.5d)*	*N/A*	*.(D+4.5d)*	*N/A*	m²
Corner column:	*(b+2.25d).(h+2.25d)*	*N/A*	*D+2.25d)²*	*N/A*	m²
Effective shear force, $V_{eff,3}$ = 1.00 . V_3				N/A	kN
Note $V_{eff,3}$ = 1.00 . V_3 because no moment effects assumed;					
Column base second perimeter, u_3				N/A	mm
	Rectangular		Circular		
Internal column:	*2.(b+h)+18d*	*N/A*	*4D+18d*	*N/A*	mm
Edge column:	*2b+h+9d or 2h+b+9d*	*N/A*	*3D+9d*	*N/A*	mm
Corner column:	*(b+h)+4.5d*	*N/A*	*2D+4.5d*	*N/A*	mm
Shear stress at column base second perimeter, $v_3 = V_{eff,3} / u_3d$				**N/A**	N/mm²

Case $v_3 < v_c$ — **N/A**

No links required.

Case $v_c < v_3 < 1.6v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{(v - v_c)ud}{0.95 f_{yv}}$$ N/A >= N/A mm²

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $1.6v_c < v_3 < 2.0v_c$ — **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95 f_{yv}}$$ N/A >= N/A mm²

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case $v_3 > 2.0v_c$ — **N/A**

Second shear perimeter shear utilisation — **N/A** — **N/A**

Note a negative shear stress v_2 and/or v_3 on a correctly specified column (wrt internal, edge or corner) indicates that the shear perimeter is beyond the physical extremes of the foundation and as such punching shear failure is not critical;

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Shear Design for Bending in Plane of Width

Shear force at column base centreline, $V_{x,ult} = q_{w,ULS} . L_{com} . B_{com}/2$	**N/A**	kN	
Shear force at column base centreline per metre, $V_{x,ult}/L_{com}$	**N/A**	kN/m	
Shear force at $1.0d_{x,s}$ from column base centreline, $V_x = q_{w,ULS} . L_{com} . (B_{com}/$	**N/A**	kN	
Shear force at $1.0d_{x,s}$ from column base centreline per metre, V_x/L_{com}	**N/A**	kN/m	
Note the above shear forces are for bending in plane of width;			
Ultimate shear stress for bending in plane of width, $v_{ult,x}=(V_{x,ult}/L_{com})/(1000.c$	**N/A**	N/mm^2	
Ultimate shear stress for bending in plane of width utilisation	**N/A**		**N/A**
Design shear stress for bending in plane of width, $v_{d,x}=(V_x/L_{com})/(1000.d_{x,s})$	**N/A**	N/mm^2	
(Shear capacity enhancement by calculating v_d at d from "support" and comparing against			
unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and*			
comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)			
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
$\rho_w = 100A_{s,prov,x,s}/(1000.d_{x,s})$	N/A	%	
$v_{c,x} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{x,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{x,s})^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_{d,x} < v_{c,x}$ for no links	**N/A**		
Concrete shear capacity $v_{c,x}.(1000.d_{x,s})$	**N/A**	kN/m	
Check $v_{c,x} < v_{d,x} < 0.4 + v_{c,x}$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_{c,x}).(1000.d_{x,s})$	**N/A**	kN/m	
Check $v_{d,x} > 0.4 + v_{c,x}$ for design links	**N/A**		
Provide shear links $A_{sv} / S > 1000.(v_{d,x}-v_{c,x})/(0.95f_{yv})$ i.e. A_{sv} / S >	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov,x}/S_x).(0.95f_{yv}).d_x$	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov,x}$	N/A	mm^2/m	
Tried $A_{sv,prov,x} / S_x$ value	N/A	$mm^2/mm/m$	
Design shear resistance for bending in plane of width utilisation	**N/A**		**N/A**

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Shear Design for Bending in Plane of Length

Shear force at column 1 base left face, $V_{y,1} = q_{w,ULS} . B_{com} . (L_{com,1}-h_1/2)$	N/A	kN	
Shear force at column 1 base right face, $V_{y,2} = F_{col,v,1,uls} - q_{w,ULS} . B_{com} . (L_{com,}$	N/A	kN	
Shear force at column 2 base left face, $V_{y,3} = F_{col,v,2,uls} - q_{w,ULS} . B_{com} . (L_{com,2}-$	N/A	kN	
Shear force at column 2 base right face, $V_{y,4} = q_{w,ULS} . B_{com} . (L_{com,2}-h_2/2)$	N/A	kN	
Shear force at critical column base face, $V_{y,ult} = MAX(V_{y,1}, V_{y,2}, V_{y,3}, V_{y,4})$	**N/A**	kN	
Shear force at critical column base face per metre, $V_{y,ult}/B_{com}$	**N/A**	kN/m	
Shear force at $1.0d_{y,s}$ from critical column base face, $V_y = V_{y,ult} - q_{w,ULS} . B_{com}$	**N/A**	kN	
Shear force at $1.0d_{y,s}$ from critical column base face per metre, V_y/B_{com}	**N/A**	kN/m	
Note the above shear forces are for bending in plane of length;			
Ultimate shear stress for bending in plane of length, $v_{ult,y}=(V_{y,ult}/B_{com})/(1000$	**N/A**	N/mm^2	
Ultimate shear stress for bending in plane of length utilisation	**N/A**		**N/A**
Design shear stress in plane of length, $v_{d,y}=(V_y/B_{com})/(1000.d_{y,s})$	**N/A**	N/mm^2	
(Shear capacity enhancement by calculating v_d at d from "support" and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 employed ***instead of*** *calculating v_d at "support" and comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)*			
Area of tensile steel reinforcement provided, $A_{s,prov,y,s}$	N/A	mm^2/m	
$\rho_w = 100A_{s,prov,y,s}/(1000.d_{y,s})$	N/A	%	
$v_{c,y} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{y,s})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{y,s})^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_{d,y} < v_{c,y}$ for no links	N/A		
Concrete shear capacity $v_{c,y}.(1000.d_{y,s})$	**N/A**	kN/m	
Check $v_{c,y} < v_{d,y} < 0.4 + v_{c,y}$ for nominal links	N/A		
Provide nominal links such that $A_{sv} / S > 0.4.(1000)/(0.95f_{yv})$ i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity $(0.4 + v_{c,y}).(1000.d_{y,s})$	**N/A**	kN/m	
Check $v_{d,y} > 0.4 + v_{c,y}$ for design links	N/A		
Provide shear links $A_{sv} / S > 1000.(v_{d,y}-v_{c,y})/(0.95f_{yv})$ i.e. A_{sv} / S >	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity $(A_{sv,prov,y}/S_y).(0.95f_{yv}).d_y$	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov,y}$	N/A	mm^2/m	
Tried $A_{sv,prov,y} / S_y$ value	N/A	$mm^2/mm/m$	
Design shear resistance for bending in plane of length utilisation	**N/A**		**N/A**

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Detailing Requirements

All detailing requirements met ?	N/A		
Max sagging steel reinforcement pitch in plane of width ($<3d_{x,s}$, <750mm)	N/A	mm	N/A
Max sagging steel reinforcement pitch in plane of length ($<3d_{y,s}$, <750mm)	N/A	mm	N/A
Max hogging steel reinforcement pitch in plane of length ($<3d_{y,h}$, <750mm)	N/A	mm	N/A

Maximum spacing:	0.5% Ast or less - 300mm Between 0.5% and 1.0% - 225mm 1.0% Ast or greater - 175mm

Max sagging steel reinforcement pitch in plane of width	N/A	mm	N/A
Max sagging steel reinforcement pitch in plane of length	N/A	mm	N/A
Max hogging steel reinforcement pitch in plane of length	N/A	mm	N/A
Min sagging steel reinforcement pitch in plane of width ($>100mm+\phi_{sx}$)	N/A	mm	N/A
Min sagging steel reinforcement pitch in plane of length ($>100mm+\phi_{sy}$)	N/A	mm	N/A
Min hogging steel reinforcement pitch in plane of length ($>100mm+\phi_{hy}$)	N/A	mm	N/A

Note an allowance has been made for laps in the min pitch by increasing the criteria by the bar diameter.

% Max sagging reinforcement in plane of width ($<= 0.04.1000.T_{com}$)	N/A	%	N/A
% Max sagging reinforcement in plane of length ($<= 0.04.1000.T_{com}$)	N/A	%	N/A
% Max hogging reinforcement in plane of length ($<= 0.04.1000.T_{com}$)	N/A	%	N/A
Sagging steel reinforcement diameter in plane of width, ϕ_{sx} (>=16mm)	N/A	mm	N/A
Sagging steel reinforcement diameter in plane of length, ϕ_{sy} (>=16mm)	N/A	mm	N/A
Hogging steel reinforcement diameter in plane of length, ϕ_{hy} (>=16mm)	N/A	mm	N/A

Standard Combined Footing Foundation Reinforcement Details

As per standard multi column footing reinforcement details;

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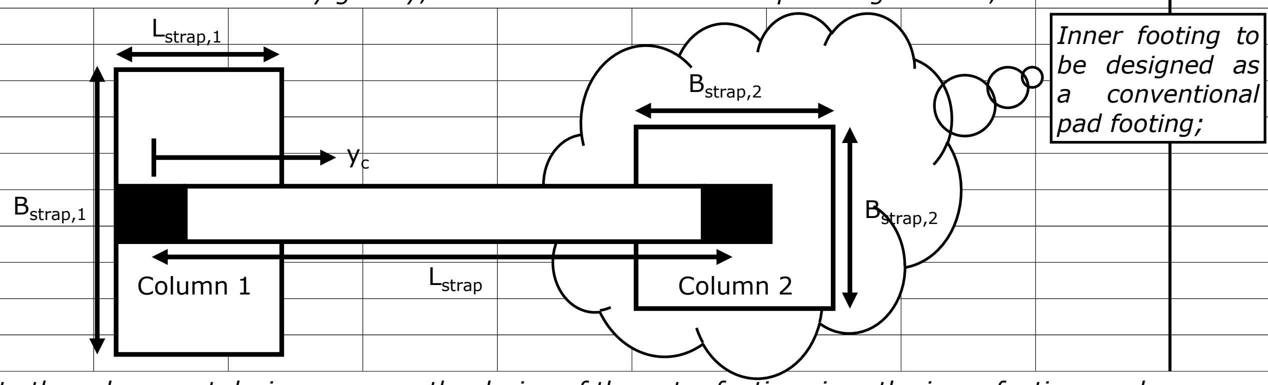
Strap Footing Foundation Dimensions

Note that the strap footing is used when a column is too close to the site boundary to allow the employment of a conventional pad footing with no eccentricity. The strap beam effectively restrains the overturning force due to the eccentric load on the outer footing. A strap footing differs from a combined footing in the fact that the proximity of the outer footing to the boundary is even more dramatic;

Width and length of inner footing, $B_{strap,2}$	2.000	m
Length of outer footing, $L_{strap,1}$	1.700	m
Length internal span, L_{strap}	3.000	m
Centroid, $y_c = (F_{col,v,2}.L_{strap})/(F_{col,v,1}+F_{col,v,2})$	N/A	m
Width of outer footing, $B_{strap,1} = (B_{strap,2}^2.L_{strap}-y_c.B_{strap,2}^2)/(y_c.L_{strap,1}-L_{strap,1}.(L_s$	N/A	m

Note the strap footing is centred on the centroid, which is a function of the loads;
Should the relative loads vary greatly, the effectiveness of the strap footing reduces;



Note the subsequent design concerns the design of the outer footing since the inner footing can be designed conventionally as a pad footing with the loads $F_{col,v,2}$ and dimensions $B_{strap,2}$, $t_{1,strap}$ and $t_{2,strap}$ defined herein;

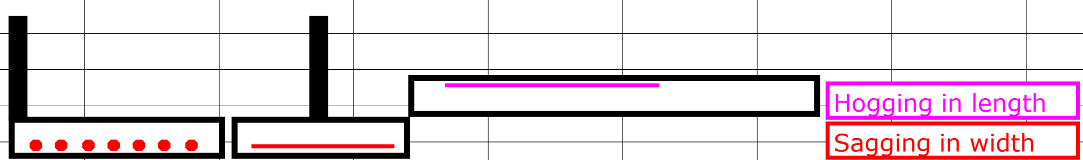
Thickness beneath base slab, $t_{1,strap}$	0.800	m
Thickness of base slab, $t_{2,strap}$ (if no base slab, then enter 0.000m)	0.000	m
Thickness of foundation, $T_{strap} = t_{1,strap} + t_{2,strap}$	N/A	m
Column base section type *(for punching shear only)*	Rectangular	
Column base location *(for punching shear only)*	Edge for Span in Width Direction	
Column 1 base depth, h (rectangular) or diameter, D	Perpendicular to Edge / 300	mm
Column 1 base width, b (rectangular) or N/A (circula	Parallel to Edge / 300	mm

Note where applicable, it is assumed that h is in same plane as $L_{strap,1}$ and that the column base is always interior and located in the centre of the strap footing $B_{strap,1}$, although not $L_{strap,1}$;

Depth of beam, h_{beam}	0.800	m
Width of beam, b_{beam}	0.300	m

Note that the strap beam must not bear on the soil, compressible void former to be specified;

Strap Footing Foundation Reinforcement



Sagging steel reinforcement diameter in width of outer footing, ϕ_{sx}	20	mm
Sagging steel reinforcement pitch for resistance in width of outer footing, p_{sx}	200	mm
Sagging steel area provided in width of outer footing, $A_{s,prov,x,s} = (\pi.\phi_{sx}^2/4)/p_s$	N/A	mm^2/m
Hogging steel reinforcement diameter in beam, ϕ_{hy}	16	mm
Hogging steel reinforcement number in beam, n_{hy}	5	
Hogging steel area provided in beam, $A_{s,prov,y,h} = n_{hy}.\pi.\phi_{hy}^2/4$	N/A	mm^2

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Shear link diameter for first shear perimeter of outer footing, $\phi_{link,2}$	None	mm	
Number of link legs for first shear perimeter of outer footing, $n_{l,2}$	30		
Area provided by all links for first shear perimeter of outer footing, $A_{sv,prov,2}$ =	**N/A**	mm²	
Shear link diameter for second shear perimeter of outer footing, $\phi_{link,3}$	None	mm	
Number of link legs for second shear perimeter of outer footing, $n_{l,3}$	30		
Area provided by all links for second shear perimeter of outer footing, $A_{sv,prov}$	**N/A**	mm²	
Shear link diameter for bending in width of outer footing, $\phi_{link,x} = \phi_{link,2}$	0	mm	
Number of link legs per metre for bending in width of outer footing, $n_{link,x}$	4	/m	
Area provided by all links per metre for bending in width of outer footing, A_{s}	**N/A**	mm²/m	
Pitch of links for bending in width of outer footing, S_x	150	mm	
Shear link diameter in beam, $\phi_{link,y}$	12	mm	
Number of links in a cross section in beam, i.e. number of legs, $n_{link,y}$	2		
Area provided by all links in a cross-section in beam, $A_{sv,prov,y} = \pi.\phi_{link,y}{}^2/4.n_{li}$	**N/A**	mm²	
Pitch of links in beam, S_y	150	mm	
Effective depth to sagging steel in width of outer footing, $d_{x,s} = T_{strap}$ - $cover_1$	N/A	mm	
Effective depth to hogging steel in beam, $d_{y,h} = h_{beam} - cover_1 - \phi_{link,y} - \phi_{hy}/2$	N/A	mm	
Estimated steel reinforcement quantity	**N/A**	kg/m³	
[7.850 . ($A_{s,prov,x,s}$) / T_{strap} + 7850 . ($A_{s,prov,y,h}$) / b_{beam} h_{beam}]; No curtailment; No laps; Links ignored; Distr			
Strap Footing Foundation SLS Loading			
SLS vertical (downward) load from column 1 and base slab (if suspended), F	650	kN	**N/A**
SLS vertical (downward) load from column 2 and base slab (if suspended), F	1000	kN	**N/A**
Strap footing (projection beneath base slab) weight of outer footing, $F_{under,stra}$	N/A	kN	
Additional soil (above footing) weight, $F_{above,soil,1} = B_{strap,1}.L_{strap,1}.$ Exclude	N/A	kN	
Note additional soil above the footing is included for embedded footings whereby the top of the footing			
is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;			
Strap footing (projection beneath base slab) weight of inner footing, $F_{under,stra}$	N/A	kN	
Additional soil (above footing) weight, $F_{above,soil,2} = B_{strap,2}{}^2$.MAX((Exclude	N/A	kN	
Note additional soil above the footing is included for embedded footings whereby the top of the footing			
is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;			
Strap beam weight, $F_{beam,strap} = b_{beam}.h_{beam}.\rho_c.L_{strap}$	N/A	kN	
Total foundation SLS vertical (downward) load of outer footing, $F_{strap,v,1}$	**N/A**	kN	
Note $F_{strap,v,1}$ = [$F_{col,v,1}.L_{strap}$ + $F_{beam,strap}.L_{strap}/2$ + ($F_{under,strap,1}$+$F_{above,soil,1}$).(L_{strap}+h/2-$L_{strap,1}$/2)]			
/ (L_{strap}+h/2-$L_{strap,1}$/2);			
Note the value of $F_{strap,v,1}$ is calculated by taking moments about the inner footing and solving for the			
reaction beneath the outer footing. This is essentially the sls reaction beneath the outer footing;			
Total foundation SLS vertical (downward) load of inner footing, $F_{strap,v,2}$	**N/A**	kN	**N/A**
Note $F_{strap,v,2}$ = $F_{col,v,1}$ + $F_{col,v,2}$ + $F_{beam,strap}$ + $F_{under,strap,1}$ + $F_{above,soil,1}$ + $F_{under,strap,2}$ + $F_{above,soil,2}$ - $F_{strap,v,1}$;			
Note the value of $F_{strap,v,2}$ is calculated by resolving vertical loads; $F_{strap,v,2}$ will be less than $F_{col,v,2}$;			
This is essentially the sls reaction beneath the inner footing and must not be negative;			
Gross working pressure under outer footing, $q_{w,1} = F_{strap,v,1} / (B_{strap,1} . L_{strap,1})$	**N/A**	kPa	
Gross working pressure under inner footing, $q_{w,2} = F_{strap,v,2} / B_{strap,2}{}^2$	**N/A**	kPa	
Strap Footing Foundation ULS Loading			
ULS vertical (downward) load from column 1 and base slab (if suspended), F	**N/A**	kN	
ULS vertical (downward) load from column 2 and base slab (if suspended), F	**N/A**	kN	

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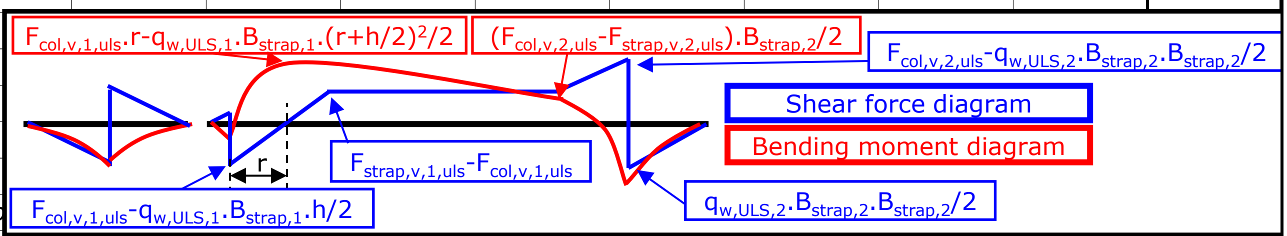
Strap Footing Foundation Reinforcement Design

Gross ULS Pressure

Total foundation ULS vertical (downward) load of outer footing, $F_{strap,v,1,uls}$	N/A	kN	
Note $F_{strap,v,1,uls}$ = [$F_{col,v,1,uls}$.L_{strap} + 1.4$F_{beam,strap}$.L_{strap}/2] / (L_{strap}+h/2-$L_{strap,1}$/2);			
Note the value of $F_{strap,v,1,uls}$ is calculated by taking moments about the inner footing and solving for the reaction beneath the outer footing. This is essentially the uls reaction beneath the outer footing;			
Gross ULS pressure under outer footing, $q_{w,ULS,1}$ = $F_{strap,v,1,uls}$ / ($B_{strap,1}$. $L_{strap,}$	N/A	kPa	
Total foundation ULS vertical (downward) load of inner footing, $F_{strap,v,2,uls}$	N/A	kN	
Note $F_{strap,v,2,uls}$ = $F_{col,v,1,uls}$ + $F_{col,v,2,uls}$ + 1.4$F_{beam,strap}$ - $F_{strap,v,1,uls}$;			
Note the value of $F_{strap,v,2,uls}$ is calculated by resolving vertical loads; $F_{strap,v,2,uls}$ will be less than $F_{col,v,2,uls}$;			
This is essentially the uls reaction beneath the inner footing and must not be negative;			
Gross ULS pressure under inner footing, $q_{w,ULS,2}$ = $F_{strap,v,2,uls}$ / $B_{strap,2}^2$	N/A	kPa	



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Sagging Bending Moment Design in Plane of Width of Outer Footing

Moment at column base face, M_x = $q_{w,ULS,1}$. $L_{strap,1}$. [($B_{strap,1}$-(b or D))/2]^2 / 2	N/A	kNm	
Moment at column base face per metre, $M_x/L_{strap,1}$	N/A	kNm/m	
Concrete moment capacity per metre, $M_{u,x}$ = 0.156f_{cu}.1000.$d_{x,s}^2$	N/A	kNm/m	
Bending stress, $[M/bd^2]_x$ = ($M_x/L_{strap,1}$) / [(1000).$d_{x,s}^2$]	N/A	N/mm^2	
Bending stress ratio, K_x = $[M/bd^2]_x$ / f_{cu} <= 0.156	N/A		N/A
Lever arm, z_x = $d_{x,s}$. [0.5 + (0.25-K_x/0.9)$^{0.5}$] <= 0.95$d_{x,s}$	N/A	mm	
Area of tension steel required, $A_{s,x}$ = ($M_x/L_{strap,1}$) / [(0.95f_y).z_x]	N/A	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
Sagging bending moment in plane of width of outer footing utilisation = $A_{s,x}$ /	N/A		N/A
Requirement to concentrate 2/3 rebar within 1.5$d_{x,s}$ frc *N/A* < *N/A*	N/A		*3.11.3.2*
[Yes if ($L_{strap,1}$-(h or D)/2)>3/4(h or D)+9/4$d_{x,s}$; No if *mm* *mm*			*BS8110*
Note that should the above requirement be applicable, it is not automatically reflected in the detailing considerations and as such should be specifically reconsidered;			
% Min sag reinforcement in plane of width of outer footing (>= 0.0024.1000	N/A	%	
% Min sag reinforcement in plane of width of outer footing utilisation	N/A		N/A

Hogging Bending Moment Design in Beam

Distance to zero shear force from column 1, $r = F_{col,v,1,uls}/(q_{w,ULS,1}.B_{strap,1})$ - h/	N/A	m	
Moment, $M_y = F_{col,v,1,uls} \cdot r - q_{w,ULS,1} \cdot B_{strap,1} \cdot (r+h/2)^2 / 2$	N/A	kNm	
Concrete moment capacity, $M_{u,y} = 0.156f_{cu}.b_{beam}.d_{y,h}^2$	N/A	kNm	
Bending stress, $[M/bd^2]_y = M_y / [b_{beam}.d_{y,h}^2]$	N/A	N/mm^2	
Bending stress ratio, $K_y = [M/bd^2]_y / f_{cu} <= 0.156$	N/A		N/A
Lever arm, $z_y = d_{y,h} \cdot [0.5 + (0.25-K_y/0.9)^{0.5}] <= 0.95d_{y,h}$	N/A	mm	
Area of tension steel required, $A_{s,y} = M_y / [(0.95f_y).z_y]$	N/A	mm^2	
Area of tensile steel reinforcement provided, $A_{s,prov,y,h}$	N/A	mm^2/m	
Hogging bending moment utilisation = $A_{s,y} / A_{s,prov,y,h}$	N/A		N/A
% Min hog reinforcement in beam (>= $0.0024.b_{beam}.h_{beam}$ G250; >= 0.0013	N/A	%	
% Min hog reinforcement in beam utilisation	N/A		N/A

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Punching Shear Design

ULS vertical (downward) load from column and base slab (if suspended), F_{col}	N/A	kN
Area of column base section, A_{c1} = b.h (rectangular) or $\pi D^2/4$ (circular)	N/A	mm^2
Average effective depth of rebar layer, $d = d_{x,s}$	N/A	mm
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m
Average area of tensile steel reinforcement provided, $A_{s,prov,s}$	N/A	mm^2/m
$\rho_w = 100A_{s,prov,s}/(1000.d)$	N/A	%
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d)^{1/4}>0.67$	N/A	N/mm^2

Column Base Face Perimeter

Shear force at column base face, $V_1 = F_{col,v,1,uls} - q_{w,ULS,1}.A_{c1}$	N/A	kN
Effective shear force, $V_{eff,1} = 1.00 . V_1$	N/A	kN
Note $V_{eff,1} = 1.00 . V_1$ because no moment effects assumed;		
Column base face perimeter, u_1	N/A	mm

	Rectangular		Circular		
Internal column:	*2.(b+h)*	*N/A*	*π.D*	*N/A*	mm
Edge column:	*2b+h or 2h+b*	*N/A*	*3/4(π.D)*	*N/A*	mm
Corner column:	*(b+h)*	*N/A*	*π.D/2*	*N/A*	mm

Shear stress at column base face perimeter, $v_1 = V_{eff,1} / u_1 d$ (< $0.8f_{cu}^{0.5}$ & 5N	N/A	N/mm^2	
Ultimate shear stress utilisation	N/A		N/A

First Shear Perimeter

Shear force 1.5d from column base face, $V_2 = F_{col,v,1,uls} - q_{w,ULS,1}.A_{c2}$	N/A	kN

	Rectangular		Circular		
Internal column:	*(b+3d).(h+3d)*	*N/A*	*$(D+3d)^2$*	*N/A*	m^2
Edge column:	*(b+1.5d).(h+3d) or (h+1.5d).(b+3d)*	*N/A*	*d).(D+3d)*	*N/A*	m^2
Corner column:	*(b+1.5d).(h+1.5d)*	*N/A*	*'D+1.5d)^2*	*N/A*	m^2

Effective shear force, $V_{eff,2} = 1.00 . V_2$	N/A	kN
Note $V_{eff,2} = 1.00 . V_2$ because no moment effects assumed;		
Column base first perimeter, u_2	N/A	mm

	Rectangular		Circular		
Internal column:	*2.(b+h)+12d*	*N/A*	*4D+12d*	*N/A*	mm
Edge column:	*2b+h+6d or 2h+b+6d*	*N/A*	*3D+6d*	*N/A*	mm
Corner column:	*(b+h)+3d*	*N/A*	*2D+3d*	*N/A*	mm

Shear stress at column base first perimeter, $v_2 = V_{eff,2} / u_2 d$	N/A	N/mm^2

*(Shear capacity enhancement by calculating v_d at 1.5d from "support" and comparing against unenhanced v_c as clause 3.7.7.6 BS8110 employed **instead of** calculating v_d at "support" and comparing against enhanced v_c within 1.5d of the "support" as clause 3.7.7.4 BS8110;)*

Case $v_2 < v_c$ N/A

No links required.

Case $v_c < v_2 < 1.6v_c$ N/A

$$\Sigma A_{sv} \sin\alpha \geq \frac{(v - v_c)ud}{0.95 f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv} \sin\alpha$ > $0.4ud/0.95f_{yv}$

Case 1.6v_c < v_2 < 2.0v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case v_2 > 2.0v_c **N/A**

First shear perimeter shear utilisation **N/A** **N/A**

Second Shear Perimeter

Shear force 2.25d from column base face, $V_3 = F_{col,v,1,uls} - q_{w,ULS,1}.A_{c3}$ N/A kN

		Rectangular		Circular		
Internal column:		*(b+4.5d).(h+4.5d)*	*N/A*	*(D+4.5d)^2*	*N/A*	m^2
Edge column:		*(b+2.25d).(h+4.5d) or (h+2.25d).(b+4.5d)*	*N/A*	*.(D+4.5d)*	*N/A*	m^2
Corner column:		*(b+2.25d).(h+2.25d)*	*N/A*	*(D+2.25d)^2*	*N/A*	m^2

Effective shear force, $V_{eff,3}$ = 1.00 . V_3 N/A kN

Note $V_{eff,3}$ = 1.00 . V_3 because no moment effects assumed;

Column base second perimeter, u_3 N/A mm

		Rectangular		Circular		
Internal column:		*2.(b+h)+18d*	*N/A*	*4D+18d*	*N/A*	mm
Edge column:		*2b+h+9d or 2h+b+9d*	*N/A*	*3D+9d*	*N/A*	mm
Corner column:		*(b+h)+4.5d*	*N/A*	*2D+4.5d*	*N/A*	mm

Shear stress at column base second perimeter, $v_3 = V_{eff,3} / u_3d$ **N/A** N/mm^2

Case v_3 < v_c **N/A**

No links required.

Case v_c < v_3 < 1.6v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{(v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case 1.6v_c < v_3 < 2.0v_c **N/A**

$$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v - v_c)ud}{0.95f_{yv}}$$ N/A >= N/A mm^2

Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$

Case v_3 > 2.0v_c **N/A**

Second shear perimeter shear utilisation **N/A** **N/A**

Note a negative shear stress v_2 and/or v_3 on a correctly specified column (wrt internal, edge or corner) indicates that the shear perimeter is beyond the physical extremes of the foundation and as such punching shear failure is not critical;

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Shear Design for Bending in Plane of Width of Outer Footing

Shear force at column base face, $V_{x,ult}$ = $q_{w,ULS,1}$. $L_{strap,1}$. [($B_{strap,1}$-(b or D))/2	**N/A**	kN	
Shear force at column base face per metre, $V_{x,ult}/L_{strap,1}$	**N/A**	kN/m	
Shear force at 1.0$d_{x,s}$ from column base face, V_x = $q_{w,ULS,1}$. $L_{strap,1}$. [($B_{strap,1}$-	**N/A**	kN	
Shear force at 1.0$d_{x,s}$ from column base face per metre, $V_x/L_{strap,1}$	**N/A**	kN/m	
Note the above shear forces are for bending in plane of width of outer footing;			
Ultimate shear stress for bending in plane of width, $v_{ult,x}$=($V_{x,ult}/L_{strap,1}$)/(1000	**N/A**	N/mm^2	
Ultimate shear stress for bending in plane of width utilisation	**N/A**		**N/A**
Design shear stress for bending in plane of width, $v_{d,x}$=($V_x/L_{strap,1}$)/(1000.$d_{x,s}$	**N/A**	N/mm^2	
(Shear capacity enhancement by calculating v_d at d from "support" and comparing against			
*unenhanced v_c as clause 3.4.5.10 BS8110 employed **instead of** calculating v_d at "support" and*			
comparing against enhanced v_c within 2d of the "support" as clause 3.4.5.8 BS8110;)			
Area of tensile steel reinforcement provided, $A_{s,prov,x,s}$	N/A	mm^2/m	
ρ_w = 100$A_{s,prov,x,s}$/(1000.$d_{x,s}$)	N/A	%	
$v_{c,x}$ = (0.79/1.25)($\rho_w f_{cu}$/25)$^{1/3}$(400/$d_{x,s}$)$^{1/4}$; ρ_w<3; f_{cu}<40; (400/$d_{x,s}$)$^{1/4}$>0.67	**N/A**	N/mm^2	
Check $v_{d,x}$ < $v_{c,x}$ for no links	**N/A**		
Concrete shear capacity $v_{c,x}$.(1000.$d_{x,s}$)	**N/A**	kN/m	
Check $v_{c,x}$ < $v_{d,x}$ < 0.4 + $v_{c,x}$ for nominal links	**N/A**		
Provide nominal links such that A_{sv} / S > 0.4.(1000)/(0.95f_{yv}) i.e.	**N/A**	$mm^2/mm/m$	
Concrete and nominal links shear capacity (0.4 + $v_{c,x}$).(1000.$d_{x,s}$)	**N/A**	kN/m	
Check $v_{d,x}$ > 0.4 + $v_{c,x}$ for design links	**N/A**		
Provide shear links A_{sv} / S > 1000.($v_{d,x}$-$v_{c,x}$)/(0.95f_{yv}) i.e. A_{sv} / S >	**N/A**	$mm^2/mm/m$	
Concrete and design links shear capacity ($A_{sv,prov,x}/S_x$).(0.95f_{yv}).d_x	**N/A**	kN/m	
Area provided by all links per metre, $A_{sv,prov,x}$	N/A	mm^2/m	
Tried $A_{sv,prov,x}$ / S_x value	N/A	$mm^2/mm/m$	
Design shear resistance for bending in plane of width of outer footing utilisati	**N/A**		**N/A**

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Shear Design in Beam

Shear force at column 1 base right centreline, $V_{y,1} = F_{col,v,1,uls} - q_{w,ULS,1} \cdot B_{strap}$	N/A	kN	
Shear force at outer footing right face, $V_{y,2} = F_{strap,v,1,uls} - F_{col,v,1,uls}$	N/A	kN	
Shear force at column 2 base left centreline, $V_{y,3} = F_{col,v,2,uls} - q_{w,ULS,2}.B_{strap,2}.B_{s}$	N/A	kN	
Shear force at column 2 base right centreline, $V_{y,4} = q_{w,ULS,2}.B_{strap,2}.B_{strap,2}/2$	N/A	kN	
Shear force at critical location, $V_y = MAX(V_{y,1}, V_{y,2}, V_{y,3}, V_{y,4})$	**N/A**	kN	
Ultimate shear stress in beam, $v_{ult,y}=V_y/(b_{beam}.d_{y,h})$ ($< 0.8f_{cu}^{0.5}$ & $5N/mm^2$)	**N/A**	N/mm^2	
Ultimate shear stress in beam utilisation	**N/A**		**N/A**
Design shear stress in beam, $v_{d,y}=V_y/(b_{beam}.d_{y,h})$	**N/A**	N/mm^2	
*(Conservatively, shear capacity enhancement by **either** calculating v_d at d from support and comparing against unenhanced v_c as clause 3.4.5.10 BS8110 **or** calculating v_d at support and comparing against enhanced v_c within 2d of the support as clause 3.4.5.8 BS8110 ignored;)*			
Area of tensile steel reinforcement provided, $A_{s,prov,y,h}$	N/A	mm^2	
Note it is assumed that $A_{s,prov,y,h}$ provided where critical shear force occurs;			
$\rho_w = 100A_{s,prov,y,h}/(b_{beam}.d_{y,h})$	N/A	%	
$v_{c,y} = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{y,h})^{1/4}$; $\rho_w<3$; $f_{cu}<40$; $(400/d_{y,h})^{1/4}>0.67$	**N/A**	N/mm^2	
Check $v_{d,y} < 0.5v_{c,y}$ for no links	**N/A**		
Concrete shear capacity $v_{c,y}.(b_{beam}.d_{y,h})$	**N/A**	kN	
Check $0.5v_{c,y} < v_{d,y} < 0.4 + v_{c,y}$ for nominal links	**N/A**		
Provide nominal links such that $A_{sv} / S > 0.4b_{beam}/(0.95f_{yv})$ i.e. A_{s}	**N/A**	mm^2/mm	
Concrete and nominal links shear capacity $(0.4 + v_{c,y}).(b_{beam}d_{y,h})$	**N/A**	kN	
Check $v_{d,y} > 0.4 + v_{c,y}$ for design links	**N/A**		
Provide shear links $A_{sv} / S > b_{beam}(v_{d,y}-v_{c,y})/(0.95f_{yv})$ i.e. $A_{sv} / S >$	**N/A**	mm^2/mm	
Concrete and design links shear capacity $(A_{sv,prov,y}/S_y).(0.95f_{yv}).d_{y}$	**N/A**	kN	
Area provided by all links in a cross-section, $A_{sv,prov,y}$	N/A	mm^2	
Tried $A_{sv,prov,y} / S_y$ value	N/A	mm^2/mm	
Design shear resistance in beam utilisation	**N/A**		**N/A**

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Detailing Requirements

All detailing requirements met ?	N/A		
Max sagging steel reinforcement pitch in plane of width of outer footing (<3d	N/A	mm	N/A
Max hogging steel reinforcement pitch in beam (<$3d_{y,h}$, <750mm)	N/A	mm	N/A

Maximum spacing: 0.5% Ast or less - 300mm
Between 0.5% and 1.0% - 225mm
1.0% Ast or greater - 175mm

Max sagging steel reinforcement pitch in plane of width of outer footing	N/A	mm	N/A
Max hogging steel reinforcement pitch in beam	N/A	mm	N/A
Min sagging steel reinforcement pitch in plane of width of outer footing (>10	N/A	mm	N/A
Min hogging steel reinforcement pitch in beam (>100mm+ϕ_{sy})	N/A	mm	N/A

Note an allowance has been made for laps in the min pitch by increasing the criteria by the bar diameter.

Sagging steel reinforcement diameter in plane of width of outer footing, ϕ_{sx} (	N/A	mm	N/A
Hogging steel reinforcement diameter in beam, ϕ_{sy} (>=16mm)	N/A	mm	N/A
% Max sagging reinforcement in plane of width of outer footing (<= 0.04.10	N/A	%	N/A
% Max hogging reinforcement in beam (<= $0.04.b_{beam}.h_{beam}$)	N/A	%	N/A
Min link diameter, $\phi_{link,y}$ (>=8mm)	N/A	mm	N/A
Link pitch, S_y (<=$0.75d_{y,h}$, <=300mm, >=MAX(100mm,50+$12.5n_{link,y}$)	N/A	mm	N/A
$A_{sv,prov,y}$ / ($b_{beam}.S_y$) (>0.10% G460; >0.17% G250)	N/A	%	N/A

Note that only single layer of reinforcement assumed for beams in calculation of pitch;

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Standard Strap Footing Foundation Reinforcement Details

As per standard multi column footing reinforcement details;

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Raft Foundation Dimensions

Note that the raft is the two-dimensional version of the combination of multi column footings, combined footings and strap footings. These two-dimensional multi-column, combined and strap footings combinations can effectively be analysed and designed as an inverted flat slab in the case of a solid raft, or as an inverted one- or two-way spanning slab in the case of a stripped raft;

Width, B_{raft} (<=L_{raft})	10.000	m	N/A
Length, L_{raft} (>=B_{raft})	10.000	m	N/A
Thickness of foundation, T_{raft}	1.000	m	

Raft Foundation SLS Loading

SLS vertical (downward) load from selected building area, F_{bdarea}	100000	kN	N/A

Note that DL of base slab is to be included and is the DL of the raft;

Note a selected building area may for instance refer to either the highly loaded core area or the normally loaded regular floor plate area, as they should be analysed and designed separately;

Additional soil (above footing) weight, $F_{above,soil}$ = $B_{raft}.L_{raft}.MAX(0$ Exclude t	N/A	kN	

Note additional soil above the footing is included for embedded footings whereby the top of the footing is below ground level and backfilled, for conservatism the saturated soil density is adopted, and $\rho_c \approx \gamma_{sat}$;

Total foundation SLS vertical (downward) load, $F_{raft,v}$ = F_{bdarea} + $F_{above,soil}$	N/A	kN
Gross working pressure, q_w = $F_{raft,v}$ / (B_{raft} . L_{raft})	N/A	kPa

Raft Foundation (Inverted) ULS Loading

SLS vertical (downward) load from selected building area minus raft DL, $F_{raft,}$	N/A	kN
ULS vertical (downward) load from selected building area minus raft DL, $F_{raft,}$	N/A	kN
ULS vertical (downward) pressure from selected building area minus raft DL,	N/A	kPa

Note the above ULS vertical (downward) pressure can be applied on an inverted flat, one- or two-way spanning slab analysis as the ULS pressure for the determination of effects for the design of the solid or stripped raft foundation;

Raft Foundation Reinforcement Design

Design reinforcement based on the combination of multi column footings, combined footings and strap footings reinforcement designs, culminating in the reinforcement design of a two-dimensional inverted flat, one- or two-way spanning slab as the raft foundation;

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Standard Raft Foundation Reinforcement Details

Design reinforcement based on the combination of standard multi column footing, standard combined footing and standard strap footing reinforcement details, culminating in the standard reinforcement detailing of a two-dimensional inverted flat, one- or two-way spanning slab as the raft foundation;

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Building Regulations Minimum Dimensions

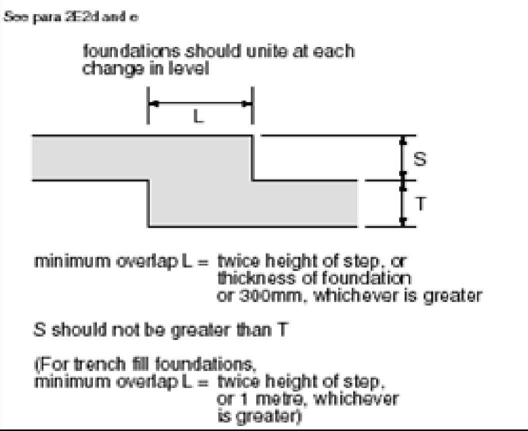
Minimum Width

Table 10 Minimum width of strip footings

Type of Ground (including engineered fill)	Condition of ground	Field test Applicable	Total load of load-bearing walling not more than (kN/linear metre) 20	30	40	50	60	70
			Minimum width of strip foundation (mm)					
I Rock	Not inferior to sandstone, limestone or firm chalk	Requires at least a pneumatic or other mechanically operated pick for excavation	In each case equal to the width of wall					
II Gravel or Sand	Medium dense	Requires pick for excavation. Wooden peg 50mm square in cross section hard to drive beyond 150mm	250	300	400	500	600	650
III Clay Sandy Clay	 Stiff Stiff	Can be indented slightly by thumb	250	300	400	500	600	650
IV Clay Sandy Clay	 Firm Firm	Thumb makes impression easily	300	350	450	600	750	850
V Sand Silty sand Clayey sand	 Loose Loose Loose	Can be excavated with a spade. Wooden peg 50mm square in cross section can be easily driven	400	600	Note Foundations on soil types V and V1 do not fall within the provisions of this section if the total load exceeds 30 kN/m.			
VI Silt Clay Sandy clay Clay or silt	 Soft Soft Soft Soft	Finger pushed in up to 10mm	450	650				
VII Silt Clay Sandy clay Clay or silt	 Very soft Very soft Very soft Very soft	Finger easily pushed in up to 25mm	Refer to specialist advice					

Stepped Foundations

Diagram 22 Elevation of stepped foundation



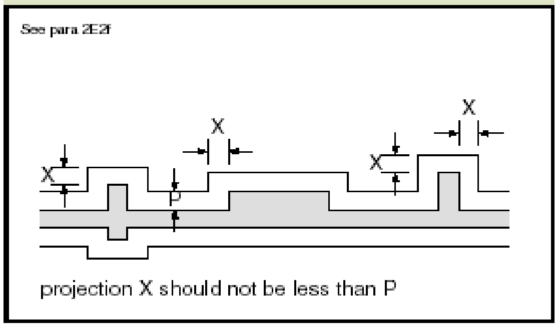
d. foundations stepped on elevation should overlap by twice the height of the step, by the thickness of the foundation, or 300mm, whichever is greater (see Diagram 22).

For trench fill foundations the overlap should be twice the height of step or 1 metre, whichever is greater.

e. steps in foundations should not be of greater height than the thickness of the foundation (see Diagram 22).

Piers and Chimneys

Diagram 23 Piers and chimneys



f. foundation of piers buttresses and chimneys should project as indicated in Diagram 23 and the projection X should never be less than the value of P where there is no local thickening of the wall.

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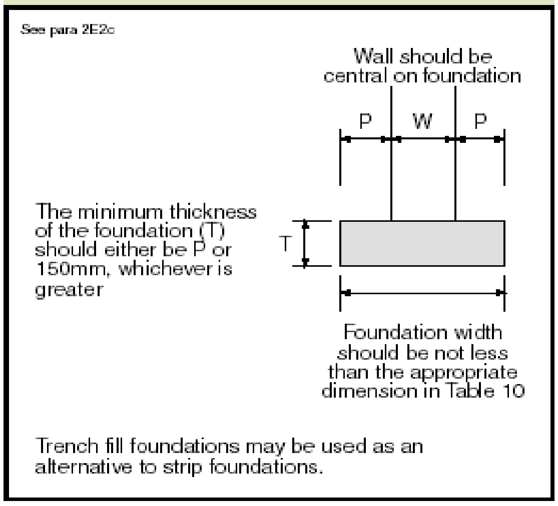
Minimum Thickness

Diagram 24 **Foundation dimensions**



c. minimum thickness T of concrete foundation should be 150mm or P, whichever is the greater where P is derived using Table 10 and Diagram 24. Trench fill foundations may be used as an acceptable alternative to strip foundations.

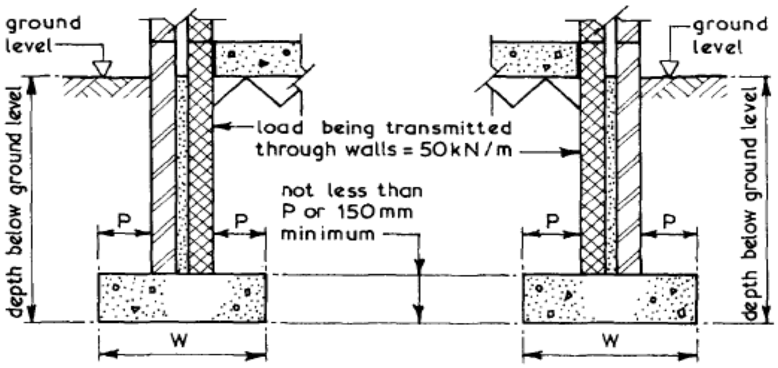
Foundations—Basic Sizing

Basic Sizing ~ the size of a foundation is basically dependent on two factors —

1. Load being transmitted.
2. Bearing capacity of subsoil under proposed foundation.

Bearing capacities for different types of subsoils may be obtained from tables such as those in BS 8004: Code of practice for foundations and BS 8103: Structural design of low rise buildings, or from soil investigation results.

Typical Examples ~



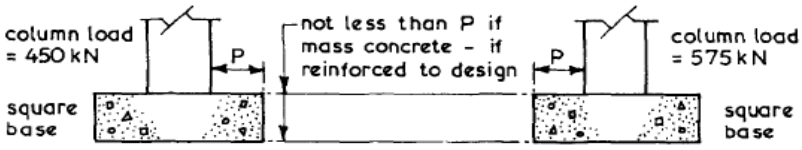
safe bearing capacity of compact gravel subsoil = 100 kN/m²

$W = \frac{\text{load}}{\text{bearing capacity}} = \frac{50}{100}$ = 500 mm minimum

safe bearing capacity of clay subsoil = 80 kN/m²

$W = \frac{\text{load}}{\text{bearing capacity}} = \frac{50}{80}$ = 625 mm minimum

The above widths may not provide adequate working space within the excavation and can be increased to give required space. Minimum width for a limited range of strip foundations can be taken direct from Table 12 in Approved Document A.



bearing capacity of subsoil 150 kN/m²

area of base = $\frac{\text{load}}{\text{bc}} = \frac{450}{150}$

$= 3\,m^2 \therefore$ side $= \sqrt{3}$ = 1·732 min.

bearing capacity of subsoil 85 kN/m²

area of base = $\frac{\text{load}}{\text{bc}} = \frac{575}{85}$

$= 6{\cdot}765\,m^2 \therefore$ side $= \sqrt{6{\cdot}765}$ = 2·6 min.

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Minimum Depth (Frost Heave in Granular or Cohesive Soils)

Minimum depth is **450mm** to avoid **frost heave**, but this is often exceeded due to other

Minimum Depth (Volume Change in Cohesive Soils Without Trees)

This is applicable to **cohesive soils only**, **not granular soils**.

For there to be no influence from trees, the following **minimum distance** must be satisfied (NHBC, 2002):-
- Low water demand trees = 0.2 x mature height
- Moderate water demand trees = 0.5 x mature height
- High water demand trees = 1.25 x mature height

Minimum depth to avoid **seasonal volume changes** due to wetting and drying of expandable and shrinkable clays are as follows (NHBC, 2002):-
- Low plasticity index (10-20%) = **750mm**
- Medium plasticity index (10-40%) = **900mm**
- High plasticity index (>40%) = **1000mm**

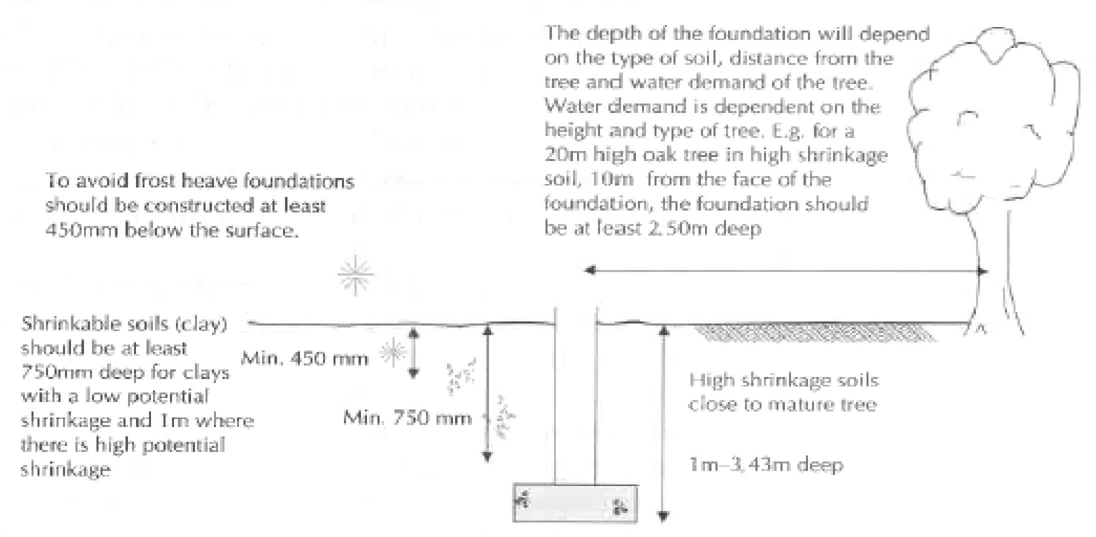


Figure 3.13 Depth of foundations and stability (information adapted from NHBC 2000).

Minimum Depth (Volume Change in Cohesive Soils With Trees)

Water demand and mature height of selected UK trees

The following common British trees are classified as having high, moderate or low water demand. Where the tree cannot be identified, assume high water demand.

Water demand	Broad leaved trees				Conifers		Broad leaf orchard trees	
	Species	Mature height* m	Species	Mature height* m	Species	Mature Height* m	Species	Mature height* m
High	Elm Eucalyptus Oak	18–24 18 16–24	Poplar Willow	25–28 16–24	Cypress	18–20		
Moderate	Acacia false Alder Ash Bay laurel Blackthorn Cherry Hawthorn Honey locust Hornbeam Horse chestnut	18 18 23 10 8 9–17 10 14 17 20	Laburnum Lime Maple Mountain ash Plane Sycamore Tree of heaven Walnut Whitebeam	12 22 8–18 11 26 22 20 18 12	Cedar Douglas fir Pine Spruce Wellingtonia Yew	20 20 20 18 30 12	Apple Cherry Pear Plum	9 15 12 10
Low	Beech Birch Holly	20 14 12	Magnolia Mulberry	9 9				

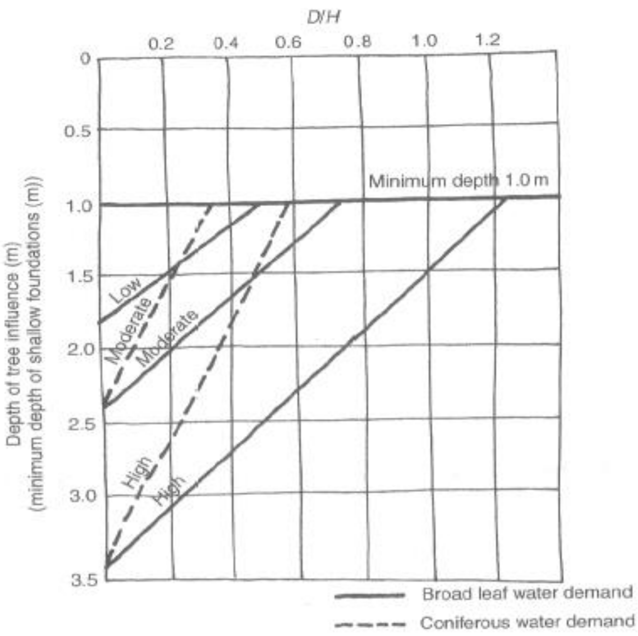
* For range of heights within species, see the full NHBC source table for full details.

Source: NHBC (2002). The information may change at any time and revisions should be checked with NHBC.

Suggested depths for foundations on cohesive soil

If *D* is the distance between the tree and the foundation, and *H* is the mature height of the tree, the following three charts (based on soil shrinkability) will estimate the required foundation depth for different water demand classifications. The full NHBC document allows for a reduction in the foundation depth for climatic reasons, for every 50 miles from the South-East of England.

Suggested depths for foundations on highly shrinkable soil



Source: NHBC (2002). The information may change at any time and revisions should be checked with NHBC.

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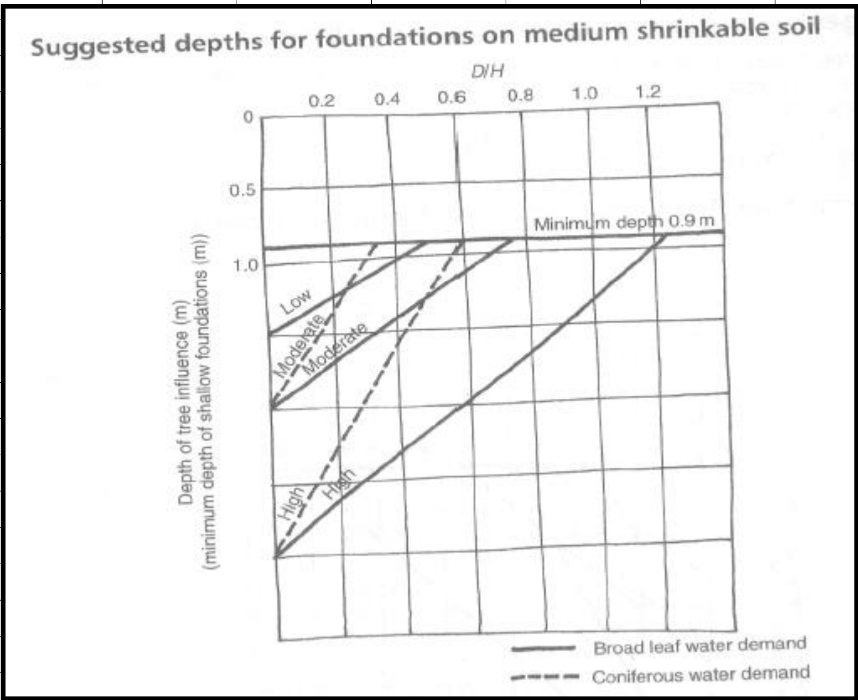

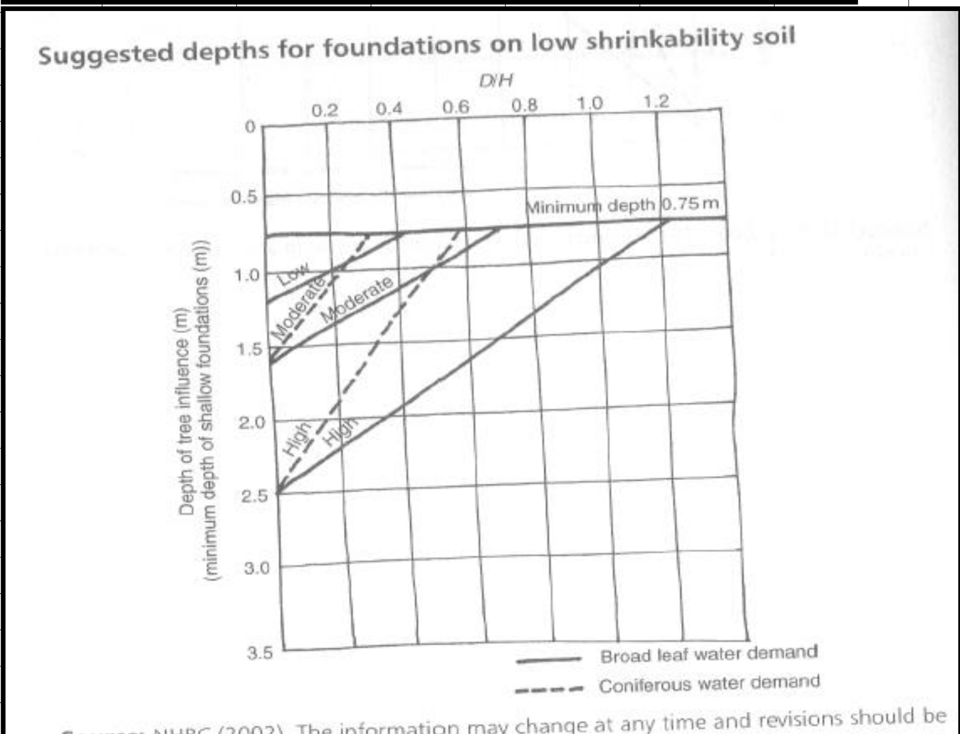

Source: NHBC (2002). The information may change at any time and revisions should be checked with NHBC.

How to determine foundation depth (m) adjacent to trees in shrinkable soils

Species	Maximum mature height (m)	Exclusion zone 1 (m)	Exclusion zone 2 (m)
High water demand trees			
Elm, Willow	24	24.0	30.0
Eucalyptus	18	18.0	22.5
Hawthorn	10	10.0	12.5
Oak, Cypress	20	20.0	25.0
Poplar	28	28.0	35.0
Moderate water demand trees			
Acacia, Alder, Monkey puzzle, Spruce	18	9.0	13.5
Apple, Bay laurel, Plum	10	5.0	7.5
Ash	23	11.5	17.3
Beech, Cedar, Douglas fir, Larch, Pine	20	10.0	15.0
Blackthorn	8	4.0	6.0
Cherry, Pear, Yew	12	6.0	9.0
Chestnut	24	12.0	18.0
Lime, Sycamore	22	11.0	16.5
Mountain ash	11	5.5	8.3
Plane	26	13.0	19.5
Wellingtonia	30	15.0	22.5
Low water demand trees			
Birch	14	2.8	7.0
Elder	10	2.0	5.0
Fig, Hazel	8	1.6	4.0
Holly, Laburnum	12	2.4	6.0
Hornbeam	17	3.4	8.5
Magnolia, Mulberry	9	1.8	4.5

Foundation depths			
Modified Plasticity Index	Volume change potential	Outside exclusion zone 1	Outside exclusion zone 2
40% and greater	High	1.50	1.00
20% to less than 40%	Medium	1.25	0.90
10% to less than 20%	Low	1.00	0.75

Notes

1 Determine whether a particular species of tree is outside exclusion zone 1 or 2.

2 Determine the foundation depth from the lower part of the table for the particular soil conditions and the appropriate exclusion zone.

3 Where the tree(s) are inside exclusion zone 1 refer to NHBC guidelines [5] on which this table is based.

Scheme Design (Cohesive Soils)

$q_{allowable} \approx 14.2N$

Spread footing on clay

$q_{allowable} = 2C_u$ Spread footing on undrained cohesive soil ($\gamma_f = 2.5$)

a) Cohesive soils (clays)

Strip footing $q_{safe} = \frac{5C_n}{F.O.S}$

Square footing $q_{safe} = \frac{6C_n}{F.O.S}$

Where q_{safe} = Safe Bearing Pressure

C_n = Undrained Shear Strength

F.O.S. = Factor of Safety (usually taken as 3.0)

5.7 Simple foundations on cohesive soils

In cohesive soils, the allowable bearing pressure can be determined from shear tests. The following equations may be used; they incorporate a factor of safety against failure of 3, which is the value normally adopted.

a) *Strip foundations*. The allowable bearing pressure, q_b (in kN/m^2), is given by the expression:

$$q_b = 1.7c$$

where

c is the undrained shear strength (in kN/m^2).

b) *Square foundations*. The allowable bearing pressure, q_b (in kN/m^2), is given by the expression:

$$q_b = 2c$$

where

c is the undrained shear strength (in kN/m^2).

It is important to note that the allowable bearing pressures derived from these expressions are not linked to any particular values of settlement.

Foundations in conhesive soils at a minimum depth of 1m below ground level

Description	Cohesive strength (kN/m^2 or kgf/cm^2 x 100)	Presumed bearing value (kN/m2 or kgf/cm2 x 100) for foundation of width		
		1m	2m	4m
Hard boulder clays, hard fissured clays (e.g. deeper London and Gault clays)	>300	800	600	400
Very stiff boulder clay, very stiff 'blue' London Clay	150-300	400-800	300-500	150-250
Stiff fissured clays (e.g. stiff 'blue' and brown London clay), stiff weathered boulder clay	75-150	200-400	150-250	75-125
Firm normally consolidated clays (at depth), fluvio-glacial and lake clays, upper weathered 'brown' London clay	40-75	100-200	75-100	50-75
Soft normally consolidated alluvial clays (e.g. marine, river and estuarine clays)	20-40	50-100	25-50	Negligible

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Scheme Design (Non Cohesive Soils)

Spread footing on gravel

$q_{allowable} = 10N$ Pad footing on dry soil ($\gamma_f = 3$)

$q_{allowable} = 7N$ Strip footing on dry soil ($\gamma_f = 3$)

$q_{wet\ allowable} = q_{allowable}/2$ Spread foundation at or below the water table

Where N is the SPT value.

b) Cohesionless soils (sands and gravels)

q_{safe} = 10 N

Where q_{safe} = safe bearing pressure

N = SPT values ('N' blows)

Foundations in non-cohesive soils at a minimum depth of 0.75m below ground level

Description of soil	N-value in standard penetration test	Presumed bearing value (kN/m² or kgf/cm² x 100) for foundation of width		
		1m	2m	4m
Very dense sands and gravels	>50	600	500	400
Dense sands and gravals	30-50	350-600	300-500	250-400
Medium-dense sands and gravels	10-30	150-350	100-300	100-250
Loose sands and gravals	5-10	50-150	50-100	50-100

The allowable bearing pressure is defined as that causing 25mm settlement under the foundation width.

If the water table is within a depth equal to the width of the foundation and the depth of the foundation is small in relation to its width, the settlements will be doubled.

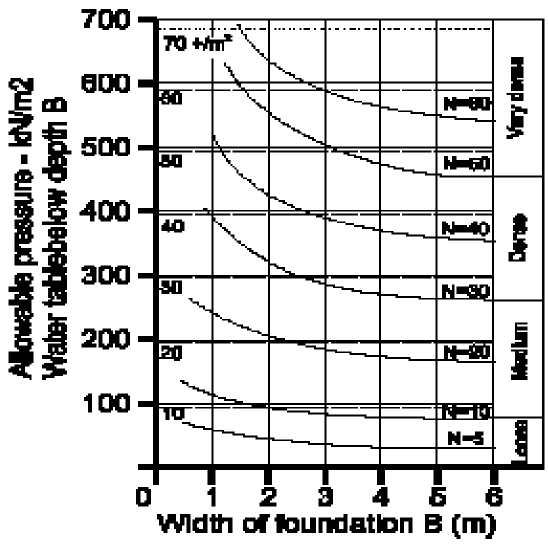
If settlements must not exceed 25mm, the allowable bearing values should be halved.

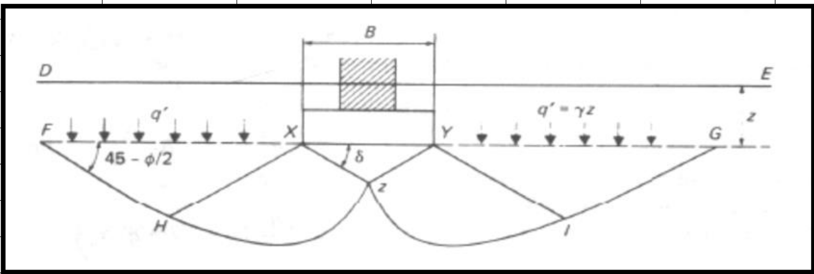
Chart for estimating allowable bearing pressure for foundations in sands

SPT 'N' values are shown as belows per 300mm

If the water table is within a depth equal to the width of the foundation and the depth of the foundation is small in relation to its width, the settlements will be doubled.

If settlements must not exceed 25mm, the allowable bearing values should be halved.





Quick estimate design methods for shallow foundations

General equation for allowable bearing capacity after Brinch Hansen

Factor of safety against bearing capacity failure, $\gamma_f = 2.0$ to 3.0, q'_o is the effective overburden pressure, γ is the unit weight of the soil, B is the width of the foundation, c is the cohesion (for the drained or undrained case under consideration) and N_c, N_q and N_γ are shallow bearing capacity factors.

Strip footings: $q_{allowable} = \dfrac{cN_c + q'_o N_q + 0.5\gamma B N_\gamma}{\gamma_f}$

Pad footings: $q_{allowable} = \dfrac{1.3cN_c + q'_o N_q + 0.4\gamma B N_\gamma}{\gamma_f}$

Approximate values for the bearing capacity factors N_c, N_q and N_γ are set out below in relation to ϕ.

Internal angle of shear ϕ	Bearing capacity factors*		
	N_c	N_q	N_γ
0	5.0	1.0	0.0
5	6.5	1.5	0.0
10	8.5	2.5	0.0
15	11.0	4.0	1.4
20	15.5	6.5	3.5
25	21.0	10.5	8.0
30	30.0	18.5	17.0
35	45.0	34.0	40.0
40	75.0	65.0	98.0

* Values from charts by Brinch Hansen (1961).

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