POMSSUP
UV DISINFECTION ROOM

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\UVROOM	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	2	(10000)
Member distributed forces	5	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	6	
Total degrees of freedom	12	
Primary load cases	4	(100)
Mass load cases	0	(100)

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	500 x 1000	S1	Not applicable	No	Standard shape

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.0000E-01	2.8610E-02	4.1667E-02	1.0417E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.500	1.000			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	CONCRETE-40	3.1975E+07	0.15	2.4500E+03	1.0000E-05	40000.00

SELF WEIGHT (kN/kg)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
1	0.000	-0.010	0.000

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

1.000 * Load case 1: D
1.300 * Load case 2: L
1.000 * Load case 3: E

Load case 7: 0.9D+E

0.900 * Load case 1: D
1.000 * Load case 3: E

case 8: 1.4D+1.7L+1.7Q

1.400 * Load case 1: D
1.700 * Load case 2: L
1.700 * Load case 4: Q

Load case 9: 0.9D+1.7Q

0.900 * Load case 1: D
1.700 * Load case 4: Q

Load case 10: 1.4D+1.7L+1.4F

1.400 * Load case 1: D
1.700 * Load case 2: L
1.400 * Load case 5: F

Load case 11: 0.9D+1.4F

0.900 * Load case 1: D
1.400 * Load case 5: F

Load case 12: 1.4D+1.7L+E+1.7Q

1.400 * Load case 1: D
1.700 * Load case 2: L
1.000 * Load case 3: E
1.700 * Load case 4: Q

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	E
4	Q
5	F
6	D+1.3L+E
7	0.9D+E
8	1.4D+1.7L+1.7Q
9	0.9D+1.7Q
10	1.4D+1.7L+1.4F
11	0.9D+1.4F
12	1.4D+1.7L+E+1.7Q

MEMBER FORCES AND MOMENTS (kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	62.228*	-51.680	0.000	0.000	0.000	71.318
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
2	10	0.000	158.724*	0.000	0.000	0.000	-121.688
2	10	0.000	-158.724#	0.000	0.000	0.000	-121.688
1	12	62.228	-81.280	0.000	0.000	0.000	135.205*
3	12	62.228	81.280	0.000	0.000	0.000	-135.205#

NODE REACTIONS (kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Nodes

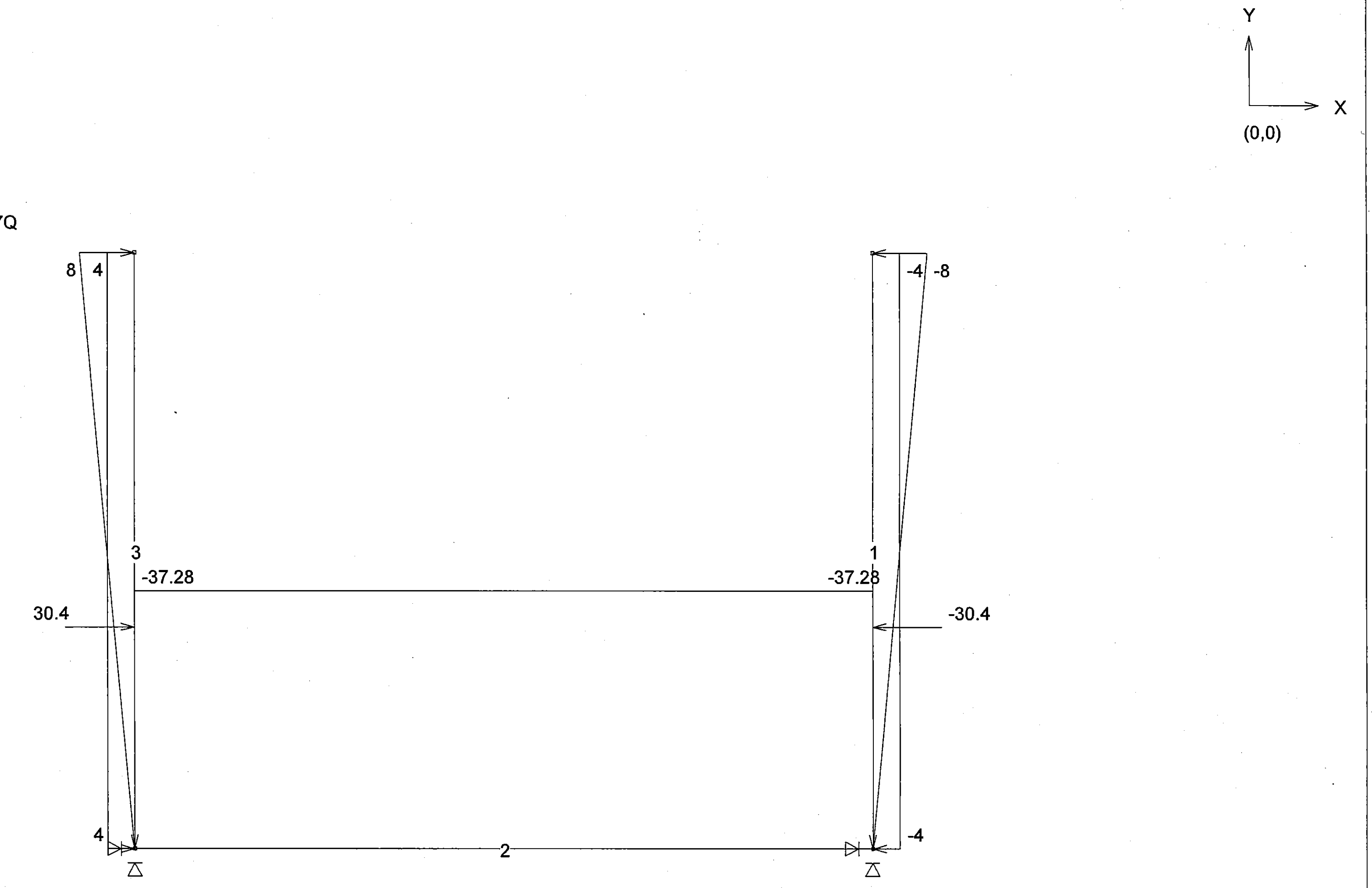
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	81.280*	100.910	0.000	0.000	0.000	-164.862
3	12	-81.280#	100.910	0.000	0.000	0.000	164.862
1	10	0.000	220.952*	0.000	0.000	0.000	-121.688
1	3	29.600	0.000#	0.000	0.000	0.000	-63.887
3	12	-81.280	100.910	0.000	0.000	0.000	164.862*
1	12	81.280	100.910	0.000	0.000	0.000	-164.862#

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	62.228*	-51.680	0.000	0.000	0.000	71.318
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
2	10	0.000	158.724*	0.000	0.000	0.000	-121.688
2	10	0.000	-158.724#	0.000	0.000	0.000	-121.688
1	12	62.228	-81.280	0.000	0.000	0.000	135.205*
3	12	62.228	81.280	0.000	0.000	0.000	-135.205#

1		(SW)	D
3			E
4			Q
5			F
6			D+1.3L+E
7			0.9D+E
8			1.4D+1.7L+1.7Q
9			0.9D+1.7Q
10			1.4D+1.7L+1.4F
11			0.9D+1.4F
12			1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

UV DISINFECTION Model

Job: UVROOM, Designer: FYP, Units: m,kN,MPa, Scale: 1:43, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
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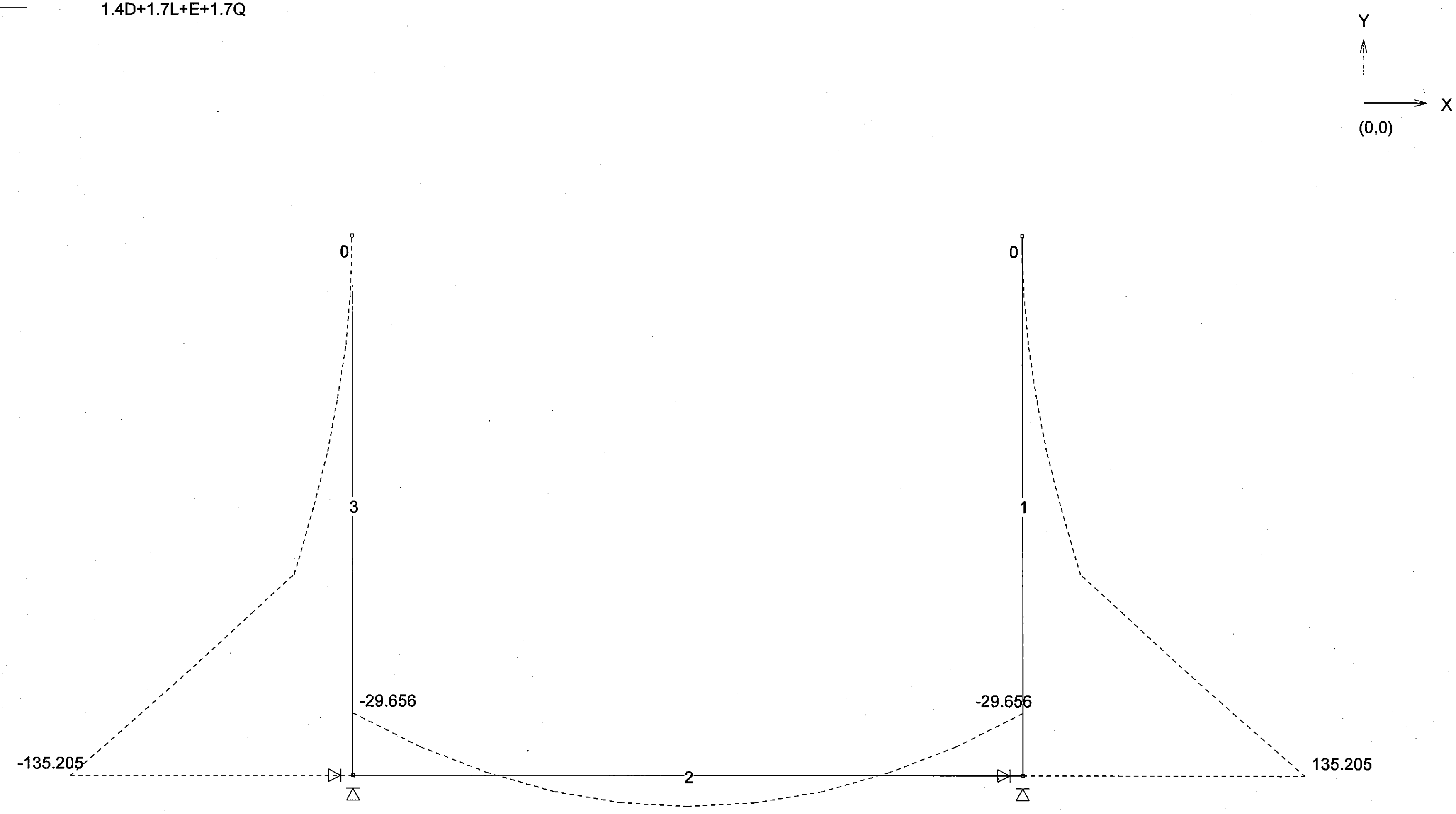
UV DISINFECTION ROOM 8 Nov 2011, 7:21 pm

12 —— 1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

Bending Moment Diagram

Job: UVROOM, Designer: FYP, Units: m,kN,MPa, Scale: 1:43, Axes: XY
Load: 1 Disp: None Moment: 3 Shear: None Axial: None
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J : Sludge Treatment Building

1. Load Breakdown and Calculation
2. Result of Structural Analysis
3. Design of Member
4. Summary

Load Breakdown & Calculation			
Items	Calculation & Description	Result	Unit
1	Ground Floor Loads		
A	DEAD LOADS		
1	Blockwall wt	4.6	kPa
4	Slab wt =	5	kPa
5	Services =	0.5	kPa
6	Superimposed DL	0.5	kPa
		6	kPa
B	LIVE LOADS		
1	Hopper	3	kPa
2	Biological Odour Control Room	260	kN per m^2
3	Polymer Dissolving Tank	300	kN per m^2
4	Hoist Block	65	kN per m^2
5	Hoist Block	30	kN per m^2
6		6	kN per m^2
7		6	kN per m^2
8	Polymer Feeder Pump	4	kN per m^2
9	Air Compressor	13	kN per m^2
10	Air Dryer	4	kN per m^2
11	Portable Water Tank	15	kN per m^2
2	First Floor Loads		
A	DEAD LOADS		
1	Suspended Timber Framed 12 THK Plasterboard Ceiling	0.396	kPa
2	12THK Plasterboard to Both sides of 75THK Timber Wall	0.9912	kPa
3	Fixed Window Glass	0	kPa
4	Slab wt	5	kPa
5	300 x 300 Non-slip Ceramic Tiles	0	kPa
6	Services	0.2	kPa
7	Superimposed DL	0.2	kPa
		6.8	kPa
B	LIVE LOADS		
1	Electrical Room	5	kPa
2	Office	3	kPa
3	General Room	10	kPa
4	Sludge Cake Hopper	62.5	kN
5	Sludge Cake Conveyor	17.5	kN
		70	kN
6	Hoist Block	30	kN
3	Roof Loads (Trafficable Roof)		
A	DEAD LOADS		
1	Colorbond Roof Sheeting	0.05	kPa
2	12THK Plasterboard Ceiling	0.29	kPa
3	Services	0.2	kPa
4	Superimposed DL	0.2	kPa
		0.74	kPa
B	LIVE LOADS	0.25	kPa

Wind Load Calculation on Roof

Items	Calculation & Description			Result	Unit
A	**Wind Load on Roof**				
1	Data				
	Return Period =			50	years
	Wind Velocity =			28	m/s
	Terrain Category =			1	
	Roof Mean Height,z =			15.5	m
	Width of Bldg =			15	m
	Length of Bldg =			20	m
	Roof Pitch, α =			10	Degrees
	Wind Angle, θ =			0	Degrees
2	Design Wind Velocity, V_z				
	V_z =	**1.35V(z/zg)^k**	**Notes for Table 4, PNGS 101-1982: Part 2**		
		k =	0.07 *zg* = 250		
		=		31.11	m/s
3	Dynamic Pressure, qz				
	q_z =	$0.6V_z$^2 x10^-3	... Dynamic Pressure ...	0.58	kPa
4	Design Pressure, P_z				
	p_z =	Cpq_z	... Design pressure...		
	h/d =	Table B2.2	α ≥ 10° h/d ≥ 0.5	1.03	
	h/b =			0.78	
	C_p =	... Slope D ...		-0.9	
	C_p =	... Slope E ...		-0.7	
	p_z =	... Slope D ...		-0.52	kPa
	=	... Slope E ...		-0.41	kPa

5 UDL on roof truss

Truss Spacing No.	Truss Spacing (m)	Truss No.	Tributary Area (m)	Windward (kN/m)			Leeward (kN/m)		
1	0.9				x-Component	y-Component		x-Component	y-Component
2	5	1	3.4	-1.777	-1.751	-0.299	-1.382	-1.362	-0.236
3	5	2	5	-2.614	-2.575	-0.440	-2.033	-2.003	-0.347
4	5	3	5	-2.614	-2.575	-0.440	-2.033	-2.003	-0.347
5	5	4	5	-2.614	-2.575	-0.440	-2.033	-2.003	-0.347
6	0.9	5	3.4	-1.777	-1.751	-0.299	-1.382	-1.362	-0.236

Wind Load Calculation on Roof

Items	Calculation & Description	Result	Unit

A Wind Load on Roof

1 Data

Return Period =	50	years
Wind Velocity =	28	m/s
Terrain Category =	1	
Roof Mean Height,z =	15.5	m
Width of Bldg =	15	m
Length of Bldg =	20	m
Roof Pitch, α =	10	Degrees
Wind Angle, θ =	90	Degrees

2 Design Wind Velocity, V_z

V_z = **1.35V(z/zg)^k** **Notes for Table 4, PNGS 101-1982: Part 2**

k = **0.07** *zg* = **250**

= 31.11 m/s

3 Dynamic Pressure, qz

q_z = $0.6V_z$^2 x10^-3 ... Dynamic Pressure ... 0.58 kPa

4 Design Pressure, P_z

p_z	=	Cpq_z ... Design pressure...		
h/d	=	Table B2.2 **α ≥ 10° h/d ≥ 0.5**	**1.03**	
h/b	=		**0.78**	
C_p	=	... Slope D ...	**-0.9**	
C_p	=	... Slope E ...	**-0.9**	
p_z	=	... Slope D ...	-0.52	kPa
	=	... Slope E ...	-0.52	kPa

5 UDL on roof truss

Truss Spacing No.	**Truss Spacing (m)**	**Truss No.**	**Tributary Area (m)**	**Windward (kN/m)**			**Leeward (kN/m)**		
1	0.9				x-Component	y-Component		x-Component	y-Component
2	5	1	3.4	-1.777	**-1.751**	**-0.299**	-1.777	**-1.751**	**-0.304**
3	5	2	5	-2.614	**-2.575**	**-0.440**	-2.614	**-2.575**	**-0.446**
4	5	3	5	-2.614	**-2.575**	**-0.440**	-2.614	**-2.575**	**-0.446**
5	5	4	5	-2.614	**-2.575**	**-0.440**	-2.614	**-2.575**	**-0.446**
6	0.9	5	3.4	-1.777	**-1.751**	**-0.299**	-1.777	**-1.751**	**-0.304**

Wind Load on Wall

Items	Calculation & Description		Result	Unit
1 Data				
Return Period =			50	years
Wind Velocity =			28	m/s
Terrain Category =			1	
Roof Mean Height,z =			15.5	m
Width of Bldg =			15	m
Length of Bldg =			20	m
Roof Pitch, α =			10	Degrees
Wind Angle, θ =			90	Degrees

2 Pressure Coefficients, Cp

h/d	=	Table B1.1		**1.03**
h/b	=			**0.78**
d/b	=			**0.75**
C_p	=	... windward external wall surface ...	A	**0.8**
C_p	=	... leeward external wall surface ...	B	**-0.5**
	=	... side external wall surface ...	C	-0.6

3 q_z = $0.6V_z$^2 x10^-3 ... Dynamic Pressure ...

Height, z, (m)	Velocity Multiplier	Design Wind Velocity , Vz (m/s)	Dynamic Pressure, qz (kPa)
6.5	***1.041***	29.148	0.510
14.2	***1.1152***	31.226	0.585

4 Design Pressure, P_z

P_z = Cpq_z

UDL Columns

Vertical Face of Wall at 90° to Wind Direction	**Spacing, (m)**	**Column No.**	**Tributary Area (m)**	Vertical Wind Load (UDL) Distribution on Columns, W = Cpqz x Tributary Area **Windward, (kN/m)**		**Leeward, (kN/m)**	
				GFL	**FFL**	**GFL**	FFL
1	5	**1**	2.5	1.020	1.170	**-0.637**	-0.731
2	5	2	5	2.039	2.340	**-1.274**	-1.463
3	5	3	5	2.039	2.340	**-1.274**	-1.463
4	5	4	5	2.039	2.340	**-1.274**	-1.463
		5	2.5	1.020	1.170	**-0.637**	-0.731

Vertical Face of Wall at 90° to Wind Direction	**Spacing, (m)**	**Column No.**	**Tributary Area (m)**	**Side, (kN/m)**	
				GFL	**FFL**
1	5	**1**	2.5	**-0.765**	**-0.878**
2	5	2	5	**-1.529**	**-1.755**
3	5	3	5	**-1.529**	**-1.755**
		4	2.5	**-0.765**	**-0.878**

LATERAL LOAD DISTRIBUTION TO RC COLUMNS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 10 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
ROOF	0.75	300	1412.5	7	9887.5	75.54857143
SUMM			1412.5		9887.5	75.54857143

V = Cd(T1) Wt from NZS 1170.5 - 2004
= factor . Wt
= 75.54857 138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T)	=	0.16
Z	=	0.13
R	=	1.80
N(T,D)	=	1.00
Sp	=	1.00
k	=	1.00
μ	=	0.70
factor	=	0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =
= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 25 KN/M^3 Note : ALL CELLS SHADED THUS REQUIRE A MANUAL INPUT
t = 0.5 m
h = 7.7 m
Wall Mass = M x t x h x L = 96.25 L (kN/m)
Floor load (1) = dl + ll/3 = 0 Kpa
Floor load (2) = dl + ll/3 = 0 Kpa
Floor load (3) = dl + ll/3 = 0 Kpa
Floor load (4) = dl + ll/3 = 0.75 Kpa

Col	LENGTH	MASS (m)	x	y	mx	my
1	0.5	48.125	0	0	0	0
2	0.5	48.125	0	5	0	241
3	0.5	48.125	0	10	0	481
4	0.5	48.125	0	15	0	722
5	0.5	48.125	5		240.625	0
6	0.5	48.125	5	5	240.625	241
7	0.5	48.125	10	10	481.25	481
8	0.5	48.125	15	15	721.875	722
9	0.5	48.125	10	0	481.25	0
10	0.5	48.125	10	5	481.25	241
11	0.5	48.125	10	10	481.25	481
12	0.5	48.125	10	15	481.25	722
13	0.5	48.125	15	0	721.875	0
14	0.5	48.125	15	5	721.875	241
15	0.5	48.125	15	10	721.875	481
16	0.5	48.125	15	15	721.875	722
17	0.5	48.125	20	0	962.5	0
18	0.5	48.125	20	5	962.5	241
19	0.5	48.125	20	10	962.5	481
20	0.5	48.125	20	15	962.5	722

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	225	10	7.5	2250	1687.5
SUM	Wt =	1187.5			12596.88	8906.25

TABLE 1

CENTRE OF MASS	Cmx =	10.607895	Cmy =	7.5

CENTRE OF RIGIDITY

COLUMN	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.002	0.002	0.000	0.000	3.777	3.777
2	0.002	0.002	0.000	0.011	3.777	3.777
3	0.002	0.002	0.000	0.021	3.777	3.777
4	0.002	0.002	0.000	0.032	3.777	3.777
5	0.002	0.002	0.011	0.000	3.777	3.777
6	0.002	0.002	0.011	0.011	3.777	3.777
7	0.002	0.002	0.021	0.021	3.777	3.777
8	0.002	0.002	0.032	0.032	3.777	3.777
9	0.002	0.002	0.021	0.000	3.777	3.777
10	0.002	0.002	0.021	0.011	3.777	3.777
11	0.002	0.002	0.021	0.021	3.777	3.777
12	0.002	0.002	0.021	0.032	3.777	3.777
13	0.002	0.002	0.032	0.000	3.777	3.777
14	0.002	0.002	0.032	0.011	3.777	3.777
15	0.002	0.002	0.032	0.021	3.777	3.777
16	0.002	0.002	0.032	0.032	3.777	3.777
17	0.002	0.002	0.043	0.000	3.777	3.777
18	0.002	0.002	0.043	0.011	3.777	3.777
19	0.002	0.002	0.043	0.021	3.777	3.777
20	0.002	0.002	0.043	0.032	3.777	3.777
SUM	0.043	0.043	0.459	0.320	75.549	75.549

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	10.75 m	Cry	=	7.5 m
ex	=	-0.14210526 m	ey	=	0 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	6 m	ex OK	however torsional
esy <=	0.3by =	4.5 m	ey OK	shears not large

edx1	=	-1.21315789 m	edy1	=	0.75 m		(ed = 1.5es + 0.05b)
edx2	=	0.85789474 m	edy2	=	-0.75 m	or	(ed = es - 0.05b)

COLUMN	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	-10	-7.5	0.33	-0.26	0.26	0.56	-0.40
2	-10	-2.5	0.23	-0.09	0.09	0.56	-0.40
3	-10	2.5	0.23	0.09	-0.09	0.56	-0.40
4	-10	7.5	0.33	0.26	-0.26	0.56	-0.40
5	-5	-7.5	0.17	-0.26	0.26	0.28	-0.20
6	-5	-2.5	0.07	-0.09	0.09	0.28	-0.20
7	-5	2.5	0.07	0.09	-0.09	0.28	-0.20
8	-5	7.5	0.17	0.26	-0.26	0.28	-0.20
9	0	-7.5	0.12	-0.26	0.26	0.00	0.00
10	0	-2.5	0.01	-0.09	0.09	0.00	0.00
11	0	2.5	0.01	0.09	-0.09	0.00	0.00
12	0	7.5	0.12	0.26	-0.26	0.00	0.00
13	5	-7.5	0.17	-0.26	0.26	-0.28	0.20
14	5	-2.5	0.07	-0.09	0.09	-0.28	0.20
15	5	2.5	0.07	0.09	-0.09	-0.28	0.20
16	5	7.5	0.17	0.26	-0.26	-0.28	0.20
17	10	-7.5	0.33	-0.26	0.26	-0.56	0.40
18	10	-2.5	0.23	-0.09	0.09	-0.56	0.40
19	10	2.5	0.23	0.09	-0.09	-0.56	0.40
20	10	7.5	0.33	0.26	-0.26	-0.56	0.40
SUMM			3.47				

TABLE 3

FINAL LOAD DISTRIBUTION

COLUMN	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	3.5	4.0	4.3	3.4	4.0	4.3
2	3.7	3.9	4.3	3.4	3.9	4.3
3	3.9	3.7	4.3	3.4	3.9	4.3
4	4.0	3.5	4.3	3.4	4.0	4.3
5	3.5	4.0	4.1	3.6	4.0	4.1
6	3.7	3.9	4.1	3.6	3.9	4.1
7	3.9	3.7	4.1	3.6	3.9	4.1
8	4.0	3.5	4.1	3.6	4.0	4.1
9	3.5	4.0	3.8	3.8	4.0	3.8
10	3.7	3.9	3.8	3.8	3.9	3.8
11	3.9	3.7	3.8	3.8	3.9	3.8
12	4.0	3.5	3.8	3.8	4.0	3.8
13	3.5	4.0	3.5	4.0	4.0	4.0
14	3.7	3.9	3.5	4.0	3.9	4.0
15	3.9	3.7	3.5	4.0	3.9	4.0
16	4.0	3.5	3.5	4.0	4.0	4.0
17	3.5	4.0	3.2	4.2	4.0	4.2
18	3.7	3.9	3.2	4.2	3.9	4.2
19	3.9	3.7	3.2	4.2	3.9	4.2
20	4.0	3.5	3.2	4.2	4.0	4.2
SUMM					79.0	81.3

TABLE 4

LATERAL LOAD DISTRIBUTION TO RC COLUMNS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 10 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
1st Floor	6.8	300	6600	7	46200	353.005714
SUMM			6600		46200	353.005714

V = Cd(T1) Wt from NZS 1170.5 - 2004

= factor . Wt

= 353.00571 138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T)	=	0.16
Z	=	0.13
R	=	1.80
N(T,D)	=	1.00
Sp	=	1.00
k	=	1.00
μ	=	0.70
factor	=	0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =

= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 25 KN/M^3 Note : ALL CELLS SHADED THUS

t = 0.4 m REQUIRE A MANUAL INPUT

h = 6.5 m

Wall Mass = M x t x h x L = 65 L (kN/m)

Floor load (1) = dl + ll/3 = 0 Kpa

Floor load (2) = dl + ll/3 = 0 Kpa

Floor load (3) = dl + ll/3 = 0 Kpa

Floor load (4) = dl + ll/3 = 13.466667 Kpa

Col	LENGTH	MASS (m)	x	y	mx	my
1	0.4	26	0	0	0	0
2	0.4	26	0	5	0	130
3	0.4	26	0	10	0	260
4	0.4	26	0	15	0	390
5	0.4	26	5		130	0
6	0.4	26	5	5	130	130
7	0.4	26	10	10	260	260
8	0.4	26	15	15	390	390
9	0.4	26	10	0	260	0
10	0.4	26	10	5	260	130
11	0.4	26	10	10	260	260
12	0.4	26	10	15	260	390
13	0.4	26	15	0	390	0
14	0.4	26	15	5	390	130
15	0.4	26	15	10	390	260
16	0.4	26	15	15	390	390
17	0.4	26	20	0	520	0
18	0.4	26	20	5	520	130
19	0.4	26	20	10	520	260
20	0.4	26	20	15	520	390

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	4040	10	7.5	40400	30300
SUM	Wt =	4560			45990	34200

TABLE 1

CENTRE OF MASS	Cmx =	10.085526	Cmy =	7.5

CENTRE OF RIGIDITY

SHEAR WALL	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.002	0.002	0.000	0.000	17.650	17.650
2	0.002	0.002	0.000	0.011	17.650	17.650
3	0.002	0.002	0.000	0.021	17.650	17.650
4	0.002	0.002	0.000	0.032	17.650	17.650
5	0.002	0.002	0.011	0.000	17.650	17.650
6	0.002	0.002	0.011	0.011	17.650	17.650
7	0.002	0.002	0.021	0.021	17.650	17.650
8	0.002	0.002	0.032	0.032	17.650	17.650
9	0.002	0.002	0.021	0.000	17.650	17.650
10	0.002	0.002	0.021	0.011	17.650	17.650
11	0.002	0.002	0.021	0.021	17.650	17.650
12	0.002	0.002	0.021	0.032	17.650	17.650
13	0.002	0.002	0.032	0.000	17.650	17.650
14	0.002	0.002	0.032	0.011	17.650	17.650
15	0.002	0.002	0.032	0.021	17.650	17.650
16	0.002	0.002	0.032	0.032	17.650	17.650
17	0.002	0.002	0.043	0.000	17.650	17.650
18	0.002	0.002	0.043	0.011	17.650	17.650
19	0.002	0.002	0.043	0.021	17.650	17.650
20	0.002	0.002	0.043	0.032	17.650	17.650
SUM	0.043	0.043	0.459	0.320	353.006	353.006

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	10.75 m	Cry	=	7.5 m
ex	=	-0.664474 m	ey	=	0 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <= 0.3bx = 6 m **ex OK** however torsional
esy <= 0.3by = 4.5 m **ey OK** shears not large

edx1	=	-1.996711 m	edy1	=	0.75 m		(ed = 1.5es + 0.05b)
edx2	=	0.335526 m	edy2	=	-0.75 m	or	(ed = es - 0.05b)

COL	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	-10	-7.5	0.33	-1.22	1.22	4.34	-0.73
2	-10	-2.5	0.23	-0.41	0.41	4.34	-0.73
3	-10	2.5	0.23	0.41	-0.41	4.34	-0.73
4	-10	7.5	0.33	1.22	-1.22	4.34	-0.73
5	-5	-7.5	0.17	-1.22	1.22	2.17	-0.36
6	-5	-2.5	0.07	-0.41	0.41	2.17	-0.36
7	-5	2.5	0.07	0.41	-0.41	2.17	-0.36
8	-5	7.5	0.17	1.22	-1.22	2.17	-0.36
9	0	-7.5	0.12	-1.22	1.22	0.00	0.00
10	0	-2.5	0.01	-0.41	0.41	0.00	0.00
11	0	2.5	0.01	0.41	-0.41	0.00	0.00
12	0	7.5	0.12	1.22	-1.22	0.00	0.00
13	5	-7.5	0.17	-1.22	1.22	-2.17	0.36
14	5	-2.5	0.07	-0.41	0.41	-2.17	0.36
15	5	2.5	0.07	0.41	-0.41	-2.17	0.36
16	5	7.5	0.17	1.22	-1.22	-2.17	0.36
17	10	-7.5	0.33	-1.22	1.22	-4.34	0.73
18	10	-2.5	0.23	-0.41	0.41	-4.34	0.73
19	10	2.5	0.23	0.41	-0.41	-4.34	0.73
20	10	7.5	0.33	1.22	-1.22	-4.34	0.73
SUMM			3.47				

TABLE 3

FINAL LOAD DISTRIBUTION

WALL	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	16.4	18.9	22.0	16.9	18.9	22.0
2	17.2	18.1	22.0	16.9	18.1	22.0
3	18.1	17.2	22.0	16.9	18.1	22.0
4	18.9	16.4	22.0	16.9	18.9	22.0
5	16.4	18.9	19.8	17.3	18.9	19.8
6	17.2	18.1	19.8	17.3	18.1	19.8
7	18.1	17.2	19.8	17.3	18.1	19.8
8	18.9	16.4	19.8	17.3	18.9	19.8
9	16.4	18.9	17.7	17.7	18.9	17.7
10	17.2	18.1	17.7	17.7	18.1	17.7
11	18.1	17.2	17.7	17.7	18.1	17.7
12	18.9	16.4	17.7	17.7	18.9	17.7
13	16.4	18.9	15.5	18.0	18.9	18.0
14	17.2	18.1	15.5	18.0	18.1	18.0
15	18.1	17.2	15.5	18.0	18.1	18.0
16	18.9	16.4	15.5	18.0	18.9	18.0
17	16.4	18.9	13.3	18.4	18.9	18.4
18	17.2	18.1	13.3	18.4	18.1	18.4
19	18.1	17.2	13.3	18.4	18.1	18.4
20	18.9	16.4	13.3	18.4	18.9	18.4
SUMM					369.3	383.4

TABLE 4

LATERAL LOAD DISTRIBUTION TO BLOCK WALLS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 4.6 KPa

FL LEVEL	Wall mass	Floor area	Wt	Hi	Wt . Hi	Fi
1st Floor	6.8	300	5724	7	40068	306.1522
SUMM			5724		40068	306.1522

V = Cd(T1) Wt from NZS 1170.5 - 2004
= factor . Wt
= 306.1522

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T) = 0.16
Z = 0.13
R = 1.80
N(T,D) = 1.00
Sp = 1.00
k = 1.00
μ = 0.70
factor = 0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =
= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 23 KN/M^3 Note : ALL CELLS SHADED THUS
t = 0.2 m REQUIRE A MANUAL INPUT
h = 3.5 m
Wall Mass = M x t x h x L = 16.1 L (kN/m)
Floor load (1) = dl + ll/3 = 0 Kpa
Floor load (2) = dl + ll/3 = 0 Kpa
Floor load (3) = dl + ll/3 = 0 Kpa
Floor load (4) = dl + ll/3 = 10.13333 Kpa

Wall	LENGTH	MASS (m)	x	y	mx	my
1	5	80.5	0	2.5	0	201.25
2	5	80.5	0	7.5	0	603.75
3	5	80.5	0	10	0	805
4	5	80.5	2.5	0	201.25	0
5	5	80.5	2.5	5	201.25	402.5
6	5	80.5	5	7.5	402.5	603.75
7	5	80.5	5	10	402.5	805
8	5	80.5	17.5	15	1408.75	1207.5

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	3040	10	7.5	30400	22800
SUM	Wt =	3684			33016.25	27428.75

TABLE 1

CENTRE OF MASS	Cmx =	8.962066	Cmy =	7.44537188

CENTRE OF RIGIDITY

Wall	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.715	0.002	0.000	0.006	38.269	38.269
2	0.715	0.002	0.000	0.018	38.269	38.269
3	0.715	0.002	0.000	0.023	38.269	38.269
4	0.715	0.002	1.786	0.000	38.269	38.269
5	0.715	0.002	1.786	0.012	38.269	38.269
6	0.715	0.002	3.573	0.018	38.269	38.269
7	0.715	0.002	3.573	0.023	38.269	38.269
8	0.715	0.002	12.505	0.035	38.269	38.269
SUM	5.717	0.019	23.224	0.134	306.152	306.152

TABLE 2

Centre of Rigidity and Calculated Eccentricities:

Crx	=	4.0625 m	Cry	=	7.1875 m
ex	=	4.899566 m	ey	=	0.257872 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	6 m	**ex**	**OK**
esy <=	0.3by =	4.5 m	**ey**	**OK**

edx1	=	8.349349 m	edy1	=	1.136808 m		(ed = 1.5es + 0.05b)
edx2	=	3.899566 m	edy2	=	-0.492128 m	or	(ed = es - 0.05b)

Wall	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	13.3493485	-15.83	127.93	0.00	0.00	7.20	3.36
2	18.3493485	-15.83	241.18	0.00	0.00	9.89	4.62
3	18.3493485	-15.83	241.18	0.00	0.00	9.89	4.62
4	25.8493485	-15.83	478.06	0.00	0.00	13.94	6.51
5	28.3493485	-15.83	574.89	0.00	0.00	15.28	7.14
6	28.3493485	-15.83	574.89	0.00	0.00	15.28	7.14
7	28.3493485	-15.83	574.89	0.00	0.00	15.28	7.14
8	28.3493485	-15.83	574.89	0.00	0.00	15.28	7.14
SUMM			3387.90				

TABLE 3

FINAL LOAD DISTRIBUTION

Wall	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	38.3	38.3	45.5	41.6	38.3	45.5
2	38.3	38.3	48.2	42.9	38.3	48.2
3	38.3	38.3	48.2	42.9	38.3	48.2
4	38.3	38.3	52.2	44.8	38.3	52.2
5	38.3	38.3	53.6	45.4	38.3	53.6
6	38.3	38.3	53.6	45.4	38.3	53.6
7	38.3	38.3	53.6	45.4	38.3	53.6
8	38.3	38.3	53.6	45.4	38.3	53.6
SUMM					306.2	408.2

TABLE 4

Torsional Moments	
Mix (kNm)	Miy (kNm)
320	52
320	55
320	55
320	59
320	61
320	61
320	61
320	61

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

Reference **Bowles, J.E Foundation Analysis & Design**
McGrall Hill, Third Edition, 1982

Design data: Compacted Material

B = [redacted] m
μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = [redacted] m
F'c = 40 MPa
ρ = 2400 kg/m^3

k's = $0.65 \sqrt[12]{(Es\,B^4)/(Ef\,If)} * (Es/(1 - \mu^2))$

ks = k's B

$\lambda = \sqrt[4]{(k's / 4Ec\,I)}$

--> $\lambda L < \pi/4$ Rigid member - Bending not influenced much by ks
$\lambda L > \pi$ Flexible member - Bending heavily localised

Es, Ef = Modulus of soil and footing respectively
B, If = Footing width and its moment of inertia based on cross section
ks = Modulus of subgrade reaction

Procedure :Conventional rigid procedure (CRP) or Beam on Elastic Foundation (Winkler)

Section	B m	I m^4	Ec GPa	Es GPa	μ MPa	k's	L m	λ	λL	Is $\lambda L < \pi/4$	Is $\lambda L > \pi$	Analysis method ?	ks = k'sB kNm
1	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	20.00	1.7730	35.4600	No	Yes	Use Winkler	355472
2	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	20.00	1.7730	35.4600	No	Yes	Use Winkler	355472
3	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	15.00	1.7730	26.5950	No	Yes	Use Winkler	355472
4	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	15.00	1.7730	26.5950	No	Yes	Use Winkler	355472
5	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	15.00	1.0338	15.5076	No	Yes	Use Winkler	41094
6	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	20.00	1.0338	20.6768	No	Yes	Use Winkler	41094
7	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	20.00	1.0338	20.6768	No	Yes	Use Winkler	41094
8	5.00	1.41E-03	31.9754	0.18	0.4	0.26729	15	1.1041	16.5616	No	Yes	Use Winkler	53458
9	4.35	1.22E-03	31.9754	0.18	0.4	0.25815	15	1.1333	16.9998	No	Yes	Use Winkler	59344
10	3.675	1.03E-03	31.9754	0.18	0.4	0.24750	15	1.1697	17.5460	No	Yes	Use Winkler	67346

Timber Roof Purlin Design

REF: PNGS 1292 - 1989

TABLE 2.2

	STRENGTH CLASSIFICATION			
Species	Moisture Condition	Strength Group(2)		Stress Grade
Hardwood	Seasoned	SD3		F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

	Basic working stress, MPa				
Stress grade	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	Modulus of Elasticity (E)
F11	11.00	6.60	8.4	1.05	10500

TABLE A

MODIFICATION FACTORS										
Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp°C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'...Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' Eq.3.21

= 5.94 MPa

Purlin Design

Loads 2.475

DL = 0.15 kN/m LL = 0.15 kN/m 0.625

Combination: DL + LL = 0.3 kN/m

Size B (mm) D (mm) Purlin Span = 2500 mm Purlin Spacing = 600 mm CTS

75 100 HWD

From Analysis

M*max	=	0.23	kNm
V*max	=	0.38	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 1.875 MPa

Working Shear Stress, fs = 3V/2BD

= 0.075 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.142401 < 1 OK

fs = 0.075 MPa < Fs = 0.945 MPa OK

Deflection Check!

From Analysis

DL = 0.15 kN/m

LL = 0.15 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 1.162574 mm Def. by DL

Allowable Defln = L/300 8.33333333 mm > 1.162574 = Defln.DL OK

Defln.LL = 1.162574 mm Def. by LL

Allowable Defln = L/360 6.94444444 mm > 1.162574 = Defln.LL OK

Roof Truss Design

REFERENCE: **REF:** **PNGS 1292 - 1989**

TABLE 2.2

STRENGTH CLASSIFICATION

Species	Moisture Condition	Strength Group(2)	Stress Grade
Hardwood	Seasoned	SD3	F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

Stress grade	Basic working stress, MPa				Modulus of Elasticity (E)
	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	
F11	11.00	6.60	8.40	1.05	10500

TABLE A

MODIFICATION FACTORS

Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp^{0}C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'..Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' Eq.3.21

= 5.94 MPa

FROM SPACE GASS ANALYSIS

MAXIMUM VALUES OF BENDING MOMENTS & AXIAL FORCES ON THE TRUSS MEMBERS

	TOP CHORD	DIAGONAL	BOTTOM CHORD
M*max (kNm)	8	2	4
P*max (+ve) (kN)	155	20	0
P*max (-ve) (kN)	0	20	78.4

CHECK FOR ADEQUACY FOR SIZE

SECTION NAME	SECTION SIZE		SECTION PROPERTIES		
	B (mm)	D (mm)	Zx = BD^2/6, mm^3	Zy= B^{2}D/6, mm^3	A = BD, mm^2
TOP CHORD	150	300	2250000	1125000	45000
WEB	75	150	281250	140625	11250
BOTTOM CHORD	150	300	2250000	1125000	45000

WORKING STRESSES

	Bending Stress, fbx = Mx/Zx (MPa)	Axial Compressive Stress, fc = P/A (MPa)	Axial Tensile Stress, ft = P/A (MPa)
TOP CHORD	3.5556	3.4444	0.0000
WEB	0.0000	1.7778	1.7778
BOTTOM CHORD	1.7778	0.0000	1.7422

TOP CHORD CHECK!

A. Combined Bending + Axial Compression

From equation 3.23, (fbx/Fbx) + (fc/Fcx) <= 1

0.270035	+	0.34257	<= 1			
		0.6126	<	1	OK	

B. Combined bending + Axial Tension

From equation 3.25, 0.6fbx + ft <= Ft

2.133333	+	0	<=	5.94	
		2.13333	<	5.94	OK

WEB CHECK!

A. Axial Stress in Compression Member

From equation 3.19, fc <= Fcx

1.777778	<	10.0548	OK

B. Axial Stress in Tension Member

From equation 3.19, ft <= Ft

0.0000	<	5.94	OK

BOTTOM CHORD CHECK!

A. Combined Bending + Axial Compression

From equation 3.23, (fbx/Fbx) + (fc/Fcx) <= 1

0.135018	+	0	<= 1		
		0.13502	<	1	OK

B. Combined bending + Axial Tension

From equation 3.25, 0.6fbx + ft <= Ft

1.066667	+	1.7422	<=	5.94	
		2.8089	<	5.94	OK

ROOF BEAM DESIGN

REFERENCE: **Concrete Structures - 3rd Edition** **AS 3600 - 2009**

R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	0.60 kN/m	(plus super)
Self wt	:	Wsw =	2.25 kN/m	
Blockwall	:	Wb =	0.00 kN/m	
TOTAL		=	2.85 kN/m	
LL :		Wll =	0.25 kN/m	
LL factor	- long term		0.4	
	- short term		0.7	

2 Beam Section Properties

Beam Size

bw	300 mm	D =	300 mm
tff	0 mm	Cover =	65 mm
Actual length, L =	5000 mm	Overall depth of Sect. D =	300 mm
Effective Length,Lef =	4600 mm	Area, A =	9.00E+04 mm^2
Neutral Axis, yc =	150.00 mm	Moment of Inertia, Ix-x =	6.75E+08 mm^4
Neutral Axis, xc =	610.00 mm	Moment of Inertia, Iy-y =	6.75E+08 mm^4
Effective width, beff =	1220.00 mm	Gross Area, Ag =	9.00E+04 mm^2

3 Design Parameters

F'c =	32 MPa	D =	300 mm
Fsy =	410 MPa	Ec =	28599.62293 MPa
m =	2400 kg/m^3	Es =	200000 MPa
		Gamma =	0.822

4 <u>INITIAL CHOICE OF SECTION</u>

Effective width	=	1220 mm	clause 8.8.2	
Is the section RECT or T , L (ans : 1 or 2)		=		2
k_1	=	0.045	(RECT sections)	1
	=	0.02084726	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	5.013		
defln/Lef	=	0.004	k_1/k_2 = 8.005245775	
d =		76 mm		
take d =		193 mm	Distance from d to extreme fibre of beam (do) =	107 mm

5 <u>Forces from an Elastic Analysis by Space Gass :</u>

Mmax -ve	=	40.00 kNm	(left support)
Mmax +ve	=	26.00 kNm	(mid-span)
M -ve	=	40.00 kNm	(right support)
Vmax	=	29.00 kN	
V (other end)	=	29.00 kN	
N*	=	31.00 kN	Axial Load

6 **FLECTURAL STRENGTH STEEL**

Determine -ve steel required : (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm d = 193

Estimate Area of main steel Ast = 660.780762 mm^2

3 Y 20 Ast = 942.4777961 mm^2

16 p = 0.016277682

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.010417717 < pd = 0.021813

p-pc = 0.005859966 pd=0.4*0.85*µ*f'c/fsy

Check M* <= phiMu

ku = 0.107458 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 47 kNm > M* = 40.00 kNm **OK**

Determine -ve steel at right support :

M -ve = 40.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm dst = 193 mm

Estimate Area of main steel Ast = 660.780762 mm^2

3 Y 20 Ast = 942.4777961 mm^2

p = 0.016277682

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.010417717 < pd = 0.021813

p-pc = 0.005859966

Check M* <= phiMu

ku = 0.107458 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 47 kNm > M* = 40.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 26 kNm

Steel in one layer then code = 1

" " two " " code = 2

take dist between centroid of bar/s and soffit = 107 mm

what is the code ? 2

d = 193 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 429.507495 mm^2

3 Y 20 Ast = 942.4777961 mm^2

3 Y 16 Asc = 603.1857895 mm^2

Assume N - A in flange

Effective width : beff = 1220 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.07340018

hence dn = 14 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 65 kNm > M*+ve = 26.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	3	Y 20
	Asc	=	3	Y 16
Right support :	Ast	=	3	Y 20
	Asc	=	3	Y 16
midspan	Ast	=	3	Y 20
	Asc	=	3	Y 16

7 STEEL CURTAILMENT REQUIREMENT CHECK

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

For +ve steel : i) 1/2 Ast must be carried into outer support

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 65 mm

d_b = 16 mm # of lines of bars = 1

a_b = 146 mm distance between bars

If a_b > 2c C = 2c + d_b = 146 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 162 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 146 mm (dist between bars)

factor = 40074.8041

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 274.485 mm

take fs/Fsy = M*/Phi Mu = 0.849293177

therefore

Lsy =	233 mm	≥ 25k1db	500
say =	500 mm		

For Bottom Steel :

Cover, c = 65 mm d_b = 20 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 150 mm

if ab < 2c C = a_b + d_b = 474 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 454 mm (dist between bars)

factor = 50093.50513

Lsyt = 333.9567 mm take fs/Fsy = M*/Phi Mu = 0.399412636

therefore

Lsy =	133 mm	≥ 25k1db	400
say =	400 mm		

9 SHEAR CHECK

V* = 29.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 193 mm   dq = 85 mm

do = 215 mm

V* = 28 kN   beta 1 = 1.2925 (but not less than 1.1)

hence = 1.2925

beta 2 = 1

beta 3 = 1 Unity

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 45.31 kN

phi Vumax = 288.96 kN > V* No web crushing

phi Vumin = 72 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 300 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 219.512195

Asv.min = 76.8292683 mm^2

Asv.max = 1185 mm^2

Asv = 2 x area of stirrups = 220 mm^2 2 Ties

theta = 31.93863 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 168.42 kN

Phi Vu = 118 kN > V* = 28 kN

hence

PROVIDE :	1 Y12 LIGS	@	300	centres

Other Support :

V* = 29.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 193 mm   dq = 85 mm

do = 215 mm

V* = 28.40 kN   beta 1 = 1.2925 (but not less than 1.1)

hence = 1.2925

beta 2 = 1

beta 3 = 1

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 45.30563 kN

phi Vumax = 288.96 kN > V* No web crushing

phi Vumin = 72 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 300 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 219.512195

Asv.min = 76.8292683 mm^2

Asv.max = 1184.60922 mm^2

Asv = 2 x area of stirrups = 330 mm2   2 Ties

theta = 33.42808 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 211.6207 kN

Phi Vu = 148.1345 kN > V* = 28.40 kN

hence

PROVIDE :	1 Y12 LIGS	@	300 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	40.00 kNm	Es	=	200000 MPa
Ml	=	40.00 kNm	Ec	=	28599.62293 MPa
Mo	=	66 kNm			
Ms	=	26.00 kNm			
Ig	=	6.75E+08 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1220 mm	A_{sc} =	603.1857895 mm^2
webwidth	b_w =	300 mm	A_{st} =	942.4777961 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	107 mm
			d_{st} =	193 mm
			d =	193 mm
			D =	300 mm

n = Es/Ec = 6.99309919

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

150 dn^2 + 10205.793 dn - 1658832.15) = 0

dn = 76.5075848 mm

d = 193 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 1.34E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 15.2735065 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 2.44E+08 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	300 mm	A_{st} =	942.4777961 mm^2
d_{sc} =	85 mm	A_{sc} =	603.1857895 mm^2
d =	193 mm		

Ig = 6.75E+08 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

150 dn^2 + 10205.793 dn - 1579303.2) = 0

dn = 74.08 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.34E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 15.2735065 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.64E+08 mm^4

At left end (arbitrary)

b_w = **300** mm

d_{sc} = **85** mm

A_{st} = **942.4778** mm^2

A_{sc} = **603.18579** mm^2

d = **193** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

150 dn^2 + 10205.793 dn - 1579303.2) = 0

dn = 74.0825006 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.34E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 15.2735065 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.64E+08 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 2.04E+08 mm^4

Midspan Deflection :

Lef = 4600 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

$= 6.80297959$ mm

Total Deflection :

Long term factor	=	0.2		
Short term factor	=	0.5		
w - long term	=	2.9 kN/m		
w - short term	=	2.975 kN/m		
Defln,s,sus	=	6.631475902 mm		
At midspan :			Ast =	942.477796 mm^2
			Asc =	603.185789 mm^2
Asc/Ast =		0.64		
kcs	=	2 - (1.2 x Asc/Ast)		>= 0.8
	=	1.232		

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)
= 14.15596 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN =	Lef / 250 =	18.4 mm	
		> 0.9 x Total Defln	
	Lef / 500 =	9.2 mm	
		< 0.9 x Total Defln	

FIRST FLOOR BEAM DESIGN

REFERENCE:	**Concrete Structures - 3rd Edition**		**AS 3600 - 2009**
	R. F. Warner, B. V. Rangan, & A. S. Hall		

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	9.80 kN/m	(plus super)
Self wt	:	Wsw =	5.00 kN/m	
Blockwall	:	Wb =	16.10 kN/m	
TOTAL		=	30.90 kN/m	
LL :		Wll =	0.25 kN/m	
LL factor	- long term		0.4	
	- short term		0.7	

2 Beam Section Properties

Beam Size

bw	400 mm	D =	500 mm
tff	250 mm	Cover =	65 mm
Actual length, L =	5000 mm	Overall depth of Sect. D =	750 mm
Effective Length,Lef =	4600 mm	Area, A =	4.30E+05 mm^2
Neutral Axis, yc =	316.86 mm	Moment of Inertia, Ix-x =	1.81E+10 mm^4
Neutral Axis, xc =	660.00 mm	Moment of Inertia, Iy-y =	4.92E+10 mm^4
Effective width, beff =	1320.00 mm	Gross Area, Ag =	5.30E+05 mm^2

3 Design Parameters

F'c =	32 MPa	D =	500 mm
Fsy =	410 MPa	Ec =	28599.62293 MPa
m =	2400 kg/m^3	Es =	200000 MPa
		Gamma =	0.822

4 INITIAL CHOICE OF SECTION

Effective width	=	1320 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)		=		1	
k_1	=	0.045	(RECT sections)		1
	=	0.02226344	(T and L sections)		2
what type of span?		5	interior span =		5
			end span =		6
k_2	=	0.0026042			
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef	=	51.015			
defln/Lef	=	0.004	k_1/k_2 =	17.27977882	
d =		124 mm			
take d =		643 mm	Distance from d to extreme fibre of beam (do) =	107 mm	

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	310.00 kNm	(left support)
Mmax +ve	=	162.00 kNm	(mid-span)
M -ve	=	140.00 kNm	(right support)
Vmax	=	310.00 kN	
V (other end)	=	310.00 kN	
N*	=	17.00 kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm d = 643

Estimate Area of main steel Ast = 1537.111703 mm^2

6 Y 20 Ast = 1884.955592 mm^2

16 p = 0.007328754

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.002345201 < pd = 0.021813

p-pc = 0.004983553 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0.091387 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

=	417	kNm	>	M* =	310.00 kNm	OK

Determine - ve steel at right support :

M -ve = 140.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm dst = 643 mm

Estimate Area of main steel Ast = 694.1794786 mm^2

6 Y 20 Ast = 1884.955592 mm^2

p = 0.007328754

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.002345201 < pd = 0.021813

p-pc = 0.004983553

Check M* <= phiMu

ku = 0.091387 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

=	417	kNm	>	M* =	140.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 162 kNm

Steel in one layer then code = 1 take dist between centroid of

" " two " " code = 2 bar/s and soffit = 107 mm

what is the code ? 2 d = 643 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 803.264825 mm^2

6 Y 20 Ast = 1884.955592 mm^2

3 Y 16 Asc = 603.1857895 mm^2

Assume N - A in flange

Effective width : beff = 1320 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.04072484

hence dn = 26 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

=	440 kNm	>	M*+ve =	162.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 20
	Asc	=	3	Y 16
Right support :	Ast	=	6	Y 20
	Asc	=	3	Y 16
midspan	Ast	=	6	Y 20
	Asc	=	3	Y 16

7 STEEL CURTAILMENT REQUIREMENT CHECK

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

For +ve steel : i) 1/2 Ast must be carried into outer support

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 65 mm

d_b = 16 mm # of lines of bars = 1

a_b = 246 mm distance between bars

If a_b > 2c C = 2c + d_b = 146 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 262 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 246 mm (dist between bars)

factor = 40074.8041

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 274.485 mm

take fs/Fsy = M*/Phi Mu = 0.744285263

therefore

Lsy =	204 mm
say =	500 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 65 mm d_b = 20 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 150 mm

if ab < 2c C = a_b + d_b = 574 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 554 mm (dist between bars)

factor = 50093.50513

Lsyt = 333.9567 mm take fs/Fsy = M*/Phi Mu = 0.368389511

therefore

Lsy =	123 mm
say =	400 mm

≥ 25k1db 400

9 SHEAR CHECK

V* = 310.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 643 mm

dq = 85 mm

do = 415 mm

V* = 290 kN beta 1 = 1.1925 (but not less than 1.1)

hence = 1.1925

beta 2 = 1

beta 3 = 1 Unity

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 98.91 kN

phi Vumax	=	743.68 kN	>	V*	No web crushing
phi Vumin	=	169 kN	>	V*	SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 195.121951

Asv.min = 68.2926829 mm^2

Asv.max = 1083 mm^2

Asv = 2 x area of stirrups = 220 mm^2 2 Ties

theta = 32.24332 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 438.02 kN

Phi Vu = 307 kN > V* = 290 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**200**	**centres**

Other Support :

V* = 310.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 643 mm

dq = 85 mm

do = 415 mm

V* = 289.97 kN beta 1 = 1.1925 (but not less than 1.1)

hence = 1.1925

beta 2 = 1

beta 3 = 1

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 98.91422 kN

phi Vumax	=	743.68 kN	>	V*	No web crushing
phi Vumin	=	169 kN	>	V*	SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 195.121951

Asv.min = 68.2926829 mm^2

Asv.max = 1082.68465 mm^2

Asv = 2 x area of stirrups = 330 mm2 2 Ties

theta = 33.86991 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 559.5767 kN

Phi Vu = 391.7037 kN > V* = 289.97 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**200**	**centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	310.00 kNm	Es	=	200000 MPa
Ml	=	140.00 kNm	Ec	=	28599.62293 MPa
Mo	=	387 kNm			
Ms	=	162.00 kNm			
Ig	=	1.81E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1320 mm	A_{sc} =	603.1857895 mm^2
webwidth	b_w =	400 mm	A_{st} =	1884.955592 mm^2
Flange thickness	t_{ff} =	250 mm	d_{sc} =	107 mm
			d_{st} =	643 mm
			d =	643 mm
			D =	750 mm

n = Es/Ec = 6.99309919

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 246796.634 dn - 37612621.05) = 0

dn = 137.158083 mm

d = 643 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 4.95E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 127.279221 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 1.13E+10 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	400 mm	A_{st} =	1884.955592 mm^2
d_{sc} =	85 mm	A_{sc} =	603.1857895 mm^2
d =	643 mm		

Ig = 1.81E+10 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 16796.6337 dn - 8783092.1) = 0

dn = 171.73 mm

d = 643 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.60E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.5685425 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 3.69E+09 mm^4

At left end (arbitrary)

b_w = **400** mm

d_{sc} = **85** mm

A_{st} = **1884.9556** mm^2

A_{sc} = **603.18579** mm^2

d = **643** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 16796.6337 dn - 8783092.1) = 0

dn = 171.7343 mm

d = 643 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.60E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.5685425 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 4.56E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 7.72E+09 mm^4

Midspan Deflection :

Lef = 4600 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 1.16837561 mm

Total Deflection :

Long term factor	=	0.2			
Short term factor	=	0.5			
w - long term	=	30.95 kN/m			
w - short term	=	31.025 kN/m			
Defln,s,sus	=	1.165551177 mm			
At midspan :			Ast	=	1884.95559 mm^2
			Asc	=	603.185789 mm^2
Asc/Ast =		0.32			
kcs	=	2 - (1.2 x Asc/Ast)	>= 0.8		
	=	1.616			

therefore

Total Defln	=	Dfln,s + (kcs x Defln,s,sus)
	=	2.74672 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN	=	Lef / 250 =	18.4 mm
			> 0.9 x Total Defln
		Lef / 500 =	9.2 mm
			> 0.9 x Total Defln

7 **Ground Floor Slab Design**

REFERENCE: **Concrete Structures - 3rd Edition**
R. F. Warner, B. V. Rangan, & A. S. Hall

Code of Practice: **Australian Standard (AS) 3600 - 2009**

DESIGN AS A TWO WAY SLAB

Design parameters

f'c	=	32 MPa		Short term live load factor, y s=	1.00	Table 9.2
c	=	30 mm		Long term live load factor, y l =	0.80	Table 9.2
fsy	=	410 MPa		K_1 =	1.70	Cl 16.1
Lx	=	5000 mm				
Ly	=	5000 mm				
Ec	=	24E+3 MPa				
ρc	=	2400 kg/m^3				

Design Loading

Dead load

Slab thickness	=	300 mm
Self weight, wt	=	7.20 kN/m^2
Ceiling & Services	=	1.80 kN/m^2
Super	=	0.00 kN/m^2
	Total =	9.00
Total DL		9.00 kN/m^2
Live load	=	4.00 kN/m^2
Service Load	=	58 kN/m^2

HENCE

$$L_x / D_s =< 105 [(L_x/L_y)^2 / wk]^{0.33}$$

$$= \frac{5000}{D_s} =< 22.76294$$

Ds = 219.655 mm

Is these a domestic building? 2

Enter 1 = yes
2 = no

No it is not a domestic building

-

-

Hence minimum thickness = 219.6553 mm
Take Ds = 250 mm

Design moments and shears:

Design load, Fd = 1.4G + 1.7Q

= 19.40 kN/m^2

Load combination

DL = 1.40

LL = 1.70

Design moment :

				Design moments in central regions					
	Ly/Lx =	1.00		Short span direction			Long span direction		
				Midspan	Cont. edge	Discont. edge	Midspan	Cont. edge	Discont. edge
Panel	Ly/Lx	βx	βy	Mx* kNm/m	1.33Mx* kNm/m	0.5Mx* kNm/m	My* kNm/m	1.33My* kNm/m	0.5My* kNm/m
A	1.00	0.0350	0.0350	16.98	22.58	8.49	16.98	22.58	8.49
B	1.00	0.0350	0.0350	16.98	22.58	8.49	16.98	22.58	8.49
C	1.00	0.0350	0.0350	16.98	22.58	8.49	16.98	22.58	8.49
D	1.00	0.0350	0.0350	16.98	22.58	8.49	16.98	22.58	8.49
E	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
F	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
G	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
H	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
I	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
J	1.00	0.0280	0.0280	13.58	18.06	6.79	13.58	18.06	6.79
K	1.00	0.0240	0.0240	11.64	15.48	5.82	11.64	15.48	5.82
L	1.00	0.0240	0.0240	11.64	15.48	5.82	11.64	15.48	5.82

Maximum design unit shear force in the slab = 48.5 kN/m

Check Flexral steel:

Design parameters

gamma, g = 0.85   pmin = 0.001951   Reinf. Y 16

short span d = 212 mm   f = 0.8

Long span d = 196 mm

Short span

f bd^2 Fsy p = -14.7E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 113.3E+9

Min. As = 0.8/fsy *b * d

= 413.6585 mm^2/m

Long span

f bd^2 Fsy p = -12.6E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 12.9E+9

Min. As = 0.8/fsy *b * d

= 382.439 mm^2/m

f Mu > M*   Mu = f bd^2 Fsy p (1 - 0.6pFsy/F'c)

Midspan

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	17.0E+6	0.0012	413.66	16	200.96	485.811	300

Continious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	22.6E+6	0.0015	413.66	16	200.96	485.811	300

Discontinious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	8.5E+6	0.0006	413.66	16	200.96	485.811	300

Midspan

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	17.0E+6	0.0013	413.66	16	200.96	485.811	300

Continious edge

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	22.6E+6	0.0018	413.66	16	200.96	485.811	300

Discontinous edge

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	8.5E+6	0.0007	413.66	16	200.96	485.811	300

Shrinkage and Temperature:

pmin = 0.0018 Table 16.1

As = pmin b Ds reinf. y 16

= 381.600 mm^2/m As = 200.96 mm^2

spacing = 526.625 mm

say 200 mm

USE	Y	16	at	200	CTS

Check for Shear:

b1 = 1.4 - d/2000 f = 0.7

= 1.294 d = 212 mm

p = 0.001951

Vuc = b b d (p x F'c)$^{1/3}$

= 108.932 kN/m

f Vuc = 76.253 kN/m > Ok

Hence shear failure is not critical

Reinforced Masonry Block Wall Design				
REFERENCE:		Concrete Structures - 3rd Edition	AS 3600 - 2009	
		R. F. Warner, B. V. Rangan, & A. S. Hall		

Step	Calculation & Description				Result	Unit
1	Data					
	N*.max	=			515	kN
	M*.max	=			1200	kNm
	V*.max	=			515	kN
	f'c	=			15	MPa
	fsy	=			410	MPa
	εsy				0.002	
	Lw	=	... overall length of wall ...		5000	mm
	Hw	=	... unsupported height of wall ...		7000	mm
	tw	=	... wall thickness ...		200	mm
	Hwe	=	0.8 Hw, effective wall height		5600	mm
	ChecK: Hwe/tw	=	28	<	30	**OK**
2	Check maximum shear strength					
	dw	=	0.8Hw		5600	
	Θ	=	tan-1(dw/Hw)		0.674741	radians
	Θ	=		30° ≤ Θ ≤ 60°	38.7	degrees
	k3	=	(0.6+10/f'c) ≤ 0.85		1.267	
	k3f'c	=			19	MPa
	Vu.max	=	k3f'ctwdwsinΘcosΘ/(1.14+0.68cot^2Θ)		4713	kN
	Θvu.max	=			3299	kN
	Θvu.max	>	V* **OK**			
3	Vertical steel in the wall					
	Gross concrete area of the wall cross-section					
	Ag	=			1120000	mm^2
	N*/Ag	=			0.46	MPa
	In design, substituting for Vu from ϕVu ≥ V*, we obtain;					
	ϕlfsy	≥	V*cotΘ/ϕtwdw - N*/Ag	ϕ = 0.7		
		≥			0.36	MPa
	Al	≥	(ϕfsy/fsy)(twLw)	881.1909 mm^2	882	mm^2
	AsV	=	Al		882	mm^2
	Using Y12 bars		area/bar =	110 mm^2		
	Now, minimum area vertical reinforcement, Ast.min = 0.0015twLw =				2500	mm^2
	Minimum Reinforcement Requirment Governs					
	# of bars	=	22.72727		23	bars
	23 bars at each wall face and the average spacing is at;					
	Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		400	mm CTS
	minimum bar spacing = 2.5tw or 500 mm whichever is less =				500	mm CTS
	Use Y12 bars at		400 mm CTS at each wall face.			
	# of bars	=			23	bars
	AsV (provided) =				2530	mm^2
	Therefore;					
	pl	=	[(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t\		0.003	
	It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.					
	In this case;					
	pl	=	0.0030 > 0.0025		**OK**	
4	Horizontal steel in the wall					
	For adequate control of cracking due to restrained shrinkage and temperature effects, the minimum value of horizontal steel ratio pt is taken as;					
	pt	=			0.0012	
	Ast.min	=	0.0015 twHw		1680	mm^2
	For Y12 bars,		area/bar =	110 mm^2		
	# of bars	=	15.27273		16	bars
	16 bars at each wall face and the average spacing is at;					
	# of bars	=			16	bars
	Spacing of bars	=	(Hw)/(# bars @ each wall face)		438	mm CTS
	Therefore, use Y16 bars		400 mm CTS at each wall face.			

5 **Check flexural strength of wall**

The flexural strength of the wall is calculated using an equivalent rectangular stress block for compression zone concrete, like columns subjected to axial compression and bending moment.
Calculate rectangular stress block parameters.

α	=	0.85-0.004(f'c-55)			1.01
γ	=	0.85-0.008(f'c-30)	0.65	<	0.97
Take γ	=				0.97
Nu	=	N*/ϕ ϕ = 0.6			858 kN

Trial depth of neutral axis, dn = 534 mm
Compressive force in concrete, Cc = 0.85γf'cbdr (include boundary element) 1321 kN
dc = γdn/2 259 mm

Raw #	# of bars	Asi (mm^2)	bar cts (mm	dsi (mm)	εsi (0.003/dn) (dn - dsi)	δsi (MPa)	Fsi (kN)	Fsidsi (kNmm)	M = Fsi(dq-dsi) (kNmm)
1	1	110	100	100	0.0024	410	45	4510	108240
2	1	110	200	300	0.0013	263	29	8676.404	63627
3	1	110	400	700	-0.0009	-373	-41	-28723.6	-73861
4	1	110	400	1100	-0.0032	-410	-45	-49610	-63140
5	1	110	400	1500	-0.0054	-410	-45	-67650	-45100
6	1	110	400	1900	-0.0077	-410	-45	-85690	-27060
7	1	110	400	2300	-0.0099	-410	-45	-103730	-9020
8	1	110	400	2700	-0.0122	-410	-45	-121770	9020
9	1	110	400	3100	-0.0144	-410	-45	-139810	27060
10	1	110	400	3500	-0.0167	-410	-45	-157850	45100
11	1	110	400	3900	-0.0189	-410	-45	-175890	63140
12	1	110	400	4300	-0.0212	-410	-45	-193930	81180
13	1	110	400	4700	-0.0234	-410	-45	-211970	99220
14	1	110	200	4900	-0.0245	-410	-45	-220990	108240
∑	14	1540	100	5000			-463	-1544427	386646

4 Axial Strength of the wall;

Nu	=	∑Fsi + Cc	[redacted] kN
dN	=	1/Nu (Ccdc + ∑Fsidsi)	-1401.76 mm
dq	=	0.5D ... plastic centroid ...	2,500 mm
e	=	dq-dN ... eccentricity ...	3,902 mm

Summing up moments of forces about the palsic centroid gives;

Mu	=		3,347 kNm
ϕMu	=		2,008 kNm
e	=	Mu/Nu	3,902 mm
Therefore, Nu	=		858 kN

ϕMu = 2,008 kNm > M* = 1200 kNm

Therefore, the flexural strength of the wall crosection is adequate to carry the design bending moment

Reinforced Masonry Block Wall Design

REFERENCE:	**Concrete Structures - 3rd Edition**		**AS 3600 - 2009**	
	R. F. Warner, B. V. Rangan, & A. S. Hall			

Step	Calculation & Description				Result	Unit
1	Data					
	N*.max	=			650	kN
	M*.max	=			2130	kNm
	V*.max	=			650	kN
	f'c	=			15	MPa
	fsy	=			410	MPa
	εsy				0.002	
	Lw	=	... overall length of wall ...		5000	mm
	Hw	=	... unsupported height of wall ...		7000	mm
	tw	=	... wall thickness ...		200	mm
	Hwe	=	0.8 Hw, effective wall height		5600	mm
	ChecK: Hwe/tw	=	28	<	30	**OK**
2	Check maximum shear strength					
	dw	=	0.8Hw		5600	
	ϴ	=	tan-1(dw/Hw)		0.674741	radians
	ϴ	=		30° ≤ ϴ ≤ 60°	38.7	degrees
	k3	=	(0.6+10/f'c) ≤ 0.85		1.267	
	k3f'c	=			19	MPa
	Vu.max	=	k3f'ctwdwsinϴcosϴ/(1.14+0.68cot^2ϴ)		4713	kN
	ϴvu.max	=			3299	kN
	ϴvu.max	>	V* **OK**			
3	Vertical steel in the wall					
	Gross concrete area of the wall cross-section					
	Ag	=			1120000	mm^2
	N*/Ag	=			0.58	MPa
	In design, substituting for Vu from ϕVu ≥ V*, we obtain;					
	ϕlfsy	≥	V*cotϴ/ϕtwdw - N*/Ag	ϕ = 0.7		
		≥			0.46	MPa
	Al	≥	(ϕfsy/fsy)(twLw)	1112.183 mm^2	1113	mm^2
	AsV	=	Al		1113	mm^2
	Using Y16		bars	area/bar = 200 mm^2		
	Now, minimum area vertical reinforcement, Ast.min = 0.0015twLw =				2500	mm^2
	Minimum Reinforcement Requirment Governs					
	# of bars	=	12.5		13	bars
	13 bars at each wall face and the average spacing is at;					
	Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		400	mm CTS
	minimum bar spacing = 2.5tw or 500 mm whichever is less =				500	mm CTS
	Use Y12 bars at		400 mm CTS at each wall face.			
	# of bars	=	2(1+ (Lw - 2b-Ast/bar)/spacing)		12.5	bars
	AsV (provided) =				2500	mm^2
	Therefore;					
	pl	=	[(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t		0.003	
	It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.					
	In this case;					
	pl	=	0.0033 > 0.0025		**OK**	
4	Horizontal steel in the wall					
	For adequate control of cracking due to restrained shrinkage and temperature effects, the minimum value of horizontal steel ratio pt is taken as;					
	pt	=			0.0012	
	Ast.min	=	0.0015 twHw		1680	mm^2
	For Y12		bars,	area/bar = 110 mm^2		
	# of bars	=	15.27273		16	bars
	16 bars at each wall face and the average spacing is at;					
	# of bars	=			16	bars
	Spacing of bars	=	(Hw)/(# bars @ each wall face)		438	mm CTS
	Therefore, use Y16 bars		400 mm CTS at each wall face.			

5 Check flexural strength of wall

The flexural strength of the wall is calculated using an equivalent rectangular stress block for compression zone concrete, like columns subjected to axial compression and bending moment.

Calculate rectangular stress block parameters.

α	=	0.85-0.004(f'c-55)			1.01
γ	=	0.85-0.008(f'c-30)	0.65	<	0.97
Take γ	=				0.97
Nu	=	N*/ϕ ϕ = 0.6			1083 kN
Trial depth of neutral axis, dn	=				747 mm
Compressive force in concrete, Cc	:	0.85γf'cbdr (include boundary element)			1848 kN
dc	=	γdn/2			362 mm

Raw #	# of bars	Asi (mm^2)	bar cts (mm	dsi (mm)	εsi (0.003/dn) (dn - dsi)	δsi (MPa)	Fsi (kN)	Fsidsi (kNmm)	M = Fsi(dq-dsi) (kNmm)
1	1	200	100	100	0.0026	410	82	8200	196800
2	1	200	200	300	0.0018	359	72	21542.17	157976
3	1	200	400	700	0.0002	76	15	10570.28	27181
4	1	200	400	1100	-0.0014	-567	-113	-124755	-158779
5	1	200	400	1500	-0.0030	-410	-82	-123000	-82000
6	1	200	400	1900	-0.0046	-410	-82	-155800	-49200
7	1	200	400	2300	-0.0062	-410	-82	-188600	-16400
8	1	200	400	2700	-0.0078	-410	-82	-221400	16400
9	1	200	400	3100	-0.0094	-410	-82	-254200	49200
10	1	200	400	3500	-0.0111	-410	-82	-287000	82000
11	1	200	400	3900	-0.0127	-410	-82	-319800	114800
12	1	200	400	4300	-0.0143	-410	-82	-352600	147600
13	1	200	400	4700	-0.0159	-410	-82	-385400	180400
14	1	200	200	4900	-0.0167	-410	-82	-401800	196800
∑	14	2800	100	5000			-765	-2774043	862778

4 Axial Strength of the wall;

Nu	=	∑Fsi + Cc	[redacted] kN
dN	=	1/Nu (Ccdc + ∑Fsidsi)	-1942.98 mm
dq	=	0.5D ... plastic centroid ...	2,500 mm
e	=	dq-dN ... eccentricity ...	4,443 mm

Summing up moments of forces about the palsic centroid gives;

Mu	=		4,813 kNm
ϕMu	=		2,888 kNm
e	=	Mu/Nu	4,443 mm
Therefore, Nu	=		1,083 kN

ϕMu = 2,888 kNm > M* = 2130 kNm

Therefore, the flexural strength of the wall crosection is adequate to carry the design bending moment

RC COLUMN DESIGN WITH MRCSECT PACKAGE

INTERACTION DIAGRAM FOR RECTANGULAR COLUMN SECTION
(NON-PRESTRESSED REINFORCEMENT ONLY)

INPUT

450 x 450 RC COLUMN DESIGN

Using RECTANGULAR STRESS BLOCK

Width (mm)...................= 450
Depth (mm)...................= 450
No of lines of reinforcement.= 3
Concrete strength (MPa)......= 32
Yield stress of steel (MPa)..= 410

STEEL DETAILS

Line no	Depth to steel	Bar diameter	No of bars
1	87	24	3
2	225	24	2
3	363	24	3

OUTPUT

Depth to Plastic Centroid (Ref Axis for Moment) = 202.701

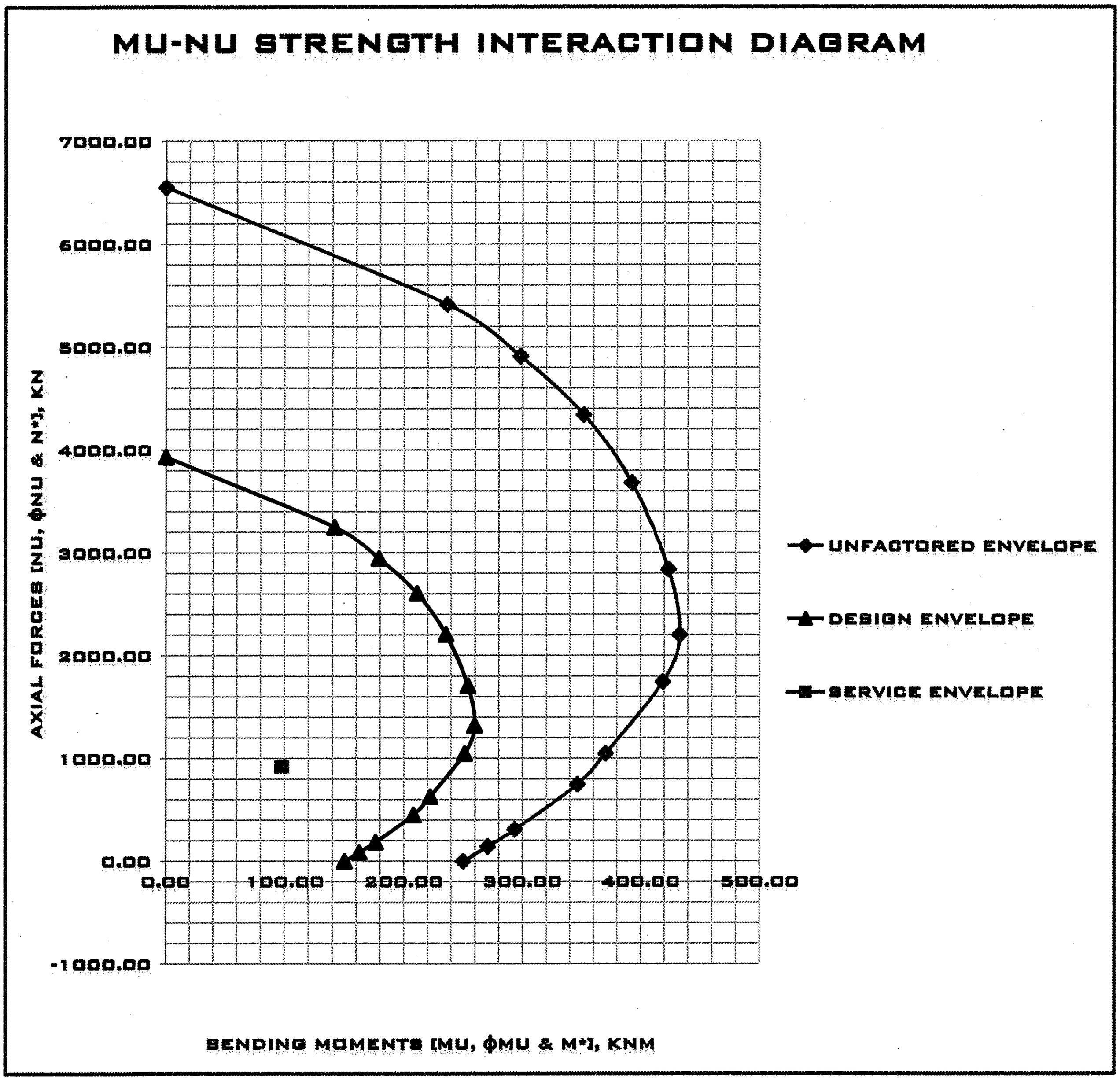
UNFACTORED ENVELOPE

Eccentricity (mm)	Force (kNm)	Moment (kNm)	Top Strain	Bottom Strain	N.A.factor ku
0.00	6554.05	0.00	0.00300	0.00300	1000.000
43.66	5414.44	236.39	0.00300	0.00000	1.000
60.73	4910.01	298.18	0.00300	-0.00030	0.909
81.01	4341.27	351.69	0.00300	-0.00070	0.811
106.62	3680.29	392.38	0.00300	-0.00130	0.865
149.07	2840.07	423.37	0.00300	-0.00220	0.715
196.51	2202.26	432.76	0.00300	-0.00300	0.620
239.83	1745.94	418.72	0.00300	-0.00390	0.566
352.91	1049.30	370.31	0.00300	-0.00580	0.480
462.68	748.85	346.48	0.00300	-0.00680	0.444
950.90	308.80	293.64	0.00300	-0.00930	0.357
1881.86	143.90	270.80	0.00300	-0.01053	0.324
	-0.87	250.14	0.00300	-0.01173	0.298

DESIGN ENVELOPE

PHI	FORCE(kN)	MOMENT(kNm)
.6	3932.43	0
.6	3248.666	141.8356
.6	2946.009	178.9065
.6	2604.762	211.

UNFACTORED ENVELOPE		NEUTRAL AXIS FACTOR		DESIGN ENVELOPE		SERVICE ENVELOPE	
Mu (kNm)	Nu (kN)	ku	phi (ϕ)	ϕMu (kNm)	ϕNu (kN)	M* (kNm)	N* (kN)
0.00	6554.05	1000.000	0.60000	0.00	3932.43	98.00	922.20
236.39	5415.44	1.000	0.60000	141.83	3249.26		
298.18	4910.01	0.909	0.60000	178.91	2946.01		
351.69	4341.27	0.811	0.60000	211.01	2604.76		
392.38	3680.29	0.865	0.60000	235.43	2208.17		
423.37	2840.07	0.715	0.60000	254.02	1704.04		
432.76	2202.26	0.620	0.60000	259.66	1321.36		
418.72	1745.94	0.566	0.60000	251.23	1047.56		
370.31	1049.30	0.480	0.60000	222.19	629.58		
346.48	748.85	0.444	0.60000	207.89	449.31		
293.64	308.80	0.357	0.60000	176.18	185.28		
270.80	143.90	0.324	0.60000	162.48	86.34		
250.14	-0.87	0.293	0.60000	150.08	-0.52		



450 x 450 RC Column is OK.

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

REFERENCE: **Concrete Structures - 3rd Edition** AS 3600 - 2009
R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	33.60 kN/m	(plus super)
Self wt	:	Wsw =	25.00 kN/m	
Blockwall	:	Wb =	0.00 kN/m	
TOTAL		=	58.60 kN/m	
LL :		Wll =	0.25 kN/m	
LL factor	- long term		0.4	
	- short term		0.7	

2 Beam Section Properties

Beam Size

bw =	5000 mm	D =	200 mm
tff =	0 mm	Cover =	30 mm
Actual length, L =	1400 mm	Overall depth of sect. D =	200 mm
Effective Length,Lef =	1000 mm	Area, A =	1.00E+06 mm^2
Neutral Axis, yc =	100.00 mm	Moment of Inertia, Ix-x =	3.33E+09 mm^4
Neutral Axis, xc =	2600.00 mm	Moment of Inertia, Iy-y =	2.08E+12 mm^4
Effective width, beff =	5200.00 mm	Gross Area, Ag =	1.00E+06 mm^2

3 Design Parameters

F'c =	40 MPa	Fsy =	410 MPa
m =	2400 kg/m^3	D =	200 mm
Ec =	31975.35051 MPa	Es =	200000 MPa
Gamma =	0.766		

4 Initial Choice of Section

Effective width =		5200 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)		=		1	
k_1	=	0.045	(RECT sections)		1
	=	0.04346021	(T and L sections)		2
what type of span?		5	interior span =		5
			end span =		6
k_2	=	0.0026042			
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef	=	96.443			
defln/Lef	=	0.004	k_1/k_2 =	17.27977882	
d	=	20 mm			
take d =		154 mm	Distance from d to extreme fibre of beam (do) =	46 mm	

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	60.30 kNm	(left support)
Mmax +ve	=	60.30 kNm	(mid-span)
M -ve	=	60.30 kNm	(right support)
Vmax	=	127.50 kN	
V (other end)	=	127.50 kN	
N*	=	0.00 kN	Axial Load

6 FLECTURAL STRENGTH STEEL

<u>Determine -ve steel required :</u> (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 1

take do = 46 mm d = 154

Estimate Area of main steel		Ast =	1248.392927 mm^2			
6	Y 16	Ast =	1206.371579 mm^2	**SPACING = 823**		
		p =	0.001566716			
2	Y 16	Asc =	402.1238597 mm^2			
		pc =	0.000522239	<	pd =	0.025409
		p-pc =	0.001044478	pd=0.4*0.85*ku*f'c/fsy		

<u>**Check M* <= phiMu**</u>

ku = 0.016442781 dsc = 38 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

=	63	kNm	>	M* =	60.30 kNm	OK

Determine -ve steel at right support:

M -ve = 60.30 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 46 mm dst = 154 mm

Estimate Area of main steel		Ast	=	1248.392927 mm^2			
6	Y 16	Ast	=	1206.371579 mm^2			
		p	=	0.001566716			
2	Y 16	Asc	=	402.1238597 mm^2			
		pc	=	0.000522239	<	pd =	0.025409
		p-pc	=	0.001044478			

Check M* <= phiMu

ku = 0.016442781 dsc = 38 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 63 kNm > M* = 60.30 kNm OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 60.3 kNm

Steel in one layer then code = 1 take dist between centroid of

" " two " " code = 2 bar/s and soffit = 46 mm

what is the code ? 1 d = 154 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1248.392927 mm^2

Bottom	6	Y 16	Ast	=	1206.371579 mm^2
Top	2	Y 16	Asc	=	402.1238597 mm^2

Assume N - A in flange

Effective width : beff = 5200 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.02371555

hence dn = 4 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 68 kNm > M*+ve = 60.30 kNm OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 16
	Asc	=	2	Y 16
Right support :	Ast	=	6	Y 16
	Asc	=	2	Y 16
midspan	Ast	=	6	Y 16
	Asc	=	2	Y 16

7 **STEEL CURTAILMENT REQUIREMENT CHECK**

The following requirements regarding curtailment are subject to stress development length requirements.

For -ve steel
- i) 1/4 Ast to continue over length of beam = 2 bars
- ii) Remainder to be curtailed at 0.3Ln from face of support. = 0.3 x le = 300 mm

For -ve steel
- i) 1/4 Ast to continue over length of beam = 2 bars
- ii) Remainder to be curtailed at 0.3Ln from face of support. = 0.3 x le = 300 mm

For +ve steel :
- i) 1/2 Ast must be carried into outer support = 2 bars
- ii) 1/4 Ast into interior support = 2 bars
- iii) Remainder curtailed at 0.1Ln from supports = 0.1 x le = 100 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 30 mm

d_b = 16 mm   # of lines of bars = 2

a_b = 2458 mm   distance between bars

If a_b > 2c   C = 2c + d_b = 76 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c   C = a_b + d_b = 2474 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 2458 mm   (dist between bars)

factor = 35843.99446

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 471.6315061 mm

take fs/Fsy = M*/Phi Mu = 0.963331

therefore

Lsy =	454	mm
say =	650	mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 30 mm d_b = 16 mm

# of lines of bars = 2

If ab > 2c   C = 2c + d_b = 76 mm

if ab < 2c   C = a_b + d_b = 2558 mm

k1 = 1 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 2542 mm   (dist between bars)

factor = 28675.19557

Lsyt = 377.3052049 mm   take fs/Fsy = M*/Phi Mu = 0.887670676

therefore

Lsy =	335	mm
say =	500	mm

≥ 25k1db 400

9 PUNCHING SHEAR CHECK

Coloumn Dimension

V* = 80.4 kN   b = 400 mm

Dist to centreline of bottom   d = 400 mm

steel from soffit   Critical shear perimeter, µ

dq = 38 mm   µ = 2(b+dom) + 2(d+dom)

dom = 162 mm   2248 mm

Area of slab: B = 1000 mm   D = 1000 mm

βh = 1   longer side/shorter side

fcv = 0.17(1+2/βh)√f'c ≤ 0.34√f'c

fcv = 13.60 MPa

ϕVuo = 3,467 kN   >   V* = 80.4 kN   OK

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	60.30 kNm	Es	=	200000 MPa
Ml	=	60.30 kNm	Ec	=	31975.35051 MPa
Mo	=	120.6 kNm			
Ms	=	60.30 kNm			
Ig	=	3.33E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	5200 mm	A_{sc} =	402.1238597 mm^2
webwidth	b_w =	5000 mm	A_{st} =	1206.371579 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	46 mm
			d_{st} =	154 mm
			d =	154 mm
			D =	200 mm

n = Es/Ec = 6.25481807

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1259229.786) = 0

dn = 20.5943193 mm

d = 154 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 1.49E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 126.4911064 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 2.95E+10 mm^4

Calculate Icr , Mcr, ILef and Iref at end supports:

b_w =	5000 mm	A_{st} =	1206.371579 mm^2
d_{sc} =	38 mm	A_{sc} =	402.1238597 mm^2
d =	154 mm		

Ig = 3.33E+09 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1242325.084) = 0

dn = 20.44 mm

d = 154 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.49E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 126.4911064 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^: = 2.95E+10 mm^4

At left end (arbitrary)

b_w =	5000 mm
d_{sc} =	38 mm
A_{st} =	1206.371579 mm^2
A_{sc} =	402.1238597 mm^2
d =	154 mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1242325.084) = 0

dn = 20.4437258 mm

d = 154 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.49E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 126.4911064 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.95E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 2.95E+10 mm^4

Midspan Deflection :

Lef = 1000 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.00531936 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 58.65 kN/m

w - short term = 58.725 kN/m

Defln,s,sus = 0.005313 mm

At midspan : Ast = 1206.371579 mm^2

Asc = 402.1238597 mm^2

Asc/Ast = 0.333333

kcs = 2 - (1.2 x Asc/Ast) >= 0.8

= 1.6

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)

= 0.01382 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 500 = 2 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

ISOLATED COLUMN FOOTING DESIGN

REFERENCE: Concrete Structures - 3rd Edition; R. F. Warner, B. V. Rangan, & A. S. Hall — Australian Standard (AS) 3600-2009

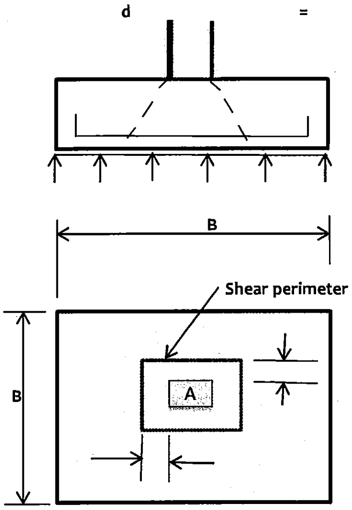
	Calculations & Description					Result	Unit
1	Data						
	DL	=	... DL from Column 1 ...			329.3	kN
	LL	=	... LL from Column 1 ...			271.3	kN
	P	=	DL + LL			600.6	kN
	P*	=	1.4 DL + 1.7 LL			922.23	kN
	qa	=			0.175	450	kPa
	f'c	=			3.85	32	MPa
	fsy	=	... reinforcement...		0.525	410	MPa
	col. Width A	=			4.55	0.45	m
	col. Width B	=				0.45	m
2	Footing Area, A						
	A	=	P*total/qa			1.334666667	m^2
	Approximate the footing dimension as:						
	B	=	short side			1.8	m
	L	=	Long side			1.8	m
						3.24	m^2
	quit	=	P^*_{total}/A			0.284638889	MPa

3 Determine the required effective depth by considering the beam shear on a critical section at distance d from the c2 mm face of the column.

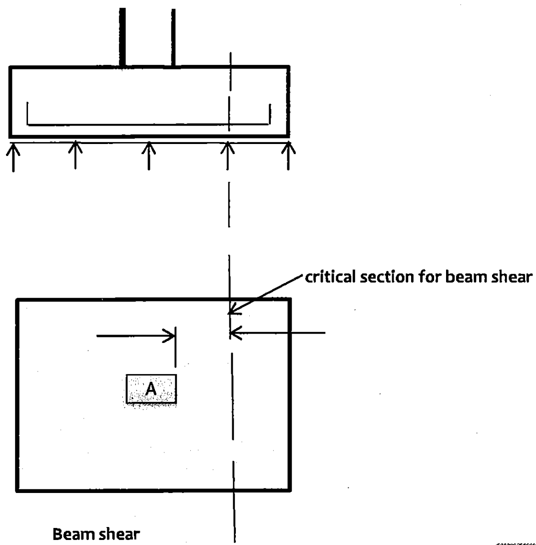
					Result	Unit
V*	=	quit x B (C-Col.w/2-d)				
V*	=	345836.25	-d	512.35		N

The shear strenght of the section depends on the amount of tensile steel used for bending. We assume a minimum quatity of:

					Result	Unit
pmin	=	1.4/fsy			0.003414634	
V_{uc}	=	β_1BD(Ast/BD x f'c)^1/3				
V_{uc}	=	946.5929 d				N
Equating V* & φVuc						
662.615 d	=	345836.25	-d	512.35		
1174.965 d	=	345836.25				
d	=				294.3374834	mm



Punching shear



Beam shear

						Result	Unit
	Cover	=				75	mm
	Assume bar dia		Y20			20	mm
	D	=	d + cover + bar dia/2			379.3374834	mm
	Take D	=				[redacted]	mm
	d	=	D-cover-bar dia/2			315	mm
	d_{om}	=	D-cover-bar dia			305	mm
4	Check Punching Shear						
	Critical shear perimeter is a rectangle of side,			450 +	305	755	mm
			and	450 +	305	755	mm
	punching shear area	=				570025	mm^2
	μ	=	2(c1+dom) + 2(c2+dom)			3020	mm
	V*	=	P* - quit x punching shear area			759.9787174	kN
	V*	=	quit(A-punching shear area)			759.9787174	kN
	f_{cv}	=	$0.17(1+2/\beta_h)\sqrt{f'c}$ ≤ $0.34\sqrt{f'c}$				
	β_h	=	Col. Length/col.width			1	
	f_{cv}	=			0.34 f'c		
	ϕV_{uo}	=	$\phi \mu d_{om} f_{cv}$			1240.105771	kN
						OK, SINCE φVuo > V*B	

5 Tensile steel for bending in longitudinal direction

The critical section for bending is at the longer face of the column, where the length of the cantilevered portion is:

l	=	0.5(B-C2)		0.675	m
M*	=	qult x B x l^2/2		116.7197344	kNm
Ast.reqd.	=	M*/ϕ0.9dfsy		1,255.21	mm^2
Ast.min.	=	1.4/fsy bd		1,936.10	mm^2
			Ast.min. Governs, Therefore, use Ast.min.		
Hence provide		7 Y20	bars in longitudinal direction		
Astprovide	=			2,170.00	mm^2

6 Tensile steel for bending in the transverse direction.

Considering a transverse strip 1 m wide:

l	=	0.5(L-C1)		0.675	m
M*	=	qult x 1m x l^2/2		64.84429688	kNm
Ast.reqd.	=	M*/ϕ0.9dfsy		697.34	mm^2/m
Ast.min.	=	1.4/fsy bd		1,075.61	mm^2/m
			Ast.min. Governs, Therefore, use Ast.min.		
Ast required for the full length				1,936.10	mm^2
Hence provide		7 Y20	bars in transverse direction		
Astprovide	=			2,170.00	mm^2

7 Spacing between the bars

Ast/bar	=			310	mm^2
S	=	[Ast/bar X beff]/Ast.prov.	... centre - to - centre spacing ...	258	mm

8 Now check 7 bars @ 258 CTS

T = fsyAst	=							889.7	kN
C = T	=							889.7	kN
A	=	C/0.85f'c						32709.6	mm^2
γkud	=	A/b						18.2	mm
a	=	γkud/2						9.1	mm
l	=	d-a						305.9	mm
Mu(prov)	=	Txl						272.2	kN
ϕMu	=	217.7 kNm		>	M*	=	116.7 kNm		OK
γ	=	0.85-0.007(f'c-28)						0.822	
ku	=	γkud/γd	0.07	<	0.4		UNDER-REINFORCED.Therefore, OK		

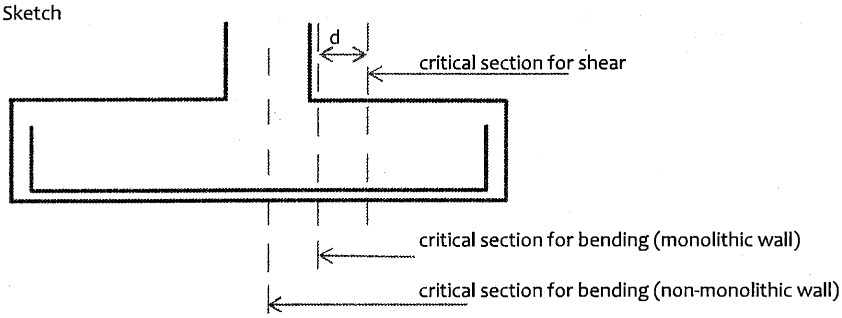
THEREFORE, USE 7 BARS @ 258 AT TOP & BOTTOM AND IN BOTH DIRECTIONS, Ast PROVIDED 2170 mm^2

STRIP FOOTING DESIGN FOR WALL

REFERENCE: **Concrete Structures - 3rd Edition** **Australian Standard (AS) 3600 -2009**

R. F. Warner, B. V. Rangan, & A. S. Hall

Step	Calculations & Description				Result	Unit
	Sketch					



Step						Result	Unit
	Strip Footing for Wall						
1	Data	RC WALL			kN/m^3 mm		
	UDL:	Self wt	=	Density x Thkness	23 200	4.6	kPa
		DL	=			27.246	kN/m
	L	=	span			1	m
	Total DL	=				31.846	kN/m
		LL	=			18	kN/m
	Conc. Load	DL	=				
		LL	=				
	P	=	DL + LL			49.846	kN
	P*	=	1.4 DL	+	1.7 LL	75.1844	kN
	qa	=				450	kPa
	f'c	=				40	MPa
	fsy	=	... reinforcement...			410	MPa
	c	=	... width of wall			0.2	m
2	Footing Area, A						
	A	=	P*total/qa			0.1107689	m^2
	Approximate the footing dimension as:						
	D	=	A/L			0.1107689	m
		=				[redacted]	m
	Mmax	=	1/8 P*(D-c)			7.51844	kNm
	qult	=	P^*_{total}/A			0.0751844	MPa
3	Determine the required effective depth by considering the beam shear on a critical section at distance d from the c mm face of the wall.						
	V*	=	qult x B (C-Col.w/2-d)				
	V*	=	30073.76	-	75.1844 d		N
	The shear strenght of the section depends on the amount of tensile steel used for bending. We assume a minimum quatity of:						
	pmin	=	1.4/fsy			0.0034146	
	V_{uc}	=	β_1BD(Ast/BD x f'c)^1/3				
	V_{uc}	=	566.4923836 d				N
	Equating V* & φVuc						
	396.5447 d	=	30073.76	-	75.1844 d		
	471.7291 d	=	30073.76				
	d	=				63.752187	mm
	Cover	=				75	mm
	Assume bar dia		Y16			16	mm
	D	=	d + cover + bar dia/2			146.75219	mm
	Take D	=				[redacted]	mm
	d	=	D-cover-bar dia/2			317	mm
4	Tensile steel for bending in longitudinal direction: Oneway Bending The critical section for bending is at the longer face of the column, where the length of the cantilevered portion is:						
	L	=	0.5(B-c)			0.4	m
	M*	=	qult x B x L/2			6.014752	kNm
	Ast.reqd.	=	M*/φ0.9dfsy			64.275004	mm^2
	Ast.min.	=	1.4/fsy bd			1082.439	mm^2
					Ast.min. Governs, Therefore, use Ast.min.		
	Hence provide		6 Y16	bars in longitudinal direction top & bottom.			
	Astprov	=		200 Ast/bar		1200	mm^2
5	Spacing between the bars						
	Ast/bar	=				200	mm^2
	S	=	[Ast/bar X beff]/Ast.prov.	... centre - to - centre spacing ...		167	mm

Wall Design

REFERENCE: **Concrete Structures - 3rd Edition** AS 3600 - 2009

R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	10.00	kN/m	(plus super)
Self wt	:	Wsw =	0.00	kN/m	
Blockwall	:	Wb =	0.00	kN/m	
TOTAL		=	10.00	kN/m	
LL :		Wll =	10	kN/m	
LL factor	- long term		0.4		
	- short term		0.7		

2 Beam Section Properties

Beam Size

bw, eff =	1000	mm	D =	400	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	5700	mm	Overall depth of Sect. D =	400	mm
Effective Length,Lef =	5400	mm	Area, A =	4.00E+05	mm^2
Neutral Axis, yc =	200.00	mm	Moment of Inertia, Ix-x =	5.33E+09	mm^4
Neutral Axis, xc =	1040.00	mm	Moment of Inertia, Iy-y =	3.33E+10	mm^4
Effective width, beff =	2080.00	mm	Gross Area, Ag =	4.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	400	mm
Fsy =	410	MPa	Ec =	31975.35051	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

4 INITIAL CHOICE OF SECTION

Effective width = 2080 mm clause 8.8.2

Is the section RECT or T , L (ans : 1 or 2) = 1

k_1	=	0.045	(RECT sections)	1
	=	0.02707672	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	29.96		
defln/Lef	=	0.004	k_1/k_2 = 17.27977882	

d = 101 mm

take d = 289 mm Distance from d to extreme fibre of beam (do) = 111 mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	472.00	kNm	(left support)	1.116
Mmax +ve	=	68.00	kNm	(mid-span)	
M -ve	=	0.00	kNm	(right support)	
Vmax	=	274.00	kN		
V (other end)	=	274.00	kN		
N*	=	96.00	kN	Axial Load	

6 FLECTURAL STRENGTH STEEL

Determine -ve steel required : (At arbitrary left support) Bar Spacing 208 mm

Number of reinforcement layers 1 or 2 : 1

take do = 111 mm d = 289

Estimate Area of main steel Ast = 5207.13532 mm^2

11	Y 24	Ast =	4976.282763 mm^2	OUTSIDE
		p =	0.017218971	
15	Y 24	Asc =	6785.840132 mm^2	INSIDE
		pc =	0.023480416	< pd = 0.025409
		p-pc =	-0.00626144	pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 99 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

=	476	kNm	>	M* =	472.00 kNm	OK

Determine - ve steel at right support :

M -ve = 0.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 111 mm dst = 289 mm

Estimate Area of main steel Ast = 0 mm^2

11 Y 24 Ast = 4976.282763 mm^2 OUTSIDE

p = 0.017218971

15 Y 24 Asc = 6785.840132 mm^2 INSIDE

pc = 0.023480416 < pd = 0.025409

p-pc = -0.00626144

Check M* <= phiMu

ku = 0 dsc = 99 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

=	476	kNm	>	M* =	0.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 68 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 123 mm

what is the code ? 2 d = 277 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 782.679307 mm^2

11 Y 24 Ast = 4976.282763 mm^2 OUTSIDE

15 Y 24 Asc = 6785.840132 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 2080 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.13596844

hence dn = 38 < flange thickness
hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

=	482 kNm	>	M*+ve =	68.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	11	Y 24
	Asc	=	15	Y 24
Right support :	Ast	=	11	Y 24
	Asc	=	15	Y 24
midspan	Ast	=	11	Y 24
	Asc	=	15	Y 24

7 STEEL CURTAILMENT REQUIREMENT CHECK

The following requirements regarding curtailment are subject to stress development length requirements.

For -ve steel
i) 1/4 Ast to continue over length of beam
= 2 bars
ii) Remainder to be curtailed at 0.3Ln from face of support.
= 0.3 x le = 1620 mm

For -ve steel
i) 1/4 Ast to continue over length of beam
= 2 bars
ii) Remainder to be curtailed at 0.3Ln from face of support.
= 0.3 x le = 1620 mm

For +ve steel :
i) 1/2 Ast must be carried into outer support
= 2 bars
ii) 1/4 Ast into interior support
= 2 bars

iii) Remainder curtailed at 0.1Ln from supports
= 0.1 x le = 540 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm
d_b = 24 mm # of lines of bars = 1
a_b = 826 mm distance between bars
If a_b > 2c C = 2c + d_b = 174 mm
C is the outside diameter of a concrete annulus
if a_b =< 2c C = a_b + d_b = 850 mm
coaxial with and surrounding a bar
k1 = 1.25 A_b = 452.3893421 mm^2
k2 = 2.2 a_b = 826 mm (dist between bars)
factor = 80648.98754
Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 463.4999 mm
take fs/Fsy = M*/Phi Mu = 0.992106757
therefore

Lsy =	460 mm
say =	750 mm

≥ 25k1db 750

For Bottom Steel :

Cover, c = 75 mm d_b = 24 mm
# of lines of bars = 1
If ab > 2c C = 2c + d_b = 174 mm
if ab < 2c C = a_b + d_b = 1198 mm
k1 = 1 A_b = 452.3893421 mm^2
k2 = 2.2 a_b = 1174 mm (dist between bars)
factor = 64519.19003
Lsyt = 370.7999 mm take fs/Fsy = M*/Phi Mu = 0.141034112
therefore

Lsy =	52 mm
say =	900 mm

≥ 25k1db 600

9 SHEAR CHECK

V* = 274.00 kN
Dist to centreline of bottom steel from soffit
critical location : d = 289 mm
dq = 93 mm
do = 307 mm
V* = 268 kN beta 1 = 1.2465 (but not less than 1.1)
hence = 1.2465
beta 2 = 1
beta 3 = 1 Unity
phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333
= 231.89 kN
phi Vumax = 1719.2 kN > V* No web crushing
phi Vumin = 361 kN > V* OK

If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 200 mm Fsyf = 410 MPa
Determine theta :
bv x s/Fsy f = 487.804878
Asv.min = 170.731707 mm^2
Asv.max = 3376 mm^2
Asv = 2 x area of stirrups = 400 mm^2 2 Ties
theta = 31.0729 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 749.03 kN
Phi Vu = 524 kN > V* = 268.22 kN
hence

PROVIDE :	**1 Y16 LIGS**	**@**	**200**	**centres**

Other Support :

V* = 274.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 289 mm   dq = 93 mm

do = 307 mm

V* = 268.22 kN   beta 1 = 1.2465 (but not less than 1.1)

hence = 1.2465

beta 2 = 1

beta 3 = 1

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 231.882 kN

phi Vumax = 1719.2 kN > V* No web crushing

phi Vumin = 361 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 487.804878

Asv.min = 170.731707 mm^2

Asv.max = 3376.08651 mm^2

Asv = 2 x area of stirrups = 600 mm2   2 Ties

theta = 32.00883 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 935.355 kN

Phi Vu = 654.7485 kN > V* = 268.22 kN

hence

PROVIDE :	**1 Y16 LIGS**	**@**	**200**	**centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	472.00 kNm	Es	=	200000 MPa
Ml	=	0.00 kNm	Ec	=	31975.35051 MPa
Mo	=	304 kNm			
Ms	=	68.00 kNm			
Ig	=	5.33E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2080 mm	A_{sc} =	6785.840132 mm^2
webwidth	b_w =	1000 mm	A_{st} =	4976.282763 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	123 mm
			d_{st} =	277 mm
			d =	289 mm
			D =	400 mm

n = Es/Ec = 6.25481807

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

500 dn^2 + 66784.0986 dn - 13381317.53) = 0

dn = 109.915508 mm

d = 277 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 1.31E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 101.192885 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	1.46E+10 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	1000 mm	**A_{st} =**	4976.282763 mm^2
d_{sc} =	99 mm	**A_{sc} =**	6785.840132 mm^2
d =	289 mm		

Ig = 5.33E+09 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

500 dn^2 + 66784.0986 dn - 12525517) = 0

dn = 105.00 mm

d = 289 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.44E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 101.192885 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.48E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **99** mm

A_{st} = **4976.2828** mm^2

A_{sc} = **6785.8401** mm^2

d = **289** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

500 dn^2 + 66784.0986 dn - 12525517) = 0

dn = 105.003997 mm

d = 289 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.44E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 101.192885 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 0.00E+00 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 7.65E+09 mm^4

Midspan Deflection :

Lef = 5400 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.25821426 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 12 kN/m

w - short term = 15 kN/m

Defln,s,sus = 0.206571409 mm

At midspan : Ast = 4976.28276 mm^2

Asc = 6785.84013 mm^2

Asc/Ast = 1.363636364

kcs = 2 - (1.2 x Asc/Ast) >= 0.8

= 0.8

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)

= 0.42347 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 250 = 21.6 mm

> 0.9 x Total Defln

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

REFERENCE: **Concrete Structures - 3rd Edition** AS 3600 - 2009
R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl	=	33.60 kN/m	(plus super)
Self wt	:	Wsw	=	25.00 kN/m	
Blockwall	:	Wb	=	0.00 kN/m	
TOTAL			=	58.60 kN/m	
LL :		Wll	=	0.25 kN/m	
LL factor	- long term			0.4	
	- short term			0.7	

2 Beam Section Properties

Beam Size

bw =	2000 mm	D =	500 mm
tff =	0 mm	Cover =	75 mm
Actual length, L =	5800 mm	Overall depth of sect. D =	500 mm
Effective Length,Lef =	5400 mm	Area, A =	1.00E+06 mm^2
Neutral Axis, yc =	250.00 mm	Moment of Inertia, Ix-x =	2.08E+10 mm^4
Neutral Axis, xc =	1540.00 mm	Moment of Inertia, Iy-y =	3.33E+11 mm^4
Effective width, beff =	3080.00 mm	Gross Area, Ag =	1.00E+06 mm^2

3 Design Parameters

F'c =	40 MPa	Fsy =	410 MPa
m =	2400 kg/m^3	D =	500 mm
Ec =	31975.35051 MPa	Es =	200000 MPa
Gamma =	0.766		

4 Initial Choice of Section

Effective width = 3080 mm clause 8.8.2
Is the section RECT or T , L (ans : 1 or 2) = 1

k_1	=	0.045	(RECT sections)	1
	=	0.03224022	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6

k_2 = 0.0026042
kcs = 1.64 (because there will be comp steel at midspan=say .5Ast)

Fdef = 96.443
defln/Lef = 0.004 k_1/k_2 = 17.27977882
d = 131 mm
take d = 405 mm Distance from d to extreme fibre of beam (do) = 95 mm

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	306.00 kNm	(left support)
Mmax +ve	=	162.00 kNm	(mid-span)
M -ve	=	311.00 kNm	(right support)
Vmax	=	127.50 kN	
V (other end)	=	127.50 kN	
N*	=	0.00 kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support)
Number of reinforcement layers 1 or 2 : 1
take do = 95 mm d = 405
Estimate Area of main steel Ast = 2408.912978 mm^2

9 Y 20 Ast = 2827.433388 mm^2 **SPACING = 206**
p = 0.003490659
9 Y 20 Asc = 2827.433388 mm^2
pc = 0.003490659 < pd = 0.025409
p-pc = 0 pd=0.4*0.85*ku*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 334 kNm > M* = 306.00 kNm OK

Determine -ve steel at right support :

M -ve = 311.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 95 mm dst = 405 mm

Estimate Area of main steel Ast = 2448.274301 mm^2

9 Y 20 Ast = 2827.433388 mm^2

p = 0.003490659

9 Y 20 Asc = 2827.433388 mm^2

pc = 0.003490659 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

= 334 kNm > M* = 311.00 kNm OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 162 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 95 mm

what is the code ? 1 d = 405 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1275.306871 mm^2

Bottom 9 Y 20 Ast = 2827.433388 mm^2

Top 9 Y 20 Asc = 2827.433388 mm^2

Assume N - A in flange

Effective width : beff = 3080 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.03568312

hence dn = 14 < flange thickness
hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 417 kNm > M*+ve = 162.00 kNm OK

Summary of reinforcement requirement :

left support :	Ast	=	9	Y 20
	Asc	=	9	Y 20
Right support :	Ast	=	9	Y 20
	Asc	=	9	Y 20
midspan	Ast	=	9	Y 20
	Asc	=	9	Y 20

7 **STEEL CURTAILMENT REQUIREMENT CHECK**

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.
= 0.3 x le = 1620 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.
= 0.3 x le = 1620 mm

For +ve steel : i) 1/2 Ast must be carried into outer support
= 2 bars

ii) 1/4 Ast into interior support
= 2 bars

iii) Remainder curtailed at 0.1Ln from supports
= 0.1 x le = 540 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm   # of lines of bars = 2

a_b = 913 mm   distance between bars

If a_b > 2c   C = 2c + d_b = 170 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c   C = a_b + d_b = 933 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 913 mm   (dist between bars)

factor = 56006.24135

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 329.4484785 mm

take fs/Fsy = M*/Phi Mu = 0.916543

therefore

Lsy = 302 mm	≥	25k1db	625
say = 650 mm			

For Bottom Steel :

Cover, c = 75 mm d_b = 20 mm

# of lines of bars = 2

If ab > 2c   C = 2c + d_b = 170 mm

if ab < 2c   C = a_b + d_b = 1107 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 1087 mm   (dist between bars)

factor = 44804.99308

Lsyt = 263.5587828 mm   take fs/Fsy = M*/Phi Mu = 0.388702665

therefore

Lsy = 102 mm	≥	25k1db	500
say = 500 mm			

9 PUNCHING SHEAR CHECK

Column Dimension

V* = 96 kN   b = 450 mm

Dist to centreline of bottom   d = 450 mm

steel from soffit   Critical shear perimeter, μ

dq = 85 mm   μ = 2(b+dom) + 2(d+dom)

dom = 415 mm   3460 mm

Area of slab: B = 1000 mm   D = 1000 mm

βh = 1   longer side/shorter side

fcv = 0.17(1+2/βh)√f'c ≤ 0.34√f'c

fcv = 13.60 MPa

φVuo = 13,670 kN   >   V* = 96 kN   **OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	306.00 kNm	Es	=	200000 MPa
Ml	=	311.00 kNm	Ec	=	31975.35051 MPa
Mo	=	470.5 kNm			
Ms	=	162.00 kNm			
Ig	=	2.08E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	3080 mm	A_{sc} =	2827.433388 mm^2
webwidth	b_w =	2000 mm	A_{st} =	2827.433388 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	95 mm
			d_{st} =	405 mm
			d =	405 mm
			D =	500 mm

n = Es/Ec = 6.25481807

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1000 dn^2 + 32542.7295 dn - 8573934.546) = 0

dn = 77.7429526 mm

d = 405 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 2.21E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 316.227766 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 1.41E+11 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	2000 mm	A_{st} =	2827.433388 mm^2
d_{sc} =	85 mm	A_{sc} =	2827.433388 mm^2
d =	405 mm		

Ig = 2.08E+10 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1000 dn^2 + 32542.7295 dn - 8425358.066) = 0

dn = 76.95 mm

d = 405 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.21E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 316.227766 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^: = 2.28E+10 mm^4

At left end (arbitrary)

b_w = **2000** mm

d_{sc} = **85** mm

A_{st} = **2827.433388** mm^2

A_{sc} = **2827.433388** mm^2

d = **405** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1000 dn^2 + 32542.7295 dn - 8425358.066) = 0

dn = 76.9494236 mm

d = 405 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.21E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 316.227766 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.18E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 8.15E+10 mm^4

Midspan Deflection :

Lef = 5400 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.11689091 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 58.65 kN/m

w - short term = 58.725 kN/m

Defln,s,sus = 0.116742 mm

At midspan : Ast = 2827.433388 mm^2

Asc = 2827.433388 mm^2

Asc/Ast = 1

kcs = 2 - (1.2 x Asc/Ast) >= 0.8

= 0.8

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)

= 0.21028 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 500 = 10.8 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

SUMMARY

A. Roof

Purlin:	75 WD x 100 DP HWD @ 600 cts	
Truss @ 2500 cts		
Truss Members		
Top Chord:	2 x 75 WD x 150 DP HWD	
Bottom Chord:	2 x 75 WD x 150 DP HWD	
Diagonal	75 WD x 150 DP HWD	

B. Roof Beam: 300 DP x 300 WD RC Beam

Reinforcements:	3 - Y20	Tensile Steel
	3 - Y16	Compressive Steel
	Y12 - 300 cts	

C. First Floor Beam: 500 DP x 450 WD RC Beam

Reinforcements:	6 - Y20	Tensile Steel
	3 - Y16	Compressive Steel
	Y12 - 200 cts	

D. First Floor Slab: 250 thk RC two-way slab

Reinforcements:	Continuous Edge:	Y16 - 200 cts Top & Bottom Eachway
	Discontinuous Edge:	Y16 - 200 cts Top & Bottom Eachway
	Mid-span:	Y16 - 200 cts Top & Bottom Eachway
T & S Reinforcements:		Y12 - 200 cts Top & Bottom Eachway

E. Non-load Bearing Block Wall

200 thk masonry blocks

Reinforcements:	Vertical:	Y 16 - 400 CTS
	Horizontal:	Y 12 - 400 CTS

F. Load Bearing Block Wall

200 thk masonry blocks

Reinforcements:	Vertical:	Y 16 - 400 CTS
	Horizontal:	Y 16 - 400 CTS

G. RC Column: 450 x 450 SQ RC Column

Main Vertical Reinf'mt:	8 - Y24
	Y12 - 200 cts Ligs

H. Ground Slab: 200 thk RC slab

Reinforcements: F92 Mesh wire Top & Bottom

J. Isolated Column Footing: 400 DP x 1800 SQ RC Pad Footing

Reinforcements:	7 - Y20 Top & Bottom Width Wise
	7 - Y20 Top & Bottom Length Wise

K. Strip Footing: 400 DP x 1000 WD RC Strip Footing

Reinforcements:	6 - Y16	Width Wise
	Y12 - 300 cts Length Wise	

L. Base Slab

500 thk RC Slab		
Reinforcement:	Y20 bars @ 200 cts	Top & Bottom Each Way
T & S	Y20 bars @ 200 cts	Top & Bottom Each Way

M. Pit Wall

400 thk RC Wall	
Reinforcement;	
Vertical:	Y24 - 200 CTS
Horizontal:	Y16 - 200 CTS

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\SLTRBUIL	
Units system	Kilonewtons, Metres	
Nodes	115	(2500)
Members	253	(5000)
Restrained nodes	20	(2500)
Nodes with spring restraints	0	(2500)
Section properties	6	(100)
Material properties	2	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	40	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	15	(10000)
Member distributed forces	265	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	0	(100)
Combination load cases	5	(100)
Load cases with titles	9	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	432	
Total degrees of freedom	570	
Primary load cases	4	(100)
Mass load cases	0	(100)

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	SLAB	S1	Not applicable	No	Standard shape
[illegible]	400DP x 400WD	S2	Not applicable	No	Standard shape
	00 x 400 COL	S3	Not applicable	No	Standard shape
	00DP x 300WD	S4	Not applicable	No	Standard shape
5	75*5 EA	S5	Single	No	AUST300.LSS
6	125*75*6 RHS	S6	Not applicable	No	AUST300.LSS

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	4.0000E-02	2.2530E-04	1.3330E-04	1.3330E-04	INFINITE	INFINITE	0.00
2	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00
3	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00
4	9.0000E-02	1.1408E-03	6.7500E-04	6.7500E-04	INFINITE	INFINITE	0.00
5	6.7200E-04	5.2800E-09	1.4700E-07	5.6300E-07	INFINITE	INFINITE	45.00
6	2.1300E-03	4.4400E-06	1.8700E-06	4.1600E-06	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.200	0.200			
2	Rectangle	0.400	0.400			
3	Rectangle	0.400	0.400			
4	Rectangle	0.300	0.300			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+08	0.25	7.8500E+03	1.1700E-05	
2	STEEL	2.0000E+08	0.25	7.8500E+03	1.1700E-05	

COMBINATION LOAD CASES

Load case 5: 1.4D+1.7L

1.400 * Load case 1: D
1.700 * Load case 2: L

Load case 6: D+1.3L+1.3W

1.000 * Load case 1: D
1.300 * Load case 2: L
1.000 * Load case 3: W

Load case 7: 0.9D+1.3W

0.900 * Load case 1: D
1.000 * Load case 3: W

Load case 8: D+1.3L+E

1.000 * Load case 1: D
1.300 * Load case 2: L
1.300 * Load case 4: E

Load case 9: 0.9+E

0.900 * Load case 1: D
1.300 * Load case 4: E

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	W
4	E
5	1.4D+1.7L
6	D+1.3L+1.3W
7	0.9D+1.3W
8	D+1.3L+E
9	0.9+E

MEMBER FORCES AND MOMENTS (kN,kNm)
------------------------- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
77	5	922.176*	-1.051	-2.314	0.066	4.843	1.865
70	4	-31.707#	0.000	24.121	0.000	-89.351	0.000
37	5	6.253	309.974*	-0.089	0.569	0.188	-309.778
36	5	4.716	-251.141#	-0.186	2.010	-0.456	-292.285
64	8	308.384	-0.242	43.788*	0.035	-143.195	0.107
73	5	320.841	1.046	-11.948#	0.025	25.612	-2.337
83	5	4.035	85.357	0.037	9.253*	-0.065	-90.564
45	5	0.867	76.871	-0.177	-8.886#	0.445	-82.967
64	8	308.384	-0.242	43.788	0.035	141.426*	-1.468
65	8	287.431	-0.610	43.777	0.023	-143.214#	0.895
44	9	0.101	-21.018	-0.018	-1.530	0.085	84.506*
37	5	6.253	309.974	-0.089	0.569	0.188	-309.778#

NODE REACTIONS (kN,kNm)
-------------- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Nodes

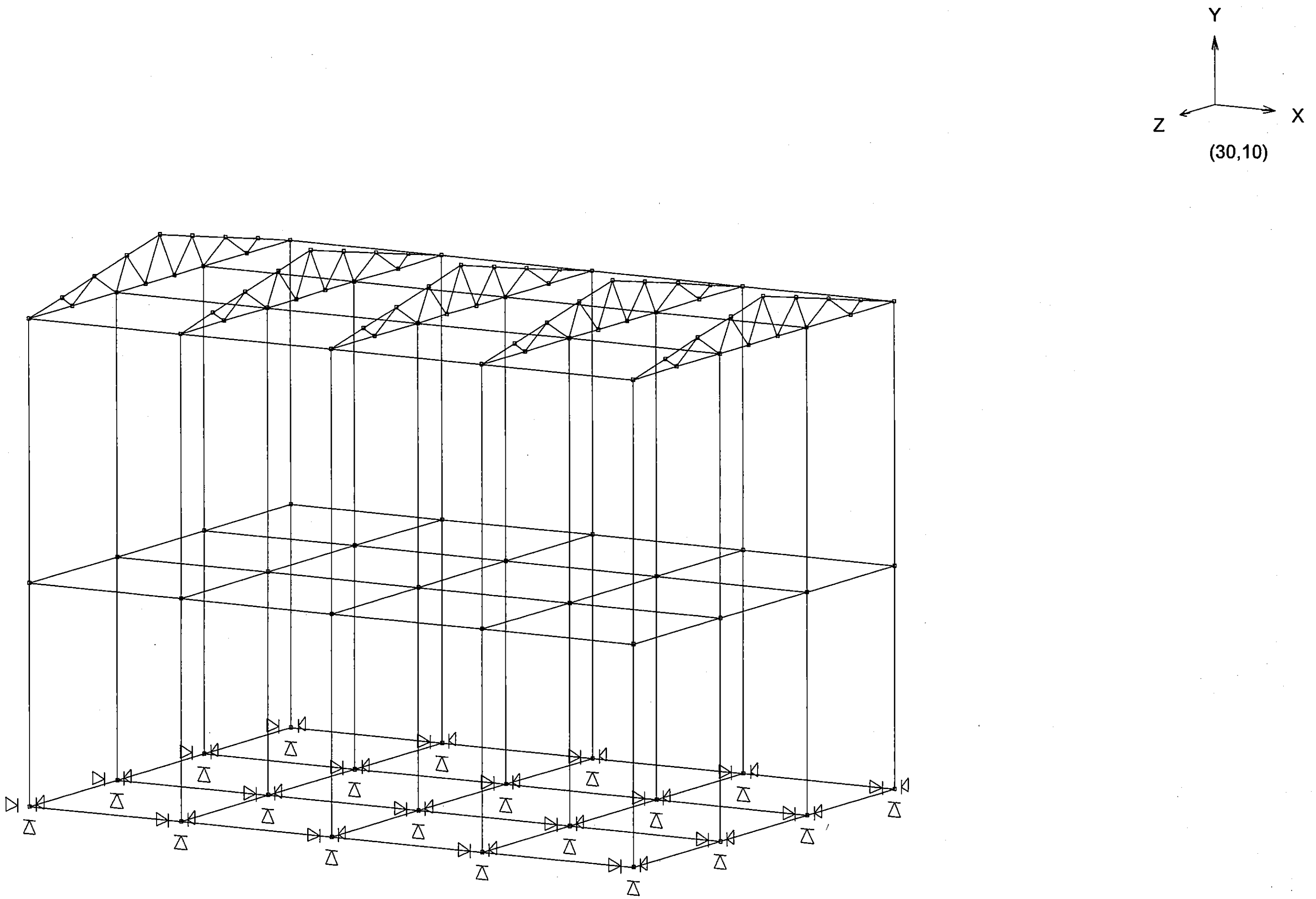
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
19	5	22.449*	435.795	-0.860	-1.546	0.073	-48.157
2	5	-15.534#	369.432	0.714	1.597	0.073	33.806
7	5	1.051	922.176*	-2.314	-4.843	0.066	-1.865
17	4	0.000	-31.707#	24.121	89.351	0.000	0.000
8	8	0.242	308.384	43.788*	143.195	0.035	-0.107
13	5	-1.046	320.841	-11.948#	-25.612	0.025	2.337
12	8	0.610	287.431	43.777	143.214*	0.023	-0.895
13	5	-1.046	320.841	-11.948	-25.612#	0.025	2.337
3	5	-13.410	346.678	-0.984	-2.085	0.086*	29.445
2	2	-5.171	132.035	0.432	0.937	-0.001#	11.189
2	5	-15.534	369.432	0.714	1.597	0.073	33.806*
19	5	22.449	435.795	-0.860	-1.546	0.073	-48.157#

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
------------------------------- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
77	5	922.176*	-1.051	-2.314	0.066	4.843	1.865
70	4	-31.707#	0.000	24.121	0.000	-89.351	0.000
37	5	6.253	309.974*	-0.089	0.569	0.188	-309.778
36	5	4.716	-251.141#	-0.186	2.010	-0.456	-292.285
64	8	308.384	-0.242	43.788*	0.035	-143.195	0.107
73	5	320.841	1.046	-11.948#	0.025	25.612	-2.337
83	5	4.035	85.357	0.037	9.253*	-0.065	-90.564
45	5	0.867	76.871	-0.177	-8.886#	0.445	-82.967
64	8	308.384	-0.242	43.788	0.035	141.426*	-1.468
65	8	287.431	-0.610	43.777	0.023	-143.214#	0.895
58	5	11.488	-21.914	0.157	0.862	0.118	212.777*
37	5	6.253	309.974	-0.089	0.569	0.188	-309.778#

8 D+1.3L+E



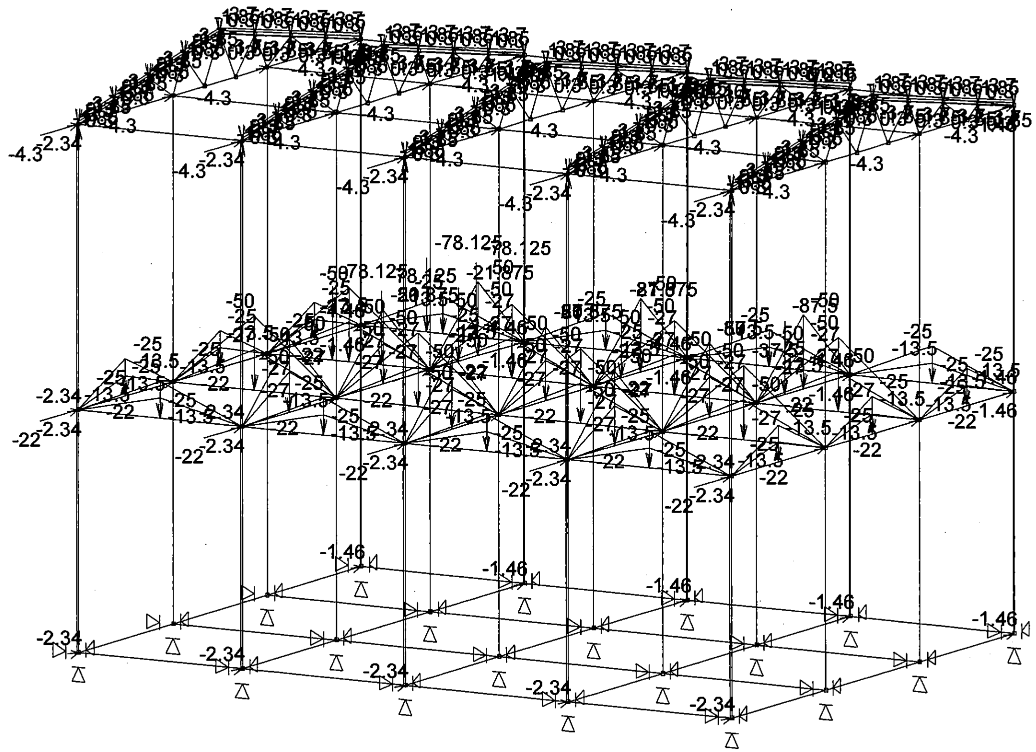
Sections:
1 SLAB
2 400DP x 400WD
3 400 x 400 COL
4 300DP x 300WD
5 75*5 EA

Sludge treatment Building Model

Job: SLTRBUIL, Designer: FYP, Units: m,kN,MPa, Scale: 1:200, Axes: XY
Load: 5 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Sludge Treatment Building 6 Nov 2011, 10:09 am

1	——	D
2	——	L
3	——	W
4	——	E
5	——	1.4D+1.7L
6	——	D+1.3L+1.3W
7	——	0.9D+1.3W
8	——	D+1.3L+E
9	——	0.9+E

Y
Z X
(30,10)



Sections:
1 SLAB
2 400DP x 400WD
3 400 x 400 COL
4 300DP x 300WD
5 75*5 EA

Model with Different Load Cases

Job: SLTRBUIL, Designer: FYP, Units: m,kN,MPa, Scale: 1:200, Axes: XY
Load: 5 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Sludge Treatment Building 6 Nov 2011, 10:09 am

POMSSUP
Sludge Treatment Bldg Pit

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\SLUDPIT	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	2	(10000)
Member distributed forces	4	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	6	
Total degrees of freedom	12	
Primary load cases	3	(100)
Mass load cases	0	(100)

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	500 x 1000	S1	Not applicable	No	Standard shape

S	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	5.0000E-01	2.8610E-02	4.1667E-02	1.0417E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.500	1.000			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	CONCRETE-40	3.1975E+07	0.15	2.4500E+03	1.0000E-05	40000.00

SELF WEIGHT (kN/kg)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
1	0.000	-0.010	0.000

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

1.000 * Load case 1: D
1.300 * Load case 2: L
1.000 * Load case 3: E

Load case 7: 0.9D+E

0.900 * Load case 1: D
1.000 * Load case 3: E

Load case 8: 1.4D+1.7L+1.7Q

1.400 * Load case 1: D
1.700 * Load case 2: L
1.700 * Load case 4: Q

Load case 9: 0.9D+1.7Q

0.900 * Load case 1: D
1.700 * Load case 4: Q

Load case 10: 1.4D+1.7L+1.4F

1.400 * Load case 1: D
1.700 * Load case 2: L
1.400 * Load case 5: F

Load case 11: 0.9D+1.4F

0.900 * Load case 1: D
1.400 * Load case 5: F

Load case 12: 1.4D+1.7L+E+1.7Q

1.400 * Load case 1: D
1.700 * Load case 2: L
1.000 * Load case 3: E
1.700 * Load case 4: Q

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	E
4	Q
5	F
6	D+1.3L+E
7	0.9D+E
8	1.4D+1.7L+1.7Q
9	0.9D+1.7Q
10	1.4D+1.7L+1.4F
11	0.9D+1.4F
12	1.4D+1.7L+E+1.7Q

MEMBER FORCES AND MOMENTS (kN,kNm)

(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	84.092*	-159.409	0.000	0.000	0.000	291.718
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
3	12	84.092	216.909*	0.000	0.000	0.000	-458.385
1	12	84.092	-216.909#	0.000	0.000	0.000	458.385
1	12	84.092	-216.909	0.000	0.000	0.000	458.385*
3	12	84.092	216.909	0.000	0.000	0.000	-458.385#

NODE REACTIONS (kN,kNm)

(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	216.909*	132.865	0.000	0.000	0.000	-505.533
3	12	-216.909#	132.865	0.000	0.000	0.000	505.533
1	8	159.409	132.865*	0.000	0.000	0.000	-338.866
1	3	57.500	0.000#	0.000	0.000	0.000	-166.667
3	12	-216.909	132.865	0.000	0.000	0.000	505.533*
1	12	216.909	132.865	0.000	0.000	0.000	-505.533#

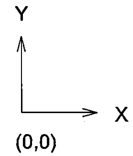
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)

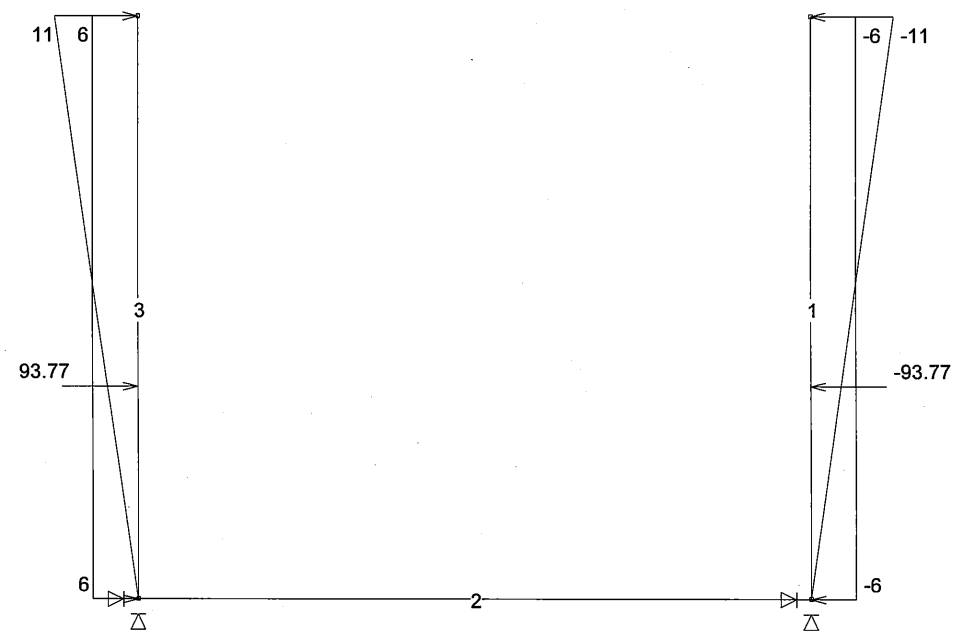
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	84.092*	-159.409	0.000	0.000	0.000	291.718
1	7	0.000#	0.000	0.000	0.000	0.000	0.000
3	12	84.092	216.909*	0.000	0.000	0.000	-458.385
1	12	84.092	-216.909#	0.000	0.000	0.000	458.385
1	12	84.092	-216.909	0.000	0.000	0.000	458.385*
3	12	84.092	216.909	0.000	0.000	0.000	-458.385#

1	———	(SW)	D
3	———		E
4	———		Q
6	———		D+1.3L+E
7	———		0.9D+E
8	———		1.4D+1.7L+1.7Q
9	———		0.9D+1.7Q
10	———		1.4D+1.7L+1.4F
11	———		0.9D+1.4F
12	———		1.4D+1.7L+E+1.7Q

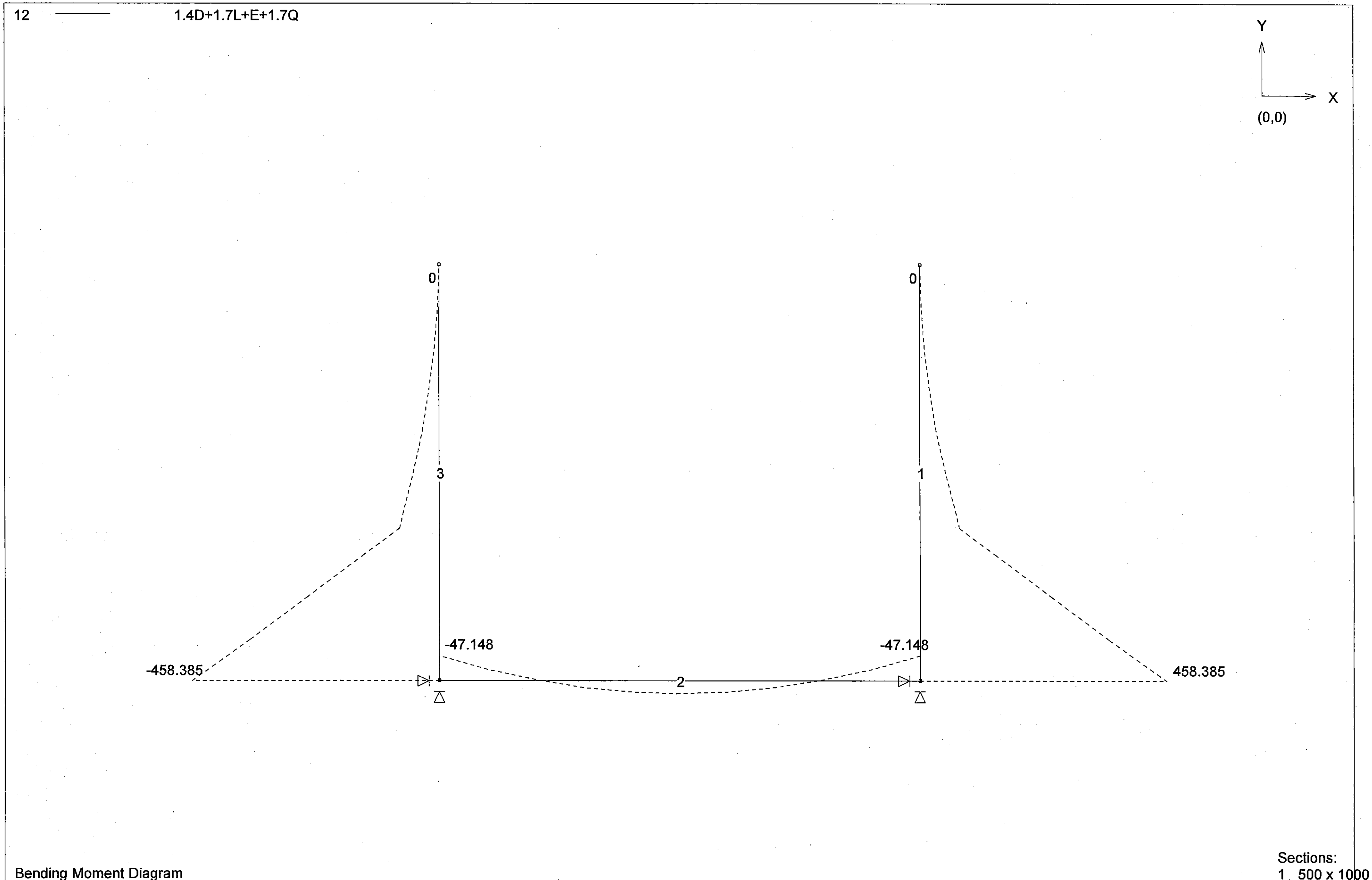




Sections:
1 500 x 1000

Sludge Treatmnt Bldg Pit Model

Job: SLUDPIT, Designer: FYP, Units: m,kN,MPa, Scale: 1:65, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Sludge Treatment Bldg Pit 13 Nov 2011, 1:11 pm



Bending Moment Diagram

Job: SLUDPIT, Designer: FYP, Units: m,kN,MPa, Scale: 1:65, Axes: XY
Load: 1 Disp: None Moment: 10 Shear: None Axial: None
POMSSUP
Sludge Treatment Bldg Pit 13 Nov 2011, 1:12 pm

K : Electrical Sub-Station

1. Load Breakdown and Calculation
2. Result of Structural Analysis
3. Design of Member
4. Summary

Load Breakdown & Calculation				
Items	Calculation & Description		Result	Unit
1	<u>Ground Floor Loads</u>			
A	DEAD LOADS			
1	Blockwall wt	=	4.6	kPa
2	300 thk plain concrete slab	=	7.2	kPa
3	150 thk RC slab	=	3.75	kPa
4	Services	=	0.05	kPa
5	Superimposed DL	=	0.5	kPa
			11.5	
B	LIVE LOADS			
1	Generator Room	=	5	kPa
2	Switch Gear Room	=	5	kPa
3	Transformer Room	=	5	kPa
4	Store Room	=	18	kPa
3	<u>Roof Loads (Trafficable Roof)</u>			
A	DEAD LOADS			
1	Colorbond Roof Sheeting	=	0.05	kPa
2	12THK Plasterboard Ceiling	=	0.29	kPa
3	Services	=	0.5	kPa
4	Superimposed DL	=	0.5	kPa
			1.34	kPa
B	LIVE LOADS	=	0.25	kPa

Wind Load Calculation for Auditorium

Items	Calculation & Description	Result	Unit
A	**Wind Load on Roof**		
1	Data		
	Return Period =	50	years
	Wind Velocity =	28	m/s
	Terrain Category =	1	
	Roof Mean Height,z =	4.9	m
	Width of Bldg =	9.3	m
	Length of Bldg =	13.6	m
	Roof Pitch, α =	3	Degrees
	Wind Angle, θ =	0	Degrees
2	Design Wind Velocity, V_z		
	V_z = ***1.35V(z/zg)^k*** **Notes for Table 4, PNGS 101-1982: Part 2**		
	k = 0.07 *zg* = 250		
	=	28.70	m/s
3	Dynamic Pressure, qz		
	q_z = $0.6V_z$^2 x10^-2 ... Dynamic Pressure ...	0.49	kPa
4	Design Pressure, P_z		
	p_z = Cpq_z ... Design pressure...		
	h/d = Table B2.2 α < 10° h/d ≥ 0.5	0.53	
	h/b =	0.36	
	C_p = ... Slope D ...	-0.9	
	C_p = ... Slope E ...	-0.9	
	p_z = ... Slope D ...	-0.44	kPa
	= ... Slope E ...	-0.44	kPa
5	UDL on roof truss		

Truss Spacing No.	Truss Spacing (m)	Truss No.	Tributary Area (m)	Windward (kN/m)			Leeward (kN/m)		
					x-Component	y-Component		x-Component	y-Component
1	0.9								
2	5	1	3.4	-1.513	-1.511	-0.081	-1.513	-1.511	-0.081
3	5	2	5	-2.225	-2.221	-0.119	-2.225	-2.221	-0.119
4	5	3	5	-2.225	-2.221	-0.119	-2.225	-2.221	-0.119
5	5	4	5	-2.225	-2.221	-0.119	-2.225	-2.221	-0.119
6	0.9	5	3.4	-1.513	-1.511	-0.081	-1.513	-1.511	-0.081

Wind Load on Wall

Items	Calculation & Description	Result	Unit
1 Data			
Return Period =		50	years
Wind Velocity =		28	m/s
Terrain Category =		1	
Roof Mean Height,z =		4.9	m
Width of Bldg =		9.3	m
Length of Bldg =		13.6	m
Roof Pitch, α =		3	Degrees
Wind Angle, θ =		90	Degrees

2 Pressure Coefficients, Cp

h/d	=	Table B1.1		0.53
h/b	=			0.36
d/b	=			0.68
C_p	=	... windward external wall surface ...	A	0.8
C_p	=	... leeward external wall surface ...	B	-0.5
	=	... side external wall surface ...	C	-0.6

3 q_z = $0.6V_z$^2 x10^-3 ... Dynamic Pressure ...

Height, z, (m)	Velocity Multiplier	Design Wind Velocity, Vz (m/s)	Dynamic Pressure, qz (kPa)
4.9	1.00	28	0.470

4 Design Pressure, P_z

P_z = Cpq_z

UDL Columns

Vertical Face of Wall at 90° to Wind Direction	Spacing, (m)	Column No.	Tributary Area (m)	Vertical Wind Load (UDL) Distribution on Columns, W = Cpqz x Tributary Area	
				Windward, (kN/m)	**Leeward, (kN/m)**
				GFL	**GFL**
1	4.4	1	2.2	0.828	**-0.517**
2	3.7	2	4.05	1.524	**-0.953**
3	5.5	3	4.6	1.731	**-1.082**
4		4	2.75	1.035	**-0.647**

Vertical Face of Wall at 90° to Wind Direction	**Spacing, (m)**	**Column No.**	**Tributary Area (m)**	**Side, (kN/m)**
				GFL
1	9.3	1	4.65	**-1.312**
2		2	4.65	**-1.312**

LATERAL LOAD DISTRIBUTION TO Block Walls

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

mass of wall = 4.6 KPa

FL LEVEL	Wall mass	floor area	Wt	Hi	Wt . Hi	Fi
ROOF	1.34	300	1538.4492	4.9	7538.40108	82.28505
SUMM			1538.4492		7538.40108	82.28505

V = Cd(T1) Wt from NZS 1170.5 - 2004

= factor . Wt

= 82.285054 138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T)	=	0.16
Z	=	0.13
R	=	1.80
N(T,D)	=	1.00
Sp	=	1.00
k	=	1.00
μ	=	0.70
factor	=	0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =

= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 23 KN/M^3 Note : ALL CELLS SHADED THUS

t = 0.2 m REQUIRE A MANUAL INPUT

h = 4.9 m

Wall Mass = M x t x h x L = 22.54 L (kN/m)

Floor load (1) = dl + ll/3 = 0 Kpa

Floor load (2) = dl + ll/3 = 0 Kpa

Floor load (3) = dl + ll/3 = 0 Kpa

Floor load (4) = dl + ll/3 = 1.34 Kpa

Wall	LENGTH	MASS (m)	x	y	mx	my
1	1.3	29.302	0	0.6	0	17.5812
2	4.2	94.668	0	5.2	0	492.2736
3	1.5	33.81	0	9.2	0	311.052
4	3	67.62	4.4	9.2	297.528	622.104
5	6.6	148.764	4.4	4.4	654.5616	654.5616
6	3	67.62	4.4	1.75	297.528	118.335
7	3.8	85.652	8	7.1	685.216	608.1292
8	4.7	105.938	8	2.35	847.504	248.9543
9	5.5	123.97	10.3	9.2	1276.891	1140.524
10	9.3	209.622	13.5	4.65	2829.897	974.7423

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	126.48	169.4832	6.8	4.65	1152.48576	788.09688
SUM	Wt =	1136.4492			8041.61136	5976.35408

TABLE 1

CENTRE OF MASS	Cmx =	7.0760852	Cmy =	5.25879562

CENTRE OF RIGIDITY

Wall	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	1.961	0.003	0.000	0.002	8.229	8.229
2	1.961	0.003	0.000	0.017	8.229	8.229
3	1.961	0.003	0.000	0.030	8.229	8.229
4	1.961	0.003	8.628	0.030	8.229	8.229
5	1.961	0.003	8.628	0.014	8.229	8.229
6	1.961	0.003	8.628	0.006	8.229	8.229
7	1.961	0.003	15.687	0.023	8.229	8.229
8	1.961	0.003	15.687	0.008	8.229	8.229
9	1.961	0.003	20.196	0.030	8.229	8.229
10	1.961	0.003	26.471	0.015	8.229	8.229
SUM	19.608	0.033	103.923	0.175	82.285	82.285

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	5.3 m	Cry	=	5.365 m
ex	=	1.776085 m	ey	=	-0.106204 m

check code eccentricities :

please enter b in x - direc'n = 13.6 m
please enter b in y - direc'n = 9.3 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	4.08 m	**ex OK**	however torsional
esy <=	0.3by =	2.79 m	**ey OK**	shears not large

edx1	=	3.344128 m	edy1	=	-0.624307 m		(ed = 1.5es + 0.05b)
edx2	=	1.096085 m	edy2	=	0.358796 m	or	(ed = es - 0.05b)

Wall	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	8.34412779	-15.83	137.34	0.000445	-0.000256	0.75	0.25
2	13.3441278	-15.83	349.97	0.000445	-0.000256	1.21	0.40
3	13.3441278	-15.83	349.97	0.000445	-0.000256	1.21	0.40
4	20.8441278	-15.83	852.75	0.000445	-0.000256	1.88	0.62
5	23.3441278	-15.83	1069.36	0.000445	-0.000256	2.11	0.69
6	23.3441278	-15.83	1069.36	0.000445	-0.000256	2.11	0.69
7	23.3441278	-15.83	1069.36	0.000445	-0.000256	2.11	0.69
8	23.3441278	-15.83	1069.36	0.000445	-0.000256	2.11	0.69
SUMM			5967.48				

TABLE 3

FINAL LOAD DISTRIBUTION

Wall	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	8.2	8.2	9.0	8.5	8.2	9.0
2	8.2	8.2	9.4	8.6	8.2	9.4
3	8.2	8.2	9.4	8.6	8.2	9.4
4	8.2	8.2	10.1	8.8	8.2	10.1
5	8.2	8.2	10.3	8.9	8.2	10.3
6	8.2	8.2	10.3	8.9	8.2	10.3
7	8.2	8.2	10.3	8.9	8.2	10.3
8	8.2	8.2	10.3	8.9	8.2	10.3
SUMM					65.8	79.3

TABLE 4

Torsional Moments	
kNm	kNm
27.5186624	-5.608118
27.5186624	-5.890357
27.5186624	-5.890357
27.5186624	-6.313716
27.5186624	-6.454836
27.5186624	-6.454836
27.5186624	-6.454836
27.5186624	-6.454836

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

Reference **Bowles, J.E Foundation Analysis & Design**
McGrall Hill, Third Edition, 1982

Design data: Compacted Material

B = 1 m, D = 0.15 m
μ = 0.4 MPa, F'c = 40 MPa
Es = 180 MPa, ρ = 2400 kg/m^3
π = 3.141593

$k's = 0.65 \sqrt[12]{(Es\ B^4)/(Ef\ If)} * (Es/(1 - \mu^2))$

--> $\lambda L < \pi/4$ Rigid member - Bending not influenced much by ks
$\lambda L > \pi$ Flexible member - Bending heavily localised

$ks = k's\ B$

$\lambda = \sqrt[4]{(k's / 4Ec\ I)}$

Es, Ef = Modulus of soil and footing respectively
B, If = Footing width and its moment of inertia based on cross section
ks = Modulus of subgrade reaction

Procedure :Conventional rigid procedure (CRP) or Beam on Elastic Foundation (Winkler)

Section	B m	I m^4	Ec GPa	Es GPa	μ MPa	k's	L m	λ	λL	Is $\lambda L < \pi/4$	Is $\lambda L > \pi$	Analysis method ?	ks = k'sB kNm
1	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	4.90	1.7730	8.6877	No	Yes	Use Winkler	355472
2	1.50	4.22E-04	31.9754	0.18	0.4	0.19784	4.90	1.3838	6.7805	No	Yes	Use Winkler	131894

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Section 1	S1	Not applicable	No	Standard shape
2	Section 2	S2	Not applicable	No	Standard shape

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	2.4000E-01	7.5125E-03	3.2000E-03	7.2000E-03	INFINITE	INFINITE	0.00
2	2.2500E-01	1.5812E-03	4.2188E-02	4.2190E-04	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.600	0.400			
2	Rectangle	0.150	1.500			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	CONCRETE-40	3.1975E+07	0.15	2.4500E+03	1.0000E-05	40000.00

SELF WEIGHT (kN/kg)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
1	0.000	-0.010	0.000

COMBINATION LOAD CASES

Load case 3: 1.4D+1.7L

1.400 * Load case 1: D
1.700 * Load case 2: L

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	1.4D+1.7L

MEMBER FORCES AND MOMENTS (kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
14	3	0.000	53.990*	0.000	-5.699	0.000	-19.586
37	3	0.000	-89.172#	0.000	0.563	0.000	-33.952
3	3	0.000	1.882	0.000	6.111*	0.000	1.691
14	3	0.000	53.990	0.000	-5.699#	0.000	-19.586
37	3	0.000	-64.882	0.000	0.563	0.000	30.646*
37	3	0.000	-89.172	0.000	0.563	0.000	-33.952#

NODE REACTIONS (kN,kNm)
(*=Maximum, #=Minimum)

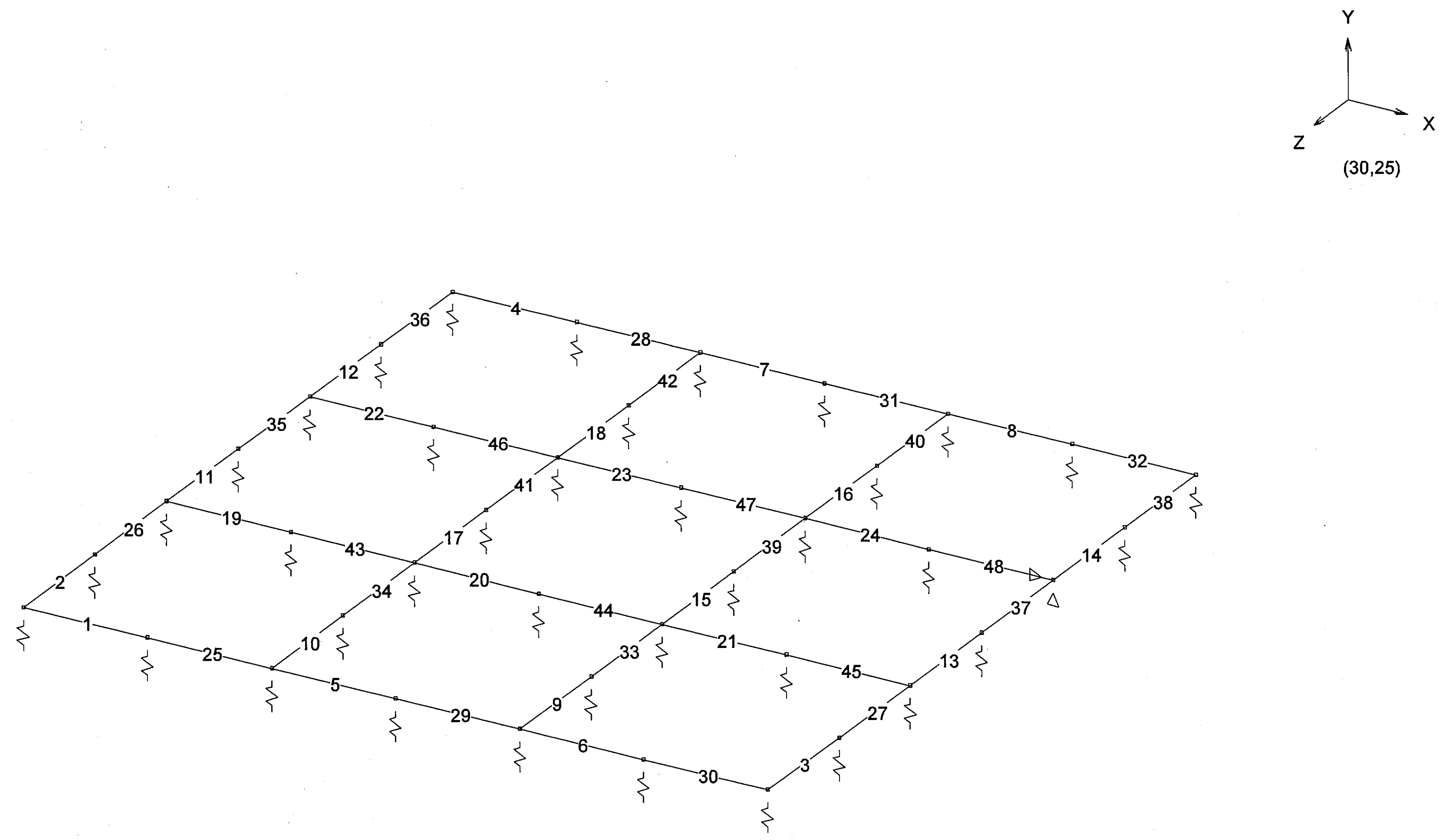
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
12	3	0.000	164.661*	0.000	-14.366	0.000	0.000
30	2	0.000	0.420#	0.000	3.157	0.000	0.000
0	3	0.000	3.107	0.000	25.523*	0.000	0.000
)	3	0.000	5.573	0.000	-55.262#	0.000	0.000

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

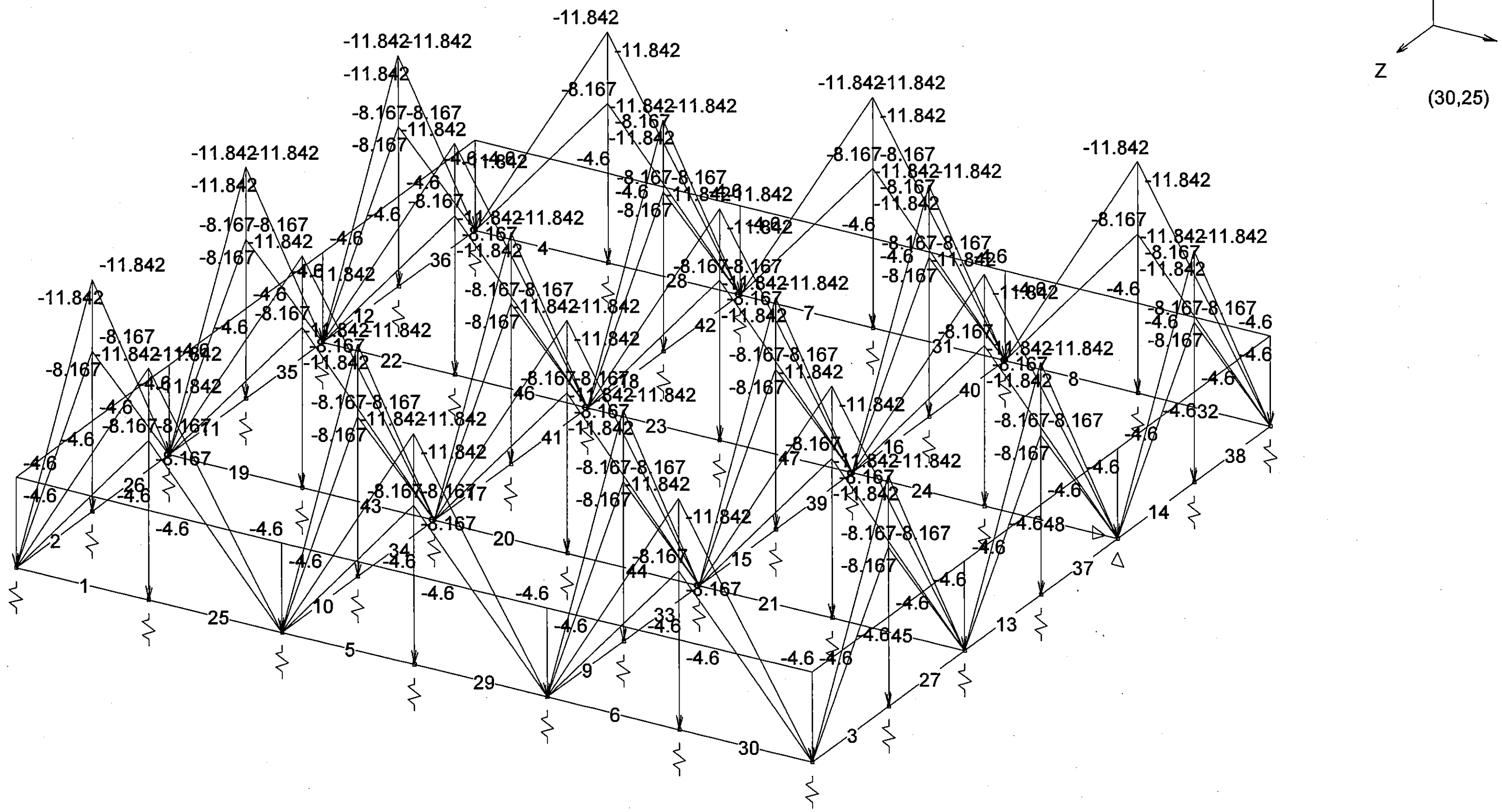
Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
14	3	0.000	53.990*	0.000	-5.699	0.000	-19.586
37	3	0.000	-89.172#	0.000	0.563	0.000	-33.952
3	3	0.000	1.882	0.000	6.111*	0.000	1.691
14	3	0.000	53.990	0.000	-5.699#	0.000	-19.586
37	3	0.000	-64.882	0.000	0.563	0.000	30.646*
37	3	0.000	-89.172	0.000	0.563	0.000	-33.952#



Ground Slab Model

Job: ESUBSTN, Designer: FYP, Units: m,kN,MPa, Scale: 1:39, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
POMSSUP
Electrical Sub Station 18 Oct 2011, 9:12 am

1 D
2 L
3 1.4D+1.7L



Ground Slab Model with Loadings

Job: ESUBSTN, Designer: FYP, Units: m,kN,MPa, Scale: 1:39, Axes: XY
Load: 0.34 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Electrical Sub Station 18 Oct 2011, 9:13 am

Timber Roof Purlin Design

REF: PNGS 1292 - 1989

TABLE 2.2

STRENGTH CLASSIFICATION

Species	Moisture Condition	Strength Group(2)	Stress Grade
Hardwood	Seasoned	SD3	F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

Stress grade	Basic working stress, MPa				Modulus of Elasticity (E)
	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	
F11	11.00	6.60	8.4	1.05	10500

TABLE A

MODIFICATION FACTORS

Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp°C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'...Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' Eq.3.21

= 5.94 MPa

Purlin Design

Loads

DL = 0.09 kN/m LL = 0.225 kN/m

Combination: DL + LL = 0.315 kN/m

Size B (mm) D (mm) Purlin Span = 1200 mm Purlin Spacing = 900 mm cts

50 75 HWD

From Analysis

M*max	=	0.06	kNm
V*max	=	0.19	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 1.2096 MPa

Working Shear Stress, fs = 3V/2BD

= 0.0756 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.091866 < 1 OK

fs = 0.0756 MPa < Fs = 0.945 MPa OK

Deflection Check!

From Analysis

DL = 0.09 kN/m

LL = 0.225 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 0.131657 mm Def. by DL

Allowable Defln = L/300 4 mm > 0.131657 = Defln.DL OK

Defln.LL = 0.329143 mm Def. by LL

Allowable Defln = L/360 3.33333333 mm > 0.329143 = Defln.LL OK

Timber Roof Rafter Design

REF: PNGS 1292 - 1989

TABLE 2.2

STRENGTH CLASSIFICATION			
Species	Moisture Condition	Strength Group(2)	Stress Grade
Hardwood	Seasoned	SD3	F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

	Basic working stress, MPa				
Stress grade	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	Modulus of Elasticity (E)
F11	11.00	6.60	8.4	1.05	10500

TABLE A

MODIFICATION FACTORS

Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp°C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'…………………………………………………Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' ……………………………………………………….Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' ……………………………………………………….Eq.3.21

= 5.94 MPa

Rafter Design

Loads

DL = 1.608 kN/m LL = 0.3 kN/m

Combination: DL + LL = 1.908 kN/m

Size B (mm) D (mm) Rafter Spacing = 1200 mm CTS Rafter Span = 4.5 m

150 150 HWD

From Analysis

M*max	=	4.83	kNm
V*max	=	4.29	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 8.586 MPa

Working Shear Stress, fs = 3V/2BD

= 0.2862 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.652084757 < 1 OK

fs = 0.2862 MPa < Fs = 0.945 MPa OK

Deflection Check!

From Analysis

DL = 1.608 kN/m

LL = 0.3 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 1.93821E-11 mm Def. by DL

Allowable Defln = L/300 15 mm > 1.93821E-11 = Defln.DL OK

Defln.LL = 3.61607E-12 mm Def. by LL

Allowable Defln = L/360 12.5 mm > 3.61607E-12 = Defln.LL OK

Reinforced Masonry Block Wall Design

REFERENCE: **Concrete Structures - 3rd Edition** **AS 3600 - 2009**
R. F. Warner, B. V. Rangan, & A. S. Hall

Step	Calculation & Description				Result	Unit
1	Data					
	N*.max	=			**79**	kN
	M*.max	=			**27**	kNm
	V*.max	=			**79**	kN
	f'c	=			**15**	MPa
	fsy	=			**410**	MPa
	εsy				**0.002**	
	Lw	=	... overall length of wall ...		**9300**	mm
	Hw	=	... unsupported height of wall ...		**4900**	mm
	tw	=	... wall thickness ...		**200**	mm
	Hwe	=	0.8 Hw, effective wall height		3920	mm
	ChecK: Hwe/tw	=	19.6	<	30	**OK**
2	Check maximum shear strength					
	dw	=	0.8Hw		3920	
	ϴ	=	tan-1(dw/Hw)		0.674741	radians
	ϴ	=		30° ≤ ϴ ≤ 60°	38.7	degrees
	k3	=	(0.6+10/f'c) ≤ 0.85		1.267	
	k3f'c	=			19	MPa
	Vu.max	=	k3f'ctwdwsinϴcosϴ/(1.14+0.68cot^2ϴ)		**3299**	kN
	ϴvu.max	=			2309	kN
	ϴvu.max	>	V* **OK**			
3	Vertical steel in the wall					
	Gross concrete area of the wall cross-section					
	Ag	=			784000	mm^2
	N*/Ag	=			0.10	MPa
	In design, substituting for Vu from ϕVu ≥ V*, we obtain;					
	ϕlfsy	≥	V*cotϴ/ϕtwdw - N*/Ag	ϕ = 0.7		
		≥			0.08	MPa
	Al	≥	(ϕfsy/fsy)(twLw)	359.1739 mm^2	**360**	mm^2
	AsV	=	Al		360	mm^2
	Using **Y16**		bars area/bar = **200** mm^2			
	Now, minimum area vertical reinforcement, Ast.min = 0.0015twLw =				4464	mm^2
	Minimum Reinforcement Requirment Governs					
	# of bars	=	22.32		**23**	bars
	23 bars at each wall face and the average spacing is at;					
	Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		404.3478	mm CTS
	minimum bar spacing = 2.5tw or 500 mm whichever is less =				500	mm CTS
	Use Y12 bars at		**400** mm CTS at each wall face.			
	# of bars	=	2(1+ (Lw - 2b-Ast/bar)/spacing)		23.25	bars
	AsV (provided) =				4650	mm^2
	Therefore;					
	pl	=	[(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t		0.003	
	It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.					
	In this case;					
	pl	=	0.0029 > 0.0025		**OK**	
4	Horizontal steel in the wall					
	For adequate control of cracking due to restrained shrinkage and temperature effects, the minimum value of horizontal steel ratio pt is taken as;					
	pt	=			0.0012	
	Ast.min	=	0.0015 twHw		1176	mm^2
	For **Y12**		bars, area/bar = **110** mm^2			
	# of bars	=	10.69091		**11**	bars
	11 bars at each wall face and the average spacing is at;					
	# of bars	=			11	bars
	Spacing of bars	=	(Hw)/(# bars @ each wall face)		445	mm CTS
	Therefore, use Y16 bars		400 mm CTS at each wall face.			

5 Check flexural strength of wall

The flexural strength of the wall is calculated using an equivalent rectangular stress block for compression zone concrete, like columns subjected to axial compression and bending moment.

Calculate rectangular stress block parameters.

α	=	0.85-0.004(f'c-55)			1.01	
γ	=	0.85-0.008(f'c-30)	0.65	<	0.97	
Take γ	=				0.97	
Nu	=	N*/ϕ	ϕ = 0.6		132 kN	135
Trial depth of neutral axis, dn	=				733 mm	

Compressive force in concrete, Cc : 0.85γf'cbdr (include boundary element) 1813 kN

dc = γdn/2 356 mm

(0.003/dn) (dn - dsi)

Raw #	# of bars	Asi (mm^2)	bar cts (mm	dsi (mm)	εsi	δsi (MPa)	Fsi (kN)	Fsidsi (kNmm)	M = Fsi(dq-dsi) (kNmm)
1	1	200	100	100	0.0026	410	82	8200	373100
2	1	200	200	300	0.0018	354	71	21266.03	308357
3	1	200	400	700	0.0001	54	11	7563.438	42679
4	1	200	400	1100	-0.0015	-601	-120	-132180.1	-426581
5	1	200	400	1500	-0.0031	-410	-82	-123000	-258300
6	1	200	400	1900	-0.0048	-410	-82	-155800	-225500
7	1	200	400	2300	-0.0064	-410	-82	-188600	-192700
8	1	200	400	2700	-0.0081	-410	-82	-221400	-159900
9	1	200	400	3100	-0.0097	-410	-82	-254200	-127100
10	1	200	400	3500	-0.0113	-410	-82	-287000	-94300
11	1	200	400	3900	-0.0130	-410	-82	-319800	-61500
12	1	200	400	4300	-0.0146	-410	-82	-352600	-28700
13	1	200	400	4700	-0.0162	-410	-82	-385400	4100
14	1	200	400	5100	-0.0179	-410	-82	-418200	36900
15	1	200	400	5500	-0.0195	-410	-82	-451000	69700
16	1	200	400	5900	-0.0211	-410	-82	-483800	102500
17	1	200	400	6300	-0.0228	-410	-82	-516600	135300
18	1	200	400	6700	-0.0244	-410	-82	-549400	168100
19	1	200	400	7100	-0.0261	-410	-82	-582200	200900
20	1	200	400	7500	-0.0277	-410	-82	-615000	233700
21	1	200	400	7900	-0.0293	-410	-82	-647800	266500
22	1	200	400	8300	-0.0310	-410	-82	-680600	299300
23	1	200	400	8700	-0.0326	-410	-82	-713400	332100
24	1	200	400	9100	-0.0342	-410	-82	-746200	364900
25	1	200	200	9300	-0.0351	-410	-82	-762600	381300
∑	25	5000	100	9400			-1678	-9549751	1744856

4 Axial Strength of the wall;

Nu	=	∑Fsi + Cc		[redacted] kN
dN	=	1/Nu (Ccdc + ∑Fsidsi)		-66158.72 mm
dq	=	0.5D	... plastic centroid ...	4,650 mm
e	=	dq-dN	... eccentricity ...	70,809 mm

Summing up moments of forces about the palsic centroid gives;

Mu	=		9,531 kNm
ϕMu	=		5,719 kNm
e	=	Mu/Nu	70,809 mm
Therefore, Nu	=		135 kN

ϕMu = 5,719 kNm > M* = 27 kNm

Therefore, the flexural strength of the wall crosection is adequate to carry the design bending moment

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

REFERENCE: **Concrete Structures - 3rd Edition** **Australian Standard (AS) 3600 - 1988**
R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	33.60 kN/m	(plus super)
Self wt	:	Wsw =	43.88 kN/m	
Blockwall	:	Wb =	0.00 kN/m	
TOTAL		=	77.48 kN/m	
LL :		Wll =	0.25 kN/m	
LL factor	- long term		0.4	
	- short term		0.7	

2 Beam Section Properties

Beam Size

bw =	3900 mm	D =	450 mm
tff =	0 mm	Cover =	65 mm
Actual length, L =	9300 mm	Overall depth of sect. D =	450 mm
Effective Length,Lef =	8900 mm	Area, A =	1.76E+06 mm^2
Neutral Axis, yc =	225.00 mm	Moment of Inertia, Ix-x =	2.96E+10 mm^4
Neutral Axis, xc =	2840.00 mm	Moment of Inertia, Iy-y =	2.22E+12 mm^4
Effective width, beff =	5680.00 mm	Gross Area, Ag =	1.76E+06 mm^2

3 Design Parameters

F'c =	32 MPa	Fsy =	410 MPa
m =	2400 kg/m^3	D =	450 mm
Ec =	28599.623 MPa	Es =	200000 MPa
Gamma =	0.822		

4 Initial Choice of Section

Effective width	=	5680 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)			=	1	
k_1	=	0.045	(RECT sections)		1
	=	0.033464	(T and L sections)		2
what type of span?		5	interior span =		5
			end span =		6
k_2	=	0.002604			
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef	=	127.398			
defln/Lef	=	0.004	k_1/k_2 =	17.279779	
d	=	201 mm			
take d =		373 mm	Distance from d to extreme fibre of beam (do) =	77 mm	

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	25.00 kNm	(left support)
Mmax +ve	=	25.00 kNm	(mid-span)
M -ve	=	25.00 kNm	(right support)
Vmax	=	65.00 kN	
V (other end)	=	65.00 kN	
N*	=	0.00 kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 1

take do = 77 mm d = 373

Estimate Area of main steel Ast = 213.69083 mm^2

2 Y 12 Ast = 226.19467 mm^2

p = 0.0001555

2 Y 12 Asc = 226.19467 mm^2

pc = 0.0001555 < pd = 0.02181

p-pc = 0 pd=0.4*0.85*µ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 71 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

= 25.21 kNm > M* = 25.00 kNm	OK

Determine - ve steel at right support :

M -ve = 25.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 77 mm dst = 373 mm

Estimate Area of main steel Ast = 213.69083 mm^2

2 Y 12 Ast = 226.19467 mm^2

p = 0.0001555

2 Y 12 Asc = 226.19467 mm^2

pc = 0.0001555 < pd = 0.02181

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 71 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

= 25 kNm > M* = 25.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 25 kNm

Steel in one layer then code = 1 take dist between centroid of

" " two " " code = 2 bar/s and soffit = 77 mm

what is the code ? 1 d = 373 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 213.69083 mm^2

Bottom 3 Y 12 Ast = 339.292007 mm^2

Top 3 Y 12 Asc = 339.292007 mm^2

Assume N - A in flange

Effective width : beff = 5680 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.002937

hence dn = 1 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 47 kNm > M*+ve = 25.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	2	Y 12
	Asc	=	2	Y 12
Right support :	Ast	=	2	Y 12
	Asc	=	2	Y 12
midspan	Ast	=	3	Y 12
	Asc	=	3	Y 12

7 **STEEL CURTAILMENT REQUIREMENT CHECK**

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 2670 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 2670 mm

For +ve steel : i) 1/2 Ast must be carried into outer support

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 890 mm

8 **STRESS DEVELOPMENT**

For Top Steel :

cover : c = 65 mm

d_b = 12 mm # of lines of bars = 2

a_b = 1873 mm distance between bars

If a_b > 2c C = 2c + d_b = 142 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 1885 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 113.09734 mm^2

k2 = 2.2 a_b = 1873 mm (dist between bars)

factor = 22542.077

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 158.74702 mm

take fs/Fsy = M*/Phi Mu = 0.9918

therefore

Lsy	=	157 mm
say	=	650 mm

≥ 25k1db 375

For Bottom Steel :

Cover, c = 65 mm d_b = 12 mm

# of lines of bars = 2

If ab > 2c C = 2c + d_b = 142 mm

if ab < 2c C = a_b + d_b = 2039 mm

k1 = 1 A_b = 113.09734 mm^2

k2 = 2.2 a_b = 2027 mm (dist between bars)

factor = 18033.662

Lsyt = 126.99762 mm take fs/Fsy = M*/Phi Mu = 0.5359888

therefore

Lsy	=	68 mm
say	=	500 mm

≥ 25k1db 300

9 PUNCHING SHEAR CHECK

			Coloumn Dimension		
V^* =	80.4 kN		b =	400 mm	
Dist to centreline of bottom			d =	400 mm	
steel from soffit			Critical shear perimeter, μ		
dq =	71 mm		μ = 2(b+dom) + 2(d+dom)		
dom =	379 mm			3116 mm	
Area of slab:	B = 1000 mm		D =	1000 mm	
βh =	1	longer side/shorter side			
fcv =	$0.17(1+2/\beta h)\sqrt{f'c} \le 0.34\sqrt{f'c}$				
fcv =	10.88 MPa				
ϕVuo =	8,994 kN	>	V^* =	80.4 kN	OK

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	25.00 kNm	Es	=	200000 MPa
Ml	=	25.00 kNm	Ec	=	28599.623 MPa
Mo	=	50 kNm			
Ms	=	25.00 kNm			
Ig	=	2.96E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	**b_{eff}** =	5680 mm	**A_{sc}** =	339.29201 mm^2
webwidth	**b_w** =	3900 mm	**A_{st}** =	339.29201 mm^2
Flange thickness	**t_{ff}** =	0 mm	**d_{sc}** =	77 mm
			d_{st} =	373 mm
			d =	373 mm
			D =	450 mm

n = Es/Ec = 6.993099

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1950 dn^2 + 4406.113 dn - 1041590.7) = 0

dn = 22.00949 mm

d = 373 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 3.06E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 446.750064 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 1.67E+14 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	3900 mm	**A_{st}** =	226.19467 mm^2
d_{sc} =	71 mm	**A_{sc}** =	226.19467 mm^2
d =	373 mm		

Ig = 2.96E+10 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1950 dn^2 + 2937.409 dn - 686260.16) = 0

dn = 18.02 mm

d = 373 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.07E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 446.750064 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mı = 1.68E+14 mm^4

At left end (arbitrary)

b_w = 3900 mm

d_{sc} = 71 mm

A_{st} = 226.19467 mm^2

A_{sc} = 226.19467 mm^2

d = 373 mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

1950 dn^2 + 2937.409 dn - 686260.16) = 0

dn = 18.02168 mm

d = 373 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.07E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 446.750064 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / MI)^3 = 1.68E+14 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.68E+14 mm^4

Midspan Deflection :

Lef = 8900 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 3.44E-05 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 77.525 kN/m

w - short term = 77.6 kN/m

Defln,s,sus = 3.4E-05 mm

At midspan : Ast = 339.292007 mm^2

Asc = 339.292007 mm^2

Asc/Ast = 1

kcs = 2 - (1.2 x Asc/Ast) >= 0.8

= 0.8

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)

= 0.00006 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 500 = 17.8 mm

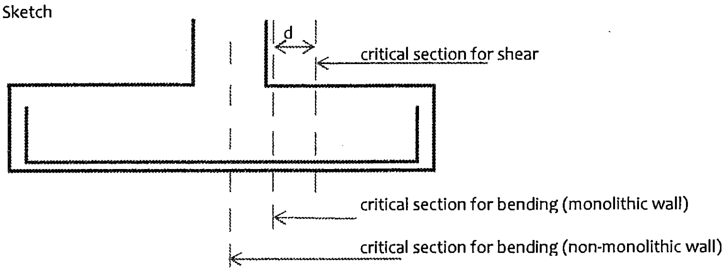
> 0.9 x Total Defln

THE SECTION IS ADEQUATE

STRIP FOOTING DESIGN FOR WALL

REFERENCE: **Concrete Structures - 3rd Edition**
R. F. Warner, B. V. Rangan, & A. S. Hall
Australian Standard (AS) 3600 - 1988

Step	Calculations & Description				Result	Unit
	Sketch					



Step	Calculations & Description				Result	Unit
	Strip Footing for Wall					
1	Data	RC WALL		kN/m^3 mm		
	UDL:	Self wt	=	Density x Thkness 23 200	46	kPa/m
		DL	=		3.015	kN/m
	L		=	span	1	m
	Total DL		=		49.015	kN/m
		LL	=		18	kN/m
	Conc. Load	DL	=			
		LL	=			
	P		=	DL + LL	67.015	kN
	P*		=	1.4 DL + 1.7 LL	99.221	kN
	qa		=		450	kPa
	f'c		=		32	MPa
	fsy		=	... reinforcement...	410	MPa
	c		=	... width of wall	0.2	m
2	Footing Area, A					
	A		=	P*total/qa	0.1489222	m^2
	Approximate the footing dimension as:					
	D		=	A/L	0.1489222	m
			=		[redacted]	m
	Mmax		=	1/8 P*(D-c)	4.96105	kNm
	qult		=	P^*_{total}/A	0.1653683	MPa
3	Determine the required effective depth by considering the beam shear on a critical section at distance d from the c mm face of the wall.					
	V*		=	qult x B (C-Col.w/2-d)		
	V*		`	19844.2 - 99.221 d		N
	The shear strenght of the section depends on the amount of tensile steel used for bending. We assume a minimum quatity of:					
	pmin		=	1.4/fsy	0.0034146	
	V_{uc}		=	β_1BD(Ast/BD x f'c)^1/3		
	V_{uc}		=	315.5309667 d		N
	Equating V* & φVuc					
	220.8717 d		=	19844.2 - 99.221 d		
	320.0927 d		=	19844.2		
	d		=		61.99517	mm
	Cover		=		75	mm
	Assume bar dia			Y16	16	mm
	D		=	d + cover + bar dia/2	144.99517	mm
	Take D		=		[redacted]	mm
	d		=	D-cover-bar dia/2	317	mm
4	Tensile steel for bending in longitudinal direction: Oneway Bending					
	The critical section for bending is at the longer face of the column, where the length of the cantilevered portion is:					
	L		=	0.5(B-c)	0.4	m
	M*		=	qult x B x L/2	7.93768	kNm
	Ast.reqd.		=	M*/φ0.9dfsy	84.823848	mm^2
	Ast.min.		=	1.4/fsy bd	649.46341	mm^2
				Ast.min. Governs, Therefore, use Ast.min.		
	Hence provide			4 Y16 bars in longitudinal direction top & bottom.		
	Astprov		=	200 Ast/bar	800	mm^2
6	Spacing between the bars					
	Ast/bar		=		200	mm^2
	S		=	[Ast/bar X beff]/Ast.prov. ... centre - to - centre spacing ...	250	mm

SUMMARY

A. Roof

Purlin: 50 WD x 50 DP HWD @ 600 cts

Rafter: 75 WD x 150 DP HWD @ 900 cts

B. Block Wall

200 thk masonry blocks

Reinforcements: Vertical: Y 16 - 400 CTS

Horizontal: Y 12 - 400 CTS

C. Base Slab

150 thk Reinforced Concrete Slab

Reinforcements: F92 Mesh Wire Top and Bottom

D. Strip Footing

400 DP x 600 WD RC Footing

Reinforcements: 4 - Y16 Top & Bottom: Width Wise

R10 - 300 CTS: Length Wise

L : Administration Building

1. Load Breakdown and Calculation
2. Result of Structural Analysis
3. Design of Member
4. Summary

Load Breakdown & Calculation				
Items	Calculation & Description		Result	Unit
1	Ground Floor Loads			
A	DEAD LOADS			
1	Blockwall wt		4.6	kPa
2	300 x 300 Non-slip Ceramic Tiles		0.53	kPa
3	Timber Wall (bath room board lining)		0.72	kPa
4	Slab wt =		5	kPa
5	Services =		0.05	kPa
6	Superimposed DL	63%	0.5	kPa
			1.8	
B	LIVE LOADS			
1	Laboratory		3	kPa
2	Compactus & Lab Equipment		14	kPa
3	Store Room & Workshop		14	kPa
4	Staff Room		3	kPa
5	Resting Room		3	kPa
6	Entrance		4	kPa
7	Toilet		2	kPa
8	Locker		2	kPa
9	Store Room		8.4	kPa
10	Stairs		4	kPa
2	First Floor Loads			
A	DEAD LOADS			
1	Suspended Timber Framed 12 THK Plasterboard Ceiling		0.396	kPa
2	12THK Plasterboard to Both sides of 75THK Timber Wall		0.9912	kPa
3	Fixed Window Glass		0.255	kPa
4	Slab wt	1.386	3.75	kPa
5	300 x 300 Non-slip Ceramic Tiles	3.4692	0.27	kPa
6	Services	0.8925	0.2	kPa
7	Superimposed DL	5.7477	0.2	kPa
			4.42	kPa
B	LIVE LOADS			
1	Stairs		4	kPa
2	Ramp		4	kPa
3	Waiting Room		3	kPa
4	Conference Room		4	kPa
5	Manager's Office		3	kPa
6	Meeting Room		4	kPa
7	WorkStaion		3	kPa
8	Store Room		8.4	kPa
9	Tea Room		2	kPa
10	Toilet		2	kPa
11	Locker		2	kPa
3	Roof Loads (Trafficable Roof)			
A	DEAD LOADS			
1	Colorbond Roof Sheeting		0.05	kPa
2	Suspended Metal Ceiling		0.25	kPa
3	12THK Plasterboard Ceiling		0.29	kPa
4	Services		0.2	kPa
5	Superimposed DL		0.2	kPa
			0.99	
B	LIVE LOADS		0.25	kPa

Wind Load Calculation on Roof		
Items Calculation & Description		Result Unit
A Wind Load on Roof		
1 Data		
Return Period =		50 years
Wind Velocity =		28 m/s
Terrain Category =		1
Roof Mean Height,z =		8.3 m
Width of Bldg =		15 m
Length of Bldg =		20 m
Roof Pitch, α =		10 Degrees
Wind Angle, θ =		0 Degrees
2 Design Wind Velocity, V_z		
V_z = **$1.35V(z/zg)^k$**	**Notes for Table 4, PNGS 101-1982: Part 2**	
k = 0.07	zg = 250	
=		29.78 m/s
3 Dynamic Pressure, qz		
q_z = $0.6V_z$^2 x10^-3	... Dynamic Pressure ...	0.53 kPa
4 Design Pressure, P_z		
p_z = Cpq_z	... Design pressure...	
h/d = Table B2.2	α ≥ 10° h/d ≥ 0.5	0.55
h/b =		0.42
C_p = ... Slope D ...		-0.9
C_p = ... Slope E ...		-0.7
p_z = ... Slope D ...		-0.48 kPa
= ... Slope E ...		-0.37 kPa

5 UDL on roof truss

Truss Spacing No.	Truss Spacing (m)	Truss No.	Tributary Area (m)	Windward (kN/m)			Leeward (kN/m)		
					x-Component	y-Component		x-Component	y-Component
1	0.9								
2	5	1	3.4	-1.629	-1.605	-0.274	-1.267	-1.248	-0.216
3	5	2	5	-2.395	-2.360	-0.403	-1.863	-1.835	-0.318
4	5	3	5	-2.395	-2.360	-0.403	-1.863	-1.835	-0.318
5	5	4	5	-2.395	-2.360	-0.403	-1.863	-1.835	-0.318
6	0.9	5	3.4	-1.629	-1.605	-0.274	-1.267	-1.248	-0.216

Wind Load Calculation on Roof

Items	Calculation & Description	Result	Unit

A Wind Load on Roof

1 Data

Return Period =	**50**	years
Wind Velocity =	**28**	m/s
Terrain Category =	**1**	
Roof Mean Height,z =	**8.3**	m
Width of Bldg =	**15**	m
Length of Bldg =	**20**	m
Roof Pitch, α =	10	Degrees
Wind Angle, θ =	**90**	Degrees

2 Design Wind Velocity, V_z

V_z = ***1.35V(z/zg)^k*** **Notes for Table 4, PNGS 101-1982: Part 2**

k = **0.07** *zg* = **250**

= 29.78 m/s

3 Dynamic Pressure, qz

q_z = $0.6V_z$^2 x10^-3 … Dynamic Pressure … 0.53 kPa

4 Design Pressure, P_z

p_z	=	Cpq_z … Design pressure…		
h/d	=	Table B2.2 **α ≥ 10° h/d ≥ 0.5**	**0.55**	
h/b	=		**0.42**	
C_p	=	… Slope D …	**-0.9**	
C_p	=	… Slope E …	**-0.9**	
p_z	=	… Slope D …	-0.48	kPa
	=	… Slope E …	-0.48	kPa

5 UDL on roof truss

Truss Spacing No.	**Truss Spacing (m)**	**Truss No.**	**Tributary Area (m)**	**Windward (kN/m)**			**Leeward (kN/m)**		
					x-Component	**y-Component**		**x-Component**	**y-Component**
1	0.9								
2	5	1	3.4	-1.629	**-1.605**	**-0.274**	-1.629	**-1.605**	**-0.278**
3	5	2	5	-2.395	**-2.360**	**-0.403**	-2.395	**-2.360**	**-0.409**
4	5	3	5	-2.395	**-2.360**	**-0.403**	-2.395	**-2.360**	**-0.409**
5	5	4	5	-2.395	**-2.360**	**-0.403**	-2.395	**-2.360**	**-0.409**
6	0.9	5	3.4	-1.629	**-1.605**	**-0.274**	-1.629	**-1.605**	**-0.278**

Wind Load on Wall

Items	Calculation & Description	Result	Unit
1 Data			
Return Period =		**50**	years
Wind Velocity =		**28**	m/s
Terrain Category =		**1**	
Roof Mean Height,z =		**8.3**	m
Width of Bldg =		**15**	m
Length of Bldg =		**20**	m
Roof Pitch, α =		10	Degrees
Wind Angle, θ =		**90**	Degrees

2 Pressure Coefficients, Cp

h/d	=	Table B1.1		**0.55**
h/b	=			**0.42**
d/b	=			**0.75**
C_p	=	... windward external wall surface ...	A	**0.8**
C_p	=	... leeward external wall surface ...	B	**-0.5**
	=	... side external wall surface ...	C	-0.6

3 q_z = $0.6V_z$^2 x10^-3 ... Dynamic Pressure ...

Height, z, (m)	Velocity Multiplier	Design Wind Velocity, Vz (m/s)	Dynamic Pressure, qz (kPa)
3	**1.00**	28	0.470
7	***1.048***	29.344	0.517

4 Design Pressure, P_z

P_z = Cpq_z

UDL Columns

Vertical Face of Wall at 90° to Wind Direction	Spacing, (m)	Column No.	Tributary Area (m)	Vertical Wind Load (UDL) Distribution on Columns, W = Cpqz x Tributary Area			
				Windward, (kN/m)		**Leeward, (kN/m)**	
				GFL	**FFL**	**GFL**	FFL
1	5	**1**	2.5	0.941	1.033	**-0.588**	-0.646
2	5	2	5	1.882	2.067	**-1.176**	-1.292
3	5	3	5	1.882	2.067	**-1.176**	-1.292
4	5	4	5	1.882	2.067	**-1.176**	-1.292
		5	2.5	0.941	1.033	**-0.588**	-0.646

Vertical Face of Wall at 90° to Wind Direction	Spacing, (m)	Column No.	Tributary Area (m)	Side, (kN/m)	
				GFL	**FFL**
1	5	**1**	2.5	**-0.706**	**-0.775**
2	5	2	5	**-1.411**	**-1.550**
3	5	3	5	**-1.411**	**-1.550**
		4	2.5	**-0.706**	**-0.775**

LATERAL LOAD DISTRIBUTION TO BLOCK WALLS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 4.6 KPa

FL LEVEL	Wall mass	Floor area	Wt	Hi	Wt . Hi	Fi
1st Floor	4.42	300	2542.2	7	17795.4	135.9714
SUMM			2542.2		17795.4	135.9714

V = Cd(T1) Wt from NZS 1170.5 - 2004

= factor . Wt

= 135.9714 138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T)	=	0.16
Z	=	0.13
R	=	1.80
N(T,D)	=	1.00
Sp	=	1.00
k	=	1.00
μ	=	0.70
factor	=	0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =

= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 23 KN/M^3 Note : ALL CELLS SHADED THUS

t = 0.2 m REQUIRE A MANUAL INPUT

h = 3.5 m

Wall Mass = M x t x h x L = 16.1 L (kN/m)

Floor load (1) = dl + ll/3 = 0 Kpa

Floor load (2) = dl + ll/3 = 0 Kpa

Floor load (3) = dl + ll/3 = 0 Kpa

Floor load (4) = dl + ll/3 = 1.8 Kpa

Wall	LENGTH	MASS (m)	x	y	mx	my
1	10	161	5	10	805	1610
2	10	161	10	5	1610	805
3	5	80.5	10	12.5	805	1006.25
4	5	80.5	17.5	14.8	1408.75	1191.4
5	5	80.5	19.8	12.5	1593.9	1006.25
6	3	48.3	19.8	12.5	956.34	603.75
7	2	32.2	19.8	5	637.56	161
8	2	32.2	19.8	2.5	637.56	80.5

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	540	10	7.5	5400	4050
SUM	Wt =	1216.2			13854.11	10514.15

TABLE 1

CENTRE OF MASS	Cmx =	11.39131	Cmy =	8.64508305

CENTRE OF RIGIDITY

Wall	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.715	0.002	3.573	0.023	33.993	33.993
2	0.715	0.002	7.146	0.012	33.993	33.993
3	0.715	0.002	7.146	0.029	33.993	33.993
4	0.715	0.002	12.505	0.035	33.993	33.993
5	0.715	0.002	14.149	0.029	33.993	33.993
6	0.715	0.002	14.149	0.029	33.993	33.993
7	0.715	0.002	14.149	0.012	33.993	33.993
8	0.715	0.002	14.149	0.006	33.993	33.993
SUM	2.858	0.009	86.965	0.175	271.943	271.943

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	30.425 m	Cry	=	18.7 m
ex	=	-19.03369 m	ey	=	-10.05492 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	6 m	**ex TOO BIG**	however torsional
esy <=	0.3by =	4.5 m	**ey TOO BIG**	shears not large

edx1	=	-29.55054 m	edy1	=	-15.83238 m		(ed = 1.5es + 0.05b)
edx2	=	-18.03369 m	edy2	=	-9.304917 m	or	(ed = es - 0.05b)

Wall	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	-24.550537	-15.83	431.28	0.06	0.03	52.37	31.96
2	-19.550537	-15.83	273.72	0.06	0.03	41.70	25.45
3	-19.550537	-15.83	273.72	0.06	0.03	41.70	25.45
4	-12.050537	-15.83	104.35	0.06	0.03	25.70	15.69
5	-9.5505365	-15.83	65.76	0.06	0.03	20.37	12.43
6	-9.5505365	-15.83	65.76	0.06	0.03	20.37	12.43
7	-9.5505365	-15.83	65.76	0.06	0.03	20.37	12.43
8	-9.5505365	-15.83	65.76	0.06	0.03	20.37	12.43
SUMM			1346.12				

TABLE 3

FINAL LOAD DISTRIBUTION

Wall	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	34.1	34.0	86.4	65.9	34.1	86.4
2	34.1	34.0	75.7	59.4	34.1	75.7
3	34.1	34.0	75.7	59.4	34.1	75.7
4	34.1	34.0	59.7	49.7	34.1	59.7
5	34.1	34.0	54.4	46.4	34.1	54.4
6	34.1	34.0	54.4	46.4	34.1	54.4
7	34.1	34.0	54.4	46.4	34.1	54.4
8	34.1	34.0	54.4	46.4	34.1	54.4
SUMM					272.4	514.9

TABLE 4

Torsional Moments	
Mix (kNm)	Miy (kNm)
-1006.2524	-1367.251
-1006.2524	-1198.403
-1006.2524	-1198.403
-1006.2524	-945.1302
-1006.2524	-860.706
-1006.2524	-860.706
-1006.2524	-860.706
-1006.2524	-860.706

LATERAL LOAD DISTRIBUTION TO RC COLUMNS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 10 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
ROOF	1.8	300	1360	7	9520	72.74057143
SUMM			1360		9520	72.74057143

V = Cd(T1) Wt from NZS 1170.5 - 2004

= factor . Wt

= 72.74057 138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T)	=	0.16
Z	=	0.13
R	=	1.80
N(T,D)	=	1.00
Sp	=	1.00
k	=	1.00
μ	=	0.70
factor	=	0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi)) Wi - Mass at FL i =

= KN Hi - Height to FL i =

CENTRE OF MASS LEVEL Roof

M = 25 KN/M^3 Note : ALL CELLS SHADED THUS

t = 0.4 m REQUIRE A MANUAL INPUT

h = 3.5 m

Wall Mass = M x t x h x L = 35 L (kN/m)

Floor load (1) = dl + ll/3 = 0 Kpa

Floor load (2) = dl + ll/3 = 0 Kpa

Floor load (3) = dl + ll/3 = 0 Kpa

Floor load (4) = dl + ll/3 = 1.8 Kpa

Col	LENGTH	MASS (m)	x	y	mx	my
1	0.4	14	0	0	0	0
2	0.4	14	0	5	0	70
3	0.4	14	0	10	0	140
4	0.4	14	0	15	0	210
5	0.4	14	5		70	0
6	0.4	14	5	5	70	70
7	0.4	14	10	10	140	140
8	0.4	14	15	15	210	210
9	0.4	14	10	0	140	0
10	0.4	14	10	5	140	70
11	0.4	14	10	10	140	140
12	0.4	14	10	15	140	210
13	0.4	14	15	0	210	0
14	0.4	14	15	5	210	70
15	0.4	14	15	10	210	140
16	0.4	14	15	15	210	210
17	0.4	14	20	0	280	0
18	0.4	14	20	5	280	70
19	0.4	14	20	10	280	140
20	0.4	14	20	15	280	210

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	540	10	7.5	5400	4050
SUM	Wt =	820			8410	6150

TABLE 1

CENTRE OF MASS	Cmx =	10.256098	Cmy =	7.5

CENTRE OF RIGIDITY

COLUMN	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.002	0.002	0.000	0.000	18.185	18.185
2	0.002	0.002	0.000	0.011	18.185	18.185
3	0.002	0.002	0.000	0.021	18.185	18.185
4	0.002	0.002	0.000	0.032	18.185	18.185
5	0.002	0.002	0.011	0.000	18.185	18.185
6	0.002	0.002	0.011	0.011	18.185	18.185
7	0.002	0.002	0.021	0.021	18.185	18.185
8	0.002	0.002	0.032	0.032	18.185	18.185
9	0.002	0.002	0.021	0.000	18.185	18.185
10	0.002	0.002	0.021	0.011	18.185	18.185
11	0.002	0.002	0.021	0.021	18.185	18.185
12	0.002	0.002	0.021	0.032	18.185	18.185
13	0.002	0.002	0.032	0.000	18.185	18.185
14	0.002	0.002	0.032	0.011	18.185	18.185
15	0.002	0.002	0.032	0.021	18.185	18.185
16	0.002	0.002	0.032	0.032	18.185	18.185
17	0.002	0.002	0.043	0.000	18.185	18.185
18	0.002	0.002	0.043	0.011	18.185	18.185
19	0.002	0.002	0.043	0.021	18.185	18.185
20	0.002	0.002	0.043	0.032	18.185	18.185
SUM	0.009	0.009	0.459	0.320	363.703	363.703

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	53.75 m	Cry	=	37.5 m
ex	=	-43.4939024 m	ey	=	-30 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	6 m	**ex TOO BIG**	however torsional
esy <=	0.3by =	4.5 m	**ey TOO BIG**	shears not large

edx1	=	-66.2408537 m	edy1	=	-45.75 m		(ed = 1.5es + 0.05b)
edx2	=	-42.4939024 m	edy2	=	-29.25 m	or	(ed = es - 0.05b)

COLUMN	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	-53.75	-37.5	9.16	2.15	1.37	4.46	2.86
2	-53.75	-32.7	8.44	1.87	1.20	4.46	2.86
3	-53.75	-27.7	7.80	1.59	1.01	4.46	2.86
4	-53.75	-22.7	7.26	1.30	0.83	4.46	2.86
5	-48.95	-37.5	8.11	2.15	1.37	4.06	2.60
6	-48.95	-32.7	7.39	1.87	1.20	4.06	2.60
7	-48.95	-27.7	6.75	1.59	1.01	4.06	2.60
8	-48.95	-22.7	6.21	1.30	0.83	4.06	2.60
9	-43.7	-37.5	7.07	2.15	1.37	3.62	2.32
10	-43.7	-32.7	6.36	1.87	1.20	3.62	2.32
11	-43.7	-27.7	5.71	1.59	1.01	3.62	2.32
12	-43.7	-22.7	5.17	1.30	0.83	3.62	2.32
13	-38.7	-37.5	6.20	2.15	1.37	3.21	2.06
14	-38.7	-32.7	5.48	1.87	1.20	3.21	2.06
15	-38.7	-27.7	4.83	1.59	1.01	3.21	2.06
16	-38.7	-22.7	4.29	1.30	0.83	3.21	2.06
17	-33.7	-37.5	5.42	2.15	1.37	2.79	1.79
18	-33.7	-32.7	4.70	1.87	1.20	2.79	1.79
19	-33.7	-27.7	4.06	1.59	1.01	2.79	1.79
20	-33.7	-22.7	3.52	1.30	0.83	2.79	1.79
SUMM			123.95				

TABLE 3

FINAL LOAD DISTRIBUTION

COLUMN	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	20.3	19.6	22.6	21.0	20.3	22.6
2	20.1	19.4	22.6	21.0	20.1	22.6
3	19.8	19.2	22.6	21.0	19.8	22.6
4	19.5	19.0	22.6	21.0	19.5	22.6
5	20.3	19.6	22.2	20.8	20.3	22.2
6	20.1	19.4	22.2	20.8	20.1	22.2
7	19.8	19.2	22.2	20.8	19.8	22.2
8	19.5	19.0	22.2	20.8	19.5	22.2
9	20.3	19.6	21.8	20.5	20.3	21.8
10	20.1	19.4	21.8	20.5	20.1	21.8
11	19.8	19.2	21.8	20.5	19.8	21.8
12	19.5	19.0	21.8	20.5	19.5	21.8
13	20.3	19.6	21.4	20.2	20.3	21.4
14	20.1	19.4	21.4	20.2	20.1	21.4
15	19.8	19.2	21.4	20.2	19.8	21.4
16	19.5	19.0	21.4	20.2	19.5	21.4
17	20.3	19.6	21.0	20.0	20.3	21.0
18	20.1	19.4	21.0	20.0	20.1	21.0
19	19.8	19.2	21.0	20.0	19.8	21.0
20	19.5	19.0	21.0	20.0	19.5	21.0
SUMM					398.2	436.3

TABLE 4

LATERAL LOAD DISTRIBUTION TO RC COLUMNS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 10 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
1st Floor	4.42	300	2932	7	20524	156.820114
SUMM			2932		20524	156.820114

V = Cd(T1) Wt   from NZS 1170.5 - 2004

= factor . Wt

= 156.82011   138

where : C = Ch(T)ZRN(T,D)Sp/kμ

Ch(T) = 0.16

Z = 0.13

R = 1.80

N(T,D) = 1.00

Sp = 1.00

k = 1.00

μ = 0.70

factor = 0.0535

Fi = V x (Wi x Hi / SUM(Wi x Hi))   Wi - Mass at FL i =

= KN   Hi - Height to FL i =

CENTRE OF MASS   LEVEL Roof

M = 25 KN/M^3   Note : ALL CELLS SHADED THUS REQUIRE A MANUAL INPUT

t = 0.4 m

h = 3.5 m

Wall Mass = M x t x h x L = 35 L (kN/m)

Floor load (1) = dl + ll/3 = 0 Kpa

Floor load (2) = dl + ll/3 = 0 Kpa

Floor load (3) = dl + ll/3 = 0 Kpa

Floor load (4) = dl + ll/3 = 4.42 Kpa

Col	LENGTH	MASS (m)	x	y	mx	my
1	0.4	14	0	0	0	0
2	0.4	14	0	5	0	70
3	0.4	14	0	10	0	140
4	0.4	14	0	15	0	210
5	0.4	14	5		70	0
6	0.4	14	5	5	70	70
7	0.4	14	10	10	140	140
8	0.4	14	15	15	210	210
9	0.4	14	10	0	140	0
10	0.4	14	10	5	140	70
11	0.4	14	10	10	140	140
12	0.4	14	10	15	140	210
13	0.4	14	15	0	210	0
14	0.4	14	15	5	210	70
15	0.4	14	15	10	210	140
16	0.4	14	15	15	210	210
17	0.4	14	20	0	280	0
18	0.4	14	20	5	280	70
19	0.4	14	20	10	280	140
20	0.4	14	20	15	280	210

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	300	1326	10	7.5	13260	9945
SUM	Wt =	1606			16270	12045

TABLE 1

CENTRE OF MASS	Cmx =	10.13076	Cmy =	7.5

CENTRE OF RIGIDITY

SHEAR WALL	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
1	0.002	0.002	0.000	0.000	39.205	39.205
2	0.002	0.002	0.000	0.011	39.205	39.205
3	0.002	0.002	0.000	0.021	39.205	39.205
4	0.002	0.002	0.000	0.032	39.205	39.205
5	0.002	0.002	0.011	0.000	39.205	39.205
6	0.002	0.002	0.011	0.011	39.205	39.205
7	0.002	0.002	0.021	0.021	39.205	39.205
8	0.002	0.002	0.032	0.032	39.205	39.205
9	0.002	0.002	0.021	0.000	39.205	39.205
10	0.002	0.002	0.021	0.011	39.205	39.205
11	0.002	0.002	0.021	0.021	39.205	39.205
12	0.002	0.002	0.021	0.032	39.205	39.205
13	0.002	0.002	0.032	0.000	39.205	39.205
14	0.002	0.002	0.032	0.011	39.205	39.205
15	0.002	0.002	0.032	0.021	39.205	39.205
16	0.002	0.002	0.032	0.032	39.205	39.205
17	0.002	0.002	0.043	0.000	39.205	39.205
18	0.002	0.002	0.043	0.011	39.205	39.205
19	0.002	0.002	0.043	0.021	39.205	39.205
20	0.002	0.002	0.043	0.032	39.205	39.205
SUM	0.009	0.009	0.459	0.320	784.101	784.101

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	53.75 m	Cry	=	37.5 m
ex	=	-43.61924 m	ey	=	-30 m

check code eccentricities :

please enter b in x - direc'n = 20 m
please enter b in y - direc'n = 15 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <= 0.3bx = 6 m **ex TOO BIG** however torsional
esy <= 0.3by = 4.5 m **ey TOO BIG** shears not large

edx1	=	-66.42886 m	edy1	=	-45.75 m		(ed = 1.5es + 0.05b)
edx2	=	-42.61924 m	edy2	=	-29.25 m	or	(ed = es - 0.05b)

COL	xi m	yi m	xi^2 . IXi + yi^2 . IYi	W"ix1 (KN)	W"ix2 (KN)	W"iy1 (KN)	W"iy2 (KN)
1	-53.75	-37.5	9.16	4.63	2.96	9.64	6.18
2	-53.75	-32.7	8.44	4.04	2.58	9.64	6.18
3	-53.75	-27.7	7.80	3.42	2.19	9.64	6.18
4	-53.75	-22.7	7.26	2.80	1.79	9.64	6.18
5	-48.95	-37.5	8.11	4.63	2.96	8.78	5.63
6	-48.95	-32.7	7.39	4.04	2.58	8.78	5.63
7	-48.95	-27.7	6.75	3.42	2.19	8.78	5.63
8	-48.95	-22.7	6.21	2.80	1.79	8.78	5.63
9	-43.7	-37.5	7.07	4.63	2.96	7.83	5.03
10	-43.7	-32.7	6.36	4.04	2.58	7.83	5.03
11	-43.7	-27.7	5.71	3.42	2.19	7.83	5.03
12	-43.7	-22.7	5.17	2.80	1.79	7.83	5.03
13	-38.7	-37.5	6.20	4.63	2.96	6.94	4.45
14	-38.7	-32.7	5.48	4.04	2.58	6.94	4.45
15	-38.7	-27.7	4.83	3.42	2.19	6.94	4.45
16	-38.7	-22.7	4.29	2.80	1.79	6.94	4.45
17	-33.7	-37.5	5.42	4.63	2.96	6.04	3.88
18	-33.7	-32.7	4.70	4.04	2.58	6.04	3.88
19	-33.7	-27.7	4.06	3.42	2.19	6.04	3.88
20	-33.7	-22.7	3.52	2.80	1.79	6.04	3.88
SUMM			123.95				

TABLE 3

FINAL LOAD DISTRIBUTION

WALL	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
1	43.8	42.2	48.8	45.4	43.8	48.8
2	43.2	41.8	48.8	45.4	43.2	48.8
3	42.6	41.4	48.8	45.4	42.6	48.8
4	42.0	41.0	48.8	45.4	42.0	48.8
5	43.8	42.2	48.0	44.8	43.8	48.0
6	43.2	41.8	48.0	44.8	43.2	48.0
7	42.6	41.4	48.0	44.8	42.6	48.0
8	42.0	41.0	48.0	44.8	42.0	48.0
9	43.8	42.2	47.0	44.2	43.8	47.0
10	43.2	41.8	47.0	44.2	43.2	47.0
11	42.6	41.4	47.0	44.2	42.6	47.0
12	42.0	41.0	47.0	44.2	42.0	47.0
13	43.8	42.2	46.1	43.7	43.8	46.1
14	43.2	41.8	46.1	43.7	43.2	46.1
15	42.6	41.4	46.1	43.7	42.6	46.1
16	42.0	41.0	46.1	43.7	42.0	46.1
17	43.8	42.2	45.2	43.1	43.8	45.2
18	43.2	41.8	45.2	43.1	43.2	45.2
19	42.6	41.4	45.2	43.1	42.6	45.2
20	42.0	41.0	45.2	43.1	42.0	45.2
SUMM					858.6	941.0

TABLE 4

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

Reference **Bowles, J.E Foundation Analysis & Design**
McGrall Hill, Third Edition, 1982

Design data: Compacted Material

B = [redacted] m
μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = [redacted] m
F'c = 40 MPa
ρ = 2400 kg/m^3

$k's = 0.65 \sqrt[12]{(Es\ B^4)/(Ef\ If)} * (Es/(1-\mu^2))$

--> $\lambda L < \pi/4$ Rigid member - Bending not influenced much by ks
$\lambda L > \pi$ Flexible member - Bending heavily localised

$ks = k's\ B$

$\lambda = \sqrt[4]{(k's / 4Ec\ I)}$

Es, Ef = Modulus of soil and footing respectively
B, If = Footing width and its moment of inertia based on cross section
ks = Modulus of subgrade reaction

Procedure :Conventional rigid procedure (CRP) or Beam on Elastic Foundation (Winkler)

Section	B m	I m^4	Ec GPa	Es GPa	μ MPa	k's	L m	λ	λL	Is $\lambda L < \pi/4$	Is $\lambda L > \pi$	Analysis method ?	ks = k'sB kNm
1	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	20.00	1.7730	35.4600	No	Yes	Use Winkler	355472
2	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	20.00	1.7730	35.4600	No	Yes	Use Winkler	355472
3	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	15.00	1.7730	26.5950	No	Yes	Use Winkler	355472
4	0.40	1.13E-04	31.9754	0.18	0.4	0.14219	15.00	1.7730	26.5950	No	Yes	Use Winkler	355472
5	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	15.00	1.0338	15.5076	No	Yes	Use Winkler	41094
6	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	20.00	1.0338	20.6768	No	Yes	Use Winkler	41094
7	7.10	2.00E-03	31.9754	0.18	0.4	0.29177	20.00	1.0338	20.6768	No	Yes	Use Winkler	41094
8	5.00	1.41E-03	31.9754	0.18	0.4	0.26729	15	1.1041	16.5616	No	Yes	Use Winkler	53458
9	4.35	1.22E-03	31.9754	0.18	0.4	0.25815	15	1.1333	16.9998	No	Yes	Use Winkler	59344
10	3.675	1.03E-03	31.9754	0.18	0.4	0.24750	15	1.1697	17.5460	No	Yes	Use Winkler	67346

Timber Roof Purlin Design

REF: PNGS 1292 - 1989

TABLE 2.2

STRENGTH CLASSIFICATION

Species	Moisture Condition	Strength Group(2)		Stress Grade
Hardwood	Seasoned	SD3		F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

	Basic working stress, MPa				
Stress grade	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	Modulus of Elasticity (E)
F11	11.00	6.60	8.4	1.05	10500

TABLE A

MODIFICATION FACTORS

Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp°C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'..Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' Eq.3.21

= 5.94 MPa

Purlin Design

Loads 2.475

DL = 0.15 kN/m LL = 0.15 kN/m 0.625

Combination: DL + LL = 0.3 kN/m

Size B (mm) D (mm) Purlin Span = 2500 mm Purlin Spacing = 600 mm CTS

75 100 HWD

From Analysis

M*max	=	0.23	kNm
V*max	=	0.38	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 1.875 MPa

Working Shear Stress, fs = 3V/2BD

= 0.075 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.142401 < 1 OK

fs = 0.075 MPa < Fs = 0.945 MPa OK

Deflection Check!

From Analysis

DL = 0.15 kN/m

LL = 0.15 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 1.162574 mm Def. by DL

Allowable Defln = L/300 8.33333333 mm > 1.162574 = Defln.DL OK

Defln.LL = 1.162574 mm Def. by LL

Allowable Defln = L/360 6.94444444 mm > 1.162574 = Defln.LL OK

POMSSUP
Roof: Admin Building

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	100 x 150 HWD	S1	Not applicable	No	Standard shape
2	400 x 400 Col	S2	Not applicable	No	Standard shape

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	1.5000E-02	2.9300E-05	2.8100E-05	1.2500E-05	INFINITE	INFINITE	0.00
2	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.100	0.150			
2	Rectangle	0.400	0.400			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	TIMBER	1.0500E+07	6.50	1.1000E+03	1.0000E-10	
2	CONCRETE-32	2.8600E+07	0.15	2.4500E+03	1.0000E-05	32000.00

COMBINATION LOAD CASES

Load case 3: 1.4D+1.7

1.400 * Load case 1: D
1.700 * Load case 2: L

Load case 5: D+1.3L+1.3W

1.000 * Load case 1: D
1.300 * Load case 2: L
1.300 * Load case 4: W

L--d case 6: 0.9D+1.3W

0.900 * Load case 1: D
1.300 * Load case 4: W

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	1.4D+1.7
4	W
5	D+1.3L+1.3W
6	0.9D+1.3W

MEMBER FORCES AND MOMENTS (kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
14	3	155.217*	-3.632	0.000	0.000	0.000	0.055
4	3	-78.351#	-0.348	0.000	0.000	0.000	1.047
29	3	39.939	76.877*	0.000	0.000	0.000	-14.150
28	3	39.939	-76.877#	0.000	0.000	0.000	14.150
29	3	50.703	76.877	0.000	0.000	0.000	139.603*
28	3	50.703	-76.877	0.000	0.000	0.000	-139.603#

NODE REACTIONS (kN,kNm)
(*=Maximum, #=Minimum)

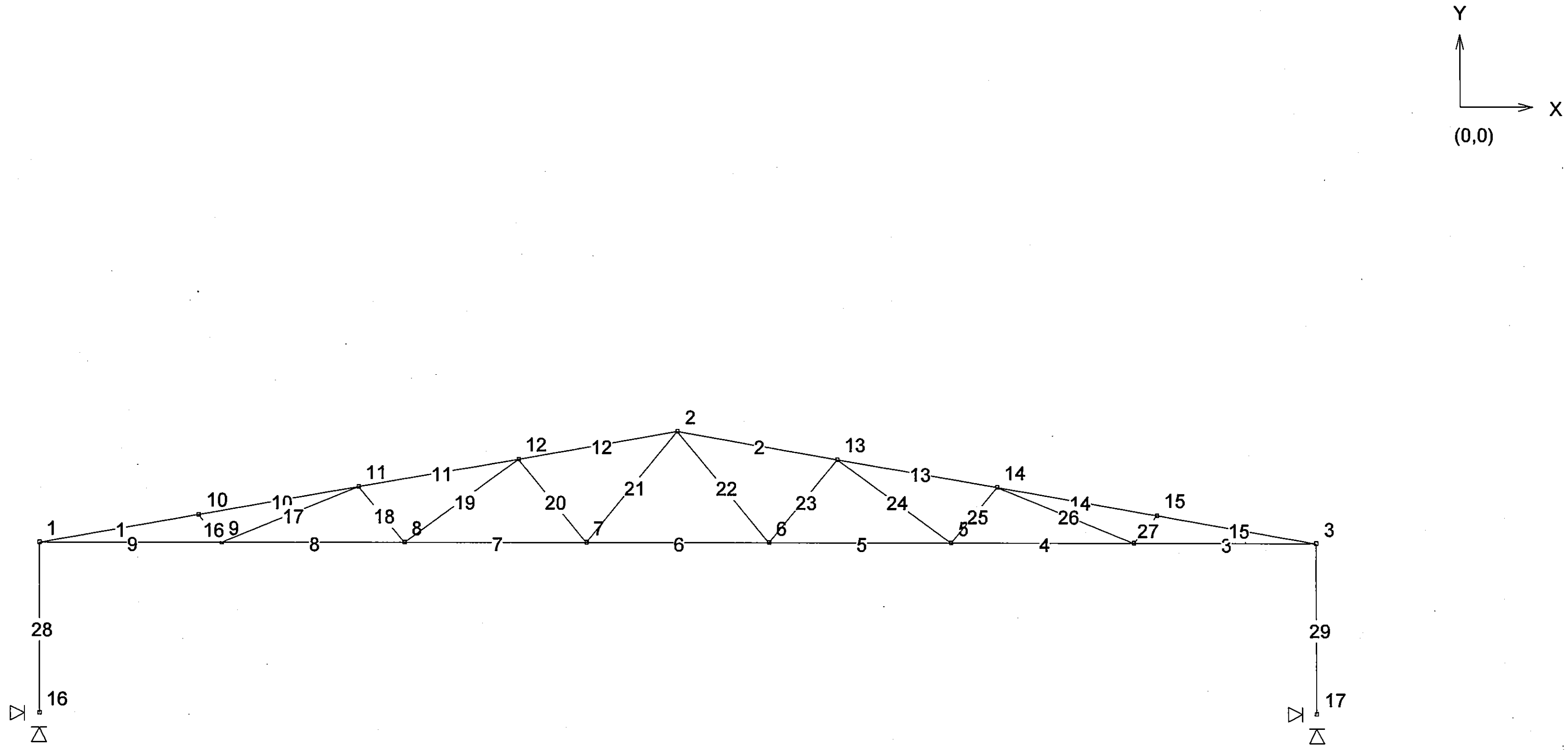
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
16	3	76.877*	50.703	0.000	0.000	0.000	-139.603
17	3	-76.877#	50.703	0.000	0.000	0.000	139.603
16	3	76.877	50.703*	0.000	0.000	0.000	-139.603
16	4	-5.897	-3.102#	0.000	0.000	0.000	10.699
17	3	-76.877	50.703	0.000	0.000	0.000	139.603*
16	3	76.877	50.703	0.000	0.000	0.000	-139.603#

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
10	3	155.217*	3.632	0.000	0.000	0.000	0.055
4	3	-78.351#	-0.348	0.000	0.000	0.000	1.047
29	3	39.939	76.877*	0.000	0.000	0.000	-14.150
28	3	39.939	-76.877#	0.000	0.000	0.000	14.150
29	3	50.703	76.877	0.000	0.000	0.000	139.603*
28	3	50.703	-76.877	0.000	0.000	0.000	-139.603#



Sections:
1 100 x 150 HWD
2 400 x 400 Col

Roof Model

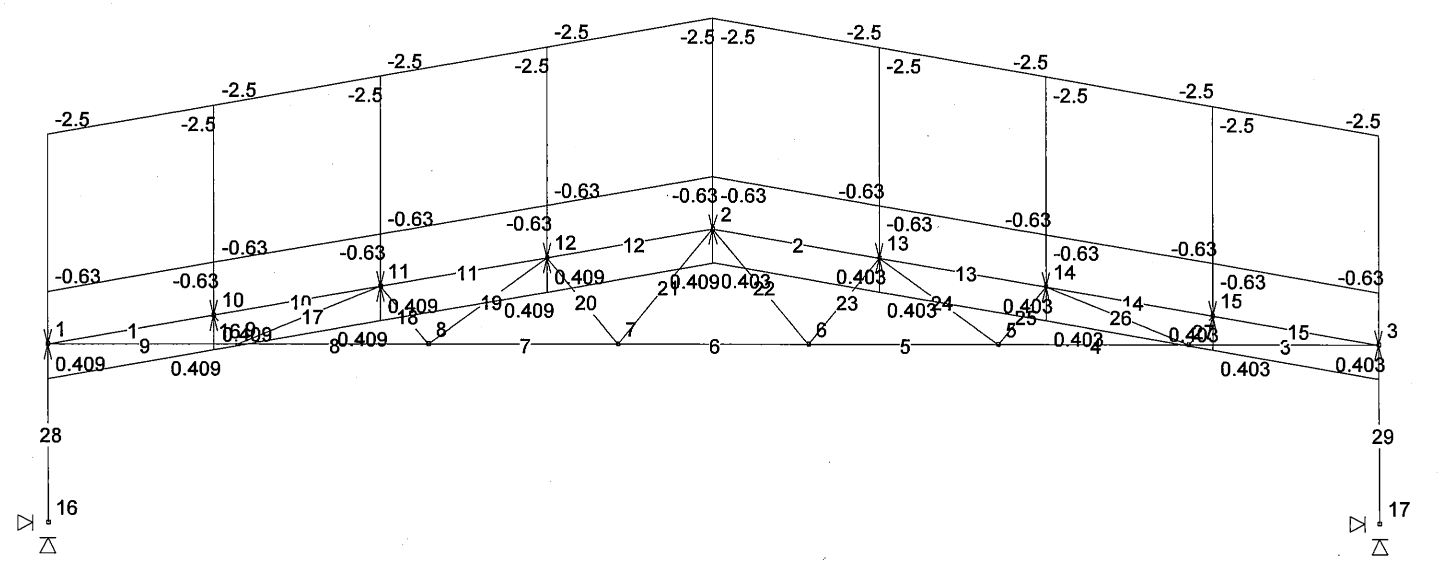
Job: ADMROOF, Designer: FYP, Units: m,kN,MPa, Scale: 1:85, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
POMSSUP
Roof: Admin Building 18 Oct 2011, 8:47 am

1	(SW)	D
2		L
3		1.4D+1.7
4		W
5		D+1.3L+1.3W
6		0.9D+1.3W

Y

X

(0,0)



Sections:
1 100 x 150 HWD
2 400 x 400 Col

Roof Model with Loadings

Job: ADMROOF, Designer: FYP, Units: m,kN,MPa, Scale: 1:85, Axes: XY
Load: 0.09 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Roof: Admin Building 18 Oct 2011, 8:48 am

Roof Truss Design

REFERENCE: REF: PNGS 1292 - 1989

TABLE 2.2			
STRENGTH CLASSIFICATION			
Species	Moisture Condition	Strength Group(2)	Stress Grade
Hardwood	Seasoned	SD3	F11

TABLE 2.3

BASIC WORKING STRESS & STIFFNESS FOR THE STRUCTURAL TIMBER (CONCERNED)

Stress grade	Basic working stress, MPa				Modulus of Elasticity (E)
	Bending (Fb')	Tension parallel to grain (Ft')	Compression parallel to grain (Fc')	Shear in Beams (Fs')	
F11	11.00	6.60	8.40	1.05	10500

TABLE A

MODIFICATION FACTORS											
Duration of Load Factor, k1	k2	k3	Partial Seasoning Factor, k4	Seasoned moisture Cont. Factor, k5	Temp^{0}C Factor, k6	Length of Bearing Factor, k7	Parallel Support Factor, k8	Grid Systems Factor, k9	Size Factor, k11	Stability Factor, k12
1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.33	1.00	1.00	1.00

PERMISSIBLE STRESSES

a. In Bending:

Fb = k1k4k5k6k8k11k12Fb'..Eq.3.1

= 13.167 MPa

b. In Compression parallel to grain:

Fc = k1k4k5k6k8k11k12Fc' Eq.3.12

= 10.0548 MPa

c. In Tension parallel to grain

Ft = k1k4k5k6k11Ft' Eq.3.21

= 5.94 MPa

FROM SPACE GASS ANALYSIS			
MAXIMUM VALUES OF BENDING MOMENTS & AXIAL FORCES ON THE TRUSS MEMBERS			
	TOP CHORD	DIAGONAL	BOTTOM CHORD
M*max (kNm)	8	2	4
P*max (+ve) (kN)	155	20	0
P*max (-ve) (kN)	0	20	78.4

CHECK FOR ADEQUACY FOR SIZE

SECTION NAME	SECTION SIZE		SECTION PROPERTIES		
	B (mm)	D (mm)	Zx = BD^2/6, mm^3	Zy= B^{2}D/6, mm^3	A = BD, mm^2
TOP CHORD	150	300	2250000	1125000	45000
WEB	75	150	281250	140625	11250
BOTTOM CHORD	150	300	2250000	1125000	45000

WORKING STRESSES

	Bending Stress, fbx = Mx/Zx (MPa)	Axial Compressive Stress, fc = P/A (MPa)	Axial Tensile Stress, ft = P/A (MPa)
TOP CHORD	3.5556	3.4444	0.0000
WEB	0.0000	1.7778	1.7778
BOTTOM CHORD	1.7778	0.0000	1.7422

TOP CHORD CHECK!

A. Combined Bending + Axial Compression

From equation 3.23, (fbx/Fbx) + (fc/Fcx) <= 1

0.270035 + 0.34257 <= 1

0.6126 < 1 OK

B. Combined bending + Axial Tension

From equation 3.25, 0.6fbx + ft <= Ft

2.133333 + 0 <= 5.94

2.13333 < 5.94 OK

WEB CHECK!

A. Axial Stress in Compression Member

From equation 3.19, fc <= Fcx

1.777778 < 10.0548 OK

B. Axial Stress in Tension Member

From equation 3.19, ft <= Ft

0.0000 < 5.94 OK

BOTTOM CHORD CHECK!

A. Combined Bending + Axial Compression

From equation 3.23, (fbx/Fbx) + (fc/Fcx) <= 1

0.135018 + 0 <= 1

0.13502 < 1 OK

B. Combined bending + Axial Tension

From equation 3.25, 0.6fbx + ft <= Ft

1.066667 + 1.7422 <= 5.94

2.8089 < 5.94 OK

Job name: ADMBDG1 Designer: FYP Date: 18 Oct 2011, 8:38 am Page: 1
POMSSUP
Administration Building

SECTION PROPERTIES (m,m^2,m^4,deg)

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	SLAB	S1	Not applicable	No	Standard shape
2	400DP x 400WD	S2	Not applicable	No	Standard shape
3	400 x 400 COL	S3	Not applicable	No	Standard shape
4	300DP x 300WD	S4	Not applicable	No	Standard shape
5	75*5 EA	S5	Single	No	AUST300.LSS
6	125*75*6 RHS	S6	Not applicable	No	AUST300.LSS

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	4.0000E-02	2.2530E-04	1.3330E-04	1.3330E-04	INFINITE	INFINITE	0.00
2	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00
3	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00
4	9.0000E-02	1.1408E-03	6.7500E-04	6.7500E-04	INFINITE	INFINITE	0.00
5	6.7200E-04	5.2800E-09	1.4700E-07	5.6300E-07	INFINITE	INFINITE	45.00
6	2.1300E-03	4.4400E-06	1.8700E-06	4.1600E-06	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.200	0.200			
2	Rectangle	0.400	0.400			
3	Rectangle	0.400	0.400			
4	Rectangle	0.300	0.300			

MATERIAL PROPERTIES (kPa,Kg/m^3)

Matl	Material Name	Young's Modulus	Poisson's Ratio	Mass Density	Coeff of Expansion	Concrete Strength
1	STEEL	2.0000E+08	0.25	7.8500E+03	1.1700E-05	
2	STEEL	2.0000E+08	0.25	7.8500E+03	1.1700E-05	

COMBINATION LOAD CASES

Load case 5: 1.4D+1.7L

1.400 * Load case 1: D
1.700 * Load case 2: L

Load case 6: D+1.3L+1.3W

1.000 * Load case 1: D
1.300 * Load case 2: L
1.000 * Load case 3: W

Load case 7: 0.9D+1.3W

0.900 * Load case 1: D
1.000 * Load case 3: W

Load case 8: D+1.3L+E

1.000 * Load case 1: D
1.300 * Load case 2: L
1.300 * Load case 4: E

Load case 9: 0.9+E

0.900 * Load case 1: D
1.300 * Load case 4: E

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	W
4	E
5	1.4D+1.7L
6	D+1.3L+1.3W
7	0.9D+1.3W
8	D+1.3L+E
9	0.9+E

MEMBER FORCES AND MOMENTS (kN,kNm)
------------------------- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
74	5	554.812*	0.836	-1.945	-0.018	2.344	-1.419
79	4	-14.338#	0.245	23.752	0.028	-48.871	-0.247
40	5	-0.872	127.956*	0.067	0.044	-0.169	-131.250
85	5	8.159	-144.776#	-0.294	3.212	-0.286	-144.573
65	8	210.888	0.587	44.606*	0.042	-79.876	-0.917
103	5	33.719	-0.189	-19.909#	0.022	44.761	0.814
59	8	4.370	50.846	-0.004	11.311*	0.045	-42.107
86	8	4.298	-46.595	-0.154	-16.992#	0.220	39.579
65	8	210.888	0.587	44.606	0.042	76.246*	1.136
66	8	216.092	1.037	44.591	0.008	-80.029#	-1.471
85	5	8.159	-107.787	-0.294	3.212	0.258	89.047*
85	5	8.159	-144.776	-0.294	3.212	-0.286	-144.573#

NODE REACTIONS (kN,kNm)
-------------- (*=Maximum, #=Minimum)

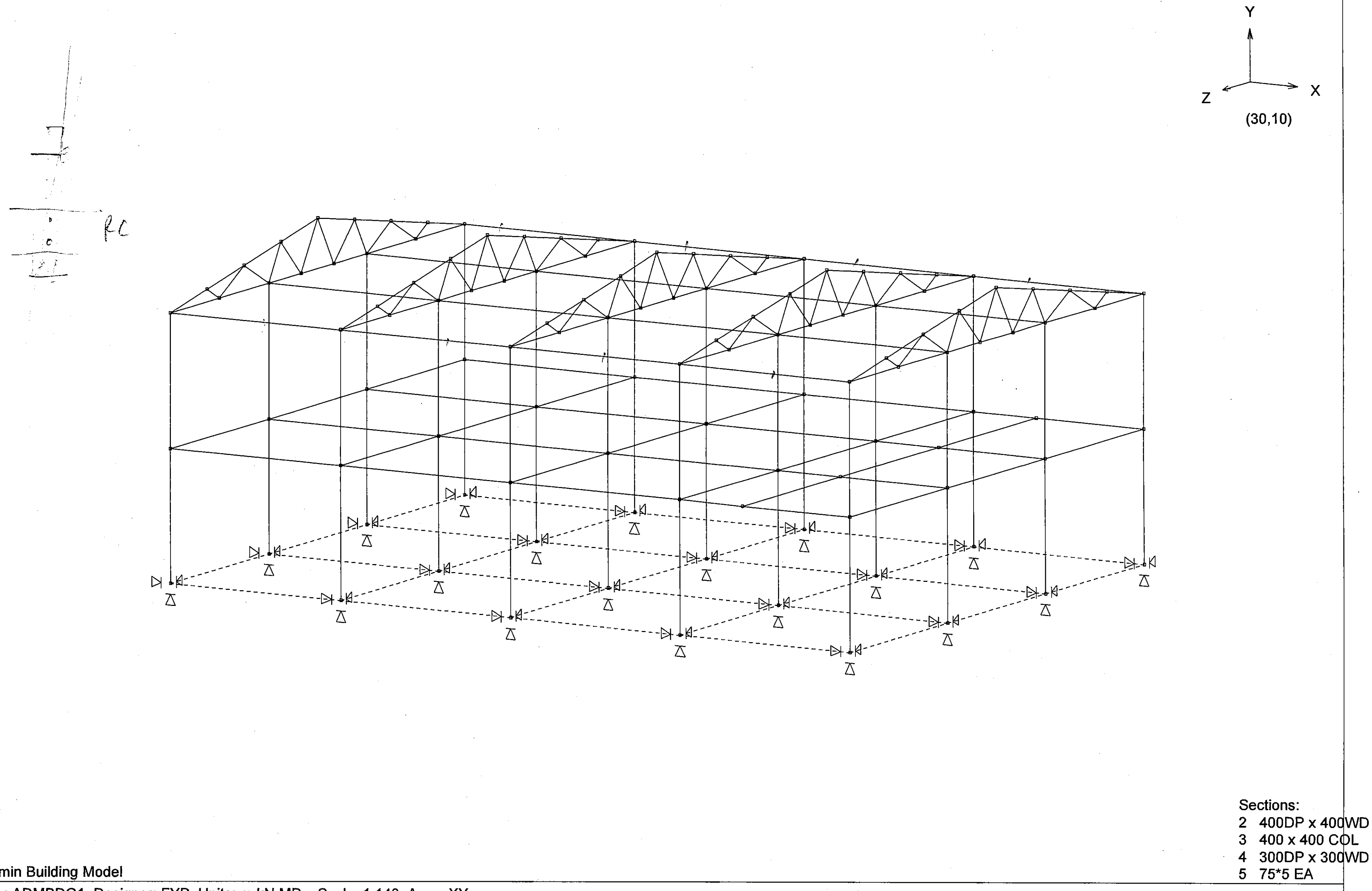
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
19	5	19.560*	273.544	-1.807	-2.179	-0.031	-22.225
3	5	-20.214#	263.316	-4.178	-5.020	-0.015	23.873
11	5	-0.836	554.812*	-1.945	-2.344	-0.018	1.419
5	4	-0.245	-14.338#	23.752	48.871	0.028	0.247
12	8	-0.587	210.888	44.606*	79.876	0.042	0.917
9	5	0.287	256.036	-18.716#	-21.942	0.019	0.007
16	8	-1.037	216.092	44.591	80.029*	0.008	1.471
9	5	0.287	256.036	-18.716	-21.942#	0.019	0.007
1	6	-7.402	81.452	-1.625	-0.496	0.147*	8.773
20	6	7.795	100.899	10.853	14.495	-0.146#	-8.812
3	5	-20.214	263.316	-4.178	-5.020	-0.015	23.873*
19	5	19.560	273.544	-1.807	-2.179	-0.031	-22.225#

INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
------------------------------- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

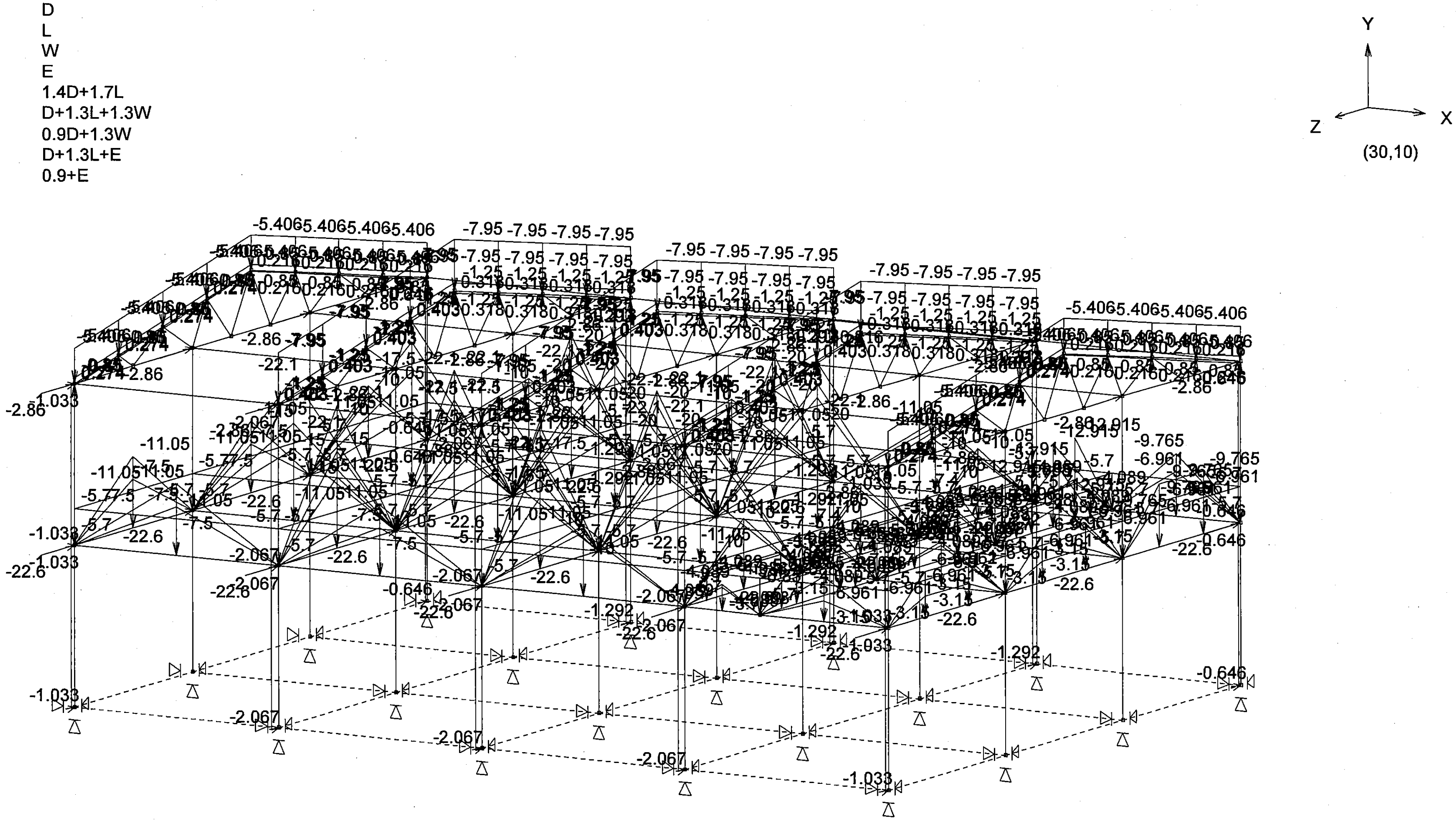
Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
74	5	554.812*	0.836	-1.945	-0.018	2.344	-1.419
79	4	-14.338#	0.245	23.752	0.028	-48.871	-0.247
40	5	-0.872	127.956*	0.067	0.044	-0.169	-131.250
85	5	8.159	-144.776#	-0.294	3.212	-0.286	-144.573
65	8	210.888	0.587	44.606*	0.042	-79.876	-0.917
103	5	33.719	-0.189	-19.909#	0.022	44.761	0.814
59	8	4.370	50.846	-0.004	11.311*	0.045	-42.107
86	8	4.298	-46.595	-0.154	-16.992#	0.220	39.579
65	8	210.888	0.587	44.606	0.042	76.246*	1.136
66	8	216.092	1.037	44.591	0.008	-80.029#	-1.471
85	5	8.159	-107.787	-0.294	3.212	0.258	89.047*
85	5	8.159	-144.776	-0.294	3.212	-0.286	-144.573#



Admin Building Model

Job: ADMBDG1, Designer: FYP, Units: m,kN,MPa, Scale: 1:140, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
POMSSUP
Administration Building 18 Oct 2011, 8:37 am

1	———	D
2	———	L
3	———	W
4	———	E
5	———	1.4D+1.7L
6	———	D+1.3L+1.3W
7	———	0.9D+1.3W
8	———	D+1.3L+E
9	———	0.9+E



Sections:
2 400DP x 400WD
3 400 x 400 COL
4 300DP x 300WD
5 75*5 EA

Admin Building Model with Loadings

Job: ADMBDG1, Designer: FYP, Units: m,kN,MPa, Scale: 1:140, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Administration Building 18 Oct 2011, 8:38 am

ROOF BEAM DESIGN

REFERENCE: **Concrete Structures - 3rd Edition** **AS 3600 - 2009**
R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	0.60	kN/m	(plus super)
Self wt	:	Wsw =	5.00	kN/m	
Blockwall	:	Wb =	0.00	kN/m	
TOTAL		=	5.60	kN/m	
LL :		Wll =	0.25	kN/m	
LL factor	- long term		0.4		
	- short term		0.7		

2 Beam Section Properties

Beam Size

bw	400	mm	D =	500	mm
tff	0	mm	Cover =	65	mm
Actual length, L =	5000	mm	Overall depth of Sect. D =	500	mm
Effective Length,Lef =	4600	mm	Area, A =	2.00E+05	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	4.17E+09	mm^4
Neutral Axis, xc =	660.00	mm	Moment of Inertia, Iy-y =	2.67E+09	mm^4
Effective width, beff =	1320.00	mm	Gross Area, Ag =	2.00E+05	mm^2

3 Design Parameters

F'c =	32	MPa	D =	500	mm
Fsy =	410	MPa	Ec =	28599.62293	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.822	

4 <u>INITIAL CHOICE OF SECTION</u>

Effective width = 1320 mm clause 8.8.2

Is the section RECT or T , L (ans : 1 or 2) = 2

k_1	=	0.045	(RECT sections)	1
	=	0.02226344	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	9.523		
defln/Lef	=	0.004	k_1/k_2 = 8.549050482	

d = 90 mm

take d = 393 mm Distance from d to extreme fibre of beam (do) = 107 mm

5 <u>Forces from an Elastic Analysis by Space Gass :</u>

Mmax -ve	=	75.30	kNm	(left support)
Mmax +ve	=	75.30	kNm	(mid-span)
M -ve	=	75.30	kNm	(right support)
Vmax	=	29.00	kN	
V (other end)	=	29.00	kN	
N*	=	31.00	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine - ve steel required :</u> (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm d = 393

Estimate Area of main steel Ast = 610.8817262 mm^2

4	Y 20		Ast =	1256.637061 mm^2		
		16	p =	0.007993874		
3	Y 16		Asc =	603.1857895 mm^2		
			pc =	0.00383706	< pd =	0.021813

p-pc = 0.004156815 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0.076226 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

= 160 kNm > M* = 75.30 kNm **OK**

Determine - ve steel at right support :

M -ve = 75.30 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 107 mm dst = 393 mm

Estimate Area of main steel Ast = 610.8817262 mm^2

4 Y 20 Ast = 1256.637061 mm^2

p = 0.007993874

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.00383706 < pd = 0.021813

p-pc = 0.004156815

Check M* <= phiMu

ku = 0.076226 dsc = 85 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d
x (d - 0.5 x 0.85 x ku x d)]

= 160 kNm > M* = 75.30 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 75.3 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 107 mm

what is the code ? 2 d = 393 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 610.881726 mm^2

4 Y 20 Ast = 1256.637061 mm^2

3 Y 16 Asc = 603.1857895 mm^2

Assume N - A in flange

Effective width : beff = 1320 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.04442081

hence dn = 17 < flange thickness
hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 179 kNm > M*+ve = 75.30 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	4	Y 20
	Asc	=	3	Y 16
Right support :	Ast	=	4	Y 20
	Asc	=	3	Y 16
midspan	Ast	=	4	Y 20
	Asc	=	3	Y 16

7 STEEL CURTAILMENT REQUIREMENT CHECK

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

For +ve steel : i) 1/2 Ast must be carried into outer support
= 2 bars
ii) 1/4 Ast into interior support
= 2 bars
iii) Remainder curtailed at 0.1Ln from supports
= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 65 mm

d_b = 16 mm # of lines of bars = 1

a_b = 246 mm distance between bars

If a_b > 2c C = 2c + d_b = 146 mm
C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 262 mm
coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 246 mm (dist between bars)

factor = 40074.8041

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 274.485 mm

take fs/Fsy = M*/Phi Mu = 0.469905721

therefore

Lsy =	129 mm
say =	500 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 65 mm d_b = 20 mm
# of lines of bars = 1

If ab > 2c C = 2c + d_b = 150 mm

if ab < 2c C = a_b + d_b = 574 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 554 mm (dist between bars)

factor = 50093.50513

Lsyt = 333.9567 mm take fs/Fsy = M*/Phi Mu = 0.420889762

therefore

Lsy =	141 mm
say =	400 mm

≥ 25k1db 400

9 SHEAR CHECK

V* = 29.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 393 mm dq = 85 mm
do = 415 mm

V* = 27 kN beta 1 = 1.1925 (but not less than 1.1)
hence = 1.1925
beta 2 = 1
beta 3 = 1 Unity

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333
= 86.42 kN

phi Vumax = 743.68 kN > V* No web crushing

phi Vumin = 156 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 195.121951

Asv.min = 68.2926829 mm^2

Asv.max = 1104 mm^2

Asv = 2 x area of stirrups = 220 mm^2 2 Ties

theta = 32.19787 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 420.70 kN

Phi Vu = 294 kN > V* = 27 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**200**	**centres**

Other Support :

V* = 29.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 393 mm

dq = 85 mm

do = 415 mm

V* = 26.70 kN beta 1 = 1.1925 (but not less than 1.1)

hence = 1.1925

beta 2 = 1

beta 3 = 1

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 86.42121 kN

phi Vumax = 743.68 kN > V* No web crushing

phi Vumin = 156 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 195.121951

Asv.min = 68.2926829 mm^2

Asv.max = 1103.6628 mm^2

Asv = 2 x area of stirrups = 330 mm2 2 Ties

theta = 33.7915 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 542.969 kN

Phi Vu = 380.0783 kN > V* = 26.70 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**200**	**centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 75.30 kNm Es = 200000 MPa

Ml = 75.30 kNm Ec = 28599.62293 MPa

Mo = 150.6 kNm

Ms = 75.30 kNm

Ig = 4.17E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	$\mathbf{b_{eff}}$ =	1320 mm	$\mathbf{A_{sc}}$ =	603.1857895 mm^2
webwidth	$\mathbf{b_w}$ =	400 mm	$\mathbf{A_{st}}$ =	1256.637061 mm^2
Flange thickness	$\mathbf{t_{ff}}$ =	0 mm	$\mathbf{d_{sc}}$ =	107 mm
			$\mathbf{d_{st}}$ =	393 mm
			d =	393 mm
			D =	500 mm

n = Es/Ec = 6.99309919

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 12402.7399 dn - 3840400.426) = 0

dn = 110.991132 mm

d = 393 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 8.81E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.5685425 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	2.27E+09 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w = 400 mm   **A_{st}** = 1256.637061 mm^2

d_{sc} = 85 mm   **A_{sc}** = 603.1857895 mm^2

d = 393 mm

Ig = 4.17E+09 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 12402.7399 dn - 3760871.476) = 0

dn = 109.58 mm

d = 393 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 8.81E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.5685425 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 2.27E+09 mm^4

At left end (arbitrary)

b_w = **400** mm

d_{sc} = **85** mm

A_{st} = **1256.6371** mm^2

A_{sc} = **603.18579** mm^2

d = **393** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

200 dn^2 + 12402.7399 dn - 3760871.476) = 0

dn = 109.583983 mm

d = 393 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 8.81E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.5685425 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.27E+09 mm^4

Iav	=	[Im + (IL + Ir)/2]/2	=	2.27E+09 mm^4

Midspan Deflection :

Lef = 4600 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

=	2.04146525 mm

Total Deflection :

Long term factor	=	0.2		
Short term factor	=	0.5		
w - long term	=	5.65 kN/m		
w - short term	=	5.725 kN/m		
Defln,s,sus	=	2.014721159 mm		
At midspan :			Ast =	1256.63706 mm^2
			Asc =	603.185789 mm^2
Asc/Ast =		0.48		
kcs	=	2 - (1.2 x Asc/Ast)	>= 0.8	
	=	1.424		

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)
= 4.62353 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 250 = 18.4 mm
> 0.9 x Total Defln
Lef / 500 = 9.2 mm
> 0.9 x Total Defln

FIRST FLOOR BEAM DESIGN

REFERENCE:	**Concrete Structures - 3rd Edition**		**AS 3600 - 2009**
	R. F. Warner, B. V. Rangan, & A. S. Hall		

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	9.80	kN/m	(plus super)
Self wt	:	Wsw =	8.25	kN/m	
Blockwall	:	Wb =	16.10	kN/m	
TOTAL		=	34.15	kN/m	
LL :		Wll =	0.25	kN/m	
LL factor	- long term		0.4		
	- short term		0.7		

2 Beam Section Properties

Beam Size

bw	550	mm	D =	600	mm
tff	150	mm	Cover =	50	mm
Actual length, L =	5000	mm	Overall depth of Sect. D =	750	mm
Effective Length,Lef =	4600	mm	Area, A =	4.68E+05	mm^2
Neutral Axis, yc =	366.35	mm	Moment of Inertia, Ix-x =	2.83E+10	mm^4
Neutral Axis, xc =	735.00	mm	Moment of Inertia, Iy-y =	4.59E+10	mm^4
Effective width, beff =	1470.00	mm	Gross Area, Ag =	5.51E+05	mm^2

3 Design Parameters

F'c =	32	MPa	D =	600	mm
Fsy =	410	MPa	Ec =	28599.62293	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.822	

4 INITIAL CHOICE OF SECTION

Effective width = 1470 mm clause 8.8.2

Is the section RECT or T , L (ans : 1 or 2) = 1

k_1	=	0.045	(RECT sections)	1
	=	0.02411424	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	56.345		
defln/Lef	=	0.004	k_1/k_2 = 17.27977882	
d =		124 mm		
take d =		658 mm	Distance from d to extreme fibre of beam (do) =	92 mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	357.00	kNm	(left support)
Mmax +ve	=	357.00	kNm	(mid-span)
M -ve	=	357.00	kNm	(right support)
Vmax	=	161.00	kN	
V (other end)	=	161.00	kN	
N*	=	17.00	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 2

take do = 92 mm d = 658

Estimate Area of main steel Ast = 1729.804532 mm^2

6 Y 20 Ast = 1884.955592 mm^2

16 p = 0.005208498

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.00166672 < pd = 0.021813

p-pc = 0.003541779 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0.064948 dsc = 70 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 434 kNm	>	M* = 357.00 kNm	OK

Determine - ve steel at right support :

M -ve = 357.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 92 mm dst = 658 mm

Estimate Area of main steel Ast = 1729.804532 mm^2

6 Y 20 Ast = 1884.955592 mm^2

p = 0.005208498

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.00166672 < pd = 0.021813

p-pc = 0.003541779

Check M* <= phiMu

ku = 0.064948 dsc = 70 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 434 kNm	>	M* = 357.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 357 kNm

Steel in one layer then code = 1 take dist between centroid of

" " two " " code = 2 bar/s and soffit = 92 mm

what is the code ? 2 d = 658 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1729.80453 mm^2

6 Y 20 Ast = 1884.955592 mm^2

3 Y 16 Asc = 603.1857895 mm^2

Assume N - A in flange

Effective width : beff = 1470 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.0357356

hence dn = 24 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 451 kNm	>	M*+ve = 357.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 20
	Asc	=	3	Y 16
Right support :	Ast	=	6	Y 20
	Asc	=	3	Y 16
midspan	Ast	=	6	Y 20
	Asc	=	3	Y 16

7 STEEL CURTAILMENT REQUIREMENT CHECK

The following requirements regarding curtailment are subject to stress development length requirements.

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

i) 1/4 Ast to continue over length of beam

For -ve steel = 2 bars

ii) Remainder to be curtailed at 0.3Ln from face of support.

= 0.3 x le = 1380 mm

For +ve steel : i) 1/2 Ast must be carried into outer support

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 50 mm

d_b = 16 mm # of lines of bars = 1

a_b = 426 mm distance between bars

If a_b > 2c C = 2c + d_b = 116 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 442 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 426 mm (dist between bars)

factor = 40074.8041

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 345.4724 mm

take fs/Fsy = M*/Phi Mu = 0.823528935

therefore

Lsy =	285 mm	≥ 25k1db 500
say =	500 mm	

For Bottom Steel :

Cover, c = 50 mm d_b = 20 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 120 mm

if ab < 2c C = a_b + d_b = 694 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 674 mm (dist between bars)

factor = 50093.50513

Lsyt = 417.4459 mm take fs/Fsy = M*/Phi Mu = 0.791663762

therefore

Lsy =	330 mm	≥ 25k1db 400
say =	400 mm	

9 SHEAR CHECK

V* = 161.00 kN — Dist to centreline of bottom steel from soffit

critical location : d = 658 mm — dq = 70 mm; do = 530 mm

V* = 138 kN — beta 1 = 1.135 (but not less than 1.1)

hence = 1.135

beta 2 = 1

beta 3 = 1 Unity

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 137.06 kN

phi Vumax = 1305.92 kN > V* No web crushing

phi Vumin = 259 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm — Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 268.292683

Asv.min = 93.902439 mm^2

Asv.max = 1537 mm^2

Asv = 2 x area of stirrups = 220 mm^2 — 2 Ties

theta = 31.31082 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 588.76 kN

Phi Vu = 412 kN > V* = 138 kN

hence

PROVIDE : 1 Y12 LIGS @ 200 centres

Other Support :

V* = 161.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 658 mm — dq = 70 mm; do = 530 mm

V* = 138.36 kN — beta 1 = 1.135 (but not less than 1.1)

hence = 1.135

beta 2 = 1

beta 3 = 1

phi Vuc = phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333

= 137.056 kN

phi Vumax = 1305.92 kN > V* No web crushing

phi Vumin = 259 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm — Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 268.292683

Asv.min = 93.902439 mm^2

Asv.max = 1536.86675 mm^2

Asv = 2 x area of stirrups = 330 mm2 — 2 Ties

theta = 32.4543 degrees

Determine Phi Vu :

Vu = Vuc + Vus

= Vuc + Asv / s x Fsyf x do cot theta

= 759.5891 kN

Phi Vu = 531.7124 kN > V* = 138.36 kN

hence

PROVIDE : 1 Y12 LIGS @ 200 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	357.00 kNm	Es	=	200000 MPa
Ml	=	357.00 kNm	Ec	=	28599.62293 MPa
Mo	=	714 kNm			
Ms	=	357.00 kNm			
Ig	=	2.83E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1470 mm	A_{sc} =	603.1857895 mm^2
webwidth	b_w =	550 mm	A_{st} =	1884.955592 mm^2
Flange thickness	t_{ff} =	150 mm	d_{sc} =	92 mm
			d_{st} =	658 mm
			d =	658 mm
			D =	750 mm

n = Es/Ec = 6.99309919

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

275 dn^2 + 154796.634 dn - 19356121.99) = 0

dn = 105.332022 mm

d = 658 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 4.63E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 175.008928 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]

= 7.41E+09 mm^4

Calculate Icr , Mcr , ILef and Iref at end supports :

b_w =	550 mm	A_{st} =	1884.955592 mm^2
d_{sc} =	70 mm	A_{sc} =	603.1857895 mm^2
d =	658 mm		

Ig = 2.83E+10 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

275 dn^2 + 16796.6337 dn - 8926593.037) = 0

dn = 152.20 mm

d = 658 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.02E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 112.005714 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 4.77E+09 mm^4

At left end (arbitrary)

b_w = 550 mm

d_{sc} = 70 mm

A_{st} = 1884.9556 mm^2

A_{sc} = 603.18579 mm^2

d = 658 mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

275 dn^2 + 16796.6337 dn - 8926593.037) = 0

dn = 152.198153 mm

d = 658 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.02E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 112.005714 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 4.77E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 6.09E+09 mm^4

Midspan Deflection :

Lef = 4600 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 3.61534373 mm

Total Deflection :

Long term factor	=	0.2			
Short term factor	=	0.5			
w - long term	=	34.2 kN/m			
w - short term	=	34.275 kN/m			
Defln,s,sus	=	3.607432691 mm			
At midspan :			Ast	=	1884.95559 mm^2
			Asc	=	603.185789 mm^2
Asc/Ast =		0.32			
kcs	=	2 - (1.2 x Asc/Ast)		>= 0.8	
	=	1.616			

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)
= 8.50046 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN =	Lef / 250 =	18.4 mm	
		> 0.9 x Total Defln	
	Lef / 500 =	9.2 mm	
		> 0.9 x Total Defln	

Reinforced Masonry Block Wall Design					
REFERENCE:		**Concrete Structures - 3rd Edition**		**AS 3600 - 2009**	
		R. F. Warner, B. V. Rangan, & A. S. Hall			

Step	Calculation & Description				Result	Unit
1	Data					
	N*.max	=			515	kN
	M*.max	=			1200	kNm
	V*.max	=			515	kN
	f'c	=			15	MPa
	fsy	=			410	MPa
	εsy				0.002	
	Lw	=	... overall length of wall ...		5000	mm
	Hw	=	... unsupported height of wall ...		7000	mm
	tw	=	... wall thickness ...		200	mm
	Hwe	=	0.8 Hw, effective wall height		5600	mm
	ChecK: Hwe/tw	=	28	<	30	**OK**
2	Check maximum shear strength					
	dw	=	0.8Hw		5600	
	Θ	=	tan-1(dw/Hw)		0.674741	radians
	Θ	=		$30^0 \le \Theta \le 60^0$	38.7	degrees
	k3	=	(0.6+10/f'c) ≤ 0.85		1.267	
	k3f'c	=			19	MPa
	Vu.max	=	k3f'ctwdwsinΘcosΘ/(1.14+0.68cot^2Θ)		**4713**	kN
	Θvu.max	=			3299	kN
	Θvu.max	>	V* **OK**			
3	Vertical steel in the wall					
	Gross concrete area of the wall cross-section					
	Ag	=			1120000	mm^2
	N*/Ag	=			0.46	MPa
	In design, substituting for Vu from ϕVu ≥ V*, we obtain;					
	ϕlfsy	≥	V*cotΘ/ϕtwdw - N*/Ag	ϕ = 0.7		
		≥			0.36	MPa
	Al	≥	(ϕfsy/fsy)(twLw)	881.1909 mm^2	882	mm^2
	AsV	=	Al		882	mm^2
	Using	Y16	bars	area/bar = 200 mm^2		
	Now, minimum area vertical reinforcement, Ast.min = 0.0015twLw =				2500	mm^2
	Minimum Reinforcement Requirment Governs					
	# of bars	=	12.5		13	bars
	13 bars at each wall face and the average spacing is at;					
	Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		384.6154	mm CTS
	minimum bar spacing = 2.5tw or 500 mm whichever is less =				500	mm CTS
	Use Y12 bars at		384.6154 mm CTS at each wall face.			
	# of bars	=	2(1+ (Lw - 2b-Ast/bar)/spacing)		13	bars
	AsV (provided) =				2600	mm^2
	Therefore;					
	pl	=	[(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t\		0.003	
	It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.					
	In this case;					
	pl	=	0.0034 >	0.0025	**OK**	
4	Horizontal steel in the wall					
	For adequate control of cracking due to restrained shrinkage and temperature effects, the minimum value of horizontal steel ratio pt is taken as;					
	pt	=			0.0012	
	Ast.min	=	0.0015 twHw		1680	mm^2
	For	Y12	bars,	area/bar = 110 mm^2		
	# of bars	=	15.27273		16	bars
	16 bars at each wall face and the average spacing is at;					
	# of bars	=			16	bars
	Spacing of bars	=	(Hw)/(# bars @ each wall face)		438	mm CTS
	Therefore, use Y16 bars		400 mm CTS at each wall face.			

5 **Check flexural strength of wall**

The flexural strength of the wall is calculated using an equivalent rectangular stress block for compression zone concrete, like columns subjected to axial compression and bending moment.

Calculate rectangular stress block parameters.

α	=	0.85-0.004(f'c-55)			1.01
γ	=	0.85-0.008(f'c-30)	0.65	<	0.97
Take γ	=				0.97
Nu	=	N*/ϕ	ϕ = 0.6		858 kN

Trial depth of neutral axis, dn = 681 mm

Compressive force in concrete, Cc = 0.85γf'cbdr (include boundary element) 1684 kN

dc = γdn/2 330 mm

(0.003/dn) (dn - dsi)

Raw #	# of bars	Asi (mm^2)	bar cts (mm	dsi (mm)	εsi	δsi (MPa)	Fsi (kN)	Fsidsi (kNmm)	M = Fsi(dq-dsi) (kNmm)
1	1	200	100	100	0.0026	410	82	8200	196800
2	1	200	200	300	0.0017	336	67	20140.97	147700
3	1	200	400	700	-0.0001	-33	-7	-4687.225	-12053
4	1	200	400	1100	-0.0018	-738	-148	-162431.7	-206731
5	1	200	400	1500	-0.0036	-410	-82	-123000	-82000
6	1	200	400	1900	-0.0054	-410	-82	-155800	-49200
7	1	200	400	2300	-0.0071	-410	-82	-188600	-16400
8	1	200	400	2700	-0.0089	-410	-82	-221400	16400
9	1	200	400	3100	-0.0107	-410	-82	-254200	49200
10	1	200	400	3500	-0.0124	-410	-82	-287000	82000
11	1	200	400	3900	-0.0142	-410	-82	-319800	114800
12	1	200	400	4300	-0.0159	-410	-82	-352600	147600
13	1	200	400	4700	-0.0177	-410	-82	-385400	180400
14	1	200	200	4900	-0.0186	-410	-82	-401800	196800
∑	14	2800	100	5000			-825	-2828378	765316

4 Axial Strength of the wall;

Nu	=	∑Fsi + Cc	[redacted] kN
dN	=	1/Nu (Ccdc + ∑Fsidsi)	-2644.26 mm
dq	=	0.5D ... plastic centroid ...	2,500 mm
e	=	dq-dN ... eccentricity ...	5,144 mm

Summing up moments of forces about the palsic centroid gives;

Mu	=		4,420 kNm
ϕMu	=		2,652 kNm
e	=	Mu/Nu	5,144 mm
Therefore, Nu	=		859 kN

ϕMu = 2,652 kNm > M* = 1200 kNm

Therefore, the flexural strength of the wall crosection is adequate to carry the design bending moment

Reinforced Masonry Block Wall Design

REFERENCE: **Reinforced concrete slab - 3rd Edition** **AS 3600 - 2009**

R. F. Warner, B. V. Rangan, & A. S. Hall

Step	Calculation & Description				Result	Unit
1	Data					
	N*.max	=			**515**	kN
	M*.max	=			**1200**	kNm
	V*.max	=			**515**	kN
	f'c	=			**15**	MPa
	fsy	=			**410**	MPa
	εsy				**0.002**	
	Lw	=	... overall length of wall ...		**5000**	mm
	Hw	=	... unsupported height of wall ...		**7000**	mm
	tw	=	... wall thickness ...		**200**	mm
	Hwe	=	0.8 Hw, effective wall height		5600	mm
	ChecK: Hwe/tw	=	28	<	30	**OK**
2	Check maximum shear strength					
	dw	=	0.8Hw		5600	
	ϴ	=	tan-1(dw/Hw)		0.674741	radians
	ϴ	=		30° ≤ ϴ ≤ 60°	38.7	degrees
	k3	=	(0.6+10/f'c) ≤ 0.85		1.267	
	k3f'c	=			19	MPa
	Vu.max	=	k3f'ctwdwsinϴcosϴ/(1.14+0.68cot^2ϴ)		**4713**	kN
	ϴvu.max	=			3299	kN
	ϴvu.max	>	V* **OK**			
3	Vertical steel in the wall					
	Gross concrete area of the wall cross-section					
	Ag	=			1120000	mm^2
	N*/Ag	=			0.46	MPa
	In design, substituting for Vu from ϕVu ≥ V*, we obtain;					
	ϕlfsy	≥	V*cotϴ/ϕtwdw - N*/Ag	ϕ = 0.7		
		≥			0.36	MPa
	Al	≥	(ϕfsy/fsy)(twLw)	881.1909 mm^2	**882**	mm^2
	AsV	=	Al		882	mm^2
	Using **Y16**		bars	area/bar = **200** mm^2		
	Now, minimum area vertical reinforcement, Ast.min = 0.0015twLw =				2500	mm^2
	Minimum Reinforcement Requirment Governs					
	# of bars	=	12.5		**13**	bars
	13 bars at each wall face and the average spacing is at;					
	Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		384.6154	mm CTS
	minimum bar spacing = 2.5tw or 500 mm whichever is less =				500	mm CTS
	Use Y12 bars at		384.6154 mm CTS at each wall face.			
	# of bars	=	2(1+ (Lw - 2b-Ast/bar)/spacing)		13	bars
	AsV (provided) =				2600	mm^2
	Therefore;					
	pl	=	[(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t		0.003	
	It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.					
	In this case;					
	pl	=	0.0034 > 0.0025		**OK**	
4	Horizontal steel in the wall					
	For adequate control of cracking due to restrained shrinkage and temperature effects, the minimum value of horizontal steel ratio pt is taken as;					
	pt	=			0.0012	
	Ast.min	=	0.0015 twHw		1680	mm^2
	For **Y16**		bars,	area/bar = **200** mm^2		
	# of bars	=	8.4		**9**	bars
	9 bars at each wall face and the average spacing is at;					
	# of bars	=			9	bars
	Spacing of bars	=	(Hw)/(# bars @ each wall face)		778	mm CTS
	Therefore, use Y16 bars		400 mm CTS at each wall face.			

5 Check flexural strength of wall

The flexural strength of the wall is calculated using an equivalent rectangular stress block for compression zone concrete, like columns subjected to axial compression and bending moment.

Calculate rectangular stress block parameters.

α	=	0.85-0.004(f'c-55)			1.01	
γ	=	0.85-0.008(f'c-30)	0.65	<	0.97	
Take γ	=				0.97	
Nu	=	N*/ϕ ϕ = 0.6			858	kN
Trial depth of neutral axis, dn	=				681	mm
Compressive force in concrete, Cc	:	0.85γf'cbdr (include boundary element)			1684	kN
dc	=	γdn/2			330	mm

Raw #	# of bars	Asi (mm^2)	bar cts (mm	dsi (mm)	εsi (0.003/dn)	δsi (MPa) (dn - dsi)	Fsi (kN)	Fsidsi (kNmm)	M = Fsi(dq-dsi) (kNmm)
1	1	200	100	100	0.0026	410	82	8200	196800
2	1	200	200	300	0.0017	336	67	20140.97	147700
3	1	200	400	700	-0.0001	-33	-7	-4687.225	-12053
4	1	200	400	1100	-0.0018	-738	-148	-162431.7	-206731
5	1	200	400	1500	-0.0036	-410	-82	-123000	-82000
6	1	200	400	1900	-0.0054	-410	-82	-155800	-49200
7	1	200	400	2300	-0.0071	-410	-82	-188600	-16400
8	1	200	400	2700	-0.0089	-410	-82	-221400	16400
9	1	200	400	3100	-0.0107	-410	-82	-254200	49200
10	1	200	400	3500	-0.0124	-410	-82	-287000	82000
11	1	200	400	3900	-0.0142	-410	-82	-319800	114800
12	1	200	400	4300	-0.0159	-410	-82	-352600	147600
13	1	200	400	4700	-0.0177	-410	-82	-385400	180400
14	1	200	200	4900	-0.0186	-410	-82	-401800	196800
∑	14	2800	100	5000			-825	-2828378	765316

4 Axial Strength of the wall;

Nu	=	∑Fsi + Cc	[redacted]	kN
dN	=	1/Nu (Ccdc + ∑Fsidsi)	-2644.26	mm
dq	=	0.5D ... plastic centroid ...	2,500	mm
e	=	dq-dN ... eccentricity ...	5,144	mm

Summing up moments of forces about the palsic centroid gives;

Mu	=		4,420	kNm
ϕMu	=		2,652	kNm
e	=	Mu/Nu	5,144	mm
Therefore, Nu	=		859	kN

ϕMu = 2,652 kNm > M* = 1200 kNm

Therefore, the flexural strength of the wall crosection is adequate to carry the design bending moment

RC COLUMN DESIGN WITH MRCSECT PACKAGE

INTERACTION DIAGRAM FOR RECTANGULAR COLUMN SECTION
(NON-PRESTRESSED REINFORCEMENT ONLY)

INPUT

400 x 400 RC COLUMN DESIGN

Using RECTANGULAR STRESS BLOCK

Width (mm)...................= 400
Depth (mm)...................= 400
No of lines of reinforcement.= 3
Concrete strength (MPa)......= 32
Yield stress of steel (MPa)..= 410

STEEL DETAILS

Line no	Depth to steel	Bar diameter	No of bars
1	64	24	3
2	250	24	2
3	336	24	3

OUTPUT

Depth to Plastic Centroid (Ref Axis for Moment) = 202.701

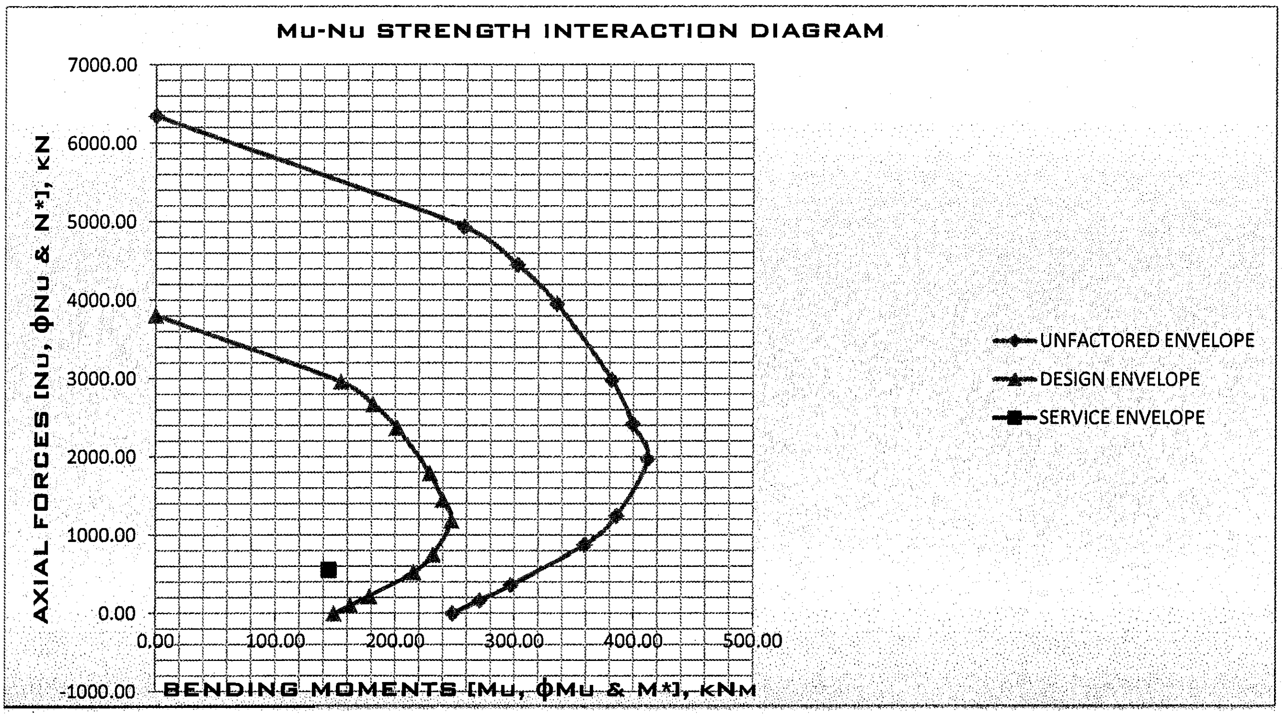
UNFACTORED ENVELOPE

Eccentricity (mm)	Force (kNm)	Moment (kNm)	Top Strain	Bottom Strain	N.A.factor ku
0.00	6346.14	0.00	0.00300	0.00300	1000.000
52.22	4934.29	257.66	0.00300	0.00000	1.000
67.91	4453.14	302.39	0.00300	-0.00030	0.909
84.85	3953.31	335.43	0.00300	-0.00070	0.965
127.88	2985.27	381.77	0.00300	-0.00160	0.776
165.22	2418.33	399.56	0.00300	-0.00230	0.696
208.67	1973.80	411.87	0.00300	-0.00290	0.636
309.20	1247.45	385.71	0.00300	-0.00450	0.524
408.36	879.16	359.02	0.00300	-0.00580	0.452
822.56	361.37	297.25	0.00300	-0.00900	0.332
1647.29	164.48	270.94	0.00300	-0.01064	0.292
	-0.62	248.03	0.00300	-0.01224	0.261

DESIGN ENVELOPE

PHI	FORCE(kN)	MOMENT(kNm)
.6	3807.687	0
.6	2960.577	154.5967
.6	2671.885	181.4348
.6	2371.985	201.2593
.6	1791.162	229.0591
.6	1450.996	239.7

UNFACTORED ENVELOPE		NEUTRAL AXIS FACTOR		DESIGN ENVELOPE		SERVICE ENVELOPE	
Mu (kNm)	Nu (kN)	ku	phi (ϕ)	ϕMu (kNm)	ϕNu (kN)	M* (kNm)	N* (kN)
0.00	6346.14	1000.000	0.60000	0.00	3807.68	144.45	554.00
257.66	4934.29	1.000	0.60000	154.60	2960.57		
302.39	4453.14	0.909	0.60000	181.43	2671.88		
335.43	3953.31	0.965	0.60000	201.26	2371.99		
381.77	2985.27	0.776	0.60000	229.06	1791.16		
399.56	2418.33	0.696	0.60000	239.74	1451.00		
411.87	1973.80	0.636	0.60000	247.12	1184.28		
385.71	1247.45	0.524	0.60000	231.43	748.47		
359.02	879.16	0.452	0.60000	215.41	527.50		
297.25	361.37	0.332	0.60000	178.35	216.82		
270.94	164.48	0.292	0.60000	162.56	98.69		
248.03	-0.62	0.261	0.60000	148.82	-0.37		


Mu-Nu STRENGTH INTERACTION DIAGRAM
AXIAL FORCES [Nu, ϕNu & N*], kN
BENDING MOMENTS [Mu, ϕMu & M*], kNm
7000.00
6000.00
5000.00
4000.00
3000.00
2000.00
1000.00
0.00
-1000.00
0.00
100.00
200.00
300.00
400.00
500.00
UNFACTORED ENVELOPE
DESIGN ENVELOPE
SERVICE ENVELOPE

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

REFERENCE: **Concrete Structures - 3rd Edition** AS 3600 - 2009
R. F. Warner, B. V. Rangan, & A. S. Hall

Design of a Continuous Beam :

1 Loads

DL	:	Wdl	=	33.60 kN/m	(plus super)
Self wt	:	Wsw	=	25.00 kN/m	
Blockwall	:	Wb	=	0.00 kN/m	
TOTAL			=	58.60 kN/m	
LL :		Wll	=	0.25 kN/m	
LL factor	- long term			0.4	
	- short term			0.7	

2 Beam Section Properties

Beam Size

bw =	5000 mm	D =	200 mm
tff =	0 mm	Cover =	30 mm
Actual length, L =	1400 mm	Overall depth of sect. D =	200 mm
Effective Length,Lef =	1000 mm	Area, A =	1.00E+06 mm^2
Neutral Axis, yc =	100.00 mm	Moment of Inertia, Ix-x =	3.33E+09 mm^4
Neutral Axis, xc =	2600.00 mm	Moment of Inertia, Iy-y =	2.08E+12 mm^4
Effective width, beff =	5200.00 mm	Gross Area, Ag =	1.00E+06 mm^2

3 Design Parameters

F'c =	40 MPa	Fsy =	410 MPa
m =	2400 kg/m^3	D =	200 mm
Ec =	31975.35051 MPa	Es =	200000 MPa
Gamma =	0.766		

4 Initial Choice of Section

Effective width = 5200 mm clause 8.8.2

Is the section RECT or T, L (ans : 1 or 2) = 1

k_1	=	0.045	(RECT sections)	1
	=	0.04346021	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6

k_2	=	0.0026042	
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)
Fdef	=	96.443	
defln/Lef	=	0.004	k_1/k_2 = 17.27977882
d	=	20 mm	
take d =		154 mm	Distance from d to extreme fibre of beam (do) = 46 mm

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	60.30 kNm	(left support)
Mmax +ve	=	60.30 kNm	(mid-span)
M -ve	=	60.30 kNm	(right support)
Vmax	=	127.50 kN	
V (other end)	=	127.50 kN	
N*	=	0.00 kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine -ve steel required : (At arbitrary left support)

Number of reinforcement layers 1 or 2 : 1

take do = 46 mm d = 154

Estimate Area of main steel		Ast =	1248.392927 mm^2	
6	Y 16	Ast =	1206.371579 mm^2	SPACING = 823
		p =	0.001566716	
2	Y 16	Asc =	402.1238597 mm^2	
		pc =	0.000522239	< pd = 0.025409
		p-pc =	0.001044478	pd=0.4*0.85*ku*f'c/fsy

Check M* <= phiMu

ku = 0.016442781 dsc = 38 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 63 kNm > M* = 60.30 kNm OK

Determine - ve steel at right support :

M -ve = 60.30 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 46 mm dst = 154 mm

Estimate Area of main steel Ast = 1248.392927 mm^2

6 Y 16 Ast = 1206.371579 mm^2

p = 0.001566716

2 Y 16 Asc = 402.1238597 mm^2

pc = 0.000522239 < pd = 0.025409

p-pc = 0.001044478

Check M* <= phiMu

ku = 0.016442781 dsc = 38 mm

phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]

= 63 kNm > M* = 60.30 kNm OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 60.3 kNm

Steel in one layer then code = 1 take dist between centroid of

" " two " " code = 2 bar/s and soffit = 46 mm

what is the code ? 1 d = 154 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1248.392927 mm^2

Bottom 6 Y 16 Ast = 1206.371579 mm^2

Top 2 Y 16 Asc = 402.1238597 mm^2

Assume N - A in flange

Effective width : beff = 5200 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)

= 0.02371555

hence dn = 4 < flange thickness

hence N -A is in flange

Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 68 kNm > M*+ve = 60.30 kNm OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 16
	Asc	=	2	Y 16
Right support :	Ast	=	6	Y 16
	Asc	=	2	Y 16
midspan	Ast	=	6	Y 16
	Asc	=	2	Y 16

7 **STEEL CURTAILMENT REQUIREMENT CHECK**

The following requirements regarding curtailment are subject to stress development length requirements.

For -ve steel
- i) 1/4 Ast to continue over length of beam = 2 bars
- ii) Remainder to be curtailed at 0.3Ln from face of support. = 0.3 x le = 300 mm ?

For -ve steel
- i) 1/4 Ast to continue over length of beam = 2 bars
- ii) Remainder to be curtailed at 0.3Ln from face of support. = 0.3 x le = 300 mm ?

For +ve steel :
- i) 1/2 Ast must be carried into outer support = 2 bars
- ii) 1/4 Ast into interior support = 2 bars
- iii) Remainder curtailed at 0.1Ln from supports = 0.1 x le = 100 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 30 mm

d_b = 16 mm # of lines of bars = 2

a_b = 2458 mm distance between bars

If a_b > 2c C = 2c + d_b = 76 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 2474 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 2458 mm (dist between bars)

factor = 35843.99446

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 471.6315061 mm

take fs/Fsy = M*/Phi Mu = 0.963331

therefore

Lsy =	454 mm
say =	650 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 30 mm d_b = 16 mm

# of lines of bars = 2

If ab > 2c C = 2c + d_b = 76 mm

if ab < 2c C = a_b + d_b = 2558 mm

k1 = 1 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 2542 mm (dist between bars)

factor = 28675.19557

Lsyt = 377.3052049 mm take fs/Fsy = M*/Phi Mu = 0.887670676

therefore

Lsy =	335 mm
say =	500 mm

≥ 25k1db 400

9 PUNCHING SHEAR CHECK

Coloumn Dimension

V* = 80.4 kN b = 400 mm

Dist to centreline of bottom d = 400 mm

steel from soffit Critical shear perimeter, μ

dq = 38 mm μ = 2(b+dom) + 2(d+dom)

dom = 162 mm 2248 mm

Area of slab: B = 1000 mm D = 1000 mm

βh = 1 longer side/shorter side

fcv = 0.17(1+2/βh)√f'c ≤ 0.34√f'c

fcv = 13.60 MPa

ϕVuo = 3,467 kN > V* = 80.4 kN OK

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 60.30 kNm Es = 200000 MPa

Ml = 60.30 kNm Ec = 31975.35051 MPa

Mo = 120.6 kNm

Ms = 60.30 kNm

Ig = 3.33E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width b_{eff} = 5200 mm A_{sc} = 402.1238597 mm^2

webwidth b_w = 5000 mm A_{st} = 1206.371579 mm^2

Flange thickness t_{ff} = 0 mm d_{sc} = 46 mm

d_{st} = 154 mm

d = 154 mm

D = 200 mm

n = Es/Ec = 6.25481807

1/2 x bw x dn^2 + (beff - bw) x t (dn - t/2) + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1259229.786) = 0

dn = 20.5943193 mm

d = 154 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2

= 1.49E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 126.4911064 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	2.95E+10 mm^4

Calculate Icr , Mcr, ILef and Iref at end supports :

b_w =	5000 mm	A_{st} =	1206.371579 mm^2	
d_{sc} =	38 mm	A_{sc} =	402.1238597 mm^2	
d =	154 mm			

Ig = 3.33E+09 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1242325.084) = 0

dn = 20.44 mm

d = 154 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.49E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 126.4911064 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^: = 2.95E+10 mm^4

At left end (arbitrary)

b_w =	**5000** mm
d_{sc} =	**38** mm
A_{st} =	**1206.371579** mm^2
A_{sc} =	**402.1238597** mm^2
d =	**154** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)

2500 dn^2 + 9658.72247 dn - 1242325.084) = 0

dn = 20.4437258 mm

d = 154 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.49E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 126.4911064 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.95E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 2.95E+10 mm^4

Midspan Deflection :

Lef = 1000 mm

Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.00531936 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 58.65 kN/m

w - short term = 58.725 kN/m

Defln,s,sus = 0.005313 mm

At midspan : Ast = 1206.371579 mm^2

Asc = 402.1238597 mm^2

Asc/Ast = 0.333333

kcs = 2 - (1.2 x Asc/Ast) >= 0.8

= 1.6

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)

= 0.01382 mm

FROM A MORE ACCURATE ANALYSIS Warner,Rangan,Hall HAVE
SHOWN THAT THIS CAN BE REDUCED BY UP TO 10%.

ALLOWABLE DEFLN = Lef / 500 = 2 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

ISOLATED COLUMN FOOTING DESIGN

REFERENCE: Concrete Structures - 3rd Edition — Australian Standard (AS) 3600 -2009
R. F. Warner, B. V. Rangan, & A. S. Hall

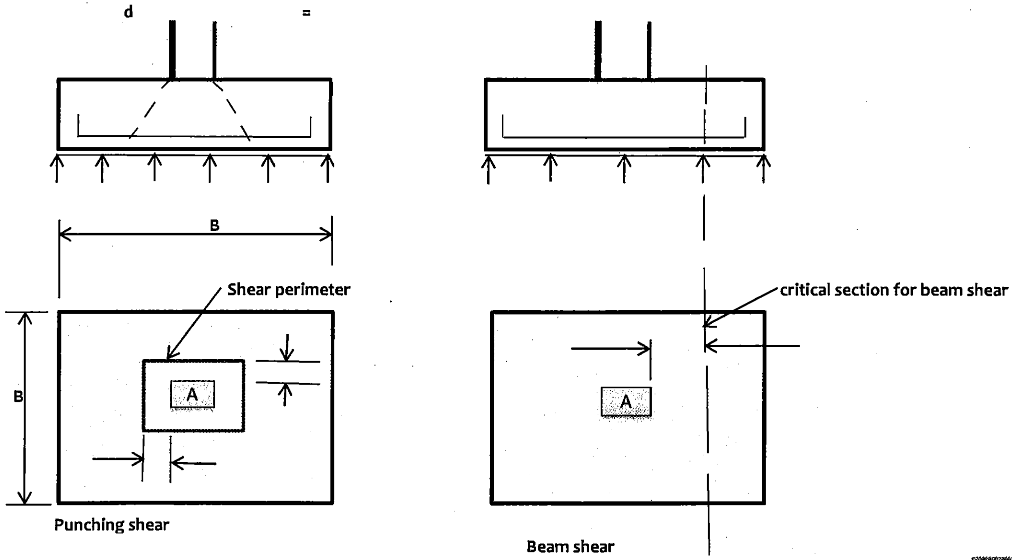
Calculations & Description						Result	Unit
1 Data							
DL	=	... DL from Column 1 ...				266	kN
LL	=	... LL from Column 1 ...				107	kN
P	=	DL + LL				373	kN
P*	=	1.2 DL	+	1.5 LL		479.7	kN
qa	=				0.175	450	kPa
f'c	=				3.85	32	MPa
fsy	=	... reinforcement...			0.525	410	MPa
col. Width A	=				4.55	0.4	m
col. Width B	=					0.4	m
2 Footing Area, A							
A	=	P*total/qa				0.828888889	m^2
Approximate the footing dimension as:							
B	=	short side				1.8	m
L	=	Long side				1.8	m
						3.24	m^2
qult	=	P^*_{total}/A				0.148055556	MPa

3 Determine the required effective depth by considering the beam shear on a critical section at distance d from the c2 mm face of the column.

V*	=	qult x B (C-Col.w/2-d)					
V*	=	186550	-d	266.5			N

The shear strenght of the section depends on the amount of tensile steel used for bending. We assume a minimum quatity of:

pmin	=	1.4/fsy				0.003414634	
V_{uc}	=	β_1BD(Ast/BD x f'c)^1/3					
V_{uc}	=	946.5929 d					N
Equating V* & φVuc							
662.615 d	=	186550	-d	266.5			
929.115 d	=	186550					
d	=					200.7824585	mm



Cover	=					75	mm
Assume bar dia		Y20				20	mm
D	=	d + cover + bar dia/2				285.7824585	mm
Take D	=					[redacted]	mm
d	=	D-cover-bar dia/2				315	mm
d_{om}	=	D-cover-bar dia				305	mm
4 Check Punching Shear							
Critical shear perimeter is a rectangle of side,			400	+	305	705	mm
		and	400	+	305	705	mm
punching shear area	=					497025	mm^2
μ	=	2(c1+dom) + 2(c2+dom)				2820	mm
V*	=	P* - qult x punching shear area				406.1126875	kN
V*	=	qult(A-punching shear area)				406.1126875	kN
f_{cv}	=	$0.17(1+2/\beta_h)\sqrt{f'c}$	≤	$0.34\sqrt{f'c}$			
β_h	=	Col. Length/col.width				1	
f_{cv}	=				0.34 f'c		
ϕV_{uo}	=	$\phi \mu d_{om} f_{cv}$				1157.979561	kN
						OK, SINCE φVuo > V*B	

5 Tensile steel for bending in longitudinal direction

The critical section for bending is at the longer face of the column, where the length of the cantilevered portion is:

l	=	0.5(B-C2)		0.7	m
M*	=	qult x B x l^2/2		65.2925	kNm
Ast.reqd.	=	M*/φ0.9dfsy		702.16	mm^2
Ast.min.	=	1.4/fsy bd		1,936.10	mm^2
			Ast.min. Governs, Therefore, use Ast.min.		
Hence provide		7 Y20	bars in longitudinal direction		
Astprovide	=			2,170.00	mm^2

6 Tensile steel for bending in the transverse direction.

Considering a transverse strip 1 m wide:

l	=	0.5(L-C1)		0.7	m
M*	=	qult x 1m x l^2/2		36.27361111	kNm
Ast.reqd.	=	M*/φ0.9dfsy		390.09	mm^2/m
Ast.min.	=	1.4/fsy bd		1,075.61	mm^2/m
			Ast.min. Governs, Therefore, use Ast.min.		
Ast required for the full length				1,936.10	mm^2
Hence provide		7 Y20	bars in transverse direction		
Astprovide	=			2,170.00	mm^2

7 Spacing between the bars

Ast/bar	=			310	mm^2
S	=	[Ast/bar X beff]/Ast.prov.	... centre - to - centre spacing ...	258	mm

8 Now check 7 bars @ 258 CTS

T = fsyAst	=							889.7	kN
C = T	=							889.7	kN
A	=	C/0.85f'c						32709.6	mm^2
γkud	=	A/b						18.2	mm
a	=	γkud/2						9.1	mm
l	=	d-a						305.9	mm
Mu(prov)	=	Txl						272.2	kN
φMu	=	217.7 kNm		>	M*	=	65.3 kNm		OK
γ	=	0.85-0.007(f'c-28)						0.822	
ku	=	γkud/γd	0.07	<	0.4	UNDER-REINFORCED.Therefore, OK			

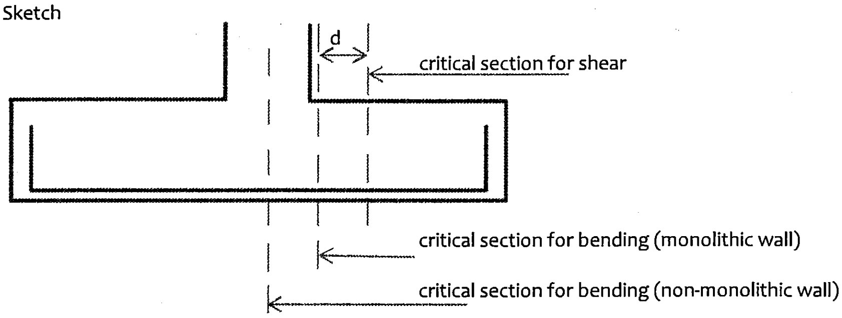
THEREFORE, USE 7 BARS @ 258 AT TOP & BOTTOM AND IN BOTH DIRECTIONS, Ast PROVIDED 2170 mm^2

STRIP FOOTING DESIGN FOR WALL

REFERENCE: **Concrete Structures - 3rd Edition** **Australian Standard (AS) 3600 -2009**

R. F. Warner, B. V. Rangan, & A. S. Hall

Step	Calculations & Description				Result	Unit
	Sketch					



Step				Description	Result	Unit
	Strip Footing for Wall					
1	Data	RC WALL		kN/m^3 mm		
	UDL:	Self wt	=	Density x Thkness 23 200	4.6	kPa
		DL	=		27.246	kN/m
	L		=	span	1	m
	Total DL		=		31.846	kN/m
		LL	=		18	kN/m
	Conc. Load	DL	=			
		LL	=			
	P		=	DL + LL	49.846	kN
	P*		=	1.4 DL + 1.7 LL	75.1844	kN
	qa		=		450	kPa
	f'c		=		40	MPa
	fsy		=	... reinforcement...	410	MPa
	c		=	... width of wall	0.2	m
2	Footing Area, A					
	A		=	P*total/qa	0.1107689	m^2
	Approximate the footing dimension as:					
	D		=	A/L	0.1107689	m
			=		[redacted]	m
	Mmax		=	1/8 P*(D-c)	7.51844	kNm
	qult		=	P^*_{total}/A	0.0751844	MPa
3	Determine the required effective depth by considering the beam shear on a critical section at distance d from the c mm face of the wall.					
	V*		=	qult x B (C-Col.w/2-d)		
	V*		=	30073.76 - 75.1844 d		N
	The shear strenght of the section depends on the amount of tensile steel used for bending. We assume a minimum quatity of:					
	pmin		=	1.4/fsy	0.0034146	
	V_{uc}		=	β_1BD(Ast/BD x f'c)^1/3		
	V_{uc}		=	566.4923836 d		N
	Equating V* & φVuc					
	396.5447 d		=	30073.76 - 75.1844 d		
	471.7291 d		=	30073.76		
	d		=		63.752187	mm
	Cover		=		75	mm
	Assume bar dia			Y16	16	mm
	D		=	d + cover + bar dia/2	146.75219	mm
	Take D		=		[redacted]	mm
	d		=	D-cover-bar dia/2	317	mm
4	Tensile steel for bending in longitudinal direction: Oneway Bending. The critical section for bending is at the longer face of the column, where the length of the cantilevered portion is:					
	L		=	0.5(B-c)	0.4	m
	M*		=	qult x B x L/2	6.014752	kNm
	Ast.reqd.		=	M*/φ0.9dfsy	64.275004	mm^2
	Ast.min.		=	1.4/fsy bd	1082.439	mm^2
				Ast.min. Governs, Therefore, use Ast.min.		
	Hence provide			6Y16 bars in longitudinal direction top & bottom.		
	Astprov		=	200 Ast/bar	1200	mm^2
5	Spacing between the bars					
	Ast/bar		=		200	mm^2
	S		=	[Ast/bar X beff]/Ast.prov. ... centre - to - centre spacing ...	167	mm