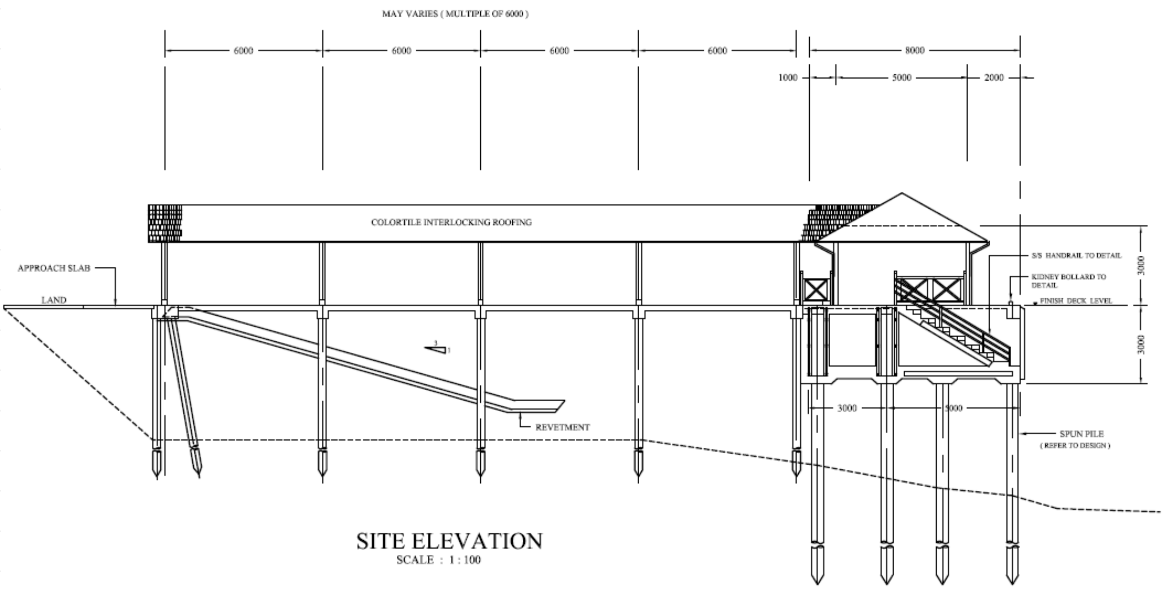
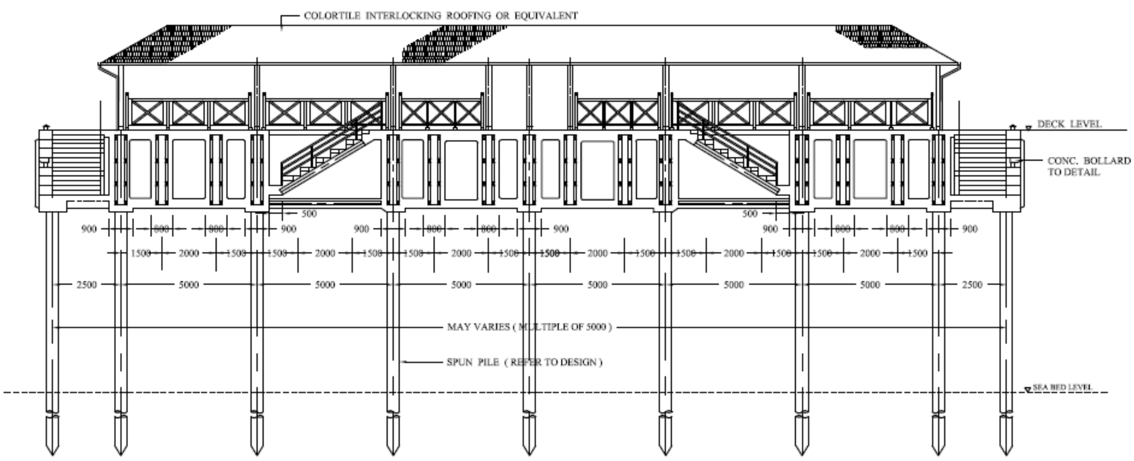
DESIGN EXAMPLE

B: ARCHITECTURE DRAWINGS

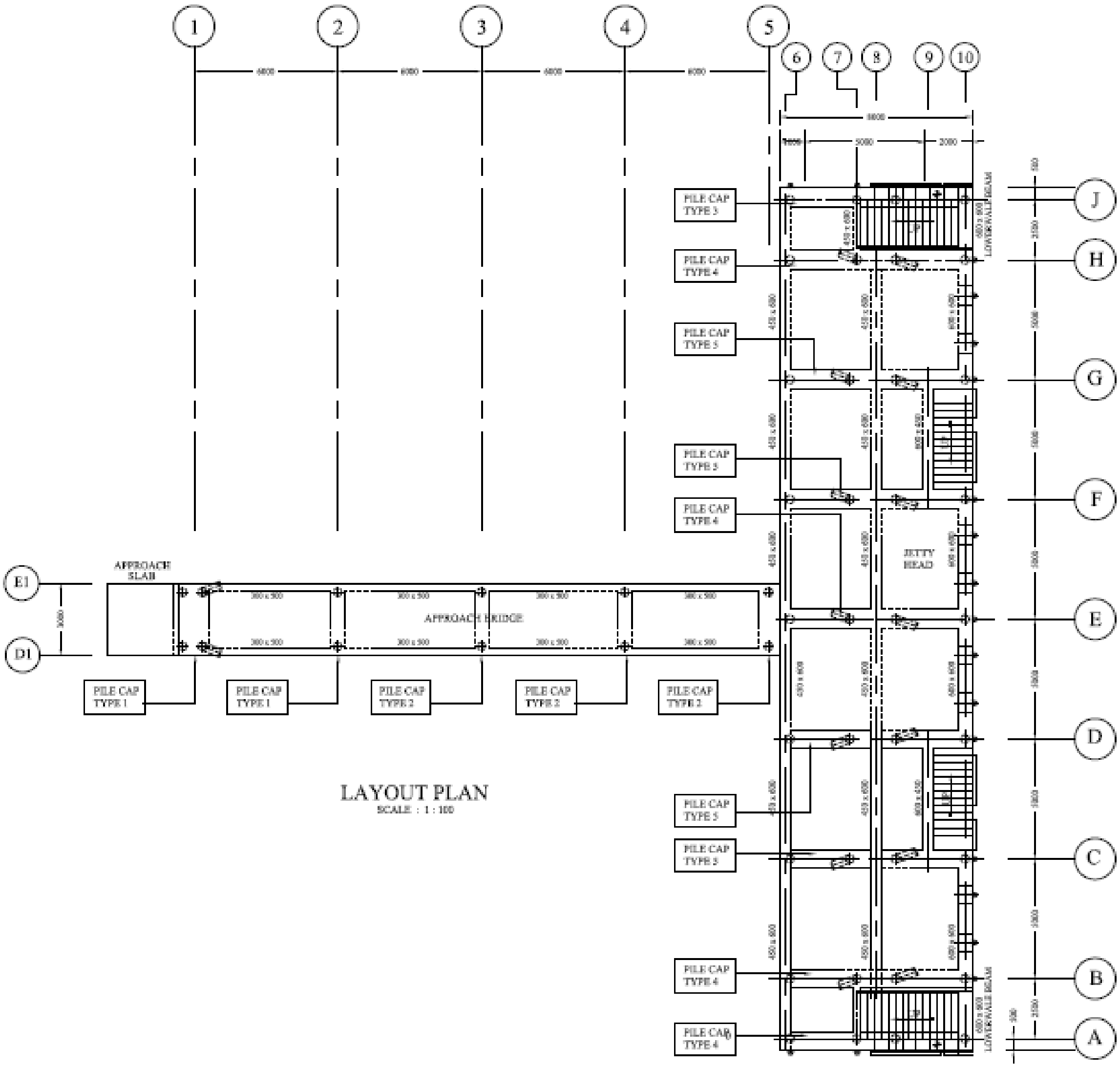


SITE ELEVATION
SCALE : 1 : 100

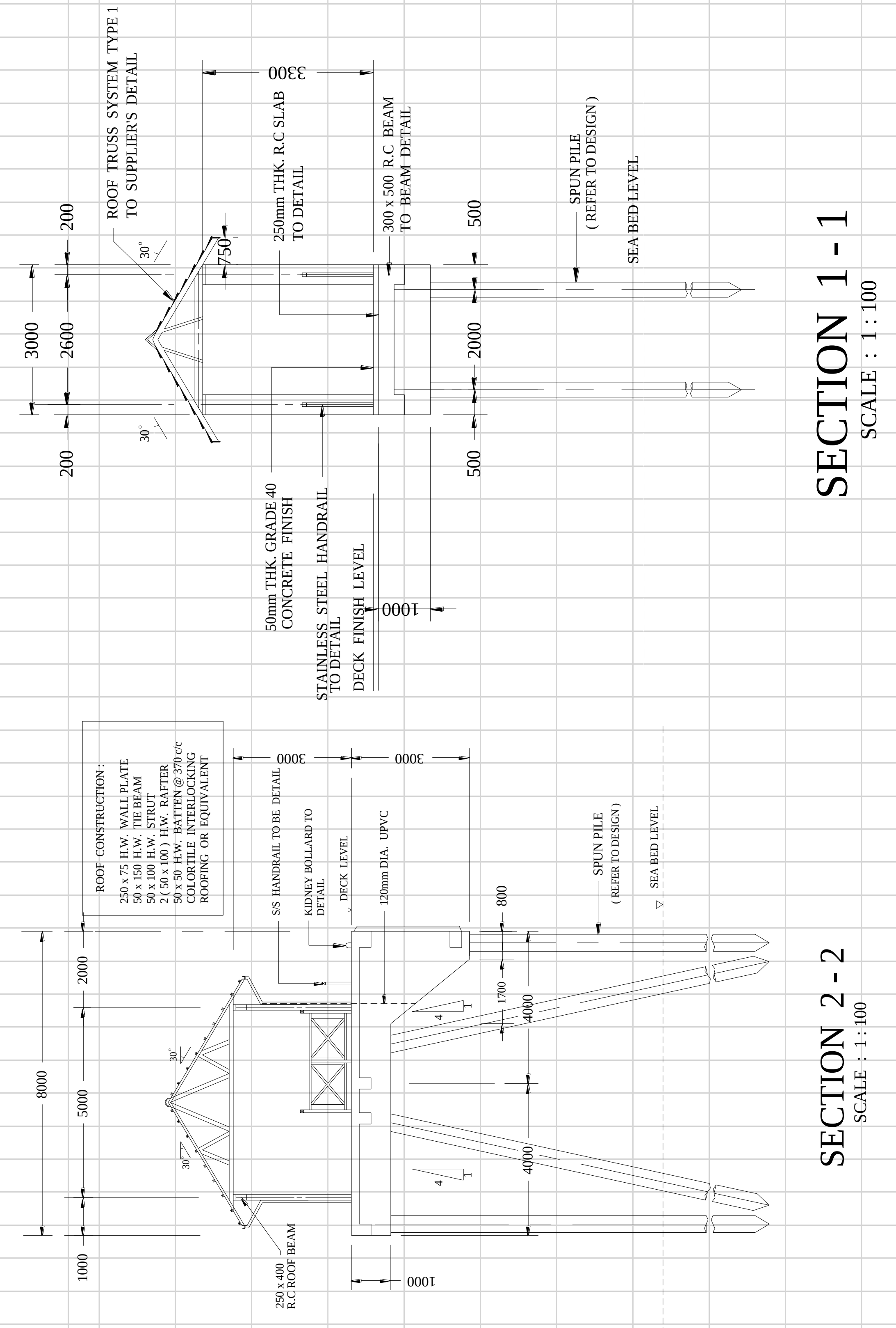


FRONT ELEVATION
SCALE : 1 : 100

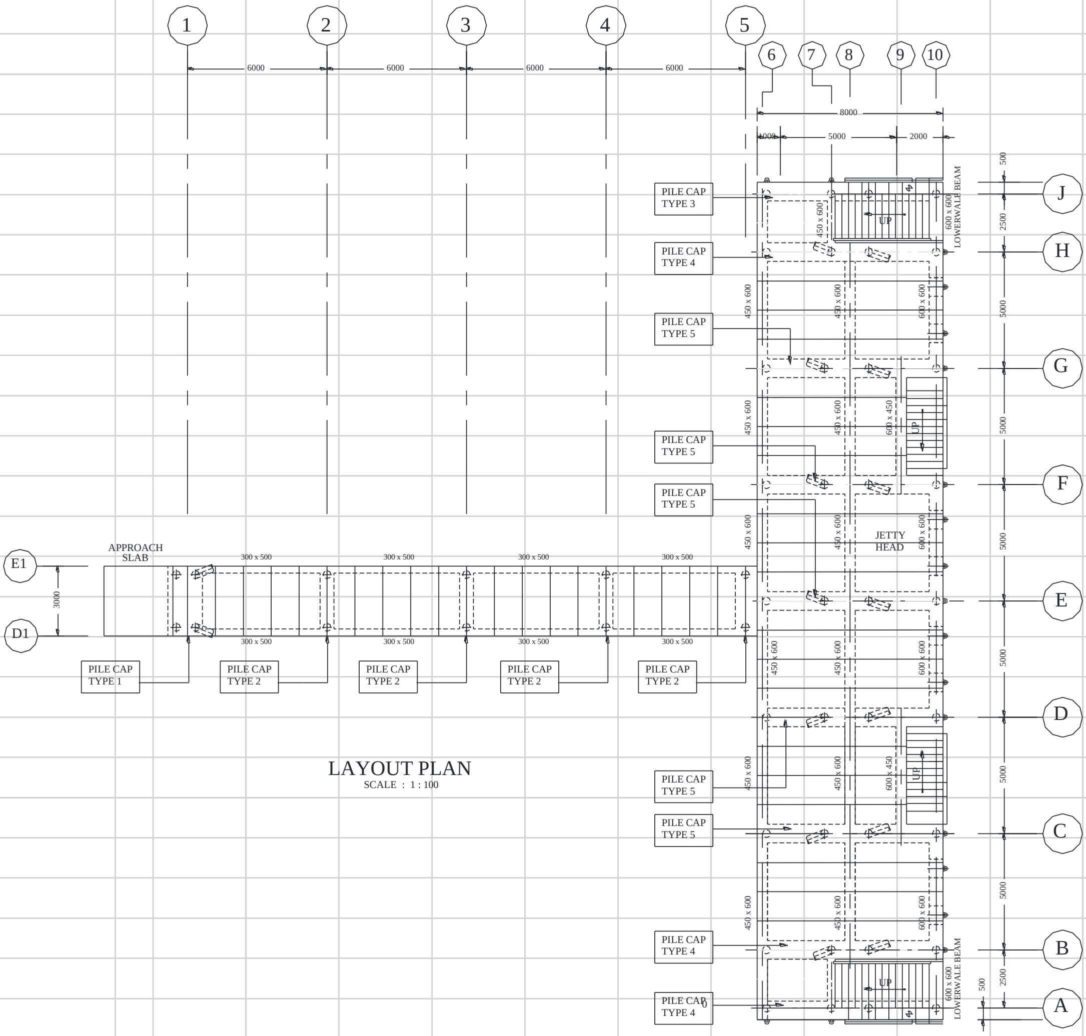
ARCHITECTURE DRAWINGS



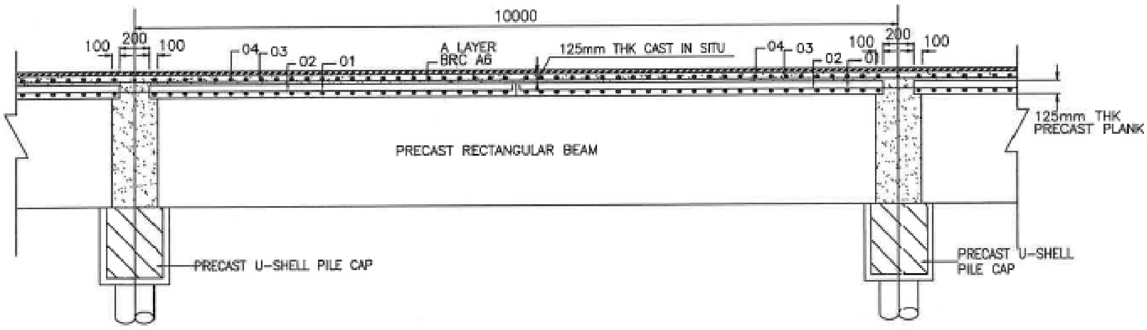
ARCHITECTURE DRAWINGS



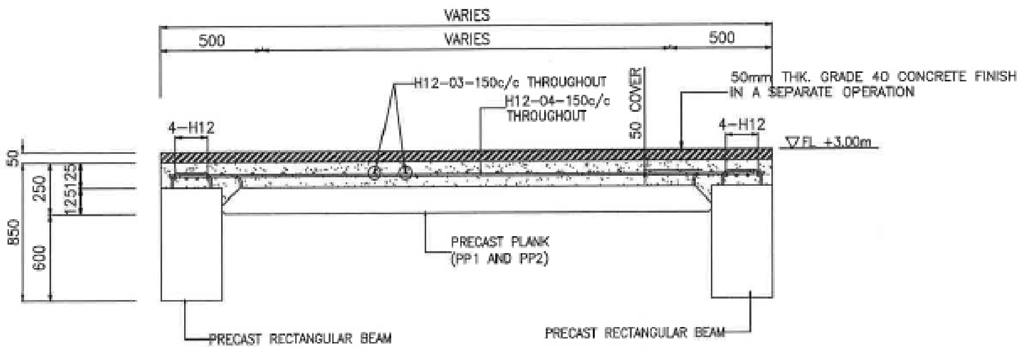
Ref.	C : STRUCTURAL LAYOUTS AND DETAILS



Ref.	STRUCTURAL LAYOUTS	

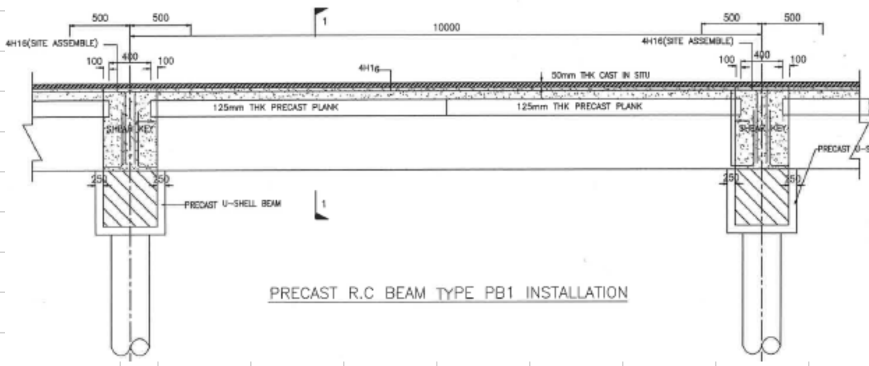


R.C PRECAST PLANK TYPE PP1, PP2, PP3, PP4, PP5 AND PP6 TYPICAL LONGITUDINAL REINFORCEMENT DETAIL

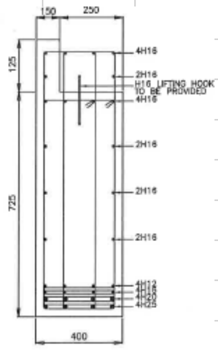


SECTION A2-A2 (TYPICAL REINFORCEMENT DETAILS)

PRECAST R.C BEAM TYPE PB2 INSTALLATION

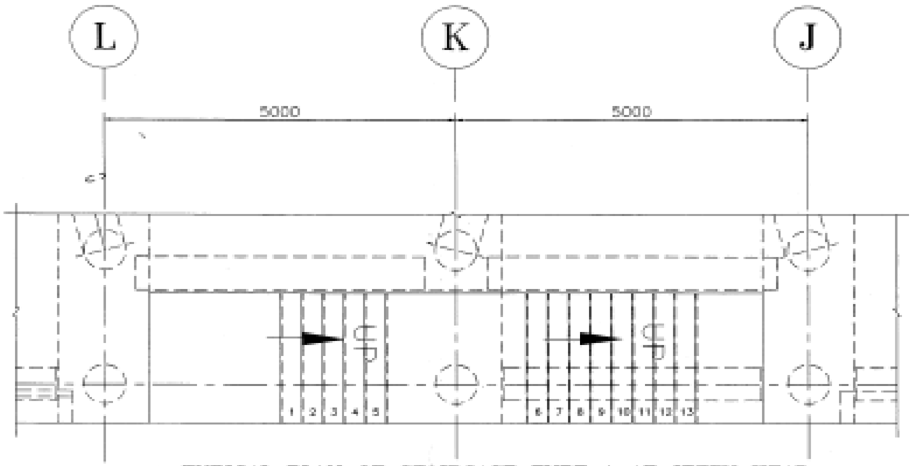


PRECAST R.C BEAM TYPE PB1 INSTALLATION

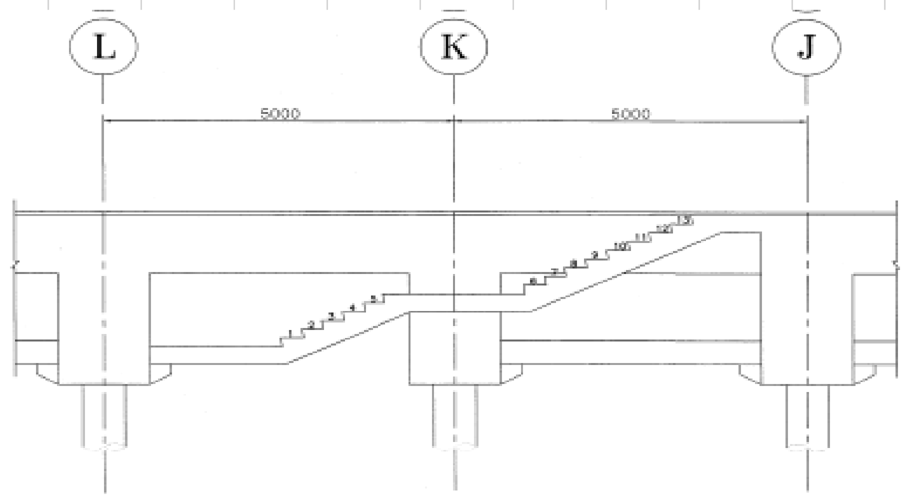


SECTION 1 - 1

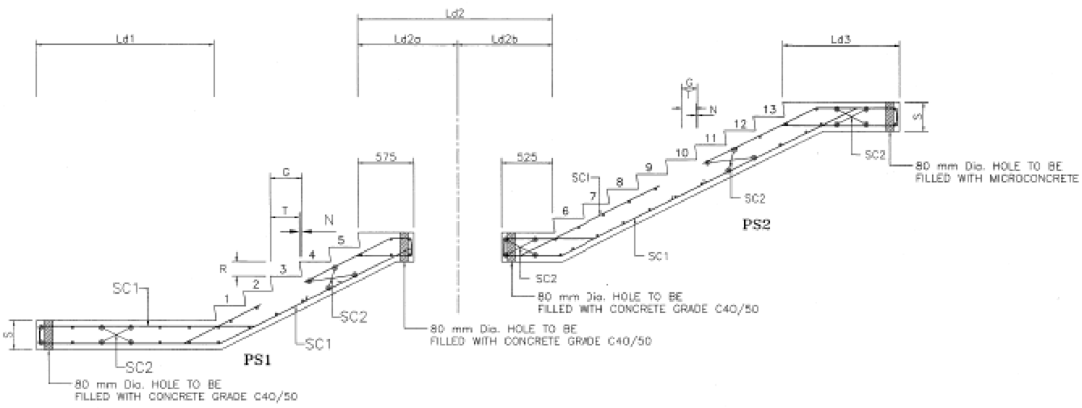
Ref.	STRUCTURAL LAYOUTS	



TYPICAL PLAN OF STAIRCASE TYPE 1 AT JETTY HEAD
SCALE 1:50



TYPICAL FRONT ELEVATION OF STAIRCASE TYPE 1
SCALE 1:50



PRECAST STAIRCASE TYPE 1 REINFORCEMENT DETAIL
SCALE 1:25

D : DESIGN SPECIFICATIONS

8.4	**STRUCTURE**
BS 6349-2	Maritime Strucrures - Suspended Deck Reinforced Concrete Jetty
	DESIGN CODES & LITERATURE
	1. BS 6349 -1-1: 2013 Maritime Works - Planning and Design for Operation
	2. BS 6349-2: 2010 Maritime Works - Design of quay walls, jetties and dolphins
	3. BS 6349-1-4: 2013 Maritime Works - Materials
	4. MS EN 1990: 2010 Eurocode - Basis of structural design
	5. MS EN 1991: 2010 Eurocode 1 - Actions on structures
	6. MS EN 1992 : 2010 Eurocode 2 - Design of concrete structures
	7. McConnel, K. et.al. "Piers, Jetties and related structures exposed to waves: Guideline for hydraulic loadings" Thomas Telford, 2004
	8. Port Work Design Manual Part 1: General design consideration for marine works, Civil Engineering Department, The Government of Hong Kong
	9. Port Work Design Manual Part 2: Guide of Piers and Dolphins Civil Engineering Department, The Government of Hong Kong
	10. Reynold, C. E., et. al. "Reynold's reinforced concrete designers's handbook"
Table 1	**DESIGN WORKING LIFE**
BS 6349-1-1	50
Table 4.1	**EXPOSURE CLASS**
EN 1992	Marine structures : Corrosion induced by chlorides from sea water
	XS3 Tidal, splash,and spray zones
	XS2 Permanently submerge
UBBL	**FIRE RESISTANCE**
	1/2 hour (Single storey buildings - general)

	MATERIALS			
EN 206	Concrete:			
Table F.1	Characteristic strength, f_{ck}	=	35	N/mm^2
	Minimum cement content	=	340	kg/m^3
3.2	Maximum w/c ratio	=	0.45	
EN 1992	Steel reinforcement:			
	Main bar - Characteristic strength, f_{yk}	=	500	N/mm^2
	Links - Characteristic strength, f_{ywk}	=	500	N/mm^2

	FOUNDATION
	Prestressed spun pile

ACTIONS

Permanent actions

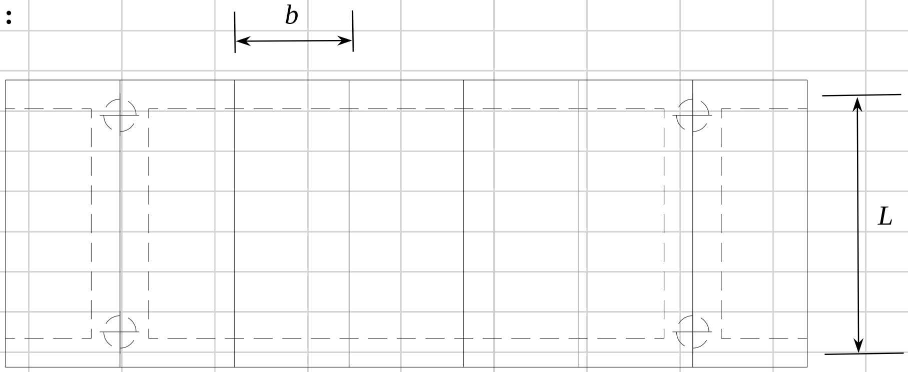
Unit weight of reinforced concrete	=	25	kN/m^3	EN 1991 Table A1
Brickwall: 125 mm thick	=	2.60	kN/m^2	Ref. [11] Table 2.13
Floor finishes:				
Rendering, screed (50 mm thk.)	=	1.15	kN/m^2	Table 2.14 Ref. [11]
Clay floor tiles	=	0.65	kN/m^2	Table 2.14 Ref. [11]
Roof : 23				
Roof tiles	=	0.67	kN/m^2	Table 2.15 Ref. [11]
Metal roof cladding	=	0.07	kN/m^2	
Insulated panel	=	0.20	kN/m^2	
Purlins	=	0.10	kN/m^2	
Suspended ceiling	=	0.15	kN/m^2	
Ceiling joists	=	0.15	kN/m^2	
Truss	=	0.10	kN/m^2	
Services	=	0.10	kN/m^2	

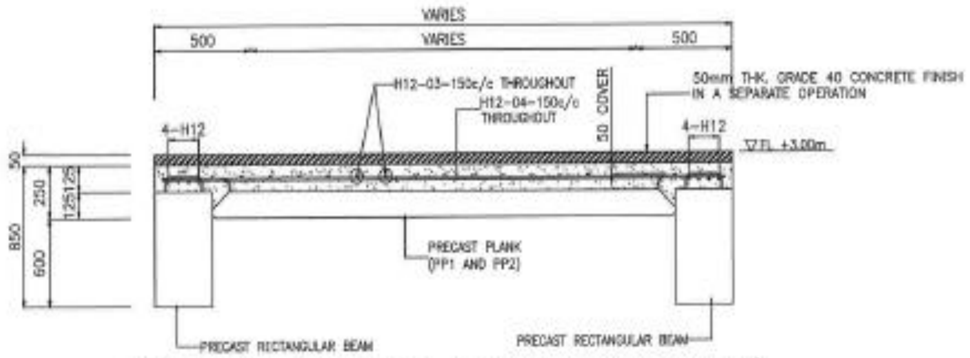
Variable actions

Imposed loads:				
Roof	=	0.25	kN/m^2	EN 1991 Table NA7
Cargo loads	=		kN/m^2	
Padestrian traffic only	=	5.00	kN/m^2	
Emergency & service vehicle	=	10.00	kN/m^2	
Ship berthing loads				
Water current loads				
Wave loading				
Mooring loads				
Force from ship propulsion				
Wind load:				
Basic wind speed	=	33.5	m/s	MS 1553

Ref.	E : DESIGN OF DECK SLAB

Data :





SECTION A2-A2 (TYPICAL REINFORCEMENT DETAILS)

Dimension

Span, L = 3000 mm   Depth, h_{pc} = 125 mm h_{all} = 175 mm

Width, b = 1000 mm $h_{in\text{-}situ}$ = 50 mm

Ref.		
Table 1.4	**Exposure**	**Limiting crack width,**
Table 7.1N	**class =** XS3	w_k = 0.3 mm

Creep coefficient, φ = 2.0

Materials :			**Durability & Bond**		
f_{ck} =	35	N/mm^2	$c_{min,b}$ =	12	mm
γ_c =	25	kN/m^3	$c_{min,dur}$ =	40	mm
E_{cm} =	34	kN/mm^2	Δc_{dev} =	10	mm
f_{yk} =	500	N/mm^2	$c_{nom} = c_{min} + \Delta c_{dev}$ =	50	mm
E_s =	200	kN/mm^2			
ϕ_{bar} =	12	mm			

Action

Premix	0 mm=	0	kN/m^2
Screed	50 mm=	1.25	kN/m^2
Services etc =		1.00	kN/m^2
Permanent =		2.25	kN/m^2
Variable =		5.00	kN/m^2

Ref.	Calculations	Output

ACTIONS

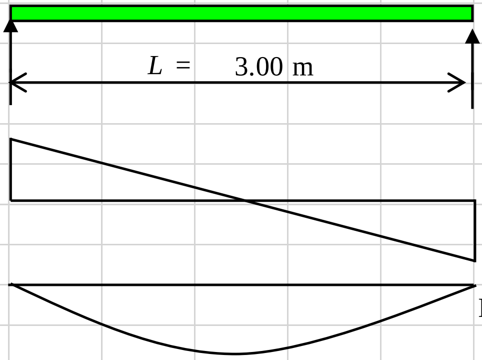
Slab selfweight = 0.175 x 25 =	4.38	kN/m^2
Permanent load (Excluding selfweight) =	2.25	kN/m^2
Characteristic permanent action, g_k =	6.63	kN/m^2
Characteristic variable action, q_k =	5.00	kN/m^2
Design action, $n_d = 1.35g_k + 1.5q_k$ =	16.44	kN/m^2
Quasi-permanent action, $n_{qp} = 1.0g_k + 0.3q_k$ =	8.125	kN/m^2

Output: Table A1.2B : EN 1990 (for the design action)

ANALYSIS

w_d = 16.44 kN/m

L = 3.00 m



M

Shear Force,

V = $w_d L/2$

= 24.7 kN/m

Bending Moment,

$M = w_d L^2/8$

= 18.5 kNm/m

6.1 **MAIN REINFORCEMENT**

Effective depth,

$d = h - C_{nom} - 0.5\phi_{bar}$

= 175 - 50 - (0.5 x 12) = 119 mm

Design moment, M_{Ed} = 18.5 kNm

$K = M / bd^2 f_{ck}$

= 18.5 x 10^6 / (1000 x 119^2 x 35)

= 0.037 < K_{bal} = 0.167

Compression reinforcement is not required

$z = d\ [\ 0.5 + \sqrt{0.25 - K/1.134}\ ]$ = 0.97 d ≤ 0.95d

$A_s = M / 0.87 f_{yk} z$

= 18 x 10^6 / (0.87 x 500 x 0.95 x 119)

= 376 mm^2/m

Main bar : H12 - 300 (377 mm^2/m)

9.2.1.1 Minimum and maximum reinforcement area,

$A_{s,min} = 0.26(f_{ctm}/f_{yk})\ bd$ = 0.26 x (3.21 / 500) x bd

= 0.0017 bd = 0.0017 x1000 x 119 = 199 mm^2/m

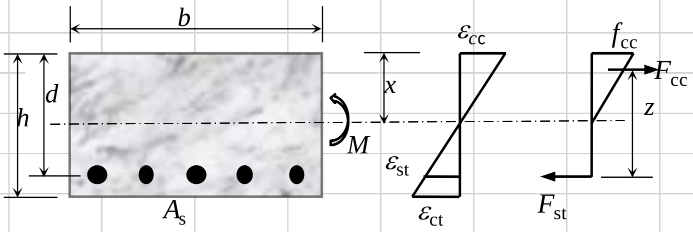
$A_{s,max}$ = 0.04A_c = 0.04 x1000 x 175 = 7000 mm^2/m

Secondary bar : H12 - 400 (283 mm^2/m)

Ref.	Calculations	Output
	SHEAR	
	Design shear force, V_{Ed} = 24.7 kN	
6.2.2	Design shear resistance,	
	$V_{Rd,c}$ = [0.12 k (100$\rho_1 f_{ck}$)$^{1/3}$] bd	
	k = 1 + (200/d)$^{1/2}$ ≤ 2.0	
	= 1+(200/ 119)$^{1/2}$ = 2.30	
	ρ_1 = A_{sl}/bd ≤ 0.02	
	= 377 / (1000 x 119) = 0.003	
	$V_{Rd,c}$ = 0.12 x 2.0 x (100 x 0.003 x 35)$^{1/3}$ x 1000 x 119	
	= 63689 N = 63.7 kN	
	V_{min} = [0.035$k^{3/2} f_{ck}^{1/2}$] bd	
	= 0.035 x 2.0$^{3/2}$ x 35$^{1/2}$ x 1000 x 119	
	= 69694 N = 69.7 kN	
	So, $V_{Rd,c}$ = 69.7 kN > V_{Ed}	Ok !
7.4	**DEFLECTION**	
	ρ = $A_{s,req}$ / bd = 376 / 1000 x 119 = 0.0032	
	ρ_o = (f_{ck})$^{1/2}$ x 10^{-3} = (35)$^{1/2}$x10^{-3} = 0.0059	
Table 7.4N	Factor for structural system, K = 1.0	
	ρ < ρ_o Use equation (1)	
	$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right] \quad (1)$$	
	= 1.0 (11 + 16.6 + 15.40) = 43.0	
	Modification factor for s span less than 7 m = 1.00	
	Modification factor for steel area provided,	
	= $A_{s,prov}/A_{s,req}$ = 377 / 376 = 1.00 < 1.5	
	Therefore allowable span-effective depth ratio,	
	$(l/d)_{allowble}$ = 26.0 x 1.00 x 1.00 = 26.06	
	Actual span-effective depth	
	$(l/d)_{actual}$ = 3000 / 119 = 25.2 < $(l/d)_{allowble}$	Ok !
	CRACKING	
	Limiting crack width, w_{max} = 0.3 mm	
7.3.3	Steel stress,	
	$$f_s = \frac{f_{yk}}{1.15} \times \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)}\frac{1}{\delta}\frac{A_{s,req}}{A_{s,prov}}$$	
	= (500 / 1.15) x [(6.6 + (0.3x 5.0)) / 16.4] x 1.0	
	= 246 N/mm^2	
	Max. allowable bar spacing = 150 mm	
	Max bar spacing, s = 300 mm > 150 mm	Fail !

Ref.	Calculations	Output

7.3.4



Section Strain Stress/Force

Reinforcement H12 - 300

Area of reinforcement, A_s = 377 mm^2/m

Effective depth, d = $h - c - \phi/2$

= 175 - 50 - 12/2 = 119 mm

Modular ratio, $\alpha_e = E_s / E_{cm,eff}$ = 200 / 11 = 17.6

Steel ratio, $\rho = A_s/bd$ = 377 / (1000 x 119) = 0.003

Depth of neutral axis,

$x = \{ -\alpha_e . \rho + [\alpha_e . \rho\ (2 + \alpha_e . \rho)]^{1/2} \}\ d$

= { (17.6 x 0.003) +

[(17.6x 0.003) x (2 + 17.6 x 0.003)]$^{1/2}$}d

= 0.28 d = 0.28 x 119 = 33.7 mm

Lever arm, z = 119 - (33.7 / 3) = 108 mm

Steel tensile stress at quasi-permanent load

$\sigma_s = M_{qp} / (A_s . z)$

= 9.1 x 10^6 / (377 x 108) = 225 N/mm^2

Note:

Allow for creep ? yes

$E_{cm,eff} = E_{cm} / (1+\varphi)$

= 34 /(1 + 2)

= 11.4 kN/mm^2

Eq. 7.11 Maximum crack spacing,

$s_{r,max} = 3.4c + 0.2125 k_1 \phi / \rho_{p,eff}$

c = 50 mm Cover to reinforcement

k_1 = 0.8 Bond coefficient

ϕ = 12 mm Bar diameter

$\rho_{p,eff}$ = Effective reinforcement ratio

= $A_s / A_{c,eff} = A_s / bh_{eff}$

Depth of effective tension area, h_{eff} = The lesser of,

$2.5(h - d)$ = 2.5 (175 - 119) = 140.0 mm

$(h - x) / 3$ = (175 - 33.7) / 3 = 47.1 mm

$h / 2$ = 175 / 2 = 87.5 mm

$\rho_{p,eff}$ = 377 / 1000 x 47.1 = 0.008

$s_{r,max}$ = (3.4 x 50) + (0.2125 x 0.8 x 12 / 0.008)

= 425 mm

Bar spacing = 300 mm

> $5(c + \phi/2)$ = 5(50 + 12 /2)= 280 mm

Upper limit of $s_{r,max} = 1.3(h - x)$ = 1.3 (175 - 33.7)

= 184 mm apply

Ref.	STRUCTURAL LAYOUTS	

Eq. 7.9

Surface strain

$(\varepsilon_{sm} - \varepsilon_{cm}) = [\sigma_s - k_t(f_{ct,eff} / \rho_{p,eff})(1 + \alpha_e\, \rho_{p,eff})] / E_s$

$\geq \; 0.6\sigma_s / E_s$

σ_s =	225	N/mm^2	Steel stress
k_t =	0.4		Duration of load factor
$f_{ct,eff}$ =	3.21	N/mm^2	Tensile strength of concrete
$\rho_{p,eff}$ =	0.0080		Effective reinforcement ratio
α_e =	17.6		Modular ratio

$(\varepsilon_{sm} - \varepsilon_{cm})$

$= \dfrac{225 - [\,0.4\;(3.21 / \;0.008)\,(1 + 17.6 \times 0.008)]}{200 / \times 10^3}$

$= (225 - 183) / 200 \times 10^3 = 0.00021$

$\geq \; 0.6\sigma_s/E_s$

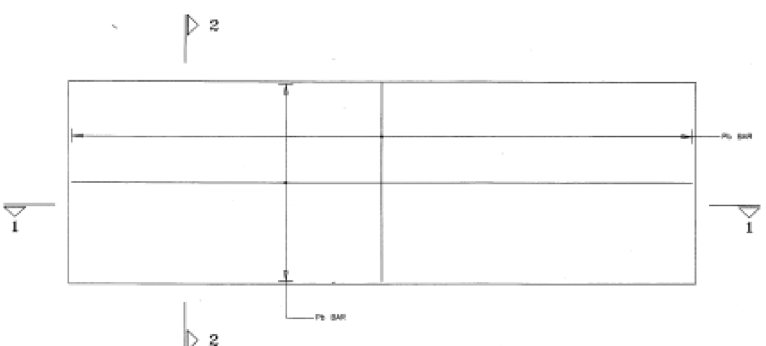
$= 0.6 \times 225 / 200 \times 10^3 = 0.00067$

Crack width,

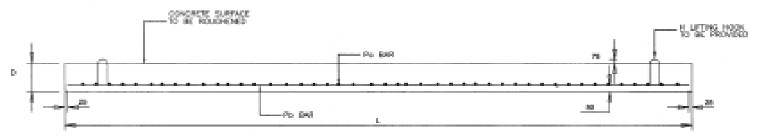
$w_k = s_{r,max}\,(\varepsilon_{sm} - \varepsilon_{cm})$

$= 184 \times 0.00067 = 0.12$ mm $<$ 0.30 mm Ok !

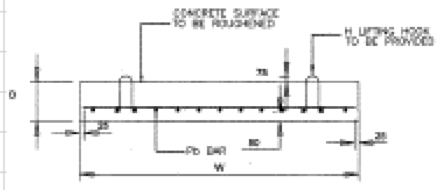
DETAILING



PLAN



SECTION 1-1



SECTION 2-2

Depth, D = 125 mm

Cover = 50 mm

Main reinforcement,

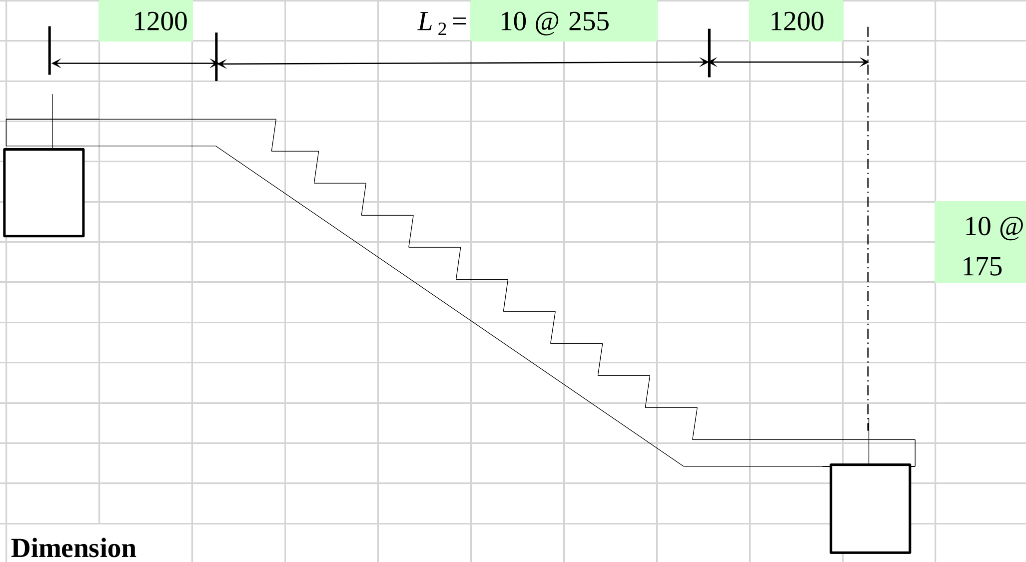
H12 - 300

Secondary reinforcement

H12 - 400

Ref.	F : DESIGN OF STAIRCASE	

Data :



Dimension

Span, L =	4950	mm	Riser, R =	175	mm
Going, G =	255	mm	Waist, h =	250	mm

Exposure class =	XS3	**Limiting crack width,** w_k =	0.3 mm
Creep coefficient, φ =	2.0		

Materials :			**Durability & Bond**		
f_{ck} =	35	N/mm^2	$c_{min,b}$ =	16	mm
γ_c =	25	kN/m^3	$c_{min,dur}$ =	45	mm
E_{cm} =	34	kN/mm^2	Δc_{dev} =	10	mm
f_{yk} =	500	N/mm^2	$c_{nom} = c_{min} + \Delta c_{dev}$ =	55	mm
E_s =	200	kN/mm^2			
ϕ_{bar} =	16	mm			

Action			**Average thickness**
Screed 50 mm=	1.25	kN/m^2	$y = h \, . \, [(G^2 + R^2)^{1/2}/G]$ = 303 mm
Finishes etc =	1.00	kN/m^2	$t = [y + (y + R)] / 2$
Permanent =	2.25	kN/m^2	= 391 mm
Variable =	5.00	kN/m^2	

<u>ACTIONS</u>

<u>Landing</u>

Slab selfweight = 0.250 x 25	=	6.25	kN/m^2	
Permanent load (Excluding selfweight)	=	2.25	kN/m^2	
Characteristic permanent action, g_k	=	8.50	kN/m^2	
Characteristic variable action, q_k	=	5.00	kN/m^2	
Design action, $n_d = 1.35g_k + 1.5q_k$	=	18.98	kN/m^2	Table A1.2B : EN 1990

Ref.	Calculations	Output

<u>Flight</u>

Slab selfweight = 0.391 x 25 = 9.77 kN/m^2

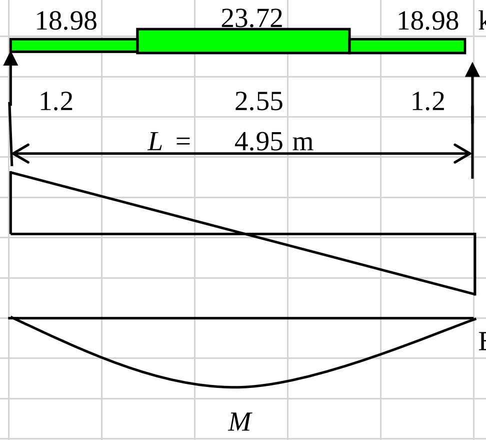
Permanent load (Excluding selfweight) = 2.25 kN/m^2

Characteristic permanent action, g_k = 12.02 kN/m^2

Characteristic variable action, q_k = 5.00 kN/m^2

Design action, $n_d = 1.35g_k + 1.5q_k$ = 23.72 kN/m^2 — Table A1.2B : EN 1990

<u>ANALYSIS</u>



Shear Force,

V_A = 53.02 kN/m

V_B = 53.02 kN/m

Bending Moment,

$M = FL/8$

= 65.6 kNm/m

6.1 **<u>MAIN REINFORCEMENT</u>**

Effective depth,

$d = h - C_{nom} - 0.5\phi_{bar}$

= 250 - 55 - (0.5 x 16) = 187 mm

Design moment, M_{Ed} = 65.6 kNm

$K = M / bd^2 f_{ck}$

= 65.6 x 10^6 / (1000 x 187^2 x 35)

= 0.054 < K_{bal} = 0.167

Compression reinforcement is not required

$z = d\,[\,0.5 + \sqrt{0.25 - K/1.134}\,]$ = 0.95 d ≤ 0.95d

$A_s = M / 0.87 f_{yk} z$

= 66 x 10^6 / (0.87 x 500 x 0.95 x 187)

= 849 mm^2/m

Main bar : H16 - 150 (1341 mm^2/m)

9.2.1.1 Minimum and maximum reinforcement area,

$A_{s,min} = 0.26(f_{ctm}/f_{yk})\,bd$ = 0.26 x (3.21 / 500) x bd

= 0.0017 bd = 0.0017 x1000 x 187 = 312 mm^2/m

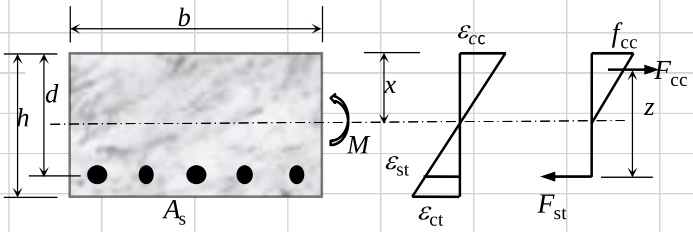
$A_{s,max}$ = 0.04A_c = 0.04 x1000 x 250 = 10000 mm^2/m

Secondary bar : H16 - 250 (804 mm^2/m)

Ref.	Calculations	Output
	SHEAR	
	Design shear force, V_{Ed} = 53.0 kN	
6.2.2	Design shear resistance,	
	$V_{Rd,c} = [\,0.12\,k\,(100\rho_1 f_{ck})^{1/3}\,]\,bd$	
	$k = 1 + (200/d)^{1/2} \leq 2.0$	
	$= 1+(200/\ 187)^{1/2} = 2.03$	
	$\rho_1 = A_{sl}/bd \leq 0.02$	
	= 1341 / (1000 x 187) = 0.007	
	$V_{Rd,c}$ = 0.12 x 2.0 x (100 x 0.007 x 35)$^{1/3}$ x 1000 x 187	
	= 131389 N = 131.4 kN	
	$V_{min} = [\,0.035k^{3/2} f_{ck}^{1/2}\,]\,bd$	
	= 0.035 x 2.0$^{3/2}$ x 35$^{1/2}$ x 1000 x 187	
	= 109519 N = 109.5 kN	
	So, $V_{Rd,c}$ = 131.4 kN > V_{Ed}	Ok !
7.4	**DEFLECTION**	
	$\rho = A_{s,req}\,/\,bd$ = 849 / 1000 x 187 = 0.0045	
	$\rho_o = (f_{ck})^{1/2} \times 10^{-3}$ = (35)$^{1/2}$x10^{-3} = 0.0059	
Table 7.4N	Factor for structural system, K = 1.0	
	ρ < ρ_o Use equation (1)	
	$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]$ (1)	
	= 1.0 (11 + 11.6 + 3.16) = 25.7	
	Modification factor for s span less than 7 m = 1.00	
	Modification factor for steel area provided,	
	$= A_{s,prov}/A_{s,req}$ = 1341 / 849 = 1.58 > 1.5	
	Therefore allowable span-effective depth ratio,	
	$(l/d)_{allowble}$ = 25.7 x 1.00 x 1.50 = 38.58	
	Actual span-effective depth	
	$(l/d)_{actual}$ = 4950 / 187 = 26.5 < $(l/d)_{allowble}$	Ok !
	CRACKING	
	Limiting crack width, w_{max} = 0.3 mm	
	Steel stress,	
	$f_s = \frac{f_{yk}}{1.15} \times \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)}\frac{1}{\delta}\frac{A_{s,req}}{A_{s,prov}}$	
	= (500 / 1.15) x [(12.0 + (0.3x 5.0)) / 23.7] x 0.6	
	= 180 N/mm^2	
	Max. allowable bar spacing = 250 mm	
	Max bar spacing, s = 150 mm < 250 mm	Ok !

Ref.	Calculations	Output

7.3.3



Section Strain Stress/Force

Reinforcement H16 - 150

Area of reinforcement, A_s = 1341 mm^2/m

Effective depth, $d = h - c - \phi/2$

= 250 - 55 - 16/2 = 187 mm

Modular ratio, $\alpha_e = E_s / E_{cm,eff}$ = 200 / 11 = 17.6

Steel ratio, $\rho = A_s/bd$ = 1341 / (1000 x 187) = 0.007

Depth of neutral axis,

$x = \{ -\alpha_e . \rho + [\alpha_e . \rho\ (2 + \alpha_e . \rho)]^{1/2} \}\ d$

= { (17.6 x 0.007) +

[(17.6x 0.007) x (2 + 17.6 x 0.007) $]^{1/2}\}d$

= 0.39 d = 0.39 x 187 = 73.3 mm

Lever arm, z = 187 - (73.3 / 3) = 163 mm

Note:
Allow for creep ? yes
$E_{cm,eff} = E_{cm} / (1+\varphi)$
= 34 /(1 + 2)
= 11.4 kN/mm^2

Steel tensile stress at quasi-permanent load

$\sigma_s = M_{qp} / (A_s\ .\ z)$

= 43.7 x 10^6 / (1341 x 163) = 201 N/mm^2

Eq. 7.11

Maximum crack spacing,

$s_{r,max} = 3.4c + 0.2125 k_1 \phi / \rho_{p,eff}$

c = 55 mm Cover to reinforcement

k_1 = 0.8 Bond coefficient

ϕ = 16 mm Bar diameter

$\rho_{p,eff}$ = Effective reinforcement ratio

= $A_s / A_{c,eff} = A_s / bh_{eff}$

Depth of effective tension area, h_{eff} = The lesser of,

$2.5(h - d)$ = 2.5 (250 - 187) = 157.5 mm

$(h - x) / 3$ = (250 - 73.3) / 3 = 58.9 mm

$h / 2$ = 250 / 2 = 125.0 mm

$\rho_{p,eff}$ = 1341 / 1000 x 58.9 = 0.0228

$s_{r,max}$ = (3.4 x 55) + (0.2125 x 0.8 x 16 / 0.0228)

= 307 mm

Bar spacing = 150 mm

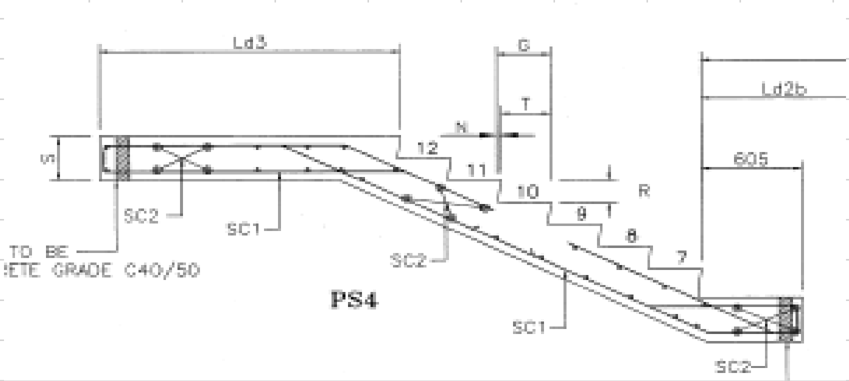
< 5(c + ϕ/2) = 5(55 + 16 /2)= 315 mm

Upper limit of $s_{r,max} = 1.3(h - x)$ = 1.3 (250 - 73.3)

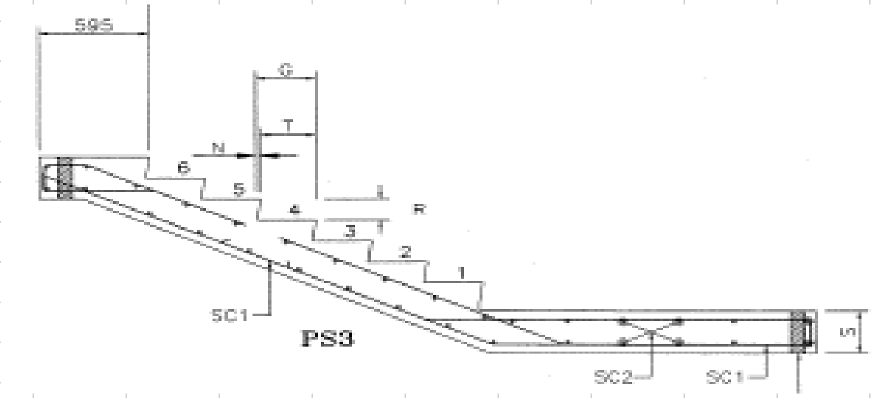
= 230 mm does not apply

Ref.	STRUCTURAL LAYOUTS	
Eq. 7.9	Surface strain	
	$(\varepsilon_{sm} - \varepsilon_{cm}) = [\sigma_s - k_t(f_{ct,eff} / \rho_{p,eff})(1 + \alpha_e \rho_{p,eff})] / E_s$	
	$\geq 0.6\sigma_s / E_s$	
	σ_s = 201 N/mm^2 Steel stress	
	k_t = 0.4 Duration of load factor	
	$f_{ct,eff}$ = 3.21 N/mm^2 Tensile strength of concrete	
	$\rho_{p,eff}$ = 0.0228 Effective reinforcement ratio	
	α_e = 17.6 Modular ratio	
	$(\varepsilon_{sm} - \varepsilon_{cm})$	
	$= \dfrac{201 - [\,0.4\ (3.21 / 0.023)\ (1 + 17.6 \times 0.023)]}{200 / \times 10^3}$	
	$= (201 - 79) / 200 \times 10^3 = 0.00061$	
	$\geq 0.6\sigma_s/E_s$	
	$= 0.6 \times 201 / 200 \times 10^3 = 0.00060$	
	Crack width,	
	$w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm})$	
	$= 307 \times 0.00061 = 0.19$ mm < 0.30 mm	Ok !

DETAILING



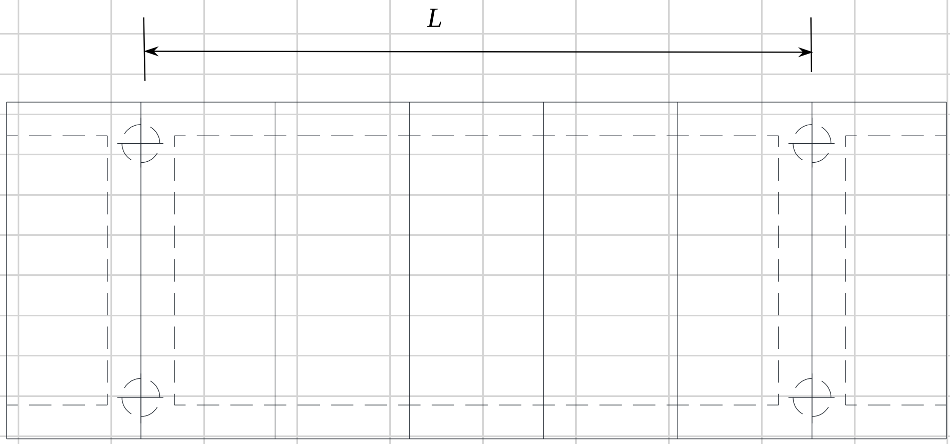
PS4

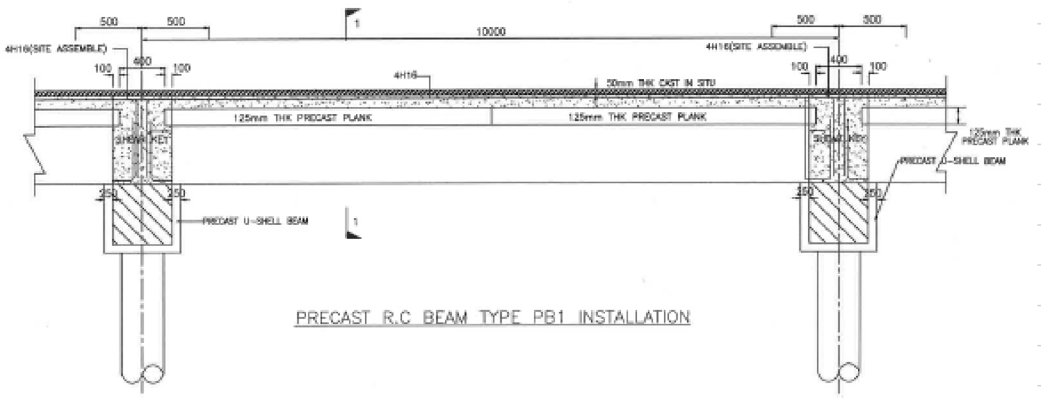


PS3

Ref.	F : DESIGN OF PRECAST BEAM

Data :





Dimension

Span, L =	10000	mm	Depth, h_{pc} =	725	mm	h_{all} =	725	mm
Width, b =	400	mm	$h_{in\text{-}situ}$ =	0	mm			

Exposure **class =** XS3

Limiting crack width, w_k = 0.3 mm

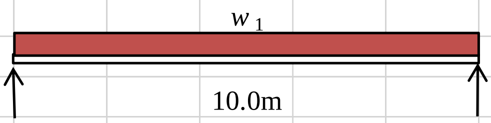
Materials :			**Durability & Bond**		
f_{ck} =	35	N/mm^2	$c_{min,b}$ =	25	mm
f_{yk} =	500	N/mm^2	$c_{min,dur}$ =	45	mm
f_{ykw} =	500	N/mm^2	Δc_{dev} =	10	mm
γ_c =	25	kN/m^3	$c_{nom} = c_{min} + \Delta c_{dev}$ =	55	mm
ϕ_{bar1} =	25	mm			
ϕ_{bar2} =	16	mm	ϕ_{link} =	10	mm

Load from slab

Permanent =	6.63	kN/m^2
Variable =	10.00	kN/m^2

Ref.	Calculations	

LOADING



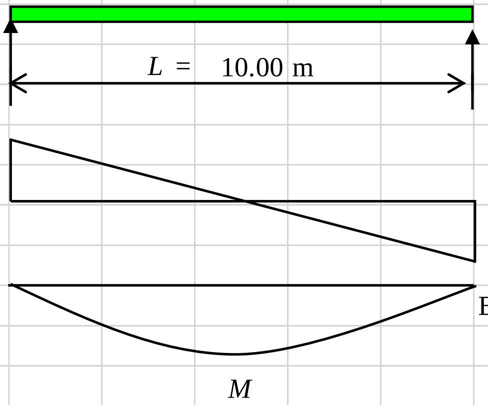
b x h x h_f :
400 x 725 x 0

w_1

Self-weight =	0.40 x	(0.73 -	0.00) x	25 =	7.25 kN/m
Dead load from slab 1=	0.50 x	6.63 x	2.60 =		8.62 kN/m
Dead load from slab 2=	0.00 x	6.63 x	0.00 =		0.00 kN/m
Permanent load, g_k			=		15.87 kN/m
Live load from slab 1=	0.50 x	10.0 x	2.60 =		13.00 kN/m
Live load from slab 2=	0.00 x	10.0 x	0.00 =		0.00 kN/m
Variable load, q_k			=		13.00 kN/m
Design load, $w = 1.35g_k + 1.5q_k$			=		40.92 kN/m

ANALYSIS

w_d = 40.92 kN/m



Shear Force,
$V = w_d L/2$
= 204.6 kN

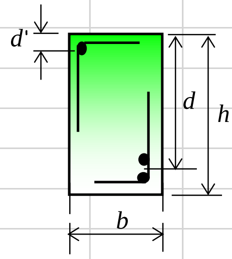
Bending Moment,
$M = w_d L^2/8$
= 511.5 kNm

6.1 **MAIN REINFORCEMENT**

Effective depth,

$d = h - C_{nom} - \phi_{link} - \phi_{bar} - 0.5\phi_{bar}$ = 623 mm

$d' = C_{nom} + \phi_{link} + \phi_{bar}/2$ = 73 mm



Design bending moment, M_{Ed} = 512 kNm

$K = M / f_{ck}bd^2$

= 512 x 10^6 / (35 x 400 x 623^2)= 0.094

Redistribution = 0 % Redistribution ratio, δ = 1.0

Using : EC2

$K_{bal} = 0.454(\delta - k_1)/k_2 - 0.182[(\delta - k_1)/k_2]^2$ k_1 = 0.44

= $0.363(\delta - k_1) - 0.116(\delta - k_1)^2$ = 0.167 k_2 = 1.25

==> $K < K_{bal}$

Compression reinforcement is not required

Ref.		
	$z = d\,[\,0.5 + \sqrt{0.25 - K/1.134}\,]$ = 1.00 d	
	$A_s = M / 0.87 f_{yk} z$	
	$= 512 \times 10^6 / (0.87 \times 500 \times 0.95 \times 623)$	
	$= 1989\ mm^2$	
	$K < K_{bal}$	
	Compression reinforcement is required	
	$z = d\,[\,0.5 + \sqrt{0.25 - K_{bal}/1.134}\,]$ = 0.82 d	
	$x = (d - z) / 0.4 = 279$ mm	
	$d'/x = 73 / 279 = 0.26 < 0.38$	
	The compression steel will have yielded $f_{sc} = 0.87 f_{yk}$	
	$A_s' = (K - K_{bal}) f_{ck} b d^2 / 0.87 f_{yk} (d - d')$	
	$= \dfrac{(0.094 \quad -0.17) \times (35 \times 400 \times 623^2)}{0.87 \times 500 \times (623 - 73)}$	
	$= -1647\ mm^2$	
	$A_s = K_{bal} f_{ck} b d^2 / 0.87 f_{yk} z_{bal} + A_{s'}$	
	$= \dfrac{0.167 \times (35 \times 400 \times 623^2)}{0.87 \times 500 \times 0.82 \times 623}$ -+ 1647	Use : Top
		3H 16
	$= 2426\ mm^2$	(603 mm^2)
	A_s = 1989 mm^2 **A_s' = 0 mm^2**	Bottom
		4H 16
9.2.1.1	Minimum and maximum reinforcement area,	4H 25
	$A_{s,min} = 0.26 (f_{ctm}/f_{yk})\, bd$	(2768 mm^2)
	$= 0.26\ (3.21\ / 500)\ bd = 0.0017\ bd$	
	$= 0.0017 \times 400 \times 622.5 = 416\ mm^2$	
	$A_{s,max} = 0.04 A_c = 0.04 \times 400 \times 725$	
	$= 11600\ mm^2$	
	<u>SHEAR REINFORCEMENT</u>	
	Design shear force, V_{Ed} = 205 kN	
6.2.3	Concrete strut capacity	
	$V_{Rd,max} = 0.36 b_w d f_{ck} (1 - f_{ck}/250) / (\cot\theta + \tan\theta)$	
	$= \dfrac{(0.36 \times 400 \times 623 \times 35\,(1 - 35/250))}{(\cot\theta + \tan\theta)}$	
	= 937 kN ⇨ θ = 22 ^{o} $\cot\theta$ = 2.5	
	= 1349 kN ⇨ θ = 45 ^{o} $\cot\theta$ = 1.0	
	$V_{Ed} < V_{Rd,max}$ $\cot\theta = 2.5$	
	$V_{Ed} < V_{Rd,max}$ $\cot\theta = 1.0$	
	⇨ Therefore angle $\theta < 22^o$	

Ref.

$\theta = 0.5\sin^{-1}[V_{Ed} / 0.18 b_w d f_{ck}(1 - f_{ck}/250)]$

$= 0.5\sin^{-1}\left\{\dfrac{205 \times 10^3}{0.18 \times 400 \times 623 \times 35(1 - 35/250)}\right\}$

$= 0.5\sin^{-1}\{0.15\} = 4.4^o$

Use : $\theta = 22.0^o$ $\tan\theta = 0.40$ $\cot\theta = 2.5$

Shear links

$A_{sw} / s = V_{Ed} / 0.78 f_{yk} d \cot\theta$

$= 204.6 \times 10^3 / (0.78 \times 500 \times 623 \times 2.5)$

$= 0.341$

Try link : H10 $A_{sw} = 157$ mm^2

Spacing, $s = 157 / 0.341 = 461$ mm

9.2.2 (6) Max. spacing, $s_{max} = 0.75d = 0.75 \times 623 = 467$ mm — Use : H10 - 300

9.2.2(5) Minimum links

$A_{sw} / s = 0.08 f_{ck}^{1/2} b_w / f_{yk}$

$= 0.08 \times (35)^{1/2} \times 400 / 500 = 0.38$

Try link : H10 $A_{sw} = 157$ mm^2

Spacing, $s = 157 / 0.38 = 415$ mm — Use : H10 - 300

$\leq 0.75d = 467$ mm

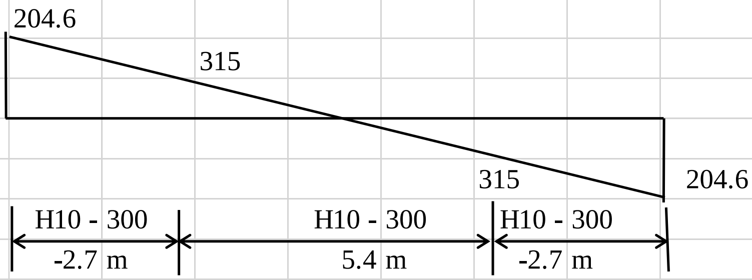
Shear resistance of minimum links

$V_{min} = (A_{sw}/s)(0.78 d f_{yk} \cot\theta)$

$= (157 / 300) \times (0.78 \times 623 \times 500 \times 2.5)$

$= 315$ kN

Links arrangement



6.2.3(7) **Additional longitudinal reinforcement**

Additional tensile force,

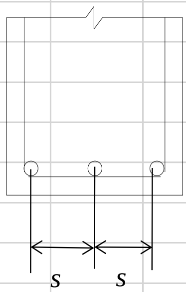
$\Delta F_{td} = 0.5 V_{Ed} \cot\theta \leq M_{E\,d,max}/z$

$= 0.5 \times 205 \times 2.48 = 253$ kN

$M_{E\,d,max}/z = 511.5 \times 10^6 / 591.4 = 865$ kN

$A_{s\,req.} = \Delta F_{td} / 0.87 f_{yk}$ — To be added to the A_s near support (after curtailment)

$= 253 \times 10^3 / (0.87 \times 500) = 582$ mm^2

Ref.	Calculations	Output
7.4	**DEFLECTION**	
	Percentage of required tension reinforcement,	
	ρ = $A_{s,req}$ / bd = 1989 / 400x 623 = 0.008	
	Reference reinforcement ratio,	
	ρ_o = $(f_{ck})^{1/2}$ x 10^{-3} = $(35)^{1/2}$x10^{-3} = 0.0059	
	Percentage of required compression reinforcement,	
	ρ' = $A_{s',req}$ / bd = 0 / 400 x 623 = 0.000	
Table 7.4N	Factor for structural system, K = 1.0	
	ρ > ρ_o	
	$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_o}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}\right]$$	
	= 1.0 (11 + 6.57 + 0.00) = 17.6	
	Modification factor for span less than 7 m	
	= 7/span = 7/10 = 0.70	
	Modification factor for steel area provided,	
	= $A_{s,prov}/A_{s,req}$ = 2768 / 1989 = 1.39 ≤ 1.5	
	Therefore allowable span-effective depth ratio,	
	$(l/d)_{allowble}$ = 17.57 x 0.70 x 1.39 = 17.1	
	Actual span-effective depth	
	$(l/d)_{actual}$ = 10000 / 622.5 = 16.1 < $(l/d)_{allowble}$	Ok !
7.3	**CRACKING**	
Table 7.1N	Limiting crack width, w_{max} = 0.3 mm	
	Steel stress,	
	$$f_s = \frac{f_{yk}}{1.15} \text{ x } \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)}\frac{1}{\delta}$$	
	= (500 / 1.15) x [(15.9 + (0.3x 13.0)) / 40.9] x 1.0	
	= 242 N/mm^2	
Table 7.4N	Max. allowable bar spacing = 200 mm	
	Bar spacing, s = [400 - 2(55) - 2(10) - (25)] / 2	
	= 122.5 mm < 200 mm	Ok !

Summary

3H 16

2H 25   4H 16   2H 25

4H 25

H10- 300   H10- 300   H10- 300

-2.7 m   5.4 m   -2.7 m

Note:

$A_{s\ req}$. near support

= 0.25 x1989 = 497 mm^2

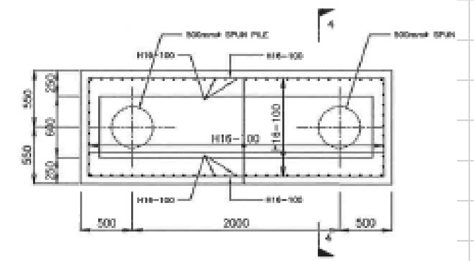
Add. A_s = 582 mm^2

Total $A_{s\ req.}$ = 1079 mm^2

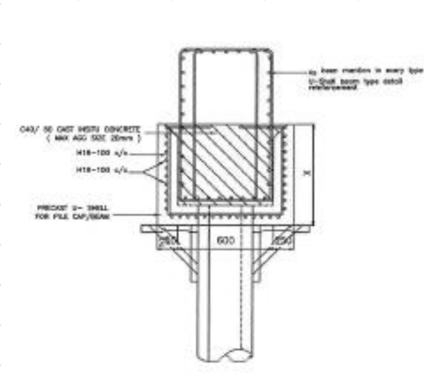
2H 25 (982 mm^2)

Ref.	F : DESIGN OF PRECAST U-SHELL

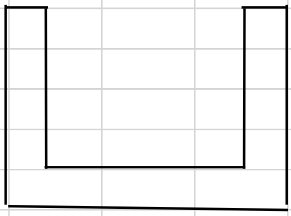
Data :

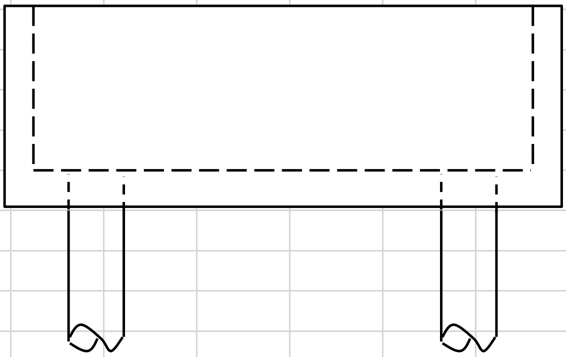


PRECAST U-SHELL BEAM (US1/US2)



ASSEMBLE DETAIL SECTION 4-4





Dimension

b_1 =	250	mm	h_1 =	500	mm	L_1 =	2000	mm
b_2 =	600	mm	h_2 =	250	mm	L_2 =	500	mm
b =	1100	mm	h =	750	mm	L =	3000	mm

Exposure
class = XS3

Limiting crack width,
w_k = 0.3 mm

Materials :

f_{ck} =	35	N/mm^2
f_{yk} =	500	N/mm^2
γ_c =	25	kN/m^3
ϕ_{bar} =	16	mm

Durability & Bond

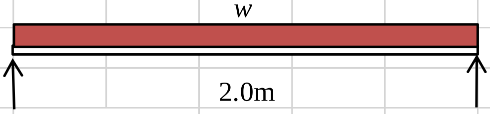
$c_{min,\,b}$ =	16	mm
$c_{min,\,dur}$ =	45	mm
Δc_{dev} =	10	mm

$c_{nom} = c_{min} + \Delta c_{dev}$ = 55 mm

Ref.	Calculations	

LOADING

b x h : 1000 x 250



Self-weight = 1.00 x 0.25 x 25 = 12.50 kN/m

SDL = 1.00 x 0.50 x 25 = 12.50 kN/m

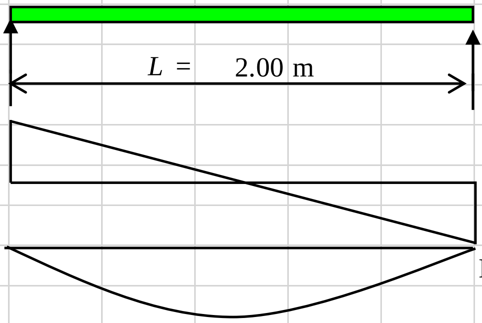
Permanent load, g_k = 25.00 kN/m

Variable load, q_k = 0.00 kN/m

Design load, $w = 1.35g_k + 1.5q_k$ = 33.75 kN/m

Quasi-permanent action, $n_{qp} = 1.0g_k + 0.3q_k$ = 25.00 kN/m

ANALYSIS



Shear Force,

$V = w_d L/2$

= 33.8 kN/m

Bending Moment,

$M = w_d L^2/8$

= 16.9 kNm/m

6.1 **MAIN REINFORCEMENT**

$d = h - C_{nom} - 0.5\phi_{bar}$ = 187 mm

Design bending moment, M_{Ed} = 17 kNm/m

$K = M / f_{ck}bd^2$

= 17 x 10^6 / (35 x 1000 x 187^2) = 0.014

Redistribution = 0 % Redistribution ratio, δ = 1.0

$K_{bal} = 0.454(\delta - k_1)/k_2 - 0.182[(\delta - k_1)/k_2]^2$

= $0.363 (\delta - k_1) - 0.116 (\delta - k_1)^2$ = 0.167

Using : EC2

k_1 = 0.44

k_2 = 1.25

==> $K < K_{bal}$

Compression reinforcement is not required

$z = d [0.5 + \sqrt{0.25 - K/1.134}]$ = 0.99 d

$A_s = M / 0.87 f_{yk} z$

= 17 x 10^6 / (0.87 x 500 x 0.95 x 187)

= 218 mm^2/m

Use : Main bar

H16- 100

(2011 mm^2/m)

9.2.1.1 Minimum and maximum reinforcement area,

$A_{s,min} = 0.26(f_{ctm}/f_{yk}) bd$

= 0.26 (3.21 / 500) bd = 0.0017 bd

= 0.0017 x 1000 x187.0 = 312 mm^2/m

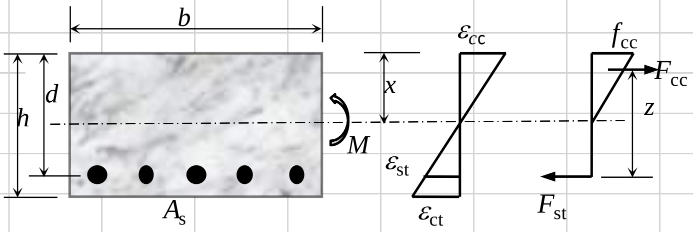
$A_{s,max}$ = 0.04A_c = 0.04 x1000 x 250= 10000 mm^2/m

Sec. bar

H16- 100

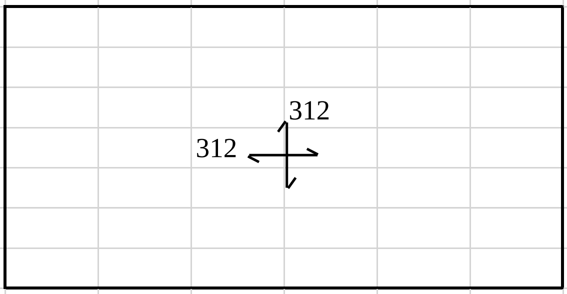
(2011 mm^2/m)

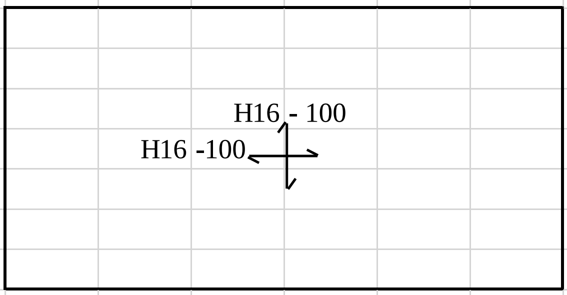
Ref.		
	SHEAR	
	Design shear force, V_{Ed} = 33.8 kN	
6.2.2	Design shear resistance,	
	$V_{Rd,c} = [\ 0.12\, k\, (100\rho_1 f_{ck})^{1/3}\ ]\ bd$	
	$k = 1 + (200/d)^{1/2} \leq 2.0$	
	$= 1+(200/\ 187)^{1/2} = 2.03$	
	$\rho_1 = A_{sl}/bd \leq 0.02$	
	= 2011 / (1000 x 187) = 0.011	
	$V_{Rd,c}$ = 0.12 x 2.0 x (100 x 0.011 x $16)^{1/3}$ x 1000 x 187	
	= 115862 N = 115.9 kN	
	$V_{min} = [\ 0.035 k^{3/2} f_{ck}^{1/2}\ ]\ bd$	
	= 0.035 x $2.0^{3/2}$ x $16^{1/2}$ x 1000 x 187	
	= 74048 N = 74.0 kN	
	So, $V_{Rd,c}$ = 115.9 kN > V_{Ed}	Ok !
7.4	**DEFLECTION**	
	$\rho = A_{s,req} / bd$ = 218 / 1000 x 187 = 0.0012	
	$\rho_o = (f_{ck})^{1/2} \times 10^{-3} = (16)^{1/2} \times 10^{-3}$ = 0.0040	
Table 7.4N	Factor for structural system, K = 1.0	
	$\rho < \rho_o$ Use equation (1)	
	$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_o}{\rho} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_o}{\rho} - 1\right)^{3/2}\right]$ (1)	
	= 1.0 (11 + 20.6 + 48.35) = 79.9	
	Modification factor for s| span less than 7 m = 1.00	
	Modification factor for steel area provided,	
	$= A_{s,prov}/A_{s,req}$ = 2011 / 218 = 9.21 > 1.5	
	Therefore allowable span-effective depth ratio,	
	$(l/d)_{allowble}$ = 26.0 x 1.00 x 1.50 = 39.00	
	Actual span-effective depth	
	$(l/d)_{actual}$ = 2000 / 187 = 10.7 < $(l/d)_{allowble}$	Ok !
	CRACKING	
	Limiting crack width, w_{max} = 0.3 mm	
	Steel stress,	
	$f_s = \frac{f_{yk}}{1.15} \times \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)}\frac{1}{\delta}\frac{A_{s,req}}{A_{s,prov}}$	
	= (500 / 1.15) x [(25.0 + (0.3x 0.0)) / 33.8] x 0.1	
	= 40 N/mm^2	
	Max. allowable bar spacing = 300 mm	
	Max bar spacing, s = 100 mm < 300 mm	Ok !

Ref.	Calculations	Output
7.3.3		
	Section Strain Stress/Force	
	Reinforcement H16 - 100	
	Area of reinforcement, A_s = 2011 mm^2/m	
	Effective depth, $d = h - c - \phi/2$	
	= 250 - 55 - 16/2 = 187 mm	
	Modular ratio, $\alpha_e = E_s/E_{cm}$ = 200 / 21 = 9.7	Note:
	Steel ratio, $\rho = A_s/bd$ = 2011 / (1000 x 187) = 0.011	Allow for creep ? yes
	Depth of neutral axis,	E_{cm} = 0.5 x 21
	$x = \{ -\alpha_e . \rho + [\alpha_e . \rho\ (2 + \alpha_e . \rho)]^{1/2} \}\ d$	= 10 kN/mm^2
	= { (19.4 x 0.011) +	α_e = 200 / 10
	[(19.4x 0.011) x (2 + 19.4 x 0.011)]$^{1/2}$}d	= 19.4
	= 0.47 d = 0.47 x 187 = 88.0 mm	
	Lever arm, z = 187 - (88.0 / 3) = 158 mm	
	Steel tensile stress at quasi-permanent load	
	$\sigma_s = M_{qp} / (A_s . z)$	
	= 12.5 x 10^6 / (2011 x 158) = 39 N/mm^2	
Eq. 7.11	Maximum crack spacing,	
	$s_{r,max} = 3.4c + 0.2125k_1\phi / \rho_{p,eff}$	
	c = 55 mm Cover to reinforcement	
	k_1 = 0.8 Bond coefficient	
	ϕ = 16 mm Bar diameter	
	$\rho_{p,eff}$ = Effective reinforcement ratio	
	= $A_s / A_{c,eff} = A_s / bh_{eff}$	
	Depth of effective tension area, h_{eff} = The lesser of,	
	$2.5(h - d)$ = 2.5 (250 - 187) = 157.5 mm	
	$(h - x) / 3$ = (250 - 88.0) / 3 = 54.0 mm	
	$h / 2$ = 250 / 2 = 125.0 mm	
	$\rho_{p,eff}$ = 2011 / 1000 x 54.0 = 0.0372	
	$s_{r,max}$ = (3.4 x 55) + (0.2125 x 0.8 x 16 / 0.0372)	
	= 260 mm	
	Bar spacing = 100 mm	
	< 5(c + ϕ/2) = 5(55 + 16 /2)= 315 mm	
	Upper limit of $s_{r,max} = 1.3(h - x)$ = 1.3 (250 - 88.0)	
	= 211 mm does not apply	

Ref.	STRUCTURAL LAYOUTS	
Eq. 7.9	Surface strain	
	$(\varepsilon_{sm} - \varepsilon_{cm}) = [\sigma_s - k_t(f_{ct,eff} / \rho_{p,eff})(1 + \alpha_e \rho_{p,eff})] / E_s$	
	$\geq 0.6\sigma_s / E_s$	
	σ_s = 39 N/mm^2 Steel stress	
	k_t = 0.4 Duration of load factor	
	$f_{ct,eff}$ = 3.21 N/mm^2 Tensile strength of concrete	
	$\rho_{p,eff}$ = 0.0372 Effective reinforcement ratio	
	α_e = 9.7 Modular ratio	
	$(\varepsilon_{sm} - \varepsilon_{cm})$	
	$= \dfrac{39 - [0.4 \ (3.21 / 0.037) (1 + 9.7 \times 0.037)]}{200 / \times 10^3}$	
	$= (39 - 47) / 200 \times 10^3 = -0.00004$	
	$\geq 0.6\sigma_s/E_s$	
	$= 0.6 \times 39 / 200 \times 10^3 = 0.00012$	
	Crack width,	
	$w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm})$	
	$= 260 \times 0.00012 = 0.03$ mm < 0.30 mm	Ok !

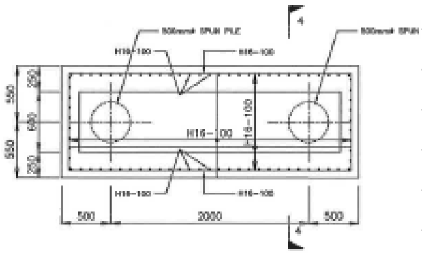
DETAILING



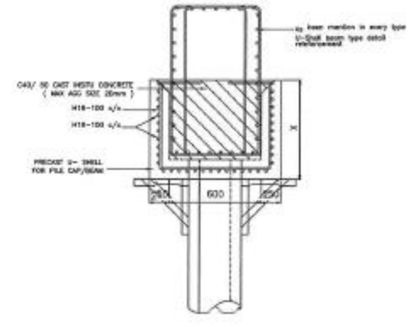


Ref.	F : DESIGN OF PILE CAP

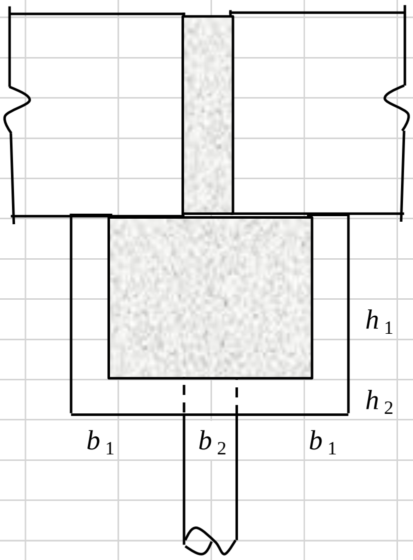
Data :

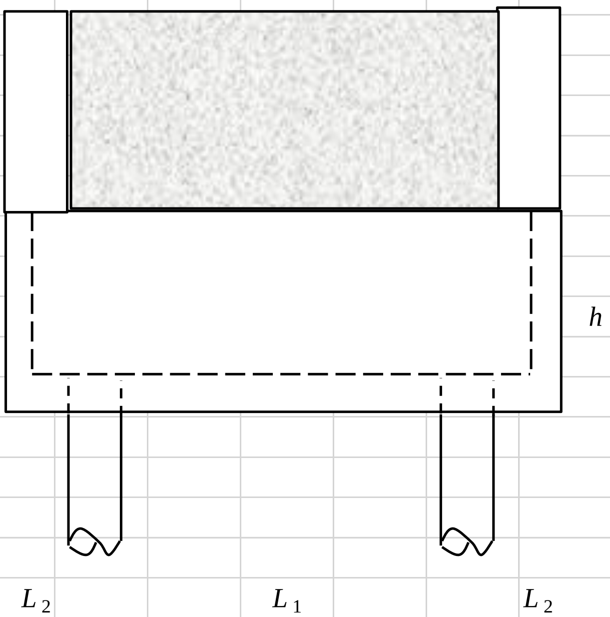


PRECAST U-SHELL BEAM (US1/US2)



ASSEMBLE DETAIL SECTION 4-4





Dimension

b_1 =	250	mm	h_1 =	500	mm	L_1 =	2000	mm
b_2 =	600	mm	h_2 =	250	mm	L_2 =	500	mm
b =	1100	mm	h =	750	mm	L =	3000	mm

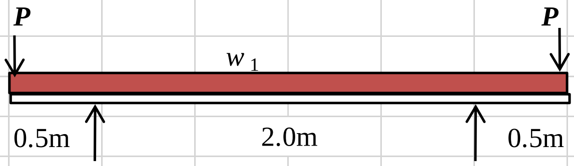
Exposure		**Limiting crack width,**		
class =	XS3	w_k =	0.3	mm

Materials : / **Durability & Bond**

f_{ck} =	35	N/mm^2	$c_{min,b}$ =	20	mm
f_{yk} =	500	N/mm^2	$c_{min,dur}$ =	45	mm
γ_c =	25	kN/m^3	Δc_{dev} =	10	mm
ϕ_{bar} =	20	mm	$c_{nom} = c_{min} + \Delta c_{dev}$ =	55	mm
ϕ_{bar2} =	16	mm	ϕ_{link} =	10	mm

Ref.	Calculations	

LOADING



b x h :
600 x 500

w

Self-weight = 1.10 x 0.75 x 25 = 20.63 kN/m

SDL = 1.10 x 0.73 x 25 = 19.94 kN/m

Permanent load, g_k = 40.56 kN/m

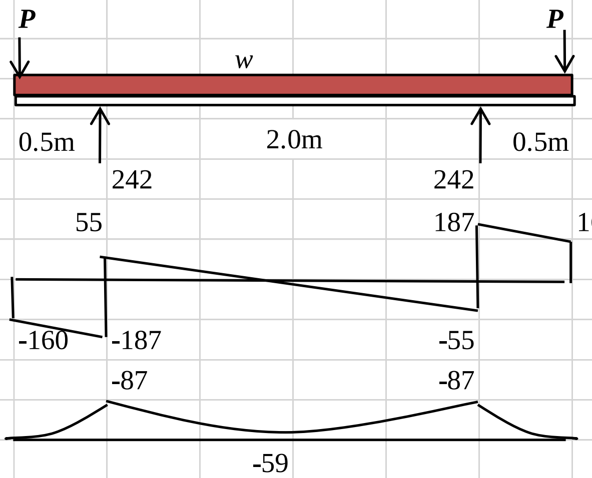
Variable load, q_k = 0.00 kN/m

Design load, w = $1.35g_k + 1.5q_k$ = 54.76 kN/m

P

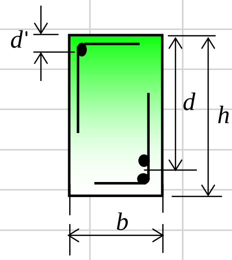
Design load, P = $1.35g_k + 1.5q_k$ = 160.00 kN

ANALYSIS



6.1

MAIN REINFORCEMENT



Effective depth,

$d = h - C_{nom} - \phi_{link} - \phi_{bar} - 0.5\phi_{bar}$ = 405 mm

$d' = C_{nom} + \phi_{link} + \phi_{bar}/2$ = 73 mm

Design bending moment, M_{Ed} = 87 kNm

$K = M / f_{ck}bd^2$

$= 87 \times 10^6 / (35 \times 600 \times 405^2) = 0.025$

Redistribution = 0 % Redistribution ratio, δ = 1.0

Using : EC2

$K_{bal} = 0.454(\delta - k_1)/k_2 - 0.182[(\delta - k_1)/k_2]^2$

k_1 = 0.44

$= 0.363(\delta - k_1) - 0.116(\delta - k_1)^2 = 0.167$

k_2 = 1.25

==> $K < K_{bal}$

Compression reinforcement is not required

Ref.		
	$z = d\,[\,0.5 + \sqrt{0.25 - K/1.134}\,] = 0.98\,d$	
	$A_s = M / 0.87 f_{yk} z$	
	$= 87 \times 10^6 / (0.87 \times 500 \times 0.95 \times 405)$	
	$= 520\ mm^2$	
	$K < K_{bal}$	
	Compression reinforcement is required	
	$z = d\,[\,0.5 + \sqrt{0.25 - K_{bal}/1.134}\,] = 0.82\,d$	
	$x = (d - z) / 0.4 = 182\ mm$	
	$d'/x = 73 / 182 = 0.40 > 0.38$	
	Compression steel is not yield $f_{sc} < 0.87 f_{yk}$	
	$A_s' = (K - K_{bal}) f_{ck} b d^2 / 0.87 f_{yk}(d - d')$	
	$= \dfrac{(0.025 \quad -0.17) \times (35 \times 600 \times 405^2)}{0.87 \times 500 \times (405 - 73)}$	
	$= -3377\ mm^2$	
	$A_s = K_{bal} f_{ck} b d^2 / 0.87 f_{yk} z_{bal} + A_{s'}$	
	$= \dfrac{0.167 \times (35 \times 600 \times 405^2)}{0.87 \times 500 \times 0.82 \times 405}$ -+ 3377	Use : Comp. bar
		4H 20
	$= 598\ mm^2$	(1257 mm^2)
	A_s = 520 mm^2 A_s' = 0 mm^2	Tension bar
		0H 16
9.2.1.1	Minimum and maximum reinforcement area,	4H 20
	$A_{s,min} = 0.26 (f_{ctm}/f_{yk})\, bd$	(1257 mm^2)
	$= 0.26\ (3.21 / 500)\ bd = 0.0017\ bd$	
	$= 0.0017 \times 600 \times 405.0 = 406\ mm^2$	
	$A_{s,max} = 0.04 A_c = 0.04 \times 600 \times 500$	
	$= 12000\ mm^2$	
	<u>SHEAR REINFORCEMENT</u>	
	Design shear force, V_{Ed} = 187 kN	
6.2.3	Concrete strut capacity	
	$V_{Rd,\,max} = 0.36 b_w d f_{ck}(1 - f_{ck}/250) / (\cot\theta + \tan\theta)$	
	$= \dfrac{(0.36 \times 600 \times 405 \times 35\,(1 - 35/250)\,)}{(\cot\theta + \tan\theta)}$	
	$= 915\ kN \Rightarrow \theta = 22^o \quad \cot\theta = 2.5$	
	$= 1317\ kN \Rightarrow \theta = 45^o \quad \cot\theta = 1.0$	
	$V_{Ed} < V_{Rd,\,max}\ \cot\theta = 2.5$	
	$V_{Ed} < V_{Rd,\,max}\ \cot\theta = 1.0$	
	$\Rightarrow$ Therefore angle $\theta < 22^o$	

Ref.	F FIRST FLOOR BEAM	

$\theta = 0.5\sin^{-1}[V_{Ed} / 0.18 b_w d f_{ck}(1 - f_{ck}/250)]$

$= 0.5\sin^{-1}\left\{\dfrac{187 \times 10^3}{0.18 \times 600 \times 405 \times 35(1 - 35/250)}\right\}$

$= 0.5\sin^{-1}\{0.14\} = 4.1^o$

Use : $\theta = 22.0^o$ $\quad \tan\theta = 0.40 \quad \cot\theta = 2.5$

Shear links

$A_{sw} / s = V_{Ed} / 0.78 f_{yk} d \cot\theta$

$= 187.0 \times 10^3 / (0.78 \times 500 \times 405 \times 2.5)$

$= 0.478$

Try link : H10 $\quad A_{sw} = 157$ mm^2

Spacing, $s = 157 / 0.478 = 328$ mm

9.2.2 (6) Max. spacing, $s_{max} = 0.75d = 0.75 \times 405 = 304$ mm — Use : H10 - 300

9.2.2(5) Minimum links

$A_{sw} / s = 0.08 f_{ck}^{1/2} b_w / f_{yk}$

$= 0.08 \times (35)^{1/2} \times 600 / 500 = 0.57$

Try link : H10 $\quad A_{sw} = 157$ mm^2

Spacing, $s = 157 / 0.57 = 277$ mm — Use : H10 - 300

$\leq 0.75d = 304$ mm

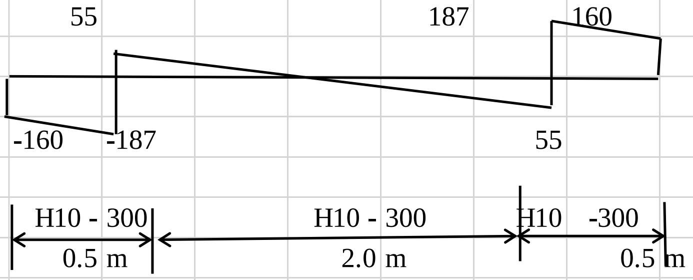
Shear resistance of minimum links

$V_{min} = (A_{sw}/s)(0.78 d f_{yk} \cot\theta)$

$= (157 / 300) \times (0.78 \times 405 \times 500 \times 2.5)$

$= 205$ kN

Links arrangement



6.2.3(7) **Additional longitudinal reinforcement**

Additional tensile force,

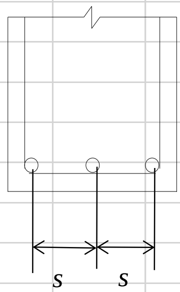
$\Delta F_{td} = 0.5 V_{Ed} \cot\theta \leq M_{E\,d,max}/z$

$= 0.5 \times 187 \times 2.48 = 231$ kN

$M_{E\,d,max}/z = 87.0 \times 10^6 / 384.8 = 226$ kN

$A_{s\,req.} = \Delta F_{td} / 0.87 f_{yk}$ — To be added to the A_s near support (after curtailment)

$= 231 \times 10^3 / (0.87 \times 500) = 532$ mm^2

Ref.	Calculations	Output
7.4	**DEFLECTION**	
	Percentage of required tension reinforcement,	
	ρ = $A_{s,req}$ / bd = 520 / 600x 405 = 0.002	
	Reference reinforcement ratio,	
	ρ_o = $(f_{ck})^{1/2}$ x 10^{-3} = $(35)^{1/2}$x10^{-3} = 0.0059	
	Percentage of required compression reinforcement,	
	ρ' = $A_{s}{}'_{,req}$ / bd = 0 / 600 x 405 = 0.000	
Table 7.4N	Factor for structural system, K = 1.0	
	ρ < ρ_o	
	$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_o}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho}}\right]$	
	= 1.0 (11 + 24.54 + 0.00) = 35.5	
	Modification factor for span less than 7 m	
	= 1.0 1.00	
	Modification factor for steel area provided,	
	= $A_{s,prov}/A_{s,req}$ = 1257 / 520 = 2.42 ≤ 1.5	
	Therefore allowable span-effective depth ratio,	
	$(l/d)_{allowble}$ = 35.54 x 1.00 x 1.50 = 53.3	
	Actual span-effective depth	
	$(l/d)_{actual}$ = 2000 / 405.0 = 4.9 < $(l/d)_{allowble}$	Ok !
7.3	**CRACKING**	
Table 7.1N	Limiting crack width, w_{max} = 0.3 mm	
	Steel stress,	
	$f_s = \frac{f_{yk}}{1.15} \times \frac{G_k + 0.3Q_k}{(1.35G_k + 1.5Q_k)}\frac{1}{\delta}\,\frac{A_{s,req}}{A_{s,prov}}$	
	= (1100 / 1.15) x [(40.6 + (0.3x 0.0)) / 54.8] x 0.4	
	= 337 N/mm^2	
Table 7.4N	Max. allowable bar spacing = 150 mm	
	Bar spacing, s = [600 - 2(55) - 2(10) - (20)] / 3	
	= 150 mm > 150 mm	Fail !

Summary

4H 20	4H 20	4H 20
2H 20	0H 16	2H 20
H10- 300	H10- 300	H10- 300
0.5 m	2.0 m	0.5 m

Note:

$A_{s\ req}$. near support

= 0.25 x520 = 130 mm^2

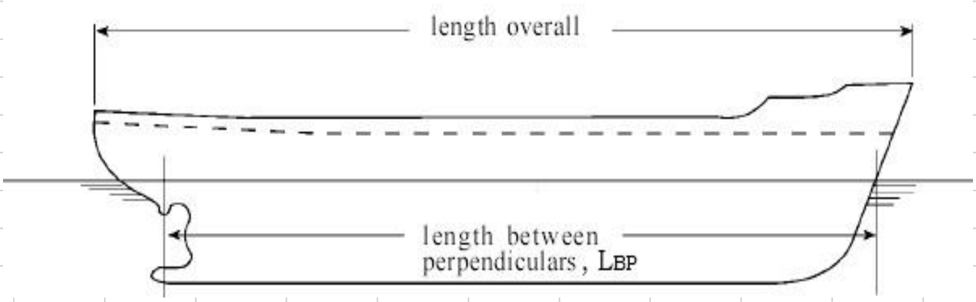
Add. A_s = 532 mm^2

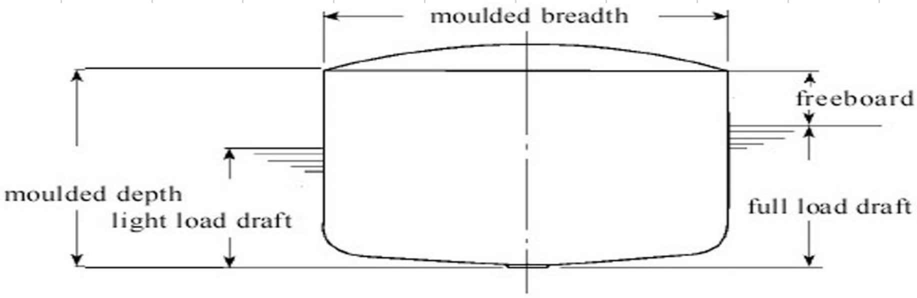
Total $A_{s\ req}$. = 662 mm^2

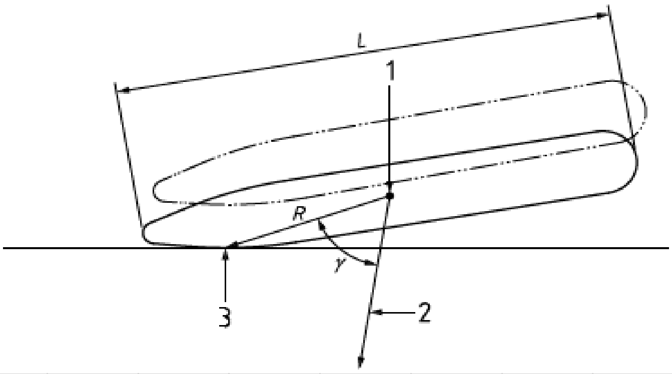
2H 20 (628 mm^2)

H : CALCULATION OF BERTHING FORCE

BS 6349-1-1	**Specification**				
Fig. D1-D10	Type of Vessel	=	General Cargo		
	Dead Weight Tonnage (DWT)	=	20000	t	
	Length between perpendiculars, L_{BP}	=	150	m	
	Moulded breadth , B	=	25	m	
	Full load draft , D_v	=	10	m	
Table D.1	Displacement Tonnage, $M_d = 2.1\ (DWT)^n$	=	26369	t	n = 0.953
BS 6349-4	Berthing Velocity, V_s	=	0.3	m/s	
Figure 9					







1 = Centre of mass
2 = Velocity vector
3 = Point of contact

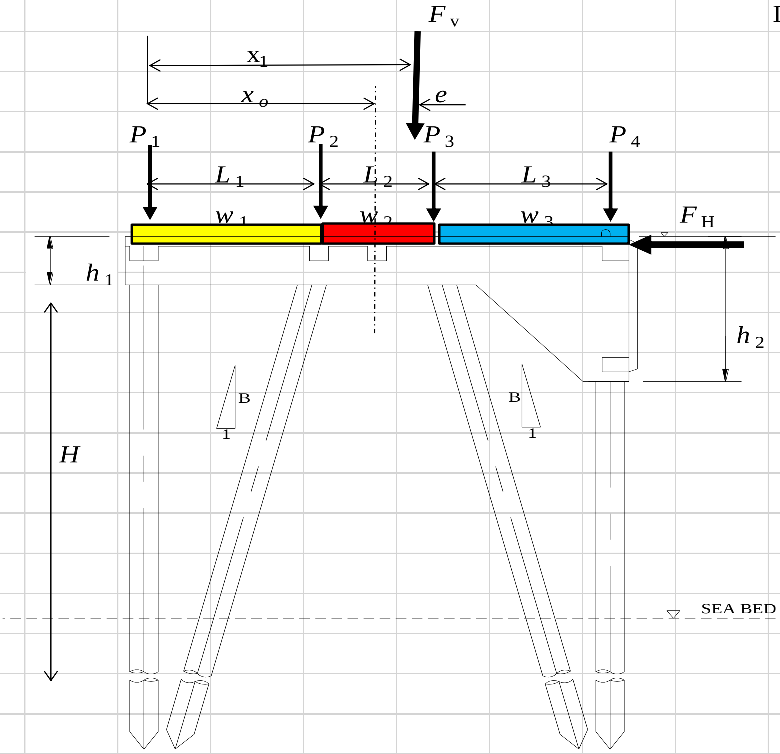
Distance of the point of contact from centre of mass

BS 6349-4	$R = [(0.5L_{BP})^2 + (0.5B)^2]^{1/2}$	=	76.0	m
5.2.5	Angle between the line, γ	=	15	^{o}

Ref.	CALCULATION OF BERTHING FORCE
	Characteristic Berthing Energy
BS 6349-4	$E_c = 0.5\ C_m\ M_d\ V_b^2\ C_E\ C_s\ C_c$
5.2	
5.2.4	C_m = Hydrodynamic mass coefficient
	$= 1 + 2D_v/B$ = 1 + (2 x 10.0/ 25.0)
	= 1.80
	M_d = Displacement of vessel
	= 26369 t
	V_b = Velocity of vessel
	= 0.3 m/s
5.2.5	C_E = Eccentricity coefficient
	$= (K^2 + R^2\cos^2\gamma) / (K^2 + R^2)$
	$K = (0.19C_b + 0.11)\ L_{BP}$
Table D.2	C_b = 0.85
BS 6349-1-1	= (0.19 x 0.85 + 0.11) x 150
	= 40.7 m
	γ = 15 ^{o}
	R = 76.0 m
	= (1659+ 5781 x 0.93) / (1659+ 5781)
BS 6349-4	= 0.95
5.2.6	C_s = Softness coefficient = 1.0
5.2.7	C_c = Berth configuration coeff. = 0.9
	E_c = 0.5 x 1.80 x 26369 x 0.30^2 x 0.95 x 1.0 x 0.9
	= 1822 kNm

I : DESIGN OF PILE FOUNDATION

Vertical and Inclined Piles



Dimension

L_1 =	5.25	m
L_2 =	0.00	m
L_3 =	3.75	m
L =	9.00	m
h_1 =	0.00	m
h_2 =	0.00	m
h =	0.00	m
x_o =	4.5	m
H =	4.8	m

Action

P_1 =	0.0	kN
P_2 =	0.0	kN
P_3 =	800.0	kN
P_4 =	0.0	kN
w_1 =	0.0	kN/m
w_2 =	0.0	kN/m
w_3 =	0.0	kN/m
F_h =	100	kN

Pile inclination

$1 : B$ = 1 : 4

N_1 N_2 N_3 N_4

$F_v = P_1 + P_2 + P_3 + P_4 + (w_1L_1) + (W_2L_2) + (w_3L_3)$ = 800 kN

$F_{v.}\ x_v = P_1(0) + P_2(L_1) + P_3(L_1 + L_{2)} + P_4(L_1 + L_2 + L_3) + (w\,1L\,1)(0.5L\,1)$

$+ (W_2L_2)(L_1 + 0.5L_2) + (w_3L_3)(L_1 + L_2 + 0.5L_3)$ = 4200 kN

x_v = 4200 / 800 = 5.25 m

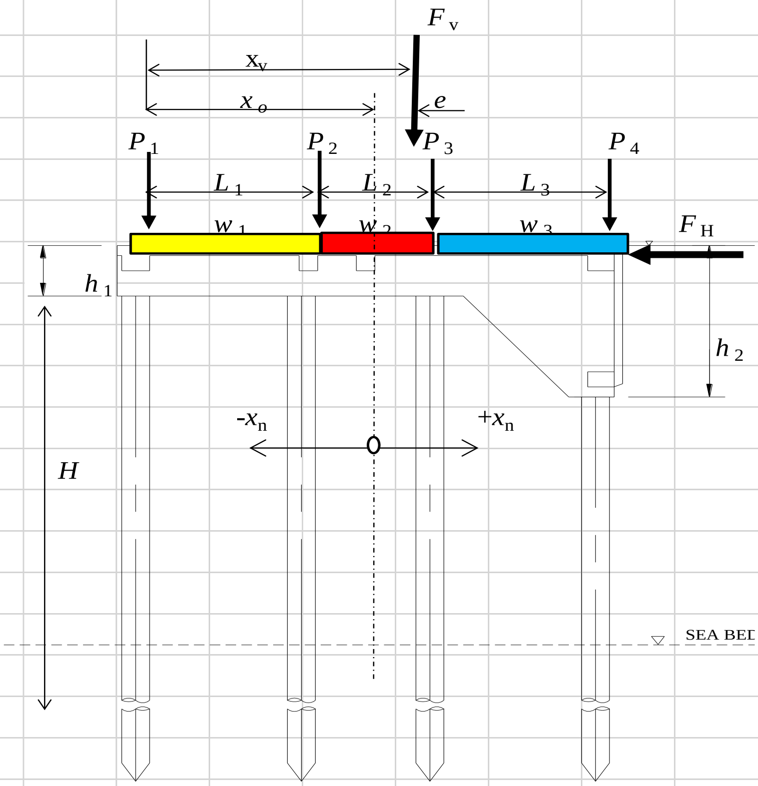
$M = F_v.\,e + F_h.\,H$ = 800 x (5.25 - 4.50) - (100 x 0.00) = 600 kNm

Pile No.	N_1	N_2	N_3	N_4	Totals
Σ_1	+ 1	+ 0.913	+ 0.913	+ 1	**+ 3.826**
Σ_5	0	+ 0.057	+ 0.057	0	**+ 0.114**
x (m)	0	+ 4.5	+ 4.5	+ 9.0	
X (m)	-4.5	0	0	+ 4.5	
$\Sigma_6 = I$	+ 20.25	0.00	0.00	+ 20.25	**+ 40.5**
k_p	+ 1	+ 0.941	+ 0.941	+ 1	
k_w	+ 0.261	+ 0.261	+ 0.261	+ 0.261	
k_h	0	+ 2.19	-2.19	0	
k_m	-0.111	0	0	+ 0.111	
Axial load	142.4 kN	402.9 kN	-9.4 kN	275.8kN	**Max. Load**
	$N_x = k_p(k_wF_v + k_hF_h + k_mM)$				**402.9 kN**

Shear force, $V = F_h/n$ = 100 / 4 = **25 kN** | Moment, $M = VH/2$ = 60 kNm

Ref.	DESIGN OF PILE FOUNDATION

Vertical Piles



Dimension

L_1	=	5.25	m
L_2	=	0.00	m
L_3	=	3.75	m
L	=	9.00	m
h_1	=	0.00	m
h_2	=	0.00	m
h	=	0.00	m
x_o	=	4.50	m
H	=	4.80	m

Action

P_1	=	0.0	kN
P_2	=	0.0	kN
P_3	=	800.0	kN
P_4	=	0.0	kN
w_1	=	0.0	kN/m
w_2	=	0.0	kN/m
w_3	=	0.0	kN/m
F_h	=	100	kN

$F_v = P_1 + P_2 + P_3 + P_4 + (w_1 L_1) + (W_2 L_2) + (w_3 L_3) =$ 800 kN

$F_{v.}\, x_v = P_1(0) + P_2(L_1) + P_3(L_1 + L_{2)} + P_4(L_1 + L_2 + L_3)$

$+\ (w_1 L_1)(0.5 L_1) + (W_2 L_2)(L_1 + 0.5 L_2) + (w_3 L_3)(L_1 + L_2 + 0.5 L_3)$

$=$ 4200 kN

$x_v =$ 4200 / 800 = 5.25 m

$M = F_{v}. e + F_h . H =$ 800 x (5.25 - 4.50) - (100 x 4.80) = 120 kNm

Calculation for loads on piles

Pile No.	n	x (m)	x^2 (m^2)	$I = nx^2$	$k_w =$ $(1/\Sigma n)$	$k_m =$ (x/nI)	Axial Load $(k_w F_v + k_m M)$ (kN)	Shear $V =$ $F_h/\Sigma n$ (kN)	BM M = $VH/2$ (kNm)
N_1	1	-4.5	20.25	20.25	0.25	-0.10	188		
N_2	1	-1.5	2.25	2.25	0.25	-0.03	196		
N_3	1	+ 1.5	2.25	2.25	0.25	0.03	204	**25**	**60**
N_4	1	+ 4.5	20.25	20.25	0.25	0.10	212		
	4			45	**Max. Load =**		**212 kN**		