4 SUMMARY

SIZE : 600 PD X 400 WD

REINFORCEMENT		
MID SPAN	3Y16	TOP
	3Y16	BOTTOM
AT SUPPORTS	3-Y16	TOP
	3-Y16	BOTTOM
LENGTH OF BEAM	2-Y16	MID DEPTH OF BEAM
STIRRUPS	Y-12 AT 200 CTS	

RC ROOF BEAMS CRB2

SIZE : 600 PD X 400 WD

REINFORCEMENT		
MID SPAN	3Y16	TOP
	3Y20	BOTTOM
AT SUPPORTS	3-Y20	TOP
	3-Y16	BOTTOM
LENGTH OF BEAM	2-Y16	MID DEPTH OF BEAM
STIRRUPS	Y-12 AT 200 CTS	

G **RC COLUMN**

SIZE: 400 X 400 SQR

REINFORCEMENT	
VERTICAL	8-Y16
STIRRUPS	Y12 AT 200 CTS

H **HOIST BEAM**

SIZE: 310 UB 40.4

BEAM TO BE BOLTED FROM ITS TOP FLANGE TO THE UNDERSIDE OF THE RC ROOF BEAM WITH 4M20 "U" BOLTS 400 EMBEDDED.

I **ROOF PURLINS**

200DP X 50WD HWD TIMBER
SPACING 900 CTS

J **SUSPENDED FLOOR SLAB**

SIZE:	275 mm	THICK
REINFORCEMENT	Y16 Top & Bottom Eachway	

POM SEWERAGE PROJECT
OXIDATION DITCH & BLOWER CONTROL ROOM
DESIGNED TT
CHECKED
DATE 8/11/2011

MEMBER SUMMARY

A BASE SLAB

THICKNESS		850 mm
REINFORCEMENT		
Y32 AT 250 CTS	TOP OF TOP REINFORCEMENT WIDTH WISE	
Y32 AT 150 CTS	BOTTOM OF TOP REINFORCEMENT LENGTHWISE	
Y32 AT 250 CTS	BTTOM OF BOTTOM REINFORCEMENT WIDTHWISE	
Y32 AT 150 CTS	TOP OF BOTTOM REINFORCEMENT LENGTHWISE	

B MAIN WALLS

HAUNCH THICKNESS		850 mm
WALL THICKNESS		500 mm
REBAR		
	VERTICAL	Y32 AT 150 CTS
	HORIZONTAL	Y24 AT 150 CTS

C FIN WALLS

HAUNCH THICKNESS		350 mm
WALL THICKNESS		300 mm
REBAR	VERTICAL	Y24 AT 200 CTS
	HORIZONTAL	Y20 AT 150 CTS

D SUSPENDED WALKWAY SLABS

SIZE	275 mm	THICK
REINFORCEMENT	Y16 AT 170 CTS	T&B EACAH WAY

BLOWER ROOM & LOCAL CONTROL ROOM

E SUSPENDED RC FLOOR BEAMS CFB1

SIZE: 900 DP X 450 WD

REINFORCEMENT		
MID SPAN	3Y20	TOP
	4Y24	BOTTOM
AT SUPPORTS	4-Y24	TOP
	3-Y20	BOTTOM
LENGTH OF BEAM	2-Y16	MID DEPTH OF BEAM
STIRRUPS	Y-12 AT 300 CTS	

F RC ROOF BEAMS CRB1

G : Final Sedimentation Tank

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

Load Breakdown & Calculation

Data

Density of Water (ρ) =	9.81	kN/m^3
Height of Water, h =	4	m
Wall Thickness, tw =	0.4	m
Height of Wall, hw =	4	m
Base Slab Thickness, tb =	0.5	m
Length of Base slab, Lb =	25	m

1 Mechanical Loading

Mechanical Load on Wall =	46.00	kN
Mechanical Load on base slab =	267.00	kN

2.a. Hydrostatic Load for per metre length for wall

Atmospheric Pressure, Patm =	0	kPa
Hydrostatic Pressure along 1 m width wall, Pw	39.24	kN/m (=kPa/m)

2.b. Hydrostatic Load Calculation for per metre width strip for base

Water Pressure on base slab, Pw = ρgh 39.24 kPa

c. Hydrostatic Load per metre width strip for the base slab, Flp = ρgh (wb = 1.0)

Flp = 39.24 kN/m

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.40	m
Height of Wall (H) =	4.50	m
Embedded Height of Wall (He) =	3.50	m
(Heel Depth) Thickness of Base (tb) =	0.50	m
Width of Base (wb) =	1.00	m
(Heel)/ Length of base (Lb) =	25.00	m
Toe Depth (td) =	0.50	m
Toe length (tl) =	0.25	m
Height of Ground Water (Hw) =	0.00	m
Slope of ground surface behind the wall (β) =	42	degrees
Slope of back face of the wall (α) =	90	degrees
Angle of wall friction (δ') =	42	degrees

MATERIAL PROPERTIES

f'c =	40	MPa
fsy =	410	MPa
Concrete Density =	25	kN/m^3
Cover (c) =	65	mm
Max. allowable crack width (W) =	0.30	mm

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (ϕ') =	42	degrees	
Design cohesion of retained mat'l (C) =	0	kN/m^3	**(Granular backfill)**
Density of retained mat'l (γ) =	20	kN/m^3	
Submerged Density of retained mat'l (γs) =	20	kN/m^3	
Design angle of int'l friction of base mat'l (ϕb) =	45	degree	
Design cohesion of base mat'l (Cb) =	45	kN/m^3	**(Cohesionless soil)**
Density of base mat'l (γb) =	20	kN/m^3	
Allowable gross ground bearing pressure (GBP) =	450	kN/m^2	

LOADINGS (Unfactored)

Surcharge load -- live (Q) =	10	kN/m^2
Surcharge load -- dead (G) =		kN/m^2

LATERAL FORCES

K_O = 0.33 K_A = (1-SIN ϕ) =

Earth Pressure coefficients, Ka & Kp;

	Active, Ka	Passive, Kp
Rankines	0.74	0.74
Coulomb	0.74	0.74

PE = 0.5γKaH^2 (B=1.0 m)

Force (kN/m)		Lever arm (m)	
Pa (Fe) =	91.04	LE =	1.167
PS(G) =	26.01	LS =	1.75
PS(Q) =	0.00	LS =	1.75
Total, Pa =	117.05	La =	1.30

Earthquake Earth Pressure

REFERENCE:

NZS 3106 : 2009 - DESIGN OF CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS
SEISMIC DESIGN OF STORAGE TANKS "Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering" : 1986

Seismic Design Data

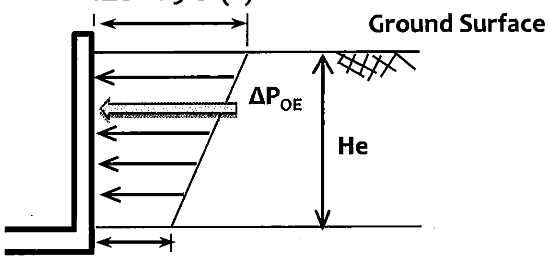
Location =	Port Moresby	
Category (pg 15) =	C	
Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15		
p =	0.002	
Tank Contents =	Non-volatile toxic chemicals	
probability factor; Fig. C2.7; pg 51 (Ap) =	1.85	
geographical coefficient representing regional seismicity; Fig. C2.1; pg 47		
β =	0.50	
peak horizontal acceleration coefficient; pg 51 (α) =	0.35	
horizontal force coefficient Ch(T) = α β Ah(T) Ap =	0.32	$A_h(T)$
normalized horizontal acceleration response =	Ah(T)	

4 **Foundation Rigid/Fixed Foundation on Weathered Mudstone**

Unit weight of engineering backfill soil (Υe) =	20.00	kN/m^3

5 **Earthquake Earth Pressure: NZS 3106, Appendix A, Cl. A2.6**

fΔe = 1.5 Ch(0) Υe He



fΔe = 0.5 Ch(0) Υe He

Increment in at-rest pressure due to earthquake

He = embedded depth =	3.5	m
Ah(0) =	1	
$C_h(0) = \alpha\ \beta\ A_h(0)\ A_p$, horizontal force coefficient =	0.324	
fΔe = 1.5 Ch(0) Υe He	34	kPa
fΔe = 0.5 Ch(0) Υe He	11	kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	1.5 Ch(0) Υe He (lw = 1)	11	kN/m	UDL
Eu2 =	0.5 Ch(0) Υe He (lw = 1)	23	kN/m	Triangular

Seismic Hydrodynamic Loads

REF ;
(1) Priestley, M. J. N. et al . (1986). "Seismic Design of Storage Tanks," Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering.
(2) NZS 3106: 2009, New Zealand Standard "Design of Concrete Structures for the Storage of Liquids".
(3) AS 3735 - 2001, Australian Standard "Concrete Structures for Retaining Liquids".

1 Design Data

Water Capacity	1718.06	m^3	Base Slab Thickness	0.5	m
Density of Water	9.81	kN/m^3	Concrete Grade	40	MPa
Inner Diameter	25	m	Concrete Density	24	kN/m^3
Height of Wall	4	m	Roof Slab Thickness	0.0000	m
Height of Water	3.5	m			
Wall Thickness	0.4	m			

Elastic/Youngs Modulus of Concrete = 5050√f'c 31939.004 MPa

2 Weight Calculations

Component		Formula	Weight (N)	Mass (kg)
Wall,	Fwall	= ρV = ρ x (πDH)(t)	3392920	3.51E+05
Base Slab,	Ffs	= ρV = ρ x (πD^2)(t)/4	5890486	6.00E+08
Water,	Fw	= PA = (ρgh) x A	16854	1.72E+06
Weight of Roof	Fr	= ρV = ρ x (πD^2)(t)/4	0.00	0.00
Mechanical Load on Wall			46.00	4.69E+03
Mechanical Load on slab			267.00	2.72E+04

3 Seismic Design Data

Location = Port Moresby
Category (pg 15) = C
Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15

Tank Contents	p
Non-volatile toxic chemicals	0.002

A_p = probability factor; Fig. C2.7; pg 51 1.85
β = geographical coefficient representing regional seismicity; Fig. C2.1; pg 47 0.50
α = peak horizontal acceleration coefficient; pg 51 0.35
$C_h(T) = \alpha\ \beta\ A_h(T)\ A_p$, horizontal force coefficient 0.32 $A_h(T)$
Ah(T) = normalized horizontal acceleration response

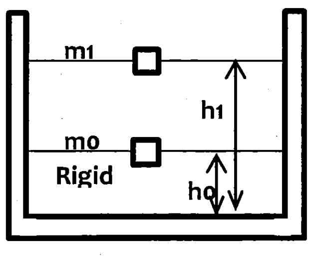
4 Foundation : Rigid/Fixed Foundation on Weathered Mudstone

Ye = Unit weight of engineering backfill soil 20.00 kN/m^3

5 Spring Mass Model for Hydrodynamic Force

h =	3.5	m	D =	25	m
For h/R =	0.28	*ratio*	mo/mι =	0.20	*constant*
mr = mo-mf ≈ 0			m1/m =	0.73	*constant*
ho/h =	0.4	*constant*	h1/h =	0.533	*constant*

mo	... Impulsive Mass ...	3.44E+05	kg
m1	... Convective Mass ...	1.25E+06	kg
ho	= hi/h x h (m)	1.40	m
h1	= hc/h x h (m)	1.87	m



6 Time Period

Impulsive Mode, $T_f = 5.61\,\pi\, H/k_h\ \sqrt{[\Upsilon/Eg]}$ tw/Rm = 0.032

kh = 0.28

Therefore, Tf = 0.04 seconds

For Convective Mode, $T_1\sqrt{(g/R)}$

H/R = 0.28 $T_1\sqrt{(g/R)}$ = 6.80

T1 = 7.7 seconds

For vertical mode;

$T_v = 5.61\,\pi\, H/k_v\ \sqrt{[\Upsilon/Eg]}$ tw/Rm = 0.032

kv = 0.2

$T_v = 5.61\,\pi\, H/k_v\ \sqrt{[\Upsilon/Eg]}$ = 0.05 seconds

7 Seismic Coefficients

Impulsive Coefficient:

Assume Damping = 2%

Tf = 0.039 seconds

Ah(Ťo) = normalized horizontal acceleration response. 1

$C_h(\check{T}o) = \alpha\,\beta\, A_h(T)\, A_p$, horizontal force coefficient 0.32 $A_h(T)$

$C_h(\check{T}o)$ = 0.32

Convective Coefficient:

Assume Damping = 0.5% Ah(T1) = 0.25

T1 = 7.7 seconds $C_h(T1) = \alpha\,\beta\, A_h(T)\, A_{p\,=}$ 0.32 $A_h(T)$

$C_h(T1)$ = 0.081

Vertical Coefficient

Damping = 5% Tv = 0.05 seconds

Av(Ťv) = 1.0

$C_v(\check{T}v) = \alpha'\,\beta\, A_v(T)\, A_p$ =

ά = 0.35 A_p = 1.85 β = 0.50

$C_v(\check{T}b) = \alpha'\,\beta\, A_v(T)\, A_{p\,=}$ 0.227

8 Vertical Distribution of Equivalent Weights

Impulsive Weight

Wo = Ch(To)mo/πD g 13.89 kN/m

Top, Wto = Wo(6ho - 2H)/2H^2 0.79 kN/m^2

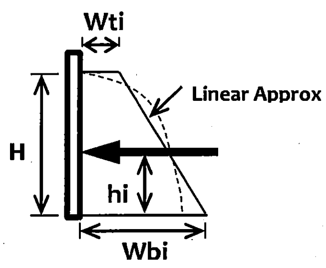
Base, Wto = Wo(4H - 6ho)/2H^2 3.18 kN/m^2

Convective Weight

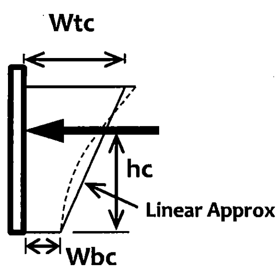
W1 = Ch(T1)m1/πD g 12.68 kN/m

Top, Wt1 = W1(6h1 - 2H)/2H^2 2.17 kN/m^2

Base, Wt1 = W1(4H - 6h1)/2H^2 1.45 kN/m^2



Impulsive



Convective/Sloshing

9 SEISMIC FORCES IN WALLS

Since RC and steel reinforcement will be varied over the wall height, the entire distribution of hoopforce and bending moment is required. Use the dimensionless charts of Appendix A3, with;

H/R = 0.28 R/T = 31.3

The general form of the HOOP TENSION FORCES is; 49%

Nθ = Nθn RP Mz = Mzn RPt

Hydrostatic

Pressure at base of wall:

Ph = ϒH =			34.34	kPa	
Nθh = Ph R Nθhn			429.19	Nθhn	kN/m
Nθhn=	0.74				
Nθh = Ph R Nθhn			317.60		kN/m
Mzh = Mzn RPt			171.68	Mzhn	kNm/m
Mzhn =			0.094		
Mzh = Mzn RPt			16.137		kNm/m
Hoop Stress fhh = Nθh/t			0.8		MPa
Vertical, Nθv = Nθnh R Cv(T) ϒ H = Nθh.Cv(T)					
Nθv =			0.227	Nθh	kN/m
Nθv =			97.26	Nθhn	kN/m
Nθv =			71.98		kN/m
Mzv =			0.227	Mzh	kNm/m
Mzv =			38.91	Mzhn	kNm/m
Mzv =			3.66		kNm/m
Hoop Stress, fhv = Nθv/t			0.18		MPa
Convective: Nθ1 = Nθn1 R x 0.837Ch(T1) ϒ R					
Pc = 0.837 Ch(T1) ϒ R			8.31		kPa
Nθ1 = Nθn1 R x 0.837Ch(T1) ϒ R			103.84	Nθ1n	kN/m
Nθ1n =			0.96		
Nθ1 = Nθn1 R x 0.837Ch(T1) ϒ R			99.69		kN/m
Mz1 = Mz1n (pc)Rt			41.54	Mz1n	kNm/m
Mz1n =			0.064		
Mz1 = Mz1n (pc)Rt			2.66		kNm/m
Hoop Stress, fh1 = Nθ1/t			0.25		MPa
Impulsive					
Nθi = Nθni R qo(0) Ch(Ťf) ϒ R					
Pi = (mo + mw + mt)/mo qo(0) ϒ R Ch(Ťo)					
qo(0) =			0.40		
Pi = (mo + mw + mt)/mo qo(0) ϒ R Ch(Ťo)			32.08		kPa
Nθi = Nθin R Pi			401.01	Nθin	kN/m
Nθin =	1.01				
Nθi = Nθin R Pi			405.02		kN/m
Mzi = Mzin (pi)Rt			160.40	Mzin	kNm/m
Mzin =			0.098		
Mzi = Mzin (pi)Rt			15.72		kNm/m
Hoop Stress, fhi = Nθi/t			1.01		MPa

10 TOTAL SEISMIC HOOP TENSION FORCES & VERTICAL MOMENTS BY SRSS

N_E = √[Nθv^2 + Nθ1^2 + Nθi^2]	423.27	kN/m
M_{ZE} = √[Mzv^2 + Mz1^2 + Mzi^2]	16.36	kNm/m
MAXIMUM TENSION STRESS FROM COMBINED HOOP STRESS		
fmax = fhh + √[fhi^2 + fh1^2 + fhv^2]	1.9	MPa

11 CONVECTIVE/ SLOSHING WAVE HEIGHT

The maximum vertical displacement of the convective waves will be;

dmax = R √[(0.84Ch(T1))^2 + (0.07 Ch(T2))^2]

Ch(T1) =			0.081	
T2 √(g/R) =			2.85	
T2 =			3.22	seconds
0.5% Damping,	0.24	0.747975		
Ah(T2) =			0.38	
C_h(T2) = α β A_h(T) A_p, horizontal force coefficient			0.12	
dmax = R√[(0.84Ch(T1))^2 + (0.07 Ch(T2))^2]			0.86	m

12 BASE SHEAR, Q

IMPULSIVE:

$Qf = Ch(Tf)(mo+mw)g$	2.20	MN

CONVECTIVE;

$Q1 = Ch(T1)\ m1\ g$	1.00	MN

The total Shear Using square root of sum of squares Rule;

$P = \sqrt{[Qf^2 + Q1^2]}$	2.42	MN

BASE/OVERTURNING MOMENT IN WALLS

$MOT = \sqrt{[(Q1h1)^2 + (QFhF)^2]}$	3.60	MNm

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

B =	0.4	m	D =	0.5	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 333.75

$\lambda L > \pi$ 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	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
2	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
3	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
4	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
5	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
6	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
7	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
8	0.40	4.17E-03	31.9754	0.18	0.4	0.10524	9.57	0.6666	6.3776	No	Yes	Use Winkler	263110
9	4.31	4.49E-02	31.9754	0.18	0.4	0.19063	18.00	0.4269	7.6834	No	Yes	Use Winkler	44230
10	4.31	4.49E-02	31.9754	0.18	0.4	0.19063	18.00	0.4269	7.6834	No	Yes	Use Winkler	44230
11	4.31	4.49E-02	31.9754	0.18	0.4	0.19063	18.00	0.4269	7.6834	No	Yes	Use Winkler	44230
12	4.31	4.49E-02	31.9754	0.18	0.4	0.19063	18.00	0.4269	7.6834	No	Yes	Use Winkler	44230
13	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
14	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
15	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
16	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
17	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
18	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410
19	4.42	4.60E-02	31.9754	0.18	0.4	0.19183	18.00	0.4249	7.6475	No	Yes	Use Winkler	43410

PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR FINAL SEDIMENTATION TANK

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	500 X 400	S1	Not applicable	No	Standard shape
2	500 X 4419	S2	Not applicable	No	Standard shape
3	500 X 4310	S3	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	2.0000E-01	5.4742E-03	2.6667E-03	4.1667E-03	INFINITE	INFINITE	0.00
2	2.2095E+00	1.7100E-01	3.5955E+00	4.6031E-02	INFINITE	INFINITE	0.00
3	2.1550E+00	1.6646E-01	3.3360E+00	4.4896E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.500	0.400			
2	Rectangle	0.500	4.419			
3	Rectangle	0.500	4.310			

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 8: 1.2G+1.2Flp [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp

Load case 9: 1.2G+1.5Fe+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 10: 1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 11: G+Eu1+Eu2+Flp [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 5: Flp

Load case 12: G+Eu1+Eu2+Fe+Fgw [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

Load case 13: 1.2G+1.5Fe+1.5FGW [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 14: 1.2G+1.5Fe+1.5Fgw [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

case 15: G+Eu1 [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)

Load case 16: G+Eu1+Fe+Fgw [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

LOAD CASE TITLES

Load Case	Title
1	G
2	Q
3	Eu1(EARTHQUKAE EARTH PRESSURE)
4	Eu2(SEISMIC HYDRODYNAMIC FORCE)
5	Flp
6	Fe
7	Fgw
8	1.2G+1.2Flp [TANKFULL]
9	1.2G+1.5Fe+1.5Fgw [TANKFULL]
10	1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11	G+Eu1+Eu2+Flp [TANKFULL]
12	G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13	1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14	1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15	G+Eu1 [TANK EMPTY]
16	G+Eu1+Fe+Fgw [TANK EMPTY]

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	8	0.000	663.545*	0.000	-69.865	0.000	-351.272
90	8	0.000	-663.545#	0.000	69.865	0.000	-351.272
13	8	0.000	663.545	0.000	69.865*	0.000	-351.272
29	8	0.000	-405.798	0.000	-69.865#	0.000	239.460
232	8	0.000	380.389	0.000	41.639	0.000	379.715*
14	8	0.000	663.545	0.000	-69.865	0.000	-351.272#

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

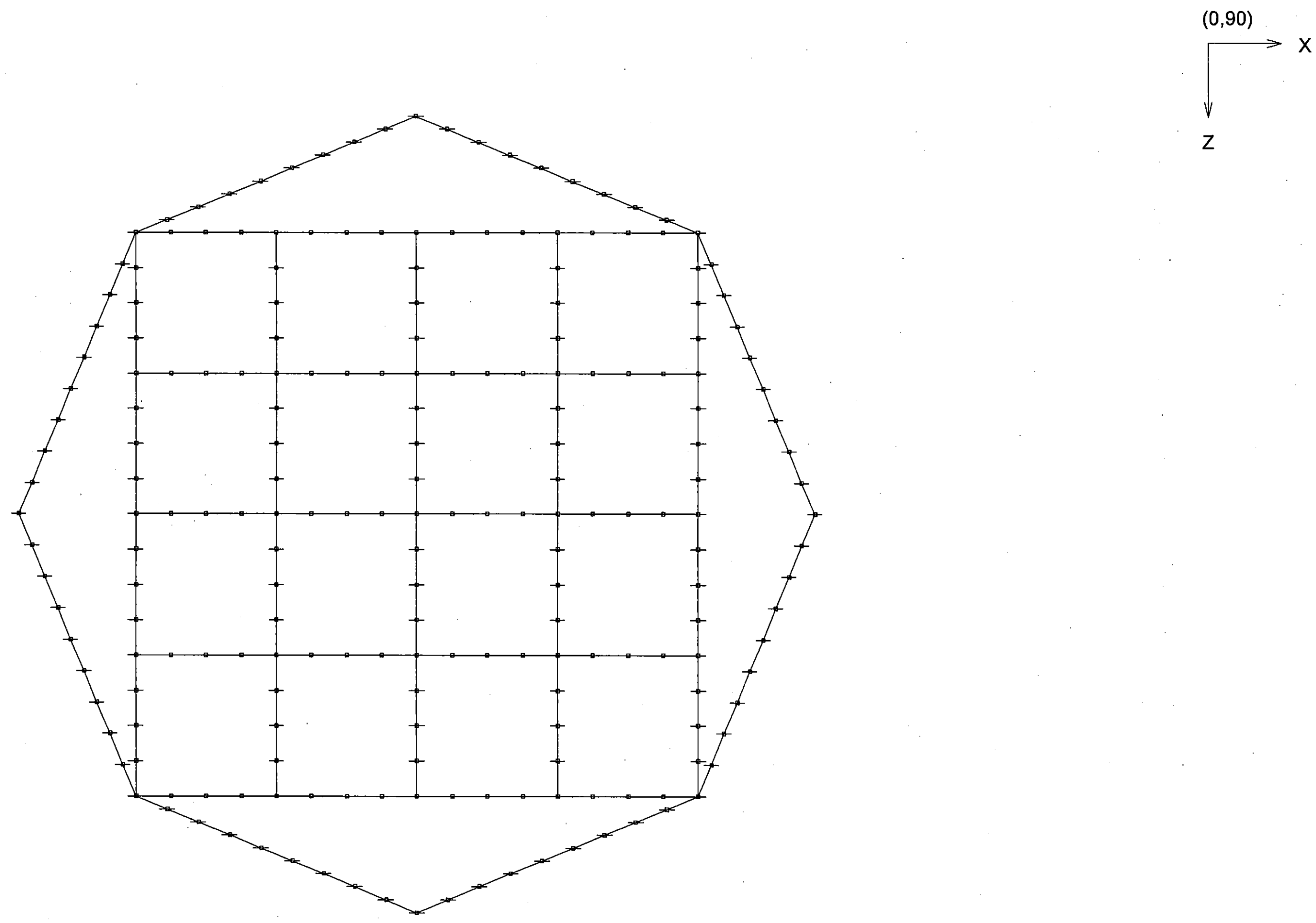
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
158	8	0.000	338.469*	0.000	55.404	0.000	0.000
210	9	0.000	-3.390#	0.000	2.412	0.000	0.000
71	8	0.000	80.360	0.000	664.621*	0.000	0.000
33	8	0.000	80.360	0.000	-664.621#	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	8	0.000	663.545*	0.000	-69.865	0.000	-351.272
90	8	0.000	-663.545#	0.000	69.865	0.000	-351.272
13	8	0.000	663.545	0.000	69.865*	0.000	-351.272
29	8	0.000	-405.798	0.000	-69.865#	0.000	239.460
232	8	0.000	380.389	0.000	41.639	0.000	379.715*
14	8	0.000	663.545	0.000	-69.865	0.000	-351.272#

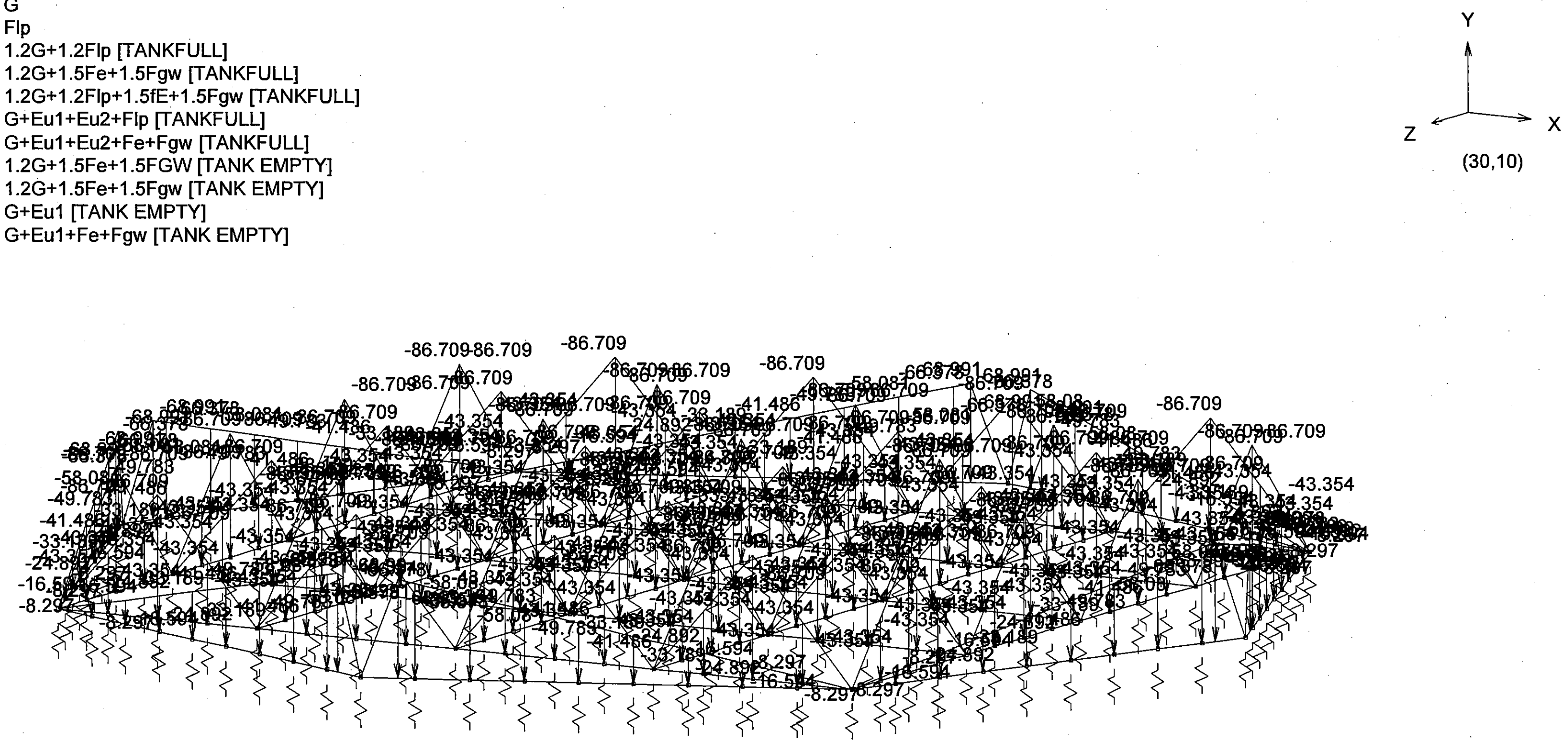


Sections:
1 500 X 400
2 500 X 4419
3 500 X 4310

Base Slab Model [Plan]

Job: FSTSLAB2, Designer: FYP, Units: m,kN,MPa, Scale: 1:230, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR FINAL SEDIMENTATION TANK 17 Oct 2011, 2:59 pm

1	———	(SW)	G
5	———		Flp
8	———		1.2G+1.2Flp [TANKFULL]
9	———		1.2G+1.5Fe+1.5Fgw [TANKFULL]
10	———		1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11	———		G+Eu1+Eu2+Flp [TANKFULL]
12	———		G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13	———		1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14	———		1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15	———		G+Eu1 [TANK EMPTY]
16	———		G+Eu1+Fe+Fgw [TANK EMPTY]



Sections:
1 500 X 400
2 500 X 4419
3 500 X 4310

Base Slab Model with Loads

Job: FSTSLAB2, Designer: FYP, Units: m,kN,MPa, Scale: 1:140, Axes: XY
Load: 5 Disp: None Moment: None Shear: None Axial: None
PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR FINAL SEDIMENTATION TANK 17 Oct 2011, 2:58 pm

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

Design of a Continuous Beam : REF. AS 3600 - 2009

1 Loads

1 Serviceability Load Combination : Long term loads

Main Load Dead Load =	33.60	kN/m	Self weight =	55.24	kN/m
Total G =	88.84	kN/m	Flp =	19.62	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	122.28	kN/m	COMB. 2	G + Flp =	108.46	kN/m
COMB. 3	G + Flp + Fgw =	108.46	kN/m		Long term Load Factors =	1	

2 Beam Section Properties

Beam Size

bw =	4419	mm	D =	500	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	18000	mm	Overall depth of sect. D =	500	mm
Effective Length, Lef =	17600	mm	Area, A =	2.21E+06	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	4.60E+10	mm^4
Neutral Axis, xc =	3959.50	mm	Moment of Inertia, Iy-y =	3.60E+12	mm^4
Effective width, beff =	7919.00	mm	Gross Area, Ag =	2.21E+06	mm^2

3 Design Parameters

F'c =	40	MPa	Fsy =	410	MPa
m =	2400	kg/m^3	D =	500	mm
Ec =	31975.4	MPa	Es =	200000	MPa
Gamma =	0.766				

4 Initial Choice of Section

Effective width = 7919 mm clause 8.8.2

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

k_1 = 0.045 (RECT sections) 1

= 0.0293685 (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 = 145.6935

defln/Lef = 0.004 k_1/k_2 = 17.28

d = 358 mm

take d = 383 mm Distance from d to extreme fibre of beam (do) = 117 mm

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	664.60	kNm	(left support)
Mmax +ve	=	664.60	kNm	(mid-span)
M -ve	=	664.60	kNm	(right support)
Vmax	=	663.50	kN	
V (other end)	=	663.50	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 : 2

take do = 117 mm d = 383

Estimate Area of main steel Ast = 5532.4 mm^2

20	Y 20	Ast =	6283.19 mm^2		**SPACING = 213** mm		
		p =	0.0037				
19	Y 20	Asc =	5969.03 mm^2				
		pc =	0.0035	<	pd	=	0.0254088
		p-pc =	0.0002	pd=0.4*0.85*μ*f'c/fsy			

Check M* <= phiMu

ku = 0.00292 dsc = 97 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)]

= 674 kNm > M* = 664.60 kNm **OK**

Determine -ve steel at right support :

M -ve = 664.60 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst 383 mm

Estimate Area of main steel Ast = 5532.4 mm^2

20	Y 20	Ast =	6283.19 mm^2				
		p =	0.0037				
19	Y 20	Asc =	5969.03 mm^2				
		pc =	0.0035	<	pd	=	0.0254088
		p-pc =	0.0002				

Check M* <= phiMu

ku = 0.00292 dsc = 97 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)]

= 674 kNm > M* = 664.60 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 664.6 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 117 mm

what is the code ? 2 d = 383 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 5532.43438 mm^2

Bottom 20 Y 20 Ast = 6283.18531 mm^2

Top 19 Y 20 Asc = 5969.02604 mm^2

Assume N - A in flange

Effective width : beff = 7919 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.0326127

hence dn = 12 < 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)]

= 877 kNm > M*+ve = 664.60 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	20	Y 20
	Asc	=	19	Y 20
Right support :	Ast	=	20	Y 20
	Asc	=	19	Y 20
midspan	Ast	=	20	Y 20
	Asc	=	19	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 = 5280 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 = 5280 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 = 1760 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 2123 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 = 2143 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.16 mm^2

k2 = 2.2 a_b = 2123 mm (dist between bars)

factor = 56006

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

= 329.448 mm

take fs/Fsy = M*/Phi Mu = 0.985645

therefore

$$\boxed{\begin{array}{l} \text{Lsy} = 325 \text{ mm} \\ \text{say} = 650 \text{ mm} \end{array}}$$

≥ 25k1db 625

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 = 2316.5 mm

k1 = 1 A_b = 314.16 mm^2

k2 = 2.2 a_b = 2296.5 mm (dist between bars)

factor = 44805

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.75790386

therefore

$$\boxed{\begin{array}{l} \text{Lsy} = 200 \text{ mm} \\ \text{say} = 500 \text{ mm} \end{array}}$$

≥ 25k1db 500

9 PUNCHING SHEAR CHECK

V* = 997.25 kN

Coloumn Dimension
b = **400** mm
d = **400** mm

Dist to centreline of bottom steel from soffit
dq = 85 mm
dom = 415 mm

Critical shear perimeter, μ
μ = 2(b+dom) + 2(d+dom)
3260 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 = 12,880 kN > V* = 997.25 kN **OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	664.60 kNm	Es	=	200000 MPa
Ml	=	664.60 kNm	Ec	=	31975.3505 MPa
Mo	=	1329.2 kNm			
Ms	=	664.60 kNm			
Ig	=	4.60E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	7919 mm	A_{sc} =	5969.02604 mm^2
webwidth	b_w =	4419 mm	A_{st} =	6283.18531 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	383 mm
			d =	383 mm
			D =	500 mm

n = Es/Ec = 6.2548181

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

2209.5 dn^2 + 70666.327 dn - 2E+07) = 0

dn = 77.437915 mm

d = 383 mm

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

= 4.35E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 698.705249 kNm

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

= 5.28E+10 mm^4

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

b_w =	4419 mm	A_{st} =	6283.2 mm^2
d_{sc} =	97 mm	A_{sc} =	5969 mm^2
d =	383 mm		

Ig = 4.60E+10 mm^4

At right end (arbitrary)

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

2209.5 dn^2 + 70666.327 dn - 2E+07) = 0

dn = 75.91 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.35E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 698.705249 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 5.28E+10 mm^4

At left end (arbitrary)

b_w = **4419** mm

d_{sc} = **97** mm

A_{st} = **6283.19** mm^2

A_{sc} = **5969.03** mm^2

d = **383** mm

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

2209.5 dn^2 + 70666.327 dn - 2E+07) = 0

dn = 75.905914 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.35E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 698.705249 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 5.28E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 5.28E+10 mm^4

Midspan Deflection :

Lef = 17600 mm

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

= 10.164764 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 122.28 kN/m

w - short term = 122.28 kN/m

Defln,s,sus = 10.16476 mm

At midspan : Ast = 6283.18531 mm^2

Asc = 5969.02604 mm^2

Asc/Ast = 0.95

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

= 0.86

therefore

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

= 18.90646 mm

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

ALLOWABLE DEFLN = Lef / 500 : 35.2 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

Job name: FSTWALL Designer: FYP Date: 17 Oct 2011, 3:08 pm Page: 1

FINAL SEDIMENTATION TANK

WALL ANALYSIS

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	400DP x 1MWD	S1	Not applicable	No	Standard shape
2	250 x 1000	S2	Not applicable	No	Standard shape
3	500 x 1000	S3	Not applicable	No	Standard shape
4	500 x 1000	S4	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	4.0000E-01	1.5969E-02	3.3333E-02	5.3333E-03	INFINITE	INFINITE	0.00
2	2.5000E-01	4.3883E-03	2.0833E-02	1.3021E-03	INFINITE	INFINITE	0.00
3	5.0000E-01	2.8610E-02	4.1667E-02	1.0417E-02	INFINITE	INFINITE	0.00
4	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	0.400	1.000			
2	Rectangle	0.250	1.000			
3	Rectangle	0.500	1.000			
4	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

COMBINATION LOAD CASES

Load case 8: 1.2G+1.2Flp [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp

Load case 9: 1.2G+1.5Fe+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 10: 1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 11: G+Eu1+Eu2+Flp [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 5: Flp

Load case 12: G+Eu1+Eu2+Fe+Fgw [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

Load case 13: 1.2G+1.5Fe+1.5FGW [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 14: 1.2G+1.5Fe+1.5Fgw [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 15: G+Eu1 [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)

case 16: G+Eu1+Fe+Fgw [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

LOAD CASE TITLES

Load Case	Title
1	G
2	Q
3	Eu1(EARTHQUKAE EARTH PRESSURE)
4	Eu2(SEISMIC HYDRODYNAMIC FORCE)
5	Flp
6	Fe
7	Fgw
8	1.2G+1.2Flp [TANKFULL]
9	1.2G+1.5Fe+1.5Fgw [TANKFULL]
10	1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11	G+Eu1+Eu2+Flp [TANKFULL]
12	G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13	1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14	1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15	G+Eu1 [TANK EMPTY]
16	G+Eu1+Fe+Fgw [TANK EMPTY]

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	135.928*	94.116	0.000	0.000	0.000	-116.420
4	9	0.000#	20.182	0.000	0.000	0.000	-14.127
5	8	0.000	302.269*	0.000	0.000	0.000	-116.420
5	10	0.000	-493.918#	0.000	0.000	0.000	-1257.159
1	9	135.928	-175.575	0.000	0.000	0.000	237.185*
5	10	0.000	-493.918	0.000	0.000	0.000	-1257.159#

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

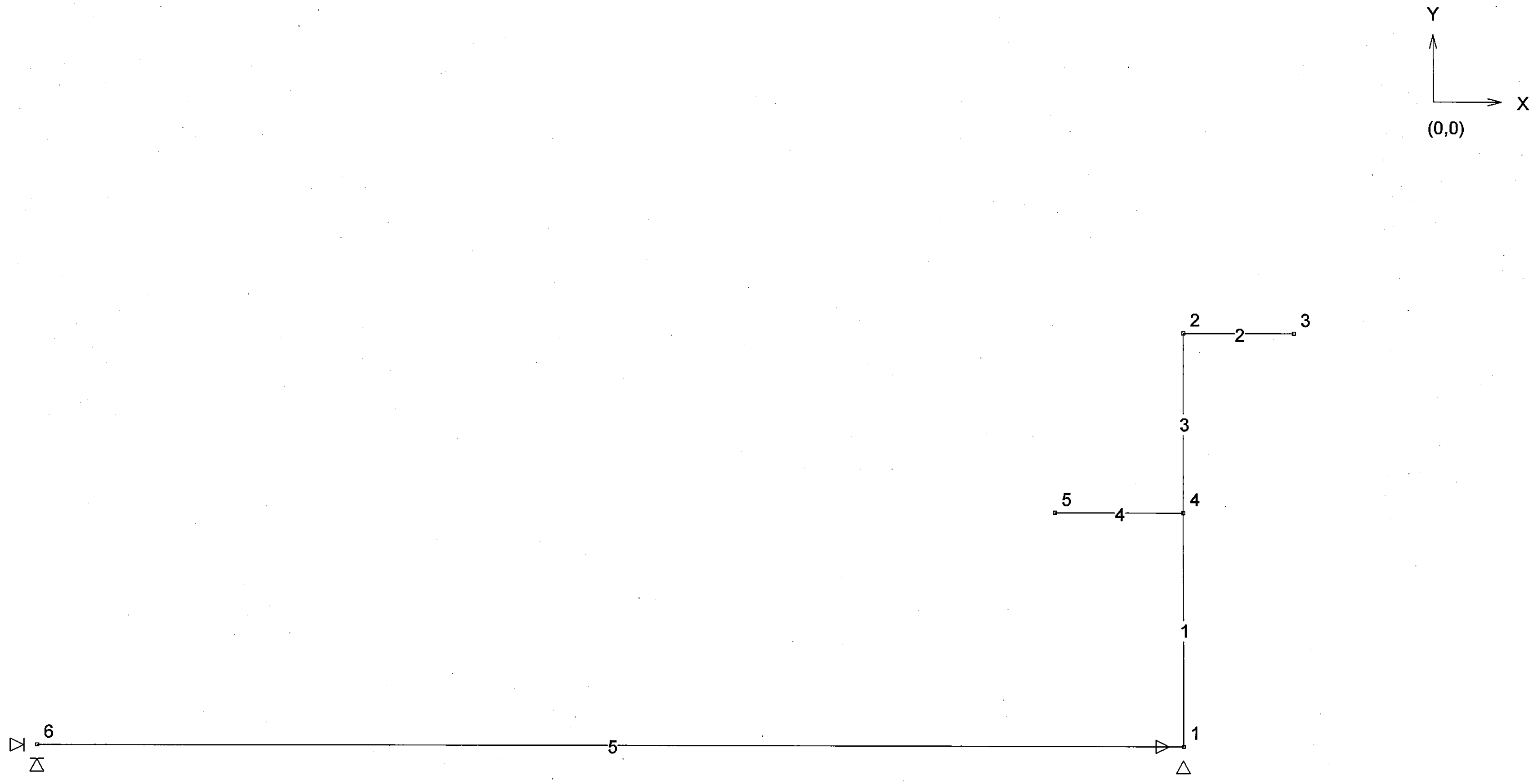
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	9	175.575*	175.040	0.000	0.000	0.000	0.000
1	8	-94.116#	438.198	0.000	0.000	0.000	0.000
6	10	0.000	493.918*	0.000	0.000	0.000	1257.159
1	6	117.050	-18.260#	0.000	0.000	0.000	0.000
6	10	0.000	493.918	0.000	0.000	0.000	1257.159*
6	4	0.000	-2.374	0.000	0.000	0.000	-9.894#

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	135.928*	94.116	0.000	0.000	0.000	-116.420
4	9	0.000#	20.182	0.000	0.000	0.000	-14.127
5	8	0.000	302.269*	0.000	0.000	0.000	-116.420
5	10	0.000	-493.918#	0.000	0.000	0.000	-1257.159
5	10	0.000	-32.639	0.000	0.000	0.000	717.428*
5	10	0.000	-493.918	0.000	0.000	0.000	-1257.159#

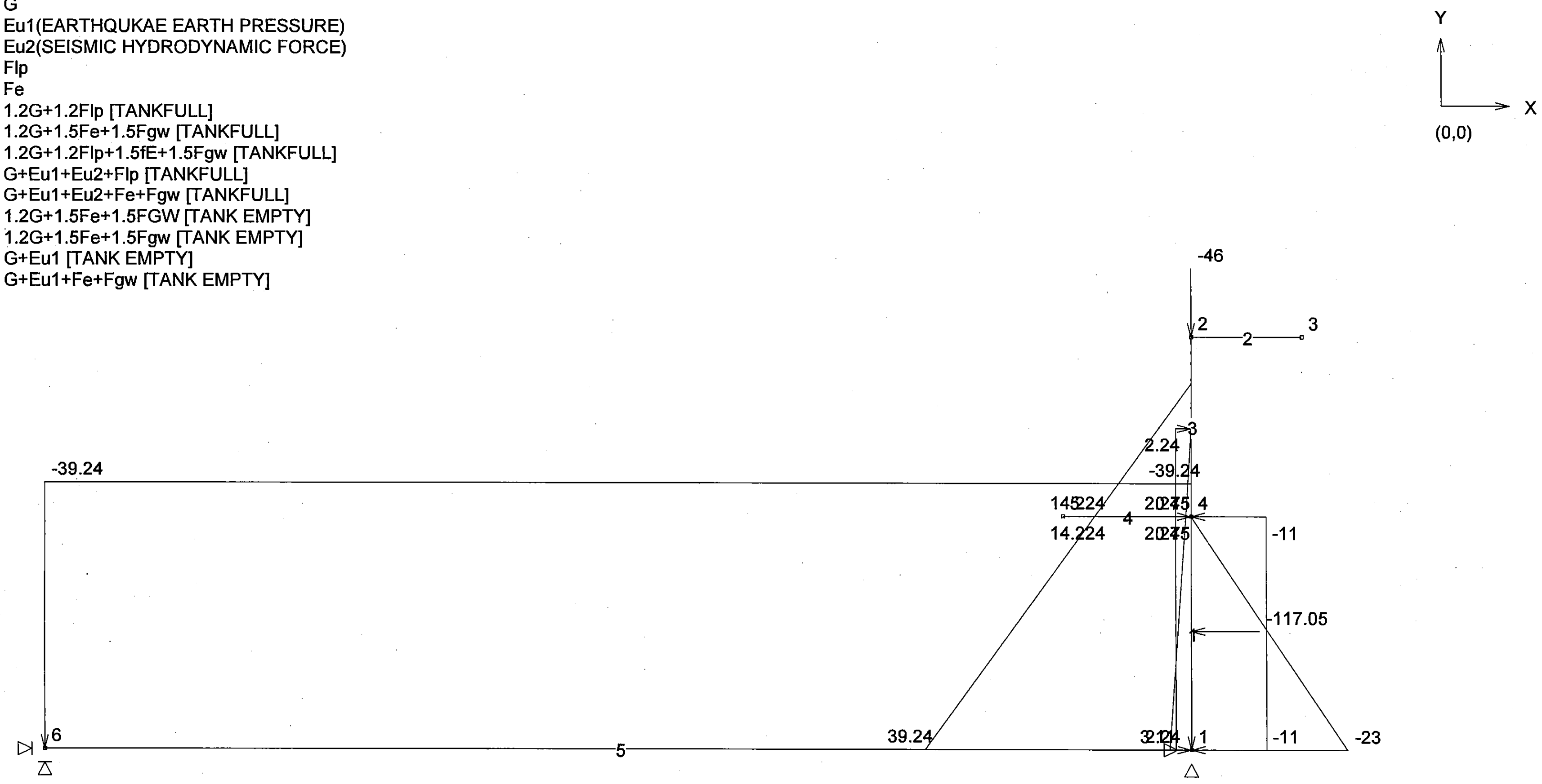


Sections:
1 400DP x 1MWD
2 250 x 1000
3 500 x 1000
4 500 x 1000

Tank Wall Model

Job: FSTWALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:74, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
FINAL SEDIMENTATION TANK
WALL ANALYSIS 17 Oct 2011, 3:06 pm

1	(SW)	G
3		Eu1(EARTHQUKAE EARTH PRESSURE)
4		Eu2(SEISMIC HYDRODYNAMIC FORCE)
5		Flp
6		Fe
8		1.2G+1.2Flp [TANKFULL]
9		1.2G+1.5Fe+1.5Fgw [TANKFULL]
10		1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11		G+Eu1+Eu2+Flp [TANKFULL]
12		G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13		1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14		1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15		G+Eu1 [TANK EMPTY]
16		G+Eu1+Fe+Fgw [TANK EMPTY]



Sections:
1 400DP x 1MWD
2 250 x 1000
3 500 x 1000
4 500 x 1000

Tank Wall with Loadings

Job: FSTWALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:74, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
FINAL SEDIMENTATION TANK
WALL ANALYSIS 17 Oct 2011, 3:06 pm

TANK WALL DESIGN AS CONTINUOUS RC BEAM

Design of a Continuous Beam : REF. AS 3600 - 2009

1 Serviceability Load Combination : Long term loads

Main Load Dead Load =	10.00	kN/m	Self weight =	15.00	kN/m
Total G =	25.00	kN/m	Flp =	19.62	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	58.44 kN/m	COMB. 2	G + Flp =	44.62	kN/m
COMB. 3	G + Flp + Fgw =	44.62 kN/m		Long term Load Factors =	1	

2 Beam Section Properties

Beam Size

bw =	1500	mm	D =	400	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	4500	mm	Overall depth of Sect. D =	400	mm
Effective Length,Lef =	4000	mm	Area, A =	6.00E+05	mm^2
Neutral Axis, yc =	200.00	mm	Moment of Inertia, Ix-x =	8.00E+09	mm^4
Neutral Axis, xc =	1150.00	mm	Moment of Inertia, Iy-y =	1.13E+11	mm^4
Effective width, beff =	2300.00	mm	Gross Area, Ag =	6.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	400	mm
Fsy =	410	MPa	Ec =	31975.351	MPa
m	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

4 Initial Choice of Section

Effective width = 2300 mm clause 8.8.2

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

k_1 = 0.045 (RECT sections) 1

= 0.032332 (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 = 41

defln/Lef = 0.004 k_1/k_2 = 17.279779

d = 81 mm

take d = 293 mm Distance from d to extreme fibre of beam (do) = 107 mm

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	237.20	kNm	(left support)
Mmax +ve	=	237.20	kNm	(mid-span)
M -ve	=	237.20	kNm	(right support)
Vmax	=	176.00	kN	
V (other end)	=	176.00	kN	
N*	=	149.00	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 = 107 mm d = 293

Estimate Area of main steel		Ast =	2581.0818 mm^2				
11	Y 20	Ast =	3455.75192 mm^2		DRY SIDE		
		p =	0.0078629				
10	Y 20	Asc =	3141.59265 mm^2		WET SIDE		
		pc =	0.0071481	<	pd	=	0.0254088
		p-pc =	0.0007148	pd=0.4*0.85*µ*f'c/fsy			

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

ku = 0.01125 dsc = 97 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)]

= 261 kNm	>	M* =	237.20 kNm	OK

Determine - ve steel at right support :

M -ve = 237.20 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 107 mm dst = 293 mm

Estimate Area of main steel		Ast	=	$2581.0818\ mm^2$			
11	Y 20	Ast	=	$3455.75192\ mm^2$		DRY SIDE	
		p	=	0.0078629			
10	Y 20	Asc	=	$3141.59265\ mm^2$		WET SIDE	
		pc	=	0.0071481	<	pd =	0.0254088
		p-pc	=	0.0007148			

Check M* <= phiMu

ku = 0.01125 dsc = 97 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)]

= 261 kNm	>	M* =	237.20 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 237.2 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 ? 1 d = 293 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = $2581.081824\ mm^2$

Bottom	11	Y 20	Ast	=	$3455.751919\ mm^2$	DRY SIDE
Top	10	Y 20	Asc	=	$3141.592654\ mm^2$	WET SIDE

Assume N - A in flange

Effective width : beff = 2300 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.080728

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)]

= 362 kNm	>	M*+ve =	237.20 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	11	Y 20
	Asc	=	10	Y 20
Right support :	Ast	=	11	Y 20
	Asc	=	10	Y 20
midspan	Ast	=	11	Y 20
	Asc	=	10	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 = 1200 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 = 1200 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 = 400 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm   # of lines of bars = 2

a_b = 663 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 = 683 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.15927 mm^2

k2 = 2.2 a_b = 663 mm   (dist between bars)

factor = 56006.241

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

= 329.448 mm

take fs/Fsy = M*/Phi Mu = 0.908756336

therefore

Lsy =	299	mm
say =	650	mm

≥ 25k1db 625

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 = 857 mm

k1 = 1 A_b = 314.15927 mm^2

k2 = 2.2 a_b = 837 mm   (dist between bars)

factor = 44804.993

Lsyt = 263.559 mm   take fs/Fsy = M*/Phi Mu = 0.655116

therefore

Lsy =	173	mm
say =	500	mm

≥ 25k1db 500

9 SHEAR CHECK

V* = 176.00 kN   Dist to centreline of bottom steel from soffit

critical location : d = 293 mm   dq = 83 mm

do = 317 mm

V* = 169 kN   beta 1 = 1.2415 (but not less than 1.1)

hence = 1.2415

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

= 273.86 kN

phi Vumax = 2662.8 kN > V* No web crushing

phi Vumin = 474 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 = 731.7073171

Asv.min = 256.097561 mm^2

Asv.max = 5252 mm^2

Asv = 2 x area of stirrups = 400 mm^2 2 Ties

theta = 30.4321 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 833.72 kN

Phi Vu = 584 kN > V* = 168.675 kN

hence

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

Other Support :

V* = 176.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 293 mm

dq = 83 mm

do = 317 mm

V* = 168.68 kN

beta 1 = 1.2415 (but not less than 1.1)

hence = 1.2415

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

= 273.858 kN

phi Vumax	=	2662.8 kN	>	V*	No web crushing
phi Vumin	=	474 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 = 731.7073171

Asv.min = 256.097561 mm^2

Asv.max = 5251.634755 mm^2

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

theta = 31.0326 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 1039.31 kN

Phi Vu = 727.516 kN > V* = 168.68 kN

hence

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

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 237.20 kNm   Es = 200000 MPa

Ml = 237.20 kNm   Ec = 31975.35 MPa

Mo = 474.4 kNm

Ms = 237.20 kNm

Ig = 8.00E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2300 mm	A_{sc} =	3141.593 mm^2
webwidth	b_w =	1500 mm	A_{st} =	3455.752 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	107 mm
			d_{st} =	293 mm
			d =	293 mm
			D =	400 mm

n = Es/Ec = 6.254818

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

750 dn^2 + 38123.6 dn - 8099633.4) = 0

dn = 81.56777 mm

d = 293 mm

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

= 1.24E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 151.7893277 kNm

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

= 3.01E+09 mm^4

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

b_w =	1500 mm	A_{st} =	3455.7519 mm^2
d_{sc} =	97 mm	A_{sc} =	3141.5927 mm^2
d =	293 mm		

Ig = 8.00E+09 mm^4

At right end (arbitrary)

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

750 dn^2 + 38123.6 dn - 7934548.5) = 0

dn = 80.53 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.24E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 151.7893277 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 3.01E+09 mm^4

At left end (arbitrary)

b_w = **1500** mm

d_{sc} = **97** mm

A_{st} = **3455.75** mm^2

A_{sc} = **3141.59** mm^2

d = **293** mm

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

750 dn^2 + 38123.6 dn - 7934548.5) = 0

dn = 80.53406 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.24E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 151.7893277 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 3.01E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 3.01E+09 mm^4

Midspan Deflection :

Lef = 4000 mm

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

= 3.286664 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 58.44 kN/m

w - short term = 58.44 kN/m

Defln,s,sus = 3.286664459 mm

At midspan : Ast = 3455.751919 mm^2

Asc = 3141.592654 mm^2

Asc/Ast = 0.909090909

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

= 0.909090909

therefore

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

= 6.27454 mm

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

ALLOWABLE DEFLN = Lef / 250 = 16 mm

> 0.9 x Total Defln

THE THICKNESS FOR THE TANK WALL IS ADEQUATE

FINAL SEDIMENTATION TANK
WALL ANALYSIS

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	400DP x 1MWD	S1	Not applicable	No	Standard shape
2	250 x 1000	S2	Not applicable	No	Standard shape
3	500 x 1000	S3	Not applicable	No	Standard shape
4	500 x 1000	S4	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	3.6000E-01	9.0996E-03	4.3200E-02	2.7000E-03	INFINITE	INFINITE	0.00
2	2.5000E-01	4.3883E-03	2.0833E-02	1.3021E-03	INFINITE	INFINITE	0.00
3	5.0000E-01	2.8610E-02	4.1667E-02	1.0417E-02	INFINITE	INFINITE	0.00
4	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	0.300	1.200			
2	Rectangle	0.250	1.000			
3	Rectangle	0.500	1.000			
4	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

case 8: 1.2G+1.2Flp [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp

Load case 9: 1.2G+1.5Fe+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 10: 1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]

1.200 * Load case 1: G
1.200 * Load case 5: Flp
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 11: G+Eu1+Eu2+Flp [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 5: Flp

Load case 12: G+Eu1+Eu2+Fe+Fgw [TANKFULL]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 4: Eu2(SEISMIC HYDRODYNAMIC FORCE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

Load case 13: 1.2G+1.5Fe+1.5FGW [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 14: 1.2G+1.5Fe+1.5Fgw [TANK EMPTY]

1.200 * Load case 1: G
1.500 * Load case 6: Fe
1.500 * Load case 7: Fgw

Load case 15: G+Eu1 [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)

Load case 16: G+Eu1+Fe+Fgw [TANK EMPTY]

1.000 * Load case 1: G
1.000 * Load case 3: Eu1(EARTHQUKAE EARTH PRESSURE)
1.000 * Load case 6: Fe
1.000 * Load case 7: Fgw

LOAD CASE TITLES

Load Case	Title
1	G
2	Q
3	Eu1(EARTHQUKAE EARTH PRESSURE)
4	Eu2(SEISMIC HYDRODYNAMIC FORCE)
5	Flp
6	Fe
7	Fgw
8	1.2G+1.2Flp [TANKFULL]
9	1.2G+1.5Fe+1.5Fgw [TANKFULL]
10	1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11	G+Eu1+Eu2+Flp [TANKFULL]
12	G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13	1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14	1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15	G+Eu1 [TANK EMPTY]
16	G+Eu1+Fe+Fgw [TANK EMPTY]

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	146.596*	94.116	0.000	0.000	0.000	-94.220
4	9	0.000#	36.039	0.000	0.000	0.000	-36.328
5	8	0.000	299.605*	0.000	0.000	0.000	-94.220
5	10	0.000	-496.582#	0.000	0.000	0.000	-1268.259
1	9	146.596	-175.575	0.000	0.000	0.000	259.386*
5	10	0.000	-496.582	0.000	0.000	0.000	-1268.259#

NODE REACTIONS (kN,kNm)

(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Nodes

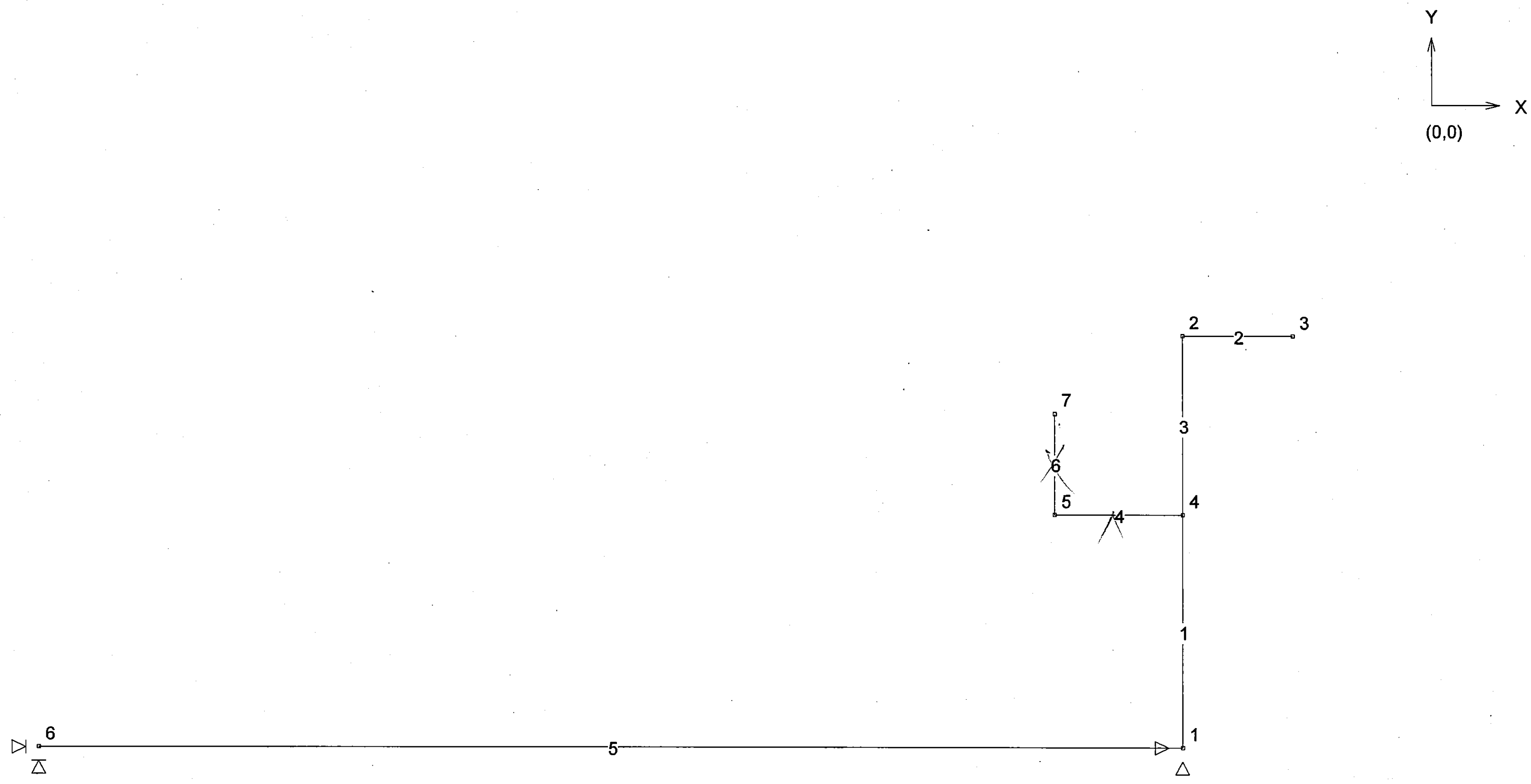
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	9	175.575*	183.044	0.000	0.000	0.000	0.000
1	8	-94.116#	446.201	0.000	0.000	0.000	0.000
6	10	0.000	496.582*	0.000	0.000	0.000	1268.259
1	6	117.050	-18.260#	0.000	0.000	0.000	0.000
6	10	0.000	496.582	0.000	0.000	0.000	1268.259*
6	4	0.000	-2.374	0.000	0.000	0.000	-9.894#

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	146.596*	94.116	0.000	0.000	0.000	-94.220
6	1	0.000#	0.000	0.000	0.000	0.000	0.000
5	8	0.000	299.605*	0.000	0.000	0.000	-94.220
5	10	0.000	-496.582#	0.000	0.000	0.000	-1268.259
5	10	0.000	-35.303	0.000	0.000	0.000	726.309*
5	10	0.000	-496.582	0.000	0.000	0.000	-1268.259#

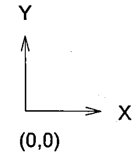


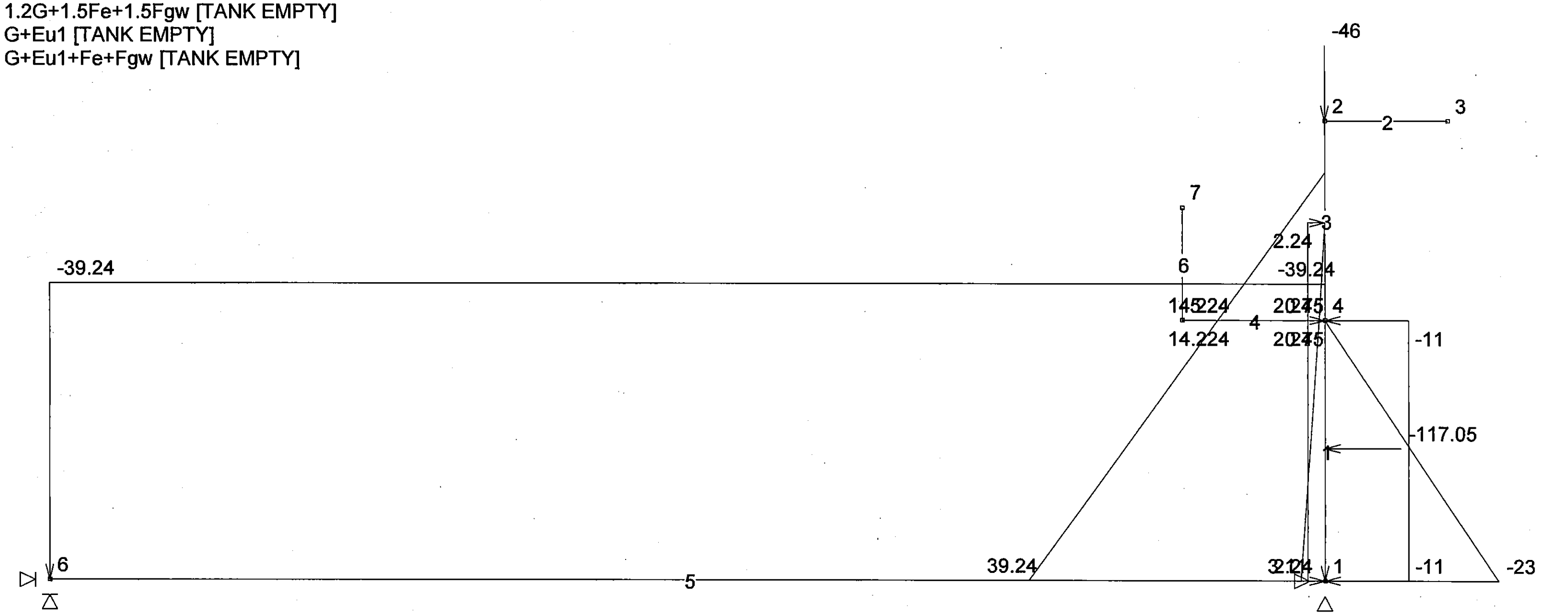
Sections:
1 400DP x 1MWD
2 250 x 1000
3 500 x 1000
4 500 x 1000

Outlet Pit Wall Model

Job: OUTLETPI, Designer: FYP, Units: m,kN,MPa, Scale: 1:74, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
FINAL SEDIMENTATION TANK
WALL ANALYSIS 17 Oct 2011, 3:11 pm

1	(SW)	G
3		Eu1(EARTHQUKAE EARTH PRESSURE)
4		Eu2(SEISMIC HYDRODYNAMIC FORCE)
5		Flp
6		Fe
8		1.2G+1.2Flp [TANKFULL]
9		1.2G+1.5Fe+1.5Fgw [TANKFULL]
10		1.2G+1.2Flp+1.5fE+1.5Fgw [TANKFULL]
11		G+Eu1+Eu2+Flp [TANKFULL]
12		G+Eu1+Eu2+Fe+Fgw [TANKFULL]
13		1.2G+1.5Fe+1.5FGW [TANK EMPTY]
14		1.2G+1.5Fe+1.5Fgw [TANK EMPTY]
15		G+Eu1 [TANK EMPTY]
16		G+Eu1+Fe+Fgw [TANK EMPTY]





Sections:
1 400DP x 1MWD
2 250 x 1000
3 500 x 1000
4 500 x 1000

Outlet Pit Wall Model with Loads

Job: OUTLETPI, Designer: FYP, Units: m,kN,MPa, Scale: 1:74, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
FINAL SEDIMENTATION TANK
WALL ANALYSIS 17 Oct 2011, 3:11 pm

OUTLET PIT TANK WALL DESIGN AS CONTINUOUS RC BEAM

Design of a Continuous Beam : REF. AS 3600 - 2009

1 Serviceability Load Combination : Long term loads

Main Load Dead Load =	10.00	kN/m	Self weight =	12.60	kN/m
Total G =	22.60	kN/m	Flp =	19.62	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	56.04	kN/m	COMB. 2	G + Flp =	42.22	kN/m
COMB. 3	G + Flp + Fgw =	42.22	kN/m		Long term Load Factors =	1	

2 Beam Section Properties

Beam Size

bw =	1200	mm	D =	420	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	4500	mm	Overall depth of Sect. D =	420	mm
Effective Length,Lef =	4000	mm	Area, A =	5.04E+05	mm^2
Neutral Axis, yc =	210.00	mm	Moment of Inertia, Ix-x =	7.41E+09	mm^4
Neutral Axis, xc =	1000.00	mm	Moment of Inertia, Iy-y =	6.05E+10	mm^4
Effective width, beff =	2000.00	mm	Gross Area, Ag =	5.04E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	420	mm
Fsy =	410	MPa	Ec =	31975.351	MPa
m	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

4 Initial Choice of Section

Effective width = 2000 mm clause 8.8.2

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

k_1 = 0.045 (RECT sections) 1

= 0.03067 (T and L sections) 2

what type of span? 5 interior span = 5

end span = 6

k_2 = 0.0026

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

Fdef = 37.064

defln/Lef = 0.004 k_1/k_2 = 17.279779

d = 82 mm

take d = 313 mm Distance from d to extreme fibre of beam (do) = 107 mm

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	260.00	kNm	(left support)
Mmax +ve	=	260.00	kNm	(mid-span)
M -ve	=	260.00	kNm	(right support)
Vmax	=	176.00	kN	
V (other end)	=	176.00	kN	
N*	=	147.00	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 = 107 mm d = 313

Estimate Area of main steel Ast = 2648.4008 mm^2

11 Y 20 Ast = 3455.75192 mm^2 DRY SIDE

p = 0.0092006

10 Y 20 Asc = 3141.59265 mm^2 WET SIDE

pc = 0.0083642 < pd = 0.0254088

p-pc = 0.0008364 pd=0.4*0.85*γ*f'c/fsy

Check M* <= phiMu

ku = 0.01317 dsc = 97 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)]

=	286	kNm	>	M*	=	260.00 kNm	OK

Determine -ve steel at right support :

M -ve : 260.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 107 mm dst = 313 mm

Estimate Area of main steel Ast = 2648.4008 mm^2

11 Y 20 Ast = 3455.75192 mm^2 DRY SIDE

p = 0.0092006

10 Y 20 Asc = 3141.59265 mm^2 WET SIDE

pc = 0.0083642 < pd = 0.0254088

p-pc = 0.0008364

Check M* <= phiMu

ku = 0.01317 dsc = 97 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)]

=	286	kNm	>	M*	=	260.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 260 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 ? 1 d = 313 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 2648.40085 mm^2

Bottom 11 Y 20 Ast = 3455.751919 mm^2 DRY SIDE

Top 10 Y 20 Asc = 3141.592654 mm^2 WET SIDE

Assume N - A in flange

Effective width : beff = 2000 mm

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

= 0.0869

hence dn = 27 < 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)]

=	386 kNm	>	M*+ve =	260.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	11	Y 20
	Asc	=	10	Y 20
Right support :	Ast	=	11	Y 20
	Asc	=	10	Y 20
midspan	Ast	=	11	Y 20
	Asc	=	10	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 = 1200 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 = 1200 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 = 400 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 513 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 = 533 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.15927 mm^2

k2 = 2.2 a_b = 513 mm (dist between bars)

factor = 56006.241

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

= 329.448 mm

take fs/Fsy = M*/Phi Mu = 0.907571202

therefore Lsy = 299 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 = 707 mm

k1 = 1 A_b = 314.15927 mm^2

k2 = 2.2 a_b = 687 mm (dist between bars)

factor = 44804.993

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.673847

therefore Lsy = 178 mm ≥ 25k1db 500

say = 500 mm

9 SHEAR CHECK

V* = 176.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 313 mm dq = 83 mm

do = 337 mm

V* = 169 kN beta 1 = 1.2315 (but not less than 1.1)

hence = 1.2315

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

= 243.84 kN

phi Vumax : 2264.64 kN > V* No web crushing

phi Vumin : 414 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 = 585.3658537

Asv.min = 204.8780488 mm^2

Asv.max = 4179 mm^2

Asv = 2 x area of stirrups = 400 mm^2 2 Ties

theta = 30.7365 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 813.08 kN

Phi Vu = 569 kN > V* = 168.9262 kN

hence

PROVIDE : 1 Y16 LIGS @ 200 centres

Other Support :

V* = 176.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 313 mm dq = 83 mm

do = 337 mm

V* = 168.93 kN beta 1 = 1.2315 (but not less than 1.1)

hence = 1.2315

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

= 243.835 kN

phi Vumax	=	2264.64 kN	>	V*	No web crushing
phi Vumin	=	414 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 = 585.3658537

Asv.min = 204.8780488 mm^2

Asv.max = 4178.713482 mm^2

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

theta = 31.4915 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 1024.98 kN

Phi Vu = 717.487 kN > V* = 168.93 kN

hence

PROVIDE 1 Y16 LIGS @ 200 centres

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	260.00 kNm	Es	=	200000 MPa
Ml	=	260.00 kNm	Ec	=	31975.35 MPa
Mo	=	520 kNm			
Ms	=	260.00 kNm			
Ig	=	7.41E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2000 mm	A_{sc} =	3141.593 mm^2
webwidth	b_w =	1200 mm	A_{st} =	3455.752 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	107 mm
			d_{st} =	313 mm
			d =	313 mm
			D =	420 mm

n = Es/Ec = 6.25482

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

600 dn^2 + 38123.6 dn - 8531935.4) = 0

dn = 91.637 mm

d = 313 mm

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

= 1.37E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 133.878187 kNm

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

= 2.19E+09 mm^4

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

b_w =	1200 mm	A_{st} =	3455.7519 mm^2
d_{sc} =	97 mm	A_{sc} =	3141.5927 mm^2
d =	313 mm		

Ig = 7.41E+09 mm^4

At right end (arbitrary)

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

600 dn^2 + 38123.6 dn - 8366850.4) = 0

dn = 90.52 mm

d = 313 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.37E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 133.878187 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 2.19E+09 mm^4

At left end (arbitrary)

b_w = **1200** mm

d_{sc} = **97** mm

A_{st} = **3455.75** mm^2

A_{sc} = **3141.59** mm^2

d = **313** mm

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

600 dn^2 + 38123.6 dn - 8366850.4) = 0

dn = 90.5171 mm

d = 313 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.37E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 133.878187 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.19E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 2.19E+09 mm^4

Midspan Deflection :

Lef = 4000 mm

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

= 4.9468 mm

Total Deflection :

Long term factor = 1

Short term factor : 1

w - long term = 56.04 kN/m

w - short term : 56.04 kN/m

Defln,s,sus = 4.946797601 mm

At midspan : Ast = 3455.751919 mm^2

Asc = 3141.592654 mm^2

Asc/Ast = 0.909090909

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

= 0.909090909

therefore

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

= 9.44389 mm

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

ALLOWABLE DEFLN = Lef / 250 = 16 mm

> 0.9 x Total Defln

THE THICKNESS FOR THE TANK WALL IS ADEQUATE

Serviceability Steel Requirement

1 DESIGN DATA

Item	Qty	Unit	
Water Capacity	0.00	m^3	
Density of Water	9.81	kN/m^3	
Inner Diameter	25.00	m	
Height of Wall	4.00	m	
Free Board	4.00	m	
Height of Water	0.40	m	
Wall Thickness	0.40	m	
Base Slab Thickness	0.50	m	
Elastic/Youngs Modulus of Concrete	0.00	MPa	
Concrete Grade	40.00	MPa	
Concrete Density	40.00	kN/m^3	
Roof Slab Thickness	0.00	m	Open Tank

3 DESIGN FOR DURABILITY

Exposure classification	B2	
Concrete cover	60	mm
f'c	40	MPa
fy	410	MPa

4 DESIGN FOR SERVICEABILITY

Bar Size	Y20	20	mm
As/bar		310	mm^2
a1		1000	mm
a2		200	mm
Ac,eff = effective concrete area		200000	mm^2
wk = allowable crack width		0.20	mm
ρp.min = minimum serviceability reo ratio		0.58	%
Ac,eff = effective concrete area		200000	mm^2
As.min = ρpmin x Ac,eff		1160	mm^2/m
As (total)		29000.00	mm^2
Total bars each side		94.00	bars
Spacing of bars		266.00	mm
Total bars on both sides		188.00	bars

5 DESIGN FOR STRENGTH

a. HOOP (OR HORIZONTAL) REINFORCEMENT

Maximum Hoop Stress, fmax	0.2	MPa
Yiled Stress of steel, fy	410	MPa

Since yield stress of steel is greater than the max. hoop stress, min. hoop reo's are required

Bar size	Y 16	201	mm^2
ρmin = 0.05f'c/fy		0.005	
As.min = ρpmin x Ac,eff		976	mm^2/m
As (total)		3904	mm^2
Total bars each side		20	bars
Spacing of bars		200.00	mm
Total bars on both sides		40	

b. **FLEXURAL REINFORCEMENT**

ρmin = √f'c/fy	0.015	
As.min = ρpmin x Ac,eff	3086	mm^2/m
As (total)	77150	mm^2
Total bars on both sides	249	bars
Total bars each side	125	
Spacing of bars	200	mm

c. **RECOMMENDATION OF MINIMUM REINFORCEMENTS & SPACINGS**

VERTICAL BARS	Y20 @ 200 CTS
HORIZONTAL BARS	Y16 @ 200 CTS

6 **DESIGN FOR STABILITY**

Inaddition to the mass of the empty structure, the design resistance against uplift and/or buoyancy may take account of:

1 Fixed Base , continuity wall and base slab connection
2 Seismic anchors/ steel dowels
3 Drainage systems
4 Pressure relief valves
5 Any combination of items 2, 3, qnd 4

SUMMARY

A. 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

B. Tank Wall

400 thk RC Wall

Reinforcements:	Vertical:	Y20 - 200 cts	Outside
		Y20 - 200 cts	Inside
	Horizontal	Y16 - 200 cts	

C. Outlet Pit Wall

420 thk RC Wall

Reinforcements:	Vertical:	Y20 - 200 cts	Outside
		Y20 - 200 cts	Inside
	Horizontal	Y16 - 200 cts	

H : Sludge Pump room

1. Load Calculations
2. Results of Structural Analysis
3. Design of Member
4. Summary

Load Breakdown & Calculation

A. Civil: Basement Floor Level

a. DL

Wall Wt, ρ(t)	10	kPa
Slab Wt = ρ(t)	15.0	kPa
Mechanical Load base slab =	71.0	kN

b. LL

Stairs	4	kPa

B. Building

1 Ground Floor Level

Wall Thickness	0.15	m
Base Slab Thickness	0.2	m
Concrete Grade	40	MPa
Concrete Density	25	kN/m^3

a. DL

Wall Wt, ρ(t)	3.75	kPa
Slab Wt = ρ(t)	5	kPa

b. LL

1 Stairs	4	kPa
2 Mechanical Load on Hoist Beam: Dynamic Load	15	kN

2 Roof Level

a. DL

1 4 mm Ply Lined Raking Ceiling	0.074	kPa
2 Colorbond Corrugated Roof Sheeting	0.0493	kPa
3 Services =	0.05	kPa
4 Superimposed DL =	0.05	kPa
Total DL =	0.3	kPa

b. LL	0.25	kPa

Earth Pressure

BASEMENT WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.40	m
Height of Wall (H) =	5.50	m
Embedded Height of Wall (He) =	4.40	m
(Heel Depth) Thickness of Base (tb) =	0.60	m
Width of Base (wb) =	1.00	m
(Heel)/ Length of base (Lb) =	16.00	m
Toe Depth (td) =	0.60	m
Toe length (tl) =	0.00	m
Height of Ground Water (Hw) =	0.00	m
Slope of ground surface behind the wall (β) =	40	degrees
Slope of back face of the wall (α) =	90	degrees
Angle of wall friction (δ') =	40	degrees

MATERIAL PROPERTIES

f'c =	40	MPa
fsy =	410	MPa
Concrete Density =	25	kN/m^3
Cover (c) =	65	mm
Max. allowable crack width (W) =	0.30	mm

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (ϕ') =	42	degrees	
Design cohesion of retained mat'l (C) =	0	kN/m^3	**(Granular backfill)**
Density of retained mat'l (γ) =	20	kN/m^3	
Submerged Density of retained mat'l (γs) =	20	kN/m^3	
Design angle of int'l friction of base mat'l (ϕb) =	45	degree	
Design cohesion of base mat'l (Cb) =	45	kN/m^3	**(Cohesionless soil)**
Density of base mat'l (γb) =	20	kN/m^3	
Allowable gross ground bearing pressure (GBP) =	450	kN/m^2	

LOADINGS (Unfactored)

Surcharge load – live (Q) =	0	kN/m^2
Surcharge load – dead (G) =	10	kN/m^2

LATERAL FORCES

K_O = 0.36 K_A = (1-SIN Ø) =

Earth Pressure coefficients, Ka & Kp;

	Active, Ka	Passive, Kp
Rankines	0.47	1.26
Coulomb	0.47	1.26

PE = 0.5γKaH^2 (B=1.0 m)

Force (kN/m)		Lever arm (m)	
Pa (Fe) =	90.38	LE =	1.467
PS(G) =	0.00	LS =	2.20
PS(Q) =	20.54	LS =	2.20
Total, Pa =	110.92	La =	1.60

EXTERNAL STABILITY (It does not require stability check because the wall is very rigid)

Earthquake Earth Pressure

REFERENCE

NZS 3106 : 2009 - DESIGN OF CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS

SEISMIC DESIGN OF STORAGE TANKS "Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering" : 1986

Seismic Design Data

Location = Port Moresby

Category (pg 15) = C

Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15

p = 0.002

Tank Contents = Non-volatile toxic chemicals

probability factor; Fig. C2.7; pg 51 (Ap) = 1.85

geographical coefficient representing regional seismicity; Fig. C2.1; pg 47

β = 0.50

peak horizontal acceleration coefficient; pg 51 (α) = 0.35

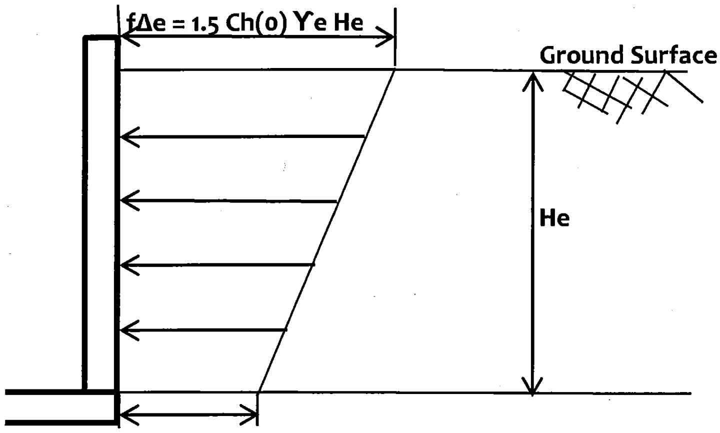
horizontal force coefficient Ch(T) = α β Ah(T) Ap = 0.32 $A_h(T)$

normalized horizontal acceleration response = Ah(T)

4 Foundation Rigid/Fixed Foundation on Weathered Mudstone

Unit weight of engineering backfill soil (Ɣe) = 20.00 kN/m^3

5 Earthquake Earth Pressure: NZS 3106, Appendix A, Cl. A2.6



Increment in at-rest pressure due to earthquake

He = embedded depth =	4.4	m
Ah(0) =	1	
$C_h(0) = \alpha\ \beta\ A_h(0)\ A_p$, horizontal force coefficient =	0.324	
fΔe = 1.5 Ch(0) Ɣe He	42.735	kPa
fΔe = 0.5 Ch(0) Ɣe He	14.245	kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	14.245	kN/m	UDL
Eu2 =	28.49	kN/m	Triangular

LATERAL LOAD DISTRIBUTION TO MASONRY BLOCK WALLS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

approx mass of column = 1 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
ROOF	1	29.2	349.84	9.05	3166.052	18.71144
SUMM			349.84		3166.052	18.71144

V = Cd(T1) Wt   from NZS 1170.5 - 2004
= factor . Wt
= 18.71144

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 REQUIRE A MANUAL INPUT
t = 0.15 m
h = 4 m
Wall Mass = M x t x h x L = 13.8 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 = 0.3 Kpa

WALL	LENGTH	MASS (m)	x	y	mx	my
w1	7.3	100.74	0	3.65	0	367.701
w2	7.3	100.74	4	3.65	402.96	367.701
w3	4	55.2	2	0	110.4	0
w4	4	55.2	2	7.3	110.4	402.96

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	29.2	8.76	2	3.65	17.52	31.974
SUM	Wt =	320.64			641.28	1170.336

TABLE 1

CENTRE OF MASS	Cmx	=	2	Cmy =	3.65

CENTRE OF RIGIDITY

SHEAR WALL	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
w1	0.8	0.001125	0	0.004106	4.677861	4.677861
w2	0.8	0.001125	3.2	0.004106	4.677861	4.677861
w3	0.8	0.001125	1.6	0	4.677861	4.677861
w4	0.8	0.001125	1.6	0.008213	4.677861	4.677861
SUM	3.2	0.0045	6.4	0.016425	18.71144	18.71144

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx	=	2 m	Cry	=	3.65 m
ex	=	0 m	ey	=	0 m

check code eccentricities :

please enter b in x - direc'n = 4 m
please enter b in y - direc'n = 7.3 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx	=	1.2 m	**ex OK**	however torsional
esy <=	0.3by	=	2.19 m	**ey OK**	shears not large

edx1	=	0.2 m	edy1	=	0.365 m		(ed = 1.5es + 0.05b)
edx2	=	-0.2 m	edy2	=	-0.365 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)
w1	0	3.65	0.014988	0.004351	-0.004351	0	0
w2	0	-3.65	0.014988	-0.004351	0.004351	0	0
w3	-2	0	3.2	0	0	-0.929045	0.929045
w4	2	3.65	3.214988	0.004351	-0.004351	0.929045	-0.929045
SUMM			6.444963				

TABLE 3

FINAL LOAD DISTRIBUTION

WALL	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
w1	4.682212	4.673509	4.677861	4.677861	4.682212	4.677861
w2	4.673509	4.682212	4.677861	4.677861	4.682212	4.677861
w3	4.677861	4.677861	3.748815	5.606906	4.677861	5.606906
w4	4.682212	4.673509	5.606906	3.748815	4.682212	5.606906
SUMM					18.7245	20.56953

TABLE 4

LATERAL LOAD DISTRIBUTION TO RC COLUMNS

Reference : Reinforced Concrete by Park and Paulay

DISTRIBUTION OF SHEAR AT EACH LEVEL

Block Wall Wt = 3.45 KPa

FL LEVEL	Column mass	floor area	Wt	Hi	Wt . Hi	Fi
ROOF	1	29.2	101.96	9.05	922.738	5.453403
SUMM			101.96		922.738	5.453403

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

= factor . Wt

= 5.453403

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 = 4 m

Col Mass = M x t x h x L = 40 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 = 0.3 Kpa

WALL	LENGTH	MASS (m)	x	y	mx	my
w1	0.4	16	0	0	0	0
w2	0.4	16	0	7.3	0	116.8
w3	0.4	16	4	0	64	0
w4	0.4	16	4	7.3	64	116.8

FLOOR	AREA	MASS (m)	x	y	mx	my
Roof	29.2	8.76	2	3.65	17.52	31.974
SUM	Wt =	72.76			145.52	265.574

TABLE 1

CENTRE OF MASS	Cmx =	2	Cmy =	3.65

CENTRE OF RIGIDITY

COL	IX m4	IY m4	x . IX	y . IY	W'ix (KN)	W'iy (KN)
C1	0.002133	0.002133	0	0	0.175916	1.363351
C2	0.002133	0.021333	0	0.155733	1.759162	1.363351
C3	0.002133	0.021333	0.008533	0	1.759162	1.363351
C4	0.002133	0.021333	0.008533	0.155733	1.759162	1.363351
SUM	0.008533	0.066133	0.017067	0.311467	5.453403	5.453403

TABLE 2

Centre of Rigidity and Calculated Eccentricities :

Crx =	2 m	Cry =	4.709677 m
ex =	0 m	ey =	-1.059677 m

check code eccentricities :

please enter b in x - direc'n = 4 m
please enter b in y - direc'n = 7.3 m

from PNG 1001 - 1982 Part 4 clause 3.4.7

esx <=	0.3bx =	1.2 m	**ex OK**	however torsional
esy <=	0.3by =	2.19 m	**ey OK**	shears not large

edx1 =	0.2 m	edy1 =	-1.954516 m		(ed = 1.5es + 0.05b)
edx2 =	-0.2 m	edy2 =	-0.694677 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)
C1	0	3.65	0.028421	-0.175526	-0.062386	0	0
C2	0	-3.65	0.284213	1.75526	0.623857	0	0
C3	-2	0	0.008533	0	0	-0.009842	0.009842
C4	2	2.590323	0.151675	-1.245668	-0.442738	0.009842	-0.009842
SUMM			0.472843				

TABLE 3

FINAL LOAD DISTRIBUTION

COL	Wix1 (KN)	Wix2 (KN)	Wiy1 (KN)	Wiy2 (KN)	FINAL LOAD DISTRIBUTION Wix (KN)	Wiy (KN)
C1	0.00039	0.113531	1.363351	1.363351	0.113531	1.363351
C2	3.514422	2.38302	1.363351	1.363351	3.514422	1.363351
C3	1.759162	1.759162	1.353509	1.373193	1.759162	1.373193
C4	0.513494	1.316425	1.373193	1.353509	1.316425	1.373193
SUMM					6.70354	5.473087

TABLE 4

Basement Floor Slab Analysis

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
μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = 0.5 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	1.00	1.04E-02	31.9754	0.18	0.4	0.13233	4.00	0.5614	2.2455	No	No	Use Conventional	132325
2	1.00	1.04E-02	31.9754	0.18	0.4	0.13233	4.00	0.5614	2.2455	No	No	Use Conventional	132325
3	1.00	1.04E-02	31.9754	0.18	0.4	0.13233	4.00	0.5614	2.2455	No	No	Use Conventional	132325
4	1.00	1.04E-02	31.9754	0.18	0.4	0.13233	4.00	0.5614	2.2455	No	No	Use Conventional	132325

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 0.1188

DL = 0.36 kN/m LL = 0.3 kN/m

Combination: DL + LL = 0.66 kN/m

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

50 50 HWD

From analysis

M*max	=	0.07	kNm
V*max	=	0.30	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 3.2076 MPa

Working Shear Stress, fs = 3V/2BD

= 0.1782 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.243609 < 1 OK

fs = 0.1782 MPa < Fs = 0.945 MPa OK

Deflection Check!

DL = 0.36 kN/m

LL = 0.3 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 0.111086 mm Def. by DL

Allowable Defln = L/300 2 mm > 0.111086 = Defln.DL OK

Defln.LL = 0.092571 mm Def. by LL

Allowable Defln = L/360 1.666667 mm > 0.092571 = 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)

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

Rafter Design

Loads

DL = 0.27 kN/m LL = 0.225 kN/m

Combination: DL + LL = 0.495 kN/m

Size B (mm) D (mm) Rafter Span = 4 m

100 150 HWD

From Space Gass Analysis

M*max	=	1.80	kNm
V*max	=	2.64	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 4.8 MPa

Working Shear Stress, fs = 3V/2BD

= 0.2644 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.364548 < 1 OK

fs = 0.2644 MPa < Fs = 0.945 MPa OK

Deflection Check!

DL = 0.27 kN/m

LL = 0.225 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 3.05E-12 mm Def. by DL

Allowable Defln = L/300 0.013333 mm > 3.05E-12 = Defln.DL OK

Defln.LL = 2.54E-12 mm Def. by LL

Allowable Defln = L/360 0.011111 mm > 2.54E-12 = Defln.LL OK

Job name: SLUDGE **Designer: FYP** **Date: 17 Oct 2011, 4:04 pm** **Page: 1**
POMSSUP
Sludge Pump Room

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	350DP x 250WD	S1	Not applicable	No	Standard shape
2	Section 2	S2	Not applicable	No	Standard shape
3	400 SQ RC COL	S3	Not applicable	No	Standard shape
4	500DP x 400WD	S4	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	7.5000E-02	7.7520E-04	3.9060E-04	5.6250E-04	INFINITE	INFINITE	0.00
2	7.5000E-03	4.9389E-06	1.5625E-06	1.4100E-05	INFINITE	INFINITE	0.00
3	1.6000E-01	3.6053E-03	2.1333E-03	2.1333E-03	INFINITE	INFINITE	0.00
4	2.0000E-01	5.4742E-03	2.6667E-03	4.1667E-03	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.300	0.250			
2	Rectangle	0.150	0.050			
3	Rectangle	0.400	0.400			
4	Rectangle	0.500	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	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 5: D+1.3L+E

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

LOAD CASE TITLES

Load Case	Title
1	D
2	L
3	1.4D+1.7L
4	E
5	D+1.3L+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
3	3	56.252*	3.110	-13.746	-0.965	13.447	-3.478
1	4	-3.876#	5.674	0.074	-0.001	-0.098	-15.458
8	3	13.309	29.796*	0.326	3.489	-0.223	-35.457
18	3	13.302	-20.967#	-0.033	-3.533	-0.325	-28.891
4	3	47.302	2.978	13.264*	0.186	-17.639	-3.111
3	3	56.252	3.110	-13.746#	-0.965	13.447	-3.478
13	3	6.870	-12.439	-0.083	4.209*	0.182	-1.854
5	3	6.863	15.234	-0.442	-4.255#	0.758	-17.202
27	3	28.465	2.978	13.264	0.186	28.785*	7.313
26	3	37.416	3.110	-13.746	-0.965	-34.665#	7.408
14	3	13.302	21.476	0.436	2.001	0.665	23.489*
8	3	13.309	29.796	0.326	3.489	-0.223	-35.457#

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

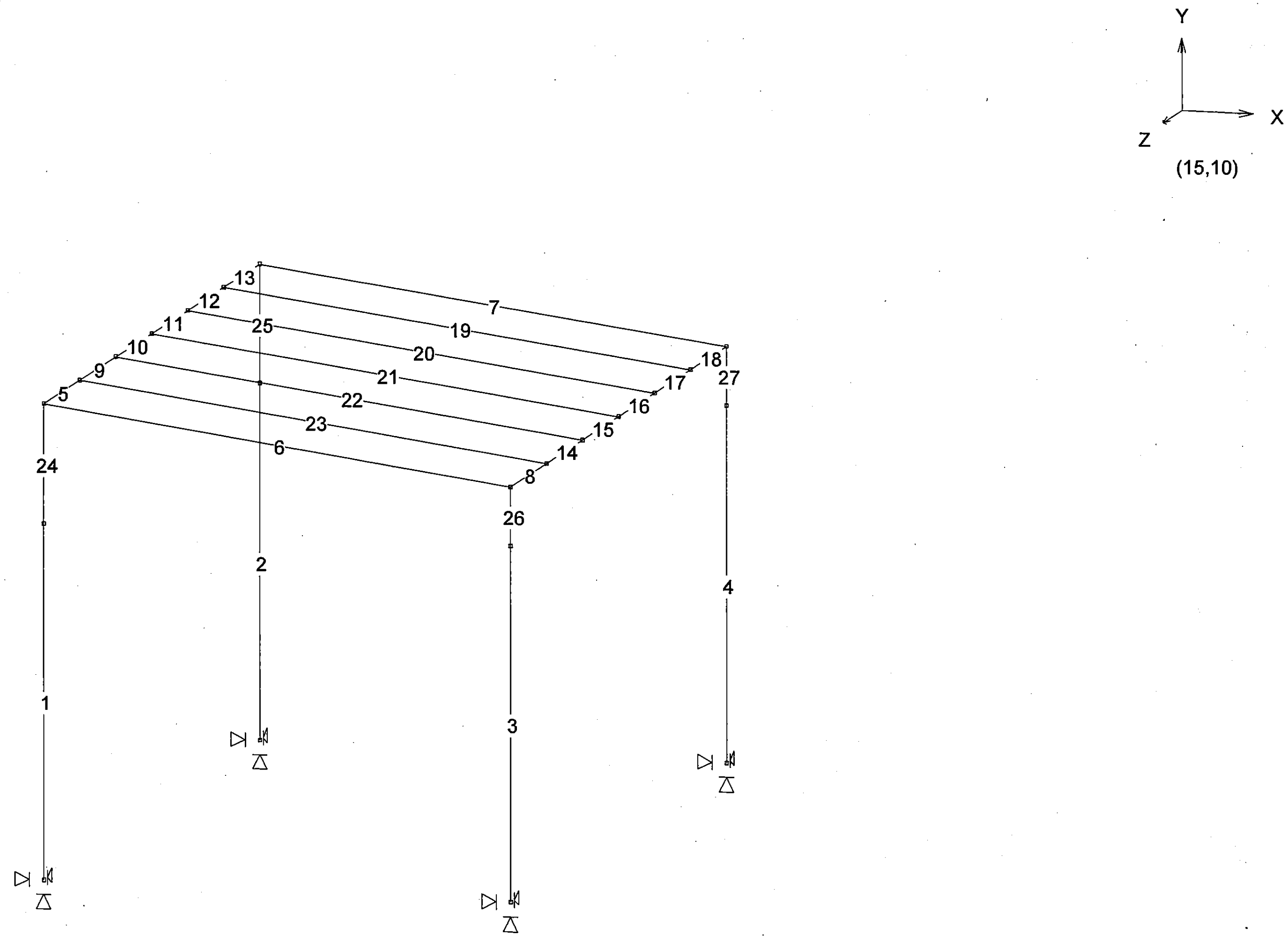
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
3	3	3.094*	43.626	6.908	9.682	-0.101	-4.360
2	5	-9.988#	44.242	-9.974	-9.783	-0.691	21.813
2	3	-3.110	56.252*	-13.746	-13.447	-0.965	3.478
1	4	-5.674	-3.876#	0.074	0.098	-0.001	15.458
4	3	-2.978	47.302	13.264*	17.639	0.186	3.111
2	3	-3.110	56.252	-13.746#	-13.447	-0.965	3.478
4	3	-2.978	47.302	13.264	17.639*	0.186	3.111
2	3	-3.110	56.252	-13.746	-13.447#	-0.965	3.478
4	3	-2.978	47.302	13.264	17.639	0.186*	3.111
2	3	-3.110	56.252	-13.746	-13.447	-0.965#	3.478
2	5	-9.988	44.242	-9.974	-9.783	-0.691	21.813*
3	3	3.094	43.626	6.908	9.682	-0.101	-4.360#

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
3	3	56.252*	3.110	-13.746	-0.965	13.447	-3.478
1	4	-3.876#	5.674	0.074	-0.001	-0.098	-15.458
8	3	13.309	29.796*	0.326	3.489	-0.223	-35.457
18	3	13.302	-20.967#	-0.033	-3.533	-0.325	-28.891
4	3	47.302	2.978	13.264*	0.186	-17.639	-3.111
3	3	56.252	3.110	-13.746#	-0.965	13.447	-3.478
13	3	6.870	-12.439	-0.083	4.209*	0.182	-1.854
5	3	6.863	15.234	-0.442	-4.255#	0.758	-17.202
27	3	28.465	2.978	13.264	0.186	28.785*	7.313
26	3	37.416	3.110	-13.746	-0.965	-34.665#	7.408
14	3	13.302	21.476	0.436	2.001	0.665	23.489*
8	3	13.309	29.796	0.326	3.489	-0.223	-35.457#

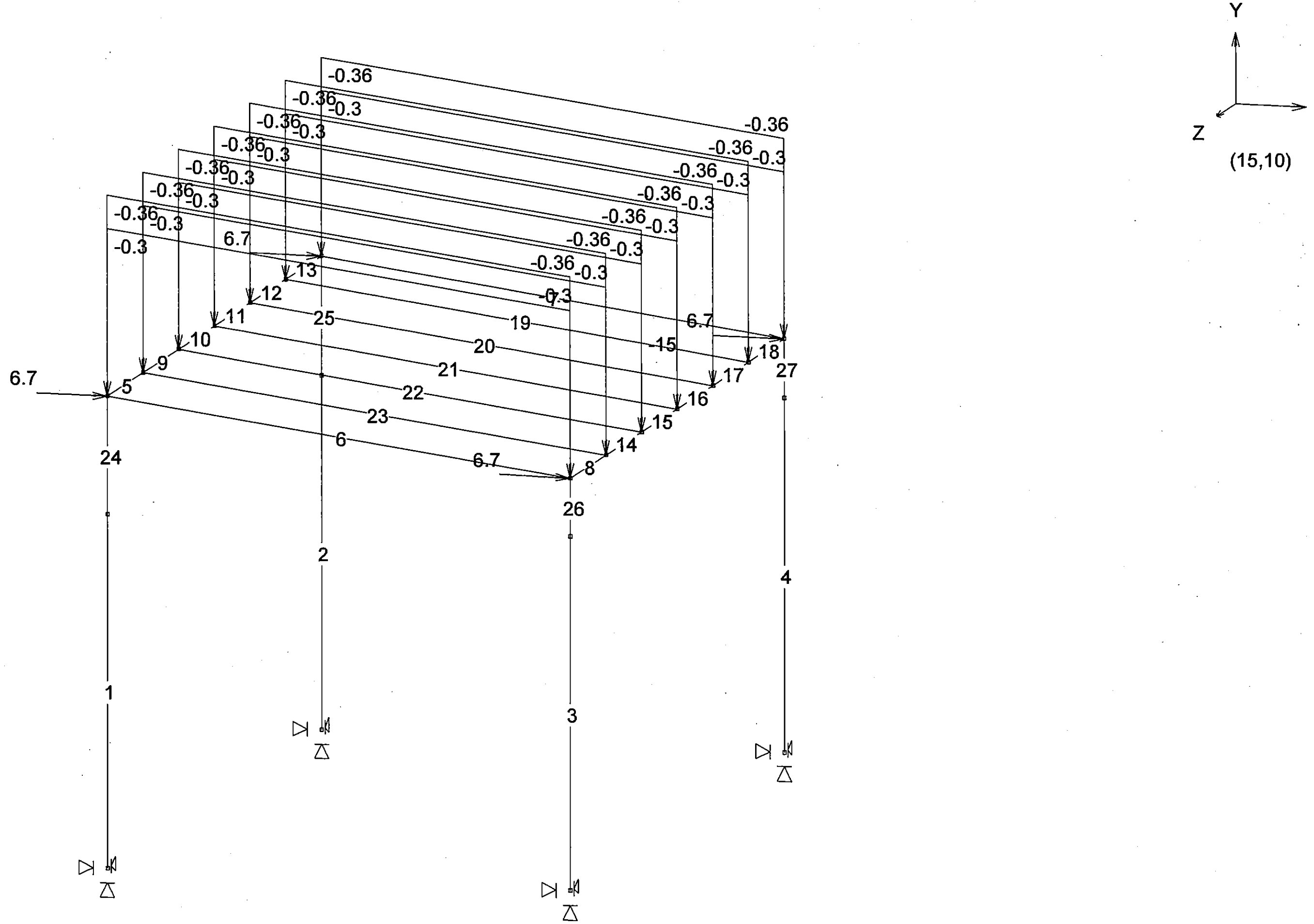


Sections:
1 350DP x 250WD
2 Section 2
3 400 SQ RC COL

Sludge Pump Room Model

Job: SLUDGE, Designer: FYP, Units: m,kN,MPa, Scale: 1:61, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
POMSSUP
Sludge Pump Room 17 Oct 2011, 4:02 pm

1	———	(SW)	D
2	———		L
3	———		1.4D+1.7L
4	———		E
5	———		D+1.3L+E



Sections:
1 350DP x 250WD
2 Section 2
3 400 SQ RC COL

Sludge Pump Room Model with Loadings

Job: SLUDGE, Designer: FYP, Units: m,kN,MPa, Scale: 1:61, Axes: XY
Load: 0.013 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Sludge Pump Room 17 Oct 2011, 4:03 pm

ROOF BEAM DESIGN

REFERENCE:	**Reinforced concrete slab - 3rd Edition**			
	R. F. Warner, B. V. Rangan, & A. S. Hall			
Design of a Continuous Beam :	**Australian Standard (AS) 3600 - 2009**			

1 Loads

DL	:	Wdl =	0.60 kN/m	(plus super)
Self wt	:	Wsw =	3.06 kN/m	
Blockwall	:	Wb =	16.74 kN/m	
TOTAL		=	20.40 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	350 mm	D =	350 mm
tff	0 mm	Cover =	65 mm
Actual length, L =	6900 mm	Overall depth of Sect. D =	350 mm
Effective Length,Lef =	6500 mm	Area, A =	1.23E+05 mm^2
Neutral Axis, yc =	175.00 mm	Moment of Inertia, Ix-x =	1.25E+09 mm^4
Neutral Axis, xc =	825.00 mm	Moment of Inertia, Iy-y =	1.25E+09 mm^4
Effective width, beff =	1650.00 mm	Gross Area, Ag =	1.23E+05 mm^2

3 Design Parameters

F'c =	32 MPa	D =	350 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 =	1650 mm	clause 8.8.2			
Is the section RECT or T, L (ans : 1 or 2)		=		1	
k_1 =	0.045	(RECT sections)			1
=	0.02003883	(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 =	33.7991				
defln/Lef =	0.004	k_1/k_2 =	17.27977882		
d =	142 mm				
take d =	253 mm	Distance from d to extreme fibre of beam (do) =		97 mm	

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	28.90 kNm	(left support)
Mmax +ve	=	23.50 kNm	(mid-span)
M -ve	=	35.50 kNm	(right support)
Vmax	=	30.00 kN	
V (other end)	=	21.00 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 =	97 mm	d =	253		
Estimate Area of main steel		Ast =	364.1933653 mm^2			
3	Y 20	Ast =	942.4777961 mm^2			
	16	p =	0.010643453			
3	Y 16	Asc =	603.1857895 mm^2			
		pc =	0.00681181	<	pd =	0.021813
		p-pc =	0.003831643	pd=0.4*0.85*µ*f'c/fsy		

Check M* <= phiMu

ku = 0.070263 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)]

= 68 kNm	>	M* =	28.90 kNm	OK

Determine -ve steel at right support :

M -ve = 35.50 kNm

Number of reinforcement layers 1 or 2 :				1			
take	do	=	97 mm	dst	=	253 mm	
Estimate Area of main steel		Ast	=	447.3655525 mm^2			
3	Y 20	Ast	=	942.4777961 mm^2			
		p	=	0.010643453			
3	Y 16	Asc	=	603.1857895 mm^2			
		pc	=	0.00681181	<	pd =	0.021813
		p-pc	=	0.003831643			

Check M* <= phiMu

ku = 0.070263 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)]

= 68 kNm > M* = 35.50 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 23.5 kNm

Steel in one layer then code = 1
" " two " " code = 2
what is the code ? 2

take dist between centroid of bar/s and soffit = 107 mm
d = 243 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 308.330365 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 = 1650 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.04310464

hence dn = 10 < 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)]

= 83 kNm > M*+ve = 23.50 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.

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 = 1950 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 = 1950 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 = 650 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 65 mm

d_b = 16 mm # of lines of bars = 1

a_b = 196 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 = 212 mm
coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

k2 = 2.2 a_b = 196 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 = 0.424238281

therefore | Lsy = 116 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 = 524 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 504 mm (dist between bars)

factor = 50093.50513

Lsyt = 333.9567 mm take fs/Fsy = M*/Phi Mu = 0.28309163

therefore | Lsy = 95 mm | ≥ 25k1db 400

| say = 400 mm |

9 **SHEAR CHECK**

V* = 30.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 253 mm dq = 93 mm

do = 257 mm

V* = 25 kN beta 1 = 1.2715 (but not less than 1.1)

hence = 1.2715

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

= 55.64 kN

phi Vumax = 402.976 kN > V* No web crushing

phi Vumin = 93 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 350 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 298.780488

Asv.min = 104.573171 mm^2

Asv.max = 1648 mm^2

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

theta = 31.12166 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 189.19 kN

Phi Vu = 132 kN > V* = 24.7749175 kN

hence

PROVIDE :	1 Y12 LIGS	@	350	centres

Other Support :

V* = 21.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 253 mm dq = 93 mm

do = 257 mm

V* = 15.77 kN beta 1 = 1.2715 (but not less than 1.1)

hence = 1.2715

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

= 55.63914 kN

phi Vumax = 402.976 kN > V* No web crushing

phi Vumin = 93 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 350 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 298.780488

Asv.min = 104.573171 mm^2

Asv.max = 1648.17718 mm^2

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

theta = 32.19059 degrees

Determine Phi Vu :

Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 237.3053 kN
Phi Vu = 166.1137 kN > V* = 15.77 kN
hence

PROVIDE : 1 Y12 LIGS @ 350 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	28.90 kNm	Es	=	200000 MPa
Ml	=	35.50 kNm	Ec	=	28599.62293 MPa
Mo	=	55.7 kNm			
Ms	=	23.50 kNm			
Ig	=	1.25E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1650 mm	A_{sc} =	603.1857895 mm^2
webwidth	b_w =	350 mm	A_{st} =	942.4777961 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	107 mm
			d_{st} =	243 mm
			d =	253 mm
			D =	350 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)
175 dn^2 + 10205.793 dn - 2054282.593) = 0
dn = 83.0414327 mm
d = 243 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2
= 2.35E+08 mm4
Mcr = 0.6sqrtF'c x bD^2/6 = 24.2537626 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]
= 1.35E+09 mm^4

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

b_w =	350 mm	A_{st} =	942.4777961 mm^2
d_{sc} =	85 mm	A_{sc} =	603.1857895 mm^2
d =	253 mm		

Ig = 1.25E+09 mm^4

At right end (arbitrary)
1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
175 dn^2 + 10205.793 dn - 1974753.643) = 0
dn = 81.00 mm
d = 253 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.57E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 24.2537626 kNm
Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 8.44E+08 mm^4

At left end (arbitrary)
b_w = 350 mm
d_{sc} = 85 mm
A_{st} = 942.4778 mm^2
A_{sc} = 603.18579 mm^2
d = 253 mm
1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
175 dn^2 + 10205.793 dn - 1974753.643) = 0
dn = 80.9976504 mm
d = 253 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.57E+08 mm^4
Mcr = 0.6sqrtF'c x bD^2/6 = 24.2537626 kNm
Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 5.74E+08 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.03E+09 mm^4

Midspan Deflection :
Lef = 6500 mm
Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 2.54831959 mm

Total Deflection :

Long term factor = 0.2
Short term factor = 0.5
w - long term = 20.4525 kN/m
w - short term = 20.5275 kN/m
Defln,s,sus = 2.539008958 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)
= 5.67638 mm

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

ALLOWABLE DEFLN = Lef / 500 = 13 mm
> 0.9 x Total Defln

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GROUND FLOOR BEAM SLUDGE PUMP ROOM

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

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

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.600	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	CONCRETE-25	2.5280E+07	0.15	2.4500E+03	1.0000E-05	25000.00

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

.R 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	3	0.000	364.833*	0.000	0.000	0.000	-419.559
1	3	0.000	-364.833#	0.000	0.000	0.000	-419.559
1	2	0.000	55.200	0.000	0.000	0.000	-63.480*
1	3	0.000	364.833	0.000	0.000	0.000	-419.559#

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

Envelope = All Load Cases
and All Nodes

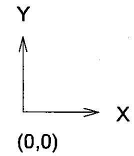
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	3	0.000	364.833*	0.000	0.000	0.000	419.559
1	2	0.000	55.200#	0.000	0.000	0.000	63.480
1	3	0.000	364.833	0.000	0.000	0.000	419.559*
2	3	0.000	364.833	0.000	0.000	0.000	-419.559#

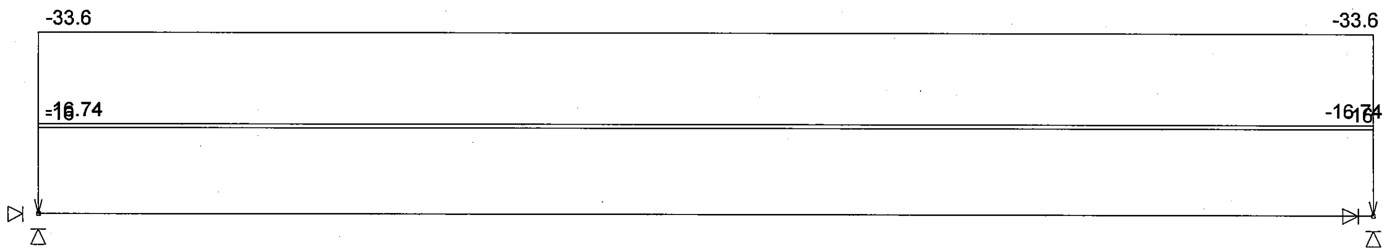
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

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
	3	0.000	364.833*	0.000	0.000	0.000	-419.559
	3	0.000	-364.833#	0.000	0.000	0.000	-419.559
1	3	0.000	0.000	0.000	0.000	0.000	209.779*
1	3	0.000	364.833	0.000	0.000	0.000	-419.559#

1 (SW) D
2 L
3 1.4D+1.7L

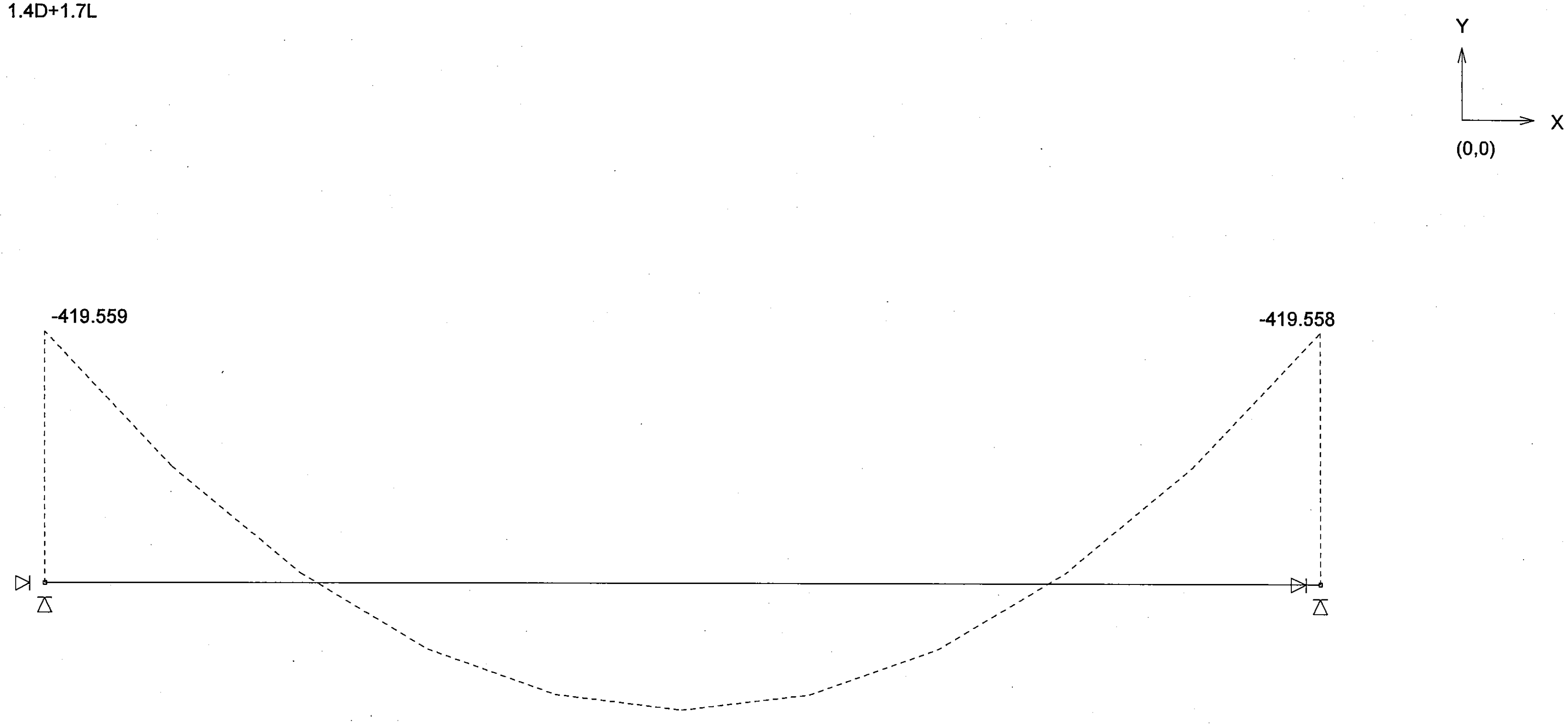




GROUND FLOOR BEAM WITH LOADINGS

Job: GB, Designer: FYP, Units: m,kN,MPa, Scale: 1:39, Axes: XY
Load: 1.4 Disp: None Moment: None Shear: None Axial: None
POMSSUP
GROUND FLOOR BEAM SLUDGE PUMP ROOM 17 Oct 2011, 4:20 pm

3 1.4D+1.7L



GROUND FLOOR BEAM ANALYSIS MODEL

Job: GB, Designer: FYP, Units: m,kN,MPa, Scale: 1:39, Axes: XY
Load: 1.4 Disp: None Moment: 12 Shear: None Axial: None
POMSSUP
GROUND FLOOR BEAM SLUDGE PUMP ROOM 17 Oct 2011, 4:21 pm

GROUNG FLOOR BEAM DESIGN

REFERENCE: **Reinforced concrete slab - 3rd Edition** AS 3600 - 2009

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	28.32	kN/m	(plus super)
Self wt	:	Wsw =	**5.00**	kN/m	
Blockwall	:	Wb =	16.74	kN/m	
TOTAL		=	50.06	kN/m	
LL :		Wll =	4	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	**200**	mm	Cover =	65	mm
Actual length, L =	**6900**	mm	Overall depth of Sect. D =	700	mm
Effective Length,Lef =	**6500**	mm	Area, A =	4.60E+05	mm^2
Neutral Axis, yc =	334.78	mm	Moment of Inertia, Ix-x =	2.49E+10	mm^4
Neutral Axis, xc =	850.00	mm	Moment of Inertia, Iy-y =	8.35E+10	mm^4
Effective width, beff =	1700.00	mm	Gross Area, Ag =	5.40E+05	mm^2

3 Design Parameters

F'c =	**40**	MPa	D =	500	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 =	1700 mm		clause 8.8.2	
Is the section RECT or T , L (ans : 1 or 2)		=	**1**	
k_1 =	0.045		(RECT sections)	**1**
=	0.0205911		(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 =	87.5224			
defln/Lef =	**0.004**	k_1/k_2 =	17.27977882	
d =	186 mm			
take d =	587 mm		Distance from d to extreme fibre of beam (do) =	**113** mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	**420.00**	kNm	(left support)
Mmax +ve	=	**210.00**	kNm	(mid-span)
M -ve	=	**420.00**	kNm	(right support)
Vmax	=	**365.00**	kN	
V (other end)	=	**365.00**	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 :			2	
take do =	**113** mm		d =	587
Estimate Area of main steel		Ast =	2281.213312	mm^2
5	**Y 24**	Ast =	**2261.946711**	mm^2

		16 p	=	0.009633504			
3	Y 20	Asc	=	**942.4777961** mm^2			
		pc	=	0.00401396	<	pd =	0.025409
		p-pc	=	0.005619544	pd=0.4*0.85*μ*f'c/fsy		

Check M* <= phiMu

ku = 0.088466 dsc = 87 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)]

=	449	kNm	>	M*	=	420.00 kNm	OK

Determine -ve steel at right support :

M -ve = 420.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 113 mm dst = 587 mm

Estimate Area of main steel		Ast	=	2281.213312 mm^2			
5	Y 24	Ast	=	**2261.946711** mm^2			
		p	=	0.009633504			
3	Y 20	Asc	=	**942.4777961** mm^2			
		pc	=	0.00401396	<	pd =	0.025409
		p-pc	=	0.005619544			

Check M* <= phiMu

ku = 0.088466 dsc = 87 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)]

=	449	kNm	>	M*	=	420.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 210 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 113 mm

what is the code ? 2 d = 587 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1140.60666 mm^2

5	**Y 24**	Ast	=	2261.946711 mm^2
3	**Y 20**	Asc	=	942.4777961 mm^2

Assume N - A in flange

Effective width : beff = 1700 mm

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

= 0.03568383

hence dn = 21 < 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)]

=	483 kNm	>	M*+ve =	210.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	5	Y 24
	Asc	=	3	Y 20
Right support :	Ast	=	5	Y 24
	Asc	=	3	Y 20
midspan	Ast	=	5	Y 24
	Asc	=	3	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 = 1950 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 = 1950 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 = 650 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 65 mm

d_b = 20 mm   # of lines of bars = 1

a_b = 246 mm   distance between bars

If a_b > 2c   C = 2c + d_b = 150 mm
C is the outside diameter of a concrete annulus

if a_b =< 2c   C = a_b + d_b = 266 mm
coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.1592654 mm^2

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

factor = 56006.24135

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 373.3749 mm

take fs/Fsy = M*/Phi Mu = 0.935531833

therefore

Lsy =	349 mm
say =	500 mm

≥ 25k1db 625

For Bottom Steel :

Cover, c = 65 mm d_b = 24 mm
# of lines of bars = 1

If ab > 2c   C = 2c + d_b = 154 mm

if ab < 2c   C = a_b + d_b = 578 mm

k1 = 1 A_b = 452.3893421 mm^2

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

factor = 64519.19003

Lsyt = 418.9558 mm   take fs/Fsy = M*/Phi Mu = 0.434559105

therefore

Lsy =	182 mm
say =	400 mm

≥ 25k1db 500

9 SHEAR CHECK

V* = 365.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 587 mm   dq = 93 mm
do = 407 mm

V* = 333 kN   beta 1 = 1.1965 (but not less than 1.1)
hence = 1.1965
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
= 112.13 kN

phi Vumax = 911.68 kN > V* No web crushing phi Vumin = 181 kN > V* SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 350 mm Fsyf = 410 MPa
Determine theta :

bv x s/Fsy f = 341.463415

Asv.min = 119.512195 mm^2

Asv.max = 2396 mm^2

Asv = 2 x area of stirrups = 220 mm^2 2 Ties
theta = 30.6622 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 337.10 kN
Phi Vu = 236 kN < V* = 333.26678 kN
hence

PROVIDE :	**1 Y12 LIGS**	**@**	**350**	**centres**

Other Support :
V* = 365.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 587 mm dq = 93 mm
do = 407 mm
V* = 333.27 kN beta 1 = 1.1965 (but not less than 1.1)
hence = 1.1965
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
= 112.1277 kN
phi Vumax = 911.68 kN > V* No web crushing
phi Vumin = 181 kN < V* SHEAR REO REQ'D
If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 350 mm Fsyf = 410 MPa
Determine theta :

bv x s/Fsy f = 341.463415

Asv.min = 119.512195 mm^2

Asv.max = 2395.73417 mm^2

Asv = 2 x area of stirrups = 330 mm2 2 Ties
theta = 31.38709 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 418.0685 kN
Phi Vu = 292.6479 kN < V* = 333.27 kN
hence

PROVIDE :	**1 Y12 LIGS**	**@**	**350 centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 420.00 kNm Es = 200000 MPa
Ml = 420.00 kNm Ec = 31975.35051 MPa
Mo = 630 kNm
Ms = 210.00 kNm
Ig = 2.49E+10 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width b_{eff} = 1700 mm A_{sc} = 942.4777961 mm^2

webwidth	b_w =	400 mm	A_{st} =	2261.946711 mm^2
Flange thickness	t_{ff} =	200 mm	d_{sc} =	113 mm
			d_{st} =	587 mm
			d =	587 mm
			D =	700 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)

200 dn^2 + 279100.614 dn - 34864552.32) = 0

dn = 115.3782 mm

d = 587 mm

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

= 4.28E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 123.961284 kNm

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

= 8.52E+09 mm^4

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

b_w =	400 mm	A_{st} =	2261.946711 mm^2
d_{sc} =	87 mm	A_{sc} =	942.4777961 mm^2
d =	587 mm		

Ig = 2.49E+10 mm^4

At right end (arbitrary)

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

200 dn^2 + 19100.6145 dn - 8735786.036) = 0

dn = 166.63 mm

d = 587 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.12E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 63.2455532 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 3.19E+09 mm^4

At left end (arbitrary)

b_w = **400** mm

d_{sc} = **87** mm

A_{st} = **2261.9467** mm^2

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

d = **587** mm

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

200 dn^2 + 19100.6145 dn - 8735786.036) = 0

dn = 166.629292 mm

d = 587 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.12E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 63.2455532 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 3.19E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 5.85E+09 mm^4

Midspan Deflection :

Lef = 6500 mm

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

= 2.96282467 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 50.86 kN/m

w - short term	=	52.06 kN/m		
Defln,s,sus	=	2.894530595 mm		
At midspan :		Ast	=	2261.94671 mm^2
		Asc	=	942.477796 mm^2
Asc/Ast =		0.416666667		
kcs	=	2 - (1.2 x Asc/Ast)		>= 0.8
	=	1.5		

therefore

Total Defln	=	Dfln,s + (kcs x Defln,s,sus)
	=	7.30462 mm

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

ALLOWABLE DEFLN	=	Lef / 500 =	13 mm
			> 0.9 x Total Defln

Ground Floor Slab Design

REFERENCE: **Reinforced concrete slab - 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	=	40 MPa	Short term live load factor, y s=	1.00	Table 9.2
c	=	55 mm	Long term live load factor, y l =	0.80	Table 9.2
fsy	=	410 MPa	K_1 =	1.70	Cl 16.1
Lx	=	7300 mm			
Ly	=	12000 mm			
Ec	=	24E+3 MPa			
ρc	=	2400 kg/m^3			

Design Loading

Dead load — Imput Load = 1 or calculated loads = 2?

Slab thickness	=	200 mm	Slelect : 1
Self weight, wt	=	4.80 kN/m^2	
Ceiling & Services	=	0.00 kN/m^2	
Super	=	1.00 kN/m^2	
	Total =	5.80	
Total DL		4.70 kN/m^2	Imput Load = 4.70 kPa
Live load	=	4.00 kN/m^2	
Service Load	=	8.7 kN/m^2	

HENCE

$Lx / Ds =< 105 [(Lx/Ly)^2 /wk]^{0.33}$

$= \frac{7300}{Ds} =< 30.7488$ Ds = 237.408 mm

Is these a domestic building? 2 Enter 1 = yes
2 = no

No it is not a domestic building

-

-

Hence minimum thickness = 237.4076 mm
Take Ds = 200 mm

Design moments and shears:

Design load, Fd = 1.2G + 1.5Q
= 11.64 kN/m^2

Load combination
DL = 1.20
LL = 1.50

Design moment :

				Design moments in central regions					
		Ly/Lx =	1.64	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	1.75	0.0960	0.0430	59.55	79.20	29.77	26.67	35.47	13.34
B	1.75	0.0960	0.0430	59.55	79.20	29.77	26.67	35.47	13.34

Maximum design unit shear force in the slab = 42.486 kN/m

Check Flexral steel:

Design parameters

gamma, g = 0.85
short span d = 137 mm
Long span d = 121 mm

pmin = 0.001951
f = 0.8

Reinf. Y 16

Short span
f bd^2 Fsy p = -6.2E+9
f bd^2 Fsy p (- 0.6pFsy / F'c) = 37.9E+9
Min. As = 0.8/fsy *b * d
= 267.3171 mm^2/m

Long span
f bd^2 Fsy p = -4.8E+9
f bd^2 Fsy p (- 0.6pFsy / F'c) = 4.9E+9
Min. As = 0.8/fsy *b * d
= 236.0976 mm^2/m

f Mu > M*

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

Midspan

Panel	Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Short	59.5E+6	0.0103	1415.07	16	200.96	142.014	140
B		59.5E+6	0.0103	1415.07	16	200.96	142.014	140

Continious edge

Panel	Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Short	79.2E+6	0.0141	1929.64	16	200.96	104.144	100
B		79.2E+6	0.0141	1929.64	16	200.96	104.144	100

Discontinious edge

Panel	Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Short	29.8E+6	0.0050	683.57	16	200.96	293.987	290
B		29.8E+6	0.0050	683.57	16	200.96	293.987	290

Midspan

Panel	Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Long	26.7E+6	0.0056	765.29	16	200.96	262.594	250
B		26.7E+6	0.0056	765.29	16	200.96	262.594	250

Continious edge

Panel	Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Long	35.5E+6	0.0074	1019.78	16	200.96	197.063	190
B		35.5E+6	0.0074	1019.78	16	200.96	197.063	190

Discontinous edge

Panel	Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
A	Long	13.3E+6	0.0028	381.55	16	200.96	526.697	350
B		13.3E+6	0.0028	381.55	16	200.96	526.697	350

Shrinkage and Temperature:

pmin = 0.0018   Table 16.1

As = pmin b Ds   reinf. y 12

= 246.600 mm^2/m As = 113.04 mm^2

spacing = 458.394 mm

say 350 mm

USE Y 12 at 350 CTS

Check for Shear:

b1 = 1.4 - d/2000   f = 0.7

= 1.3315   d = 137 mm

p = 0.0039

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

= 98.259 kN/m

f Vuc = 68.781 kN/m   >   Ok

Hence shear failure is not critical

Reinforced Masonry Block Wall Design

REFERENCE: **Reinforced concrete slab - 3rd Edition** **Australian Standard (AS) 3600 - 2009**

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

Step	Calculation & Description				Result	Unit
1	Data					
	N*	=			21	kN
	M*	=			84	kNm
	V*	=			21	kN
	f'c	=			15	MPa
	fsy	=			410	MPa
	εsy				0.002	
	Lw	=	... overall length of wall ...		7300	mm
	Hw	=	... unsupported height of wall ...		4000	mm
	tw	=	... wall thickness ...		150	mm
	Hwe	=	0.8 Hw, effective wall height		3200	mm
	ChecK: Hwe/tw	=	21.33333	<	30	**OK**

Check maximum shear strength

dw	=	0.8Hw		3200	
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)		**2020**	kN
ϴvu.max	=			1414	kN
ϴvu.max	>	V* **OK**			

Vertical steel in the wall

Gross concrete area of the wall cross-section

Ag	=			480000	mm^2
N*/Ag	=			0.04	MPa

In design, substituting for Vu from ϕVu ≥ V*, we obtain;

ϕlfsy	≥	V*cotϴ/ϕtwdw - N*/Ag	ϕ = 0.7		
	≥			0.03	MPa
Al	≥	(ϕfsy/fsy)(twLw)	91.8064 mm^2	92	mm^2
AsV	=	Al		92	mm^2
Using	Y12	bars	area/bar = 110 mm^2		
# of bars	=	0.836364		1	bars

1 bars at each wall face and the average spacing is at;

Spacing of bars	=	(Lw-2b)/(# bars @ each wall face)		7300	mm CTS
Use Y16 bars at		400 mm CTS at each wall face.			
# of bars	=	2(1+ (Lw - 2b-Ast/bar)/spacing)		38	bars
AsV (provided) =				4180	mm^2

Therefore;

pl = [(# of bars x bar dia) + (4Ast/bar dia of beam)]/(t 0.006

It is necessary to ensure that pl is not less than 0.0025 for crack control purposes.

In this case;

pl = 0.0060 > 0.0025 **OK**

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	=	1.4/fsy		0.003415	
Ast.min	=	1.4/fsy twHw		2048.78	mm^2
For	Y12	bars,	area/bar = 110 mm^2		
# of bars	=	18.62528		19	bars

10 bars at each wall face and the average spacing is at;

# of bars	=		20	bars
Spacing of bars	=	(Hw-bar area)/(# bars @ each wall face)	389	mm CTS

Therefore, use Y16 bars 400 mm CTS at each wall face.

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		35	kN

2 Trial depth of neutral axis, dn = 706 mm

Compressive force in concrete, Cc = 0.85γf'cbdn (include boundary element) 1310 kN

dc = γdn/2 342 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	110	100	100	0.0026	410	45	4510	160105
2	1	110	200	300	0.0017	345	38	11386.4	127148
3	1	110	400	700	0.0000	10	1	785.2691	3309
4	1	110	400	1100	-0.0017	-670	-74	-81032.29	-187848
5	1	110	400	1500	-0.0034	-410	-45	-67650	-96965
6	1	110	400	1900	-0.0051	-410	-45	-85690	-78925
7	1	110	400	2300	-0.0068	-410	-45	-103730	-60885
8	1	110	400	2700	-0.0085	-410	-45	-121770	-42845
9	1	110	400	3100	-0.0102	-410	-45	-139810	-24805
10	1	110	400	3500	-0.0119	-410	-45	-157850	-6765
11	1	110	400	3900	-0.0136	-410	-45	-175890	11275
12	1	110	400	4300	-0.0153	-410	-45	-193930	29315
13	1	110	400	4700	-0.0170	-410	-45	-211970	47355
14	1	110	400	5100	-0.0187	-410	-45	-230010	65395
15	1	110	400	5500	-0.0204	-410	-45	-248050	83435
16	1	110	400	5900	-0.0221	-410	-45	-266090	101475
17	1	110	400	6300	-0.0238	-410	-45	-284130	119515
18	1	110	400	6700	-0.0255	-410	-45	-302170	137555
19	1	110	400	7100	-0.0272	-410	-45	-320210	155595
20	1	110	200	7300	-0.0280	-410	-45	-329230	164615
∑	20	2200	100	7400			-666	-2973301	542440

4 Axial Strength of the wall;

Nu = ∑Fsi + Cc [redacted] kN

dN = 1/Nu (Ccdc + ∑Fsidsi) -3922.21 mm

dq = 0.5D ... plastic centroid ... 3,650 mm

e = dq-dN ... eccentricity ... 7,572 mm

Summing up moments of forces about the palsic centroid gives;

Mu = 4,874 kNm

ϕMu = 2,925 kNm

e = Mu/Nu 7,572 mm

Therefore, Nu = 644 kN

ϕMu > M*

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

PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR SLUDGE PUMP ROOM

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	600DP x 400WD	S1	Not applicable	No	Standard shape
2	600DP x3900WD	S2	Not applicable	No	Standard shape
3	600DPx2300WD	S3	Not applicable	No	Standard shape
4	600DPx1950WD	S4	Not applicable	No	Standard shape
5	600DPx1150WD	S5	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	2.3400E+00	2.5359E-01	2.9660E+00	7.0200E-02	INFINITE	INFINITE	0.00
3	1.3800E+00	1.3839E-01	6.0835E-01	4.1400E-02	INFINITE	INFINITE	0.00
4	1.1700E+00	1.1320E-01	3.7074E-01	3.5100E-02	INFINITE	INFINITE	0.00
5	6.9000E-01	5.5752E-02	7.6044E-02	2.0700E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.600	0.400			
2	Rectangle	0.600	3.900			
3	Rectangle	0.600	2.300			
4	Rectangle	0.600	1.950			
5	Rectangle	0.600	1.150			

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

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
69	3	0.000	141.862*	0.000	-0.910	0.000	-64.221
96	3	0.000	-141.862#	0.000	0.910	0.000	-64.221
57	3	0.000	65.204	0.000	25.644*	0.000	-28.818
49	3	0.000	65.204	0.000	-25.644#	0.000	-28.818
28	3	0.000	-23.676	0.000	0.000	0.000	57.866*
3	3	0.000	-136.819	0.000	0.000	0.000	-84.666#

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

Envelope = All Load Cases
and All Nodes

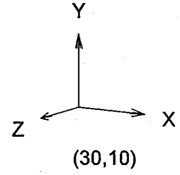
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
44	3	0.000	273.920*	0.000	0.000	0.000	0.000
7	1	0.000	29.760#	0.000	0.000	0.000	0.000
[illegible]	3	0.000	56.821	0.000	91.719*	0.000	0.000
9	3	0.000	56.821	0.000	-91.719#	0.000	0.000

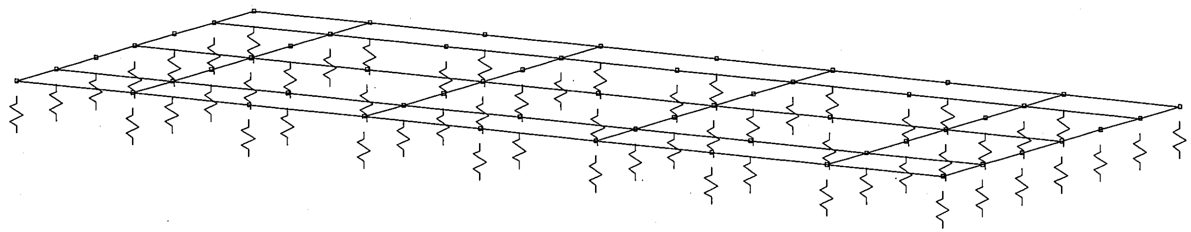
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

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
69	3	0.000	141.862*	0.000	-0.910	0.000	-64.221
96	3	0.000	-141.862#	0.000	0.910	0.000	-64.221
57	3	0.000	65.204	0.000	25.644*	0.000	-28.818
49	3	0.000	65.204	0.000	-25.644#	0.000	-28.818
28	3	0.000	-23.676	0.000	0.000	0.000	57.866*
3	3	0.000	-136.819	0.000	0.000	0.000	-84.666#

3 1.4D+1.7L



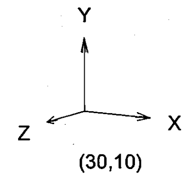


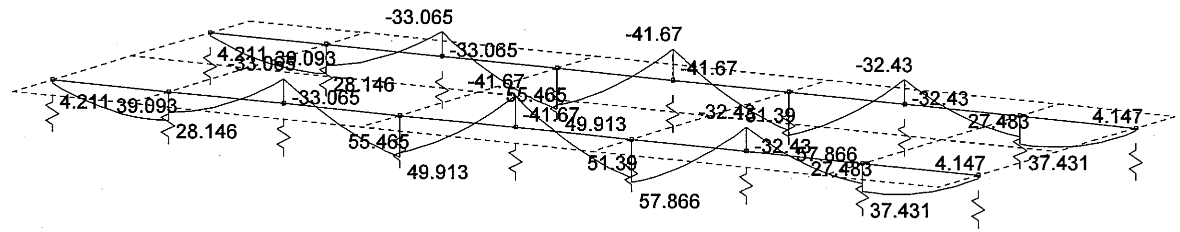
Sections:
1 600DP x 400WD
2 600DP x3900WD
3 600DPx2300WD
4 600DPx1950WD
5 600DPx1150WD

SLUDGE PUMP ROOM BASE SLAB MODEL

Job: SLDGSLB, Designer: FYP, Units: m,kN,MPa, Scale: 1:110, Axes: XY
Load: 1.9 Disp: None Moment: None Shear: None Axial: None
PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR SLUDGE PUMP ROOM 17 Oct 2011, 4:27 pm

3 1.4D+1.7L





Sections:
5 600DPx1150WD

BASE SLAB ANALYSIS MODEL

Job: SLDGSLB, Designer: FYP, Units: m,kN,MPa, Scale: 1:110, Axes: XY
Load: None Disp: None Moment: 10 Shear: None Axial: None
PORT MORESBY SEWERAGE TREATMENT UPGRADING PROJECT
BASE SLAB FOR SLUDGE PUMP ROOM 17 Oct 2011, 4:29 pm

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

REFERENCE: **Reinforced concrete slab - 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 =	58.50	kN/m	
Blockwall	:	Wb =	0.00	kN/m	
TOTAL		=	92.10	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 =	600	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	7300	mm	Overall depth of sect. D =	600	mm
Effective Length,Lef =	6900	mm	Area, A =	2.34E+06	mm^2
Neutral Axis, yc =	300.00	mm	Moment of Inertia, Ix-x =	7.02E+10	mm^4
Neutral Axis, xc =	2640.00	mm	Moment of Inertia, Iy-y =	2.97E+12	mm^4
Effective width, beff =	5280.00	mm	Gross Area, Ag =	2.34E+06	mm^2

3 **Design Parameters**

F'c =	40	MPa	Fsy =	410	MPa
m =	2400	kg/m^3	D =	600	mm
Ec =	31975.35051	MPa	Es =	200000	MPa
Gamma =	0.766				

4 **Initial Choice of Section**

Effective width = 5280 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.03522306	(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 = 151.383

defln/Lef = 0.004 k_1/k_2 = 17.27977882

d = 163 mm

take d = 483 mm Distance from d to extreme fibre of beam (do) = 117 mm

5 **Forces from an Elastic Analysis by Space Gass**

Mmax -ve	=	761.00	kNm	(left support)
Mmax +ve	=	761.00	kNm	(mid-span)
M -ve	=	761.00	kNm	(right support)
Vmax	=	940.00	kN	
V (other end)	=	940.00	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 : 2

take do = 117 mm d = 483

Estimate Area of main steel Ast = 5023.336069 mm^2

22 Y 20 Ast = 6911.503838 mm^2 **SPACING = 170**

p = 0.003669111

22 Y 20 Asc = 6911.503838 mm^2

pc = 0.003669111 < pd = 0.025409

p-pc = 0 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 97 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)]

| = 984 kNm > M* = 761.00 kNm | OK |

Determine - ve steel at right support :

M -ve = 761.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst = 483 mm

Estimate Area of main steel Ast = 5023.336069 mm^2

22 Y 20 Ast = 6911.503838 mm^2

p = 0.003669111

22 Y 20 Asc = 6911.503838 mm^2

pc = 0.003669111 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 97 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)]

| = 984 kNm > M* = 761.00 kNm | OK |

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 761 kNm

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

" " two " " code = 2 bar/s and soffit = 117 mm

what is the code ? 2 d = 483 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 5023.336069 mm^2

Bottom 22 Y 20 Ast = 6911.503838 mm^2

Top 22 Y 20 Asc = 6911.503838 mm^2

Assume N - A in flange

Effective width : beff = 5280 mm

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

= 0.0426646

hence dn = 21 < 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)]

| = 1212 kNm > M*+ve = 761.00 kNm | OK |

Summary of reinforcement requirement :

left support :	Ast	=	22	Y 20
	Asc	=	22	Y 20
Right support :	Ast	=	22	Y 20
	Asc	=	22	Y 20
midspan	Ast	=	22	Y 20
	Asc	=	22	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 = 2070 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 = 2070 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 = 690 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 1863 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 = 1883 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 1863 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 = 0.773034

therefore

Lsy =	255 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 = 2057 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 2037 mm (dist between bars)

factor = 44804.99308

Lsyt = 263.5587828 mm take fs/Fsy = M*/Phi Mu = 0.628049442

therefore

Lsy =	166 mm	≥ 25k1db 500
say =	500 mm	

9 PUNCHING SHEAR CHECK

Coloumn Dimension

V* = 80.4 kN b = 400 mm

Dist to centreline of bottom steel from soffit d = 400 mm

Critical shear perimeter, μ

dq = 85 mm μ = 2(b+dom) + 2(d+dom)

dom = 515 mm 3660 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 = 17,944 kN > V* = 80.4 kN OK

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	761.00 kNm	Es	=	200000 MPa
Ml	=	761.00 kNm	Ec	=	31975.35051 MPa
Mo	=	1522 kNm			
Ms	=	761.00 kNm			
Ig	=	7.02E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	5280 mm	A_{sc} =	6911.503838 mm^2
webwidth	b_w =	3900 mm	A_{st} =	6911.503838 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	483 mm
			d =	483 mm
			D =	600 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)

1950 dn^2 + 79548.8943 dn - 25129473.49) = 0

dn = 94.9412771 mm

d = 483 mm

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

= 7.62E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 887.967567 kNm

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

= 1.07E+11 mm^4

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

b_w =	3900 mm	A_{st} =	6911.503838 mm^2
d_{sc} =	97 mm	A_{sc} =	6911.503838 mm^2
d =	483 mm		

Ig = 7.02E+10 mm^4

At right end (arbitrary)

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

1950 dn^2 + 79548.8943 dn - 24403099.59) = 0

dn = 93.32 mm

d = 483 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.62E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 887.967567 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^: = 1.07E+11 mm^4

At left end (arbitrary)

b_w = **3900** mm

d_{sc} = **97** mm

A_{st} = **6911.503838** mm^2

A_{sc} = **6911.503838** mm^2

d = **483** mm

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

1950 dn^2 + 79548.8943 dn - 24403099.59) = 0

dn = 93.3150012 mm

d = 483 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.62E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 887.967567 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.07E+11 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.07E+11 mm^4

Midspan Deflection :

Lef = 6900 mm

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

= 0.88215567 mm

Total Deflection :

Long term factor	=	0.2
Short term factor	=	0.5
w - long term	=	92.15 kN/m
w - short term	=	92.225 kN/m
Defln,s,sus	=	0.881438 mm

At midspan : Ast = 6911.503838 mm^2

Asc = 6911.503838 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)

= 1.58731 mm

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

ALLOWABLE DEFLN = Lef / 500 = 13.8 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

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)......= 40
Yield stress of steel (MPa)..= 410

STEEL DETAILS

Line no	Depth to steel	Bar diameter	No of bars
1	64	20	3
2	250	20	2
3	336	20	3

OUTPUT

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

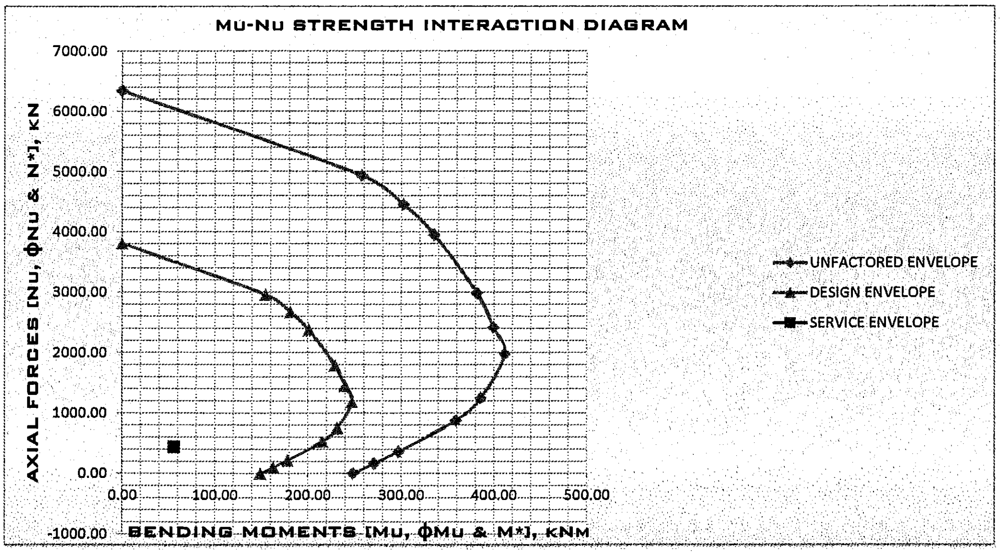
UNFACTORED ENVELOPE

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	56.00	440.00
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		



400 x 400 RC Column is OK.

Job name: BWALL1 Designer: FY Date: 17 Oct 2011, 4:58 pm Page: 1
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BASEMENT WALL FOR SLUDGE PUMP ROOM

MEMBER DATA (deg,m,kN/m,kNm/rad)
----------- (F=Fixed, R=Released) (*=Cable length)

Memb	Skew Angle	Dir Node	Dir Axis	Memb Type	Node A	Node B	Sec	Mat	Node A Fixity	Node B Fixity	Length
2	0.00			Norm	1	3	1	1	FFFFFF	FFFFFF	16.000
3	0.00			Norm	3	9	2	1	FFFFFF	FFFFFF	4.400

NODE RESTRAINTS (kN/m,kNm/rad)
--------------- (F=Fixed, R=Released, D=Deleted, S=Spring, *=General)

Node	Rest Code	X Axial Stiffness	Y Axial Stiffness	Z Axial Stiffness	X Rotation Stiffness	Y Rotation Stiffness	Z Rotation Stiffness
1	FFFFFF						
3	FFDDDR						

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

Sect	Section Name	Mark	Angle Type	Flipped	Source
1	Base Slab	S1	Not applicable	No	Standard shape
2	Basewall	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	5.0000E-01	2.8610E-02	4.1667E-02	1.0417E-02	INFINITE	INFINITE	0.00
2	4.0000E-01	1.5969E-02	3.3333E-02	5.3333E-03	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.500	1.000			
2	Rectangle	0.400	1.000			

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
3	3	1	G	1.600	-110.920	0.000	0.000
					0.000	0.000	0.000

MEMBER DISTRIBUTED FORCES (m,kN/m)

Load Case	Memb	Sub Load	Axes Sys	Start Position	Finish Position	X Start/ Finish	Y Start/ Finish	Z Start/ Finish
2	3	1	GI	0.000%	100.000%	-14.245	0.000	0.000
						-14.245	0.000	0.000
	3	2	GI	0.000%	100.000%	0.000	0.000	0.000
						-28.490	0.000	0.000

COMBINATION LOAD CASES

Load case 5: 0.9D+E

0.900 * Load case 1: D
1.000 * Load case 2: E

Load case 6: 0.9+1.7Q

0.900 * Load case 1: D
1.700 * Load case 3: Q

[illegible] CASE TITLES

Load Case	Title
1	D
2	E
3	Q
4	Qgw
5	0.9D+E
6	0.9+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
3	1	42.286*	0.000	0.000	0.000	0.000	0.000
2	1	0.000#	120.131	0.000	0.000	0.000	-384.421
2	5	0.000	138.282*	0.000	0.000	0.000	-506.852
3	6	38.058	-188.564#	0.000	0.000	0.000	301.702
2	2	0.000	30.164	0.000	0.000	0.000	321.747*
2	5	0.000	138.282	0.000	0.000	0.000	-506.852#

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

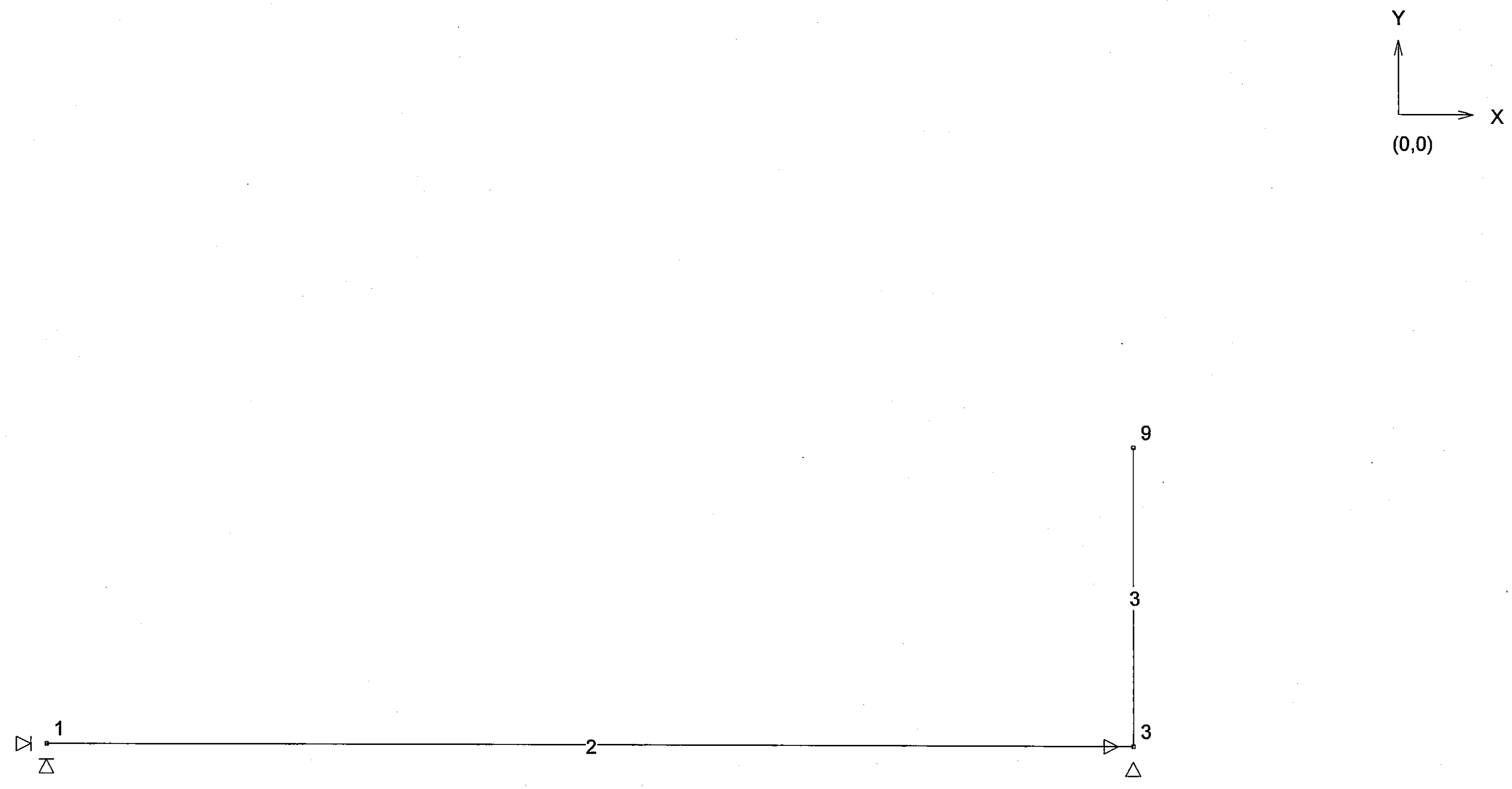
Envelope = All Load Cases
and All Nodes

Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
3	6	188.564*	74.644	0.000	0.000	0.000	0.000
1	1	0.000#	120.131	0.000	0.000	0.000	384.421
1	5	0.000	138.282*	0.000	0.000	0.000	506.852
3	2	125.356	-30.164#	0.000	0.000	0.000	0.000
1	5	0.000	138.282	0.000	0.000	0.000	506.852*
3	1	0.000	114.365	0.000	0.000	0.000	0.000#

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

Envelope = All Load Cases
and All Members
and All Sections

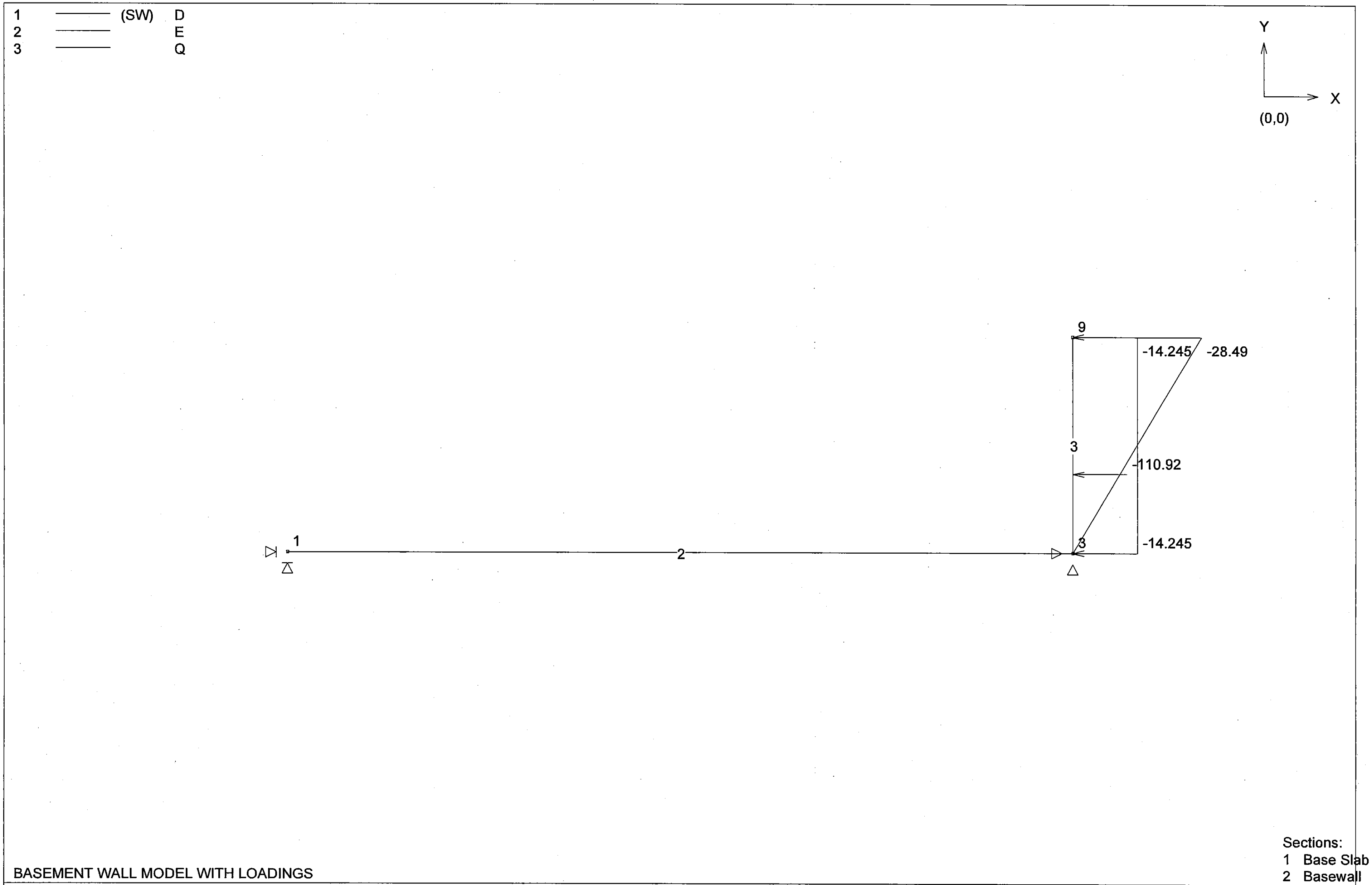
Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
3	1	42.286*	0.000	0.000	0.000	0.000	0.000
2	1	0.000#	120.131	0.000	0.000	0.000	-384.421
2	5	0.000	138.282*	0.000	0.000	0.000	-506.852
3	6	38.058	-188.564#	0.000	0.000	0.000	301.702
2	5	0.000	-0.109	0.000	0.000	0.000	377.454*
2	5	0.000	138.282	0.000	0.000	0.000	-506.852#



Sections:
1 Base Slab
2 Basewall

BASEMENT WALL MODEL

Job: BWALL1, Designer: FY, Units: m,kN,MPa, Scale: 1:110, Axes: XY
Load: None Disp: None Moment: None Shear: None Axial: None
POMSSUP
BASEMENT WALL FOR SLUDGE PUMP ROOM 17 Oct 2011, 4:55 pm



BASEMENT WALL MODEL WITH LOADINGS

Job: BWALL1, Designer: FY, Units: m,kN,MPa, Scale: 1:110, Axes: XY
Load: 1.2 Disp: None Moment: None Shear: None Axial: None
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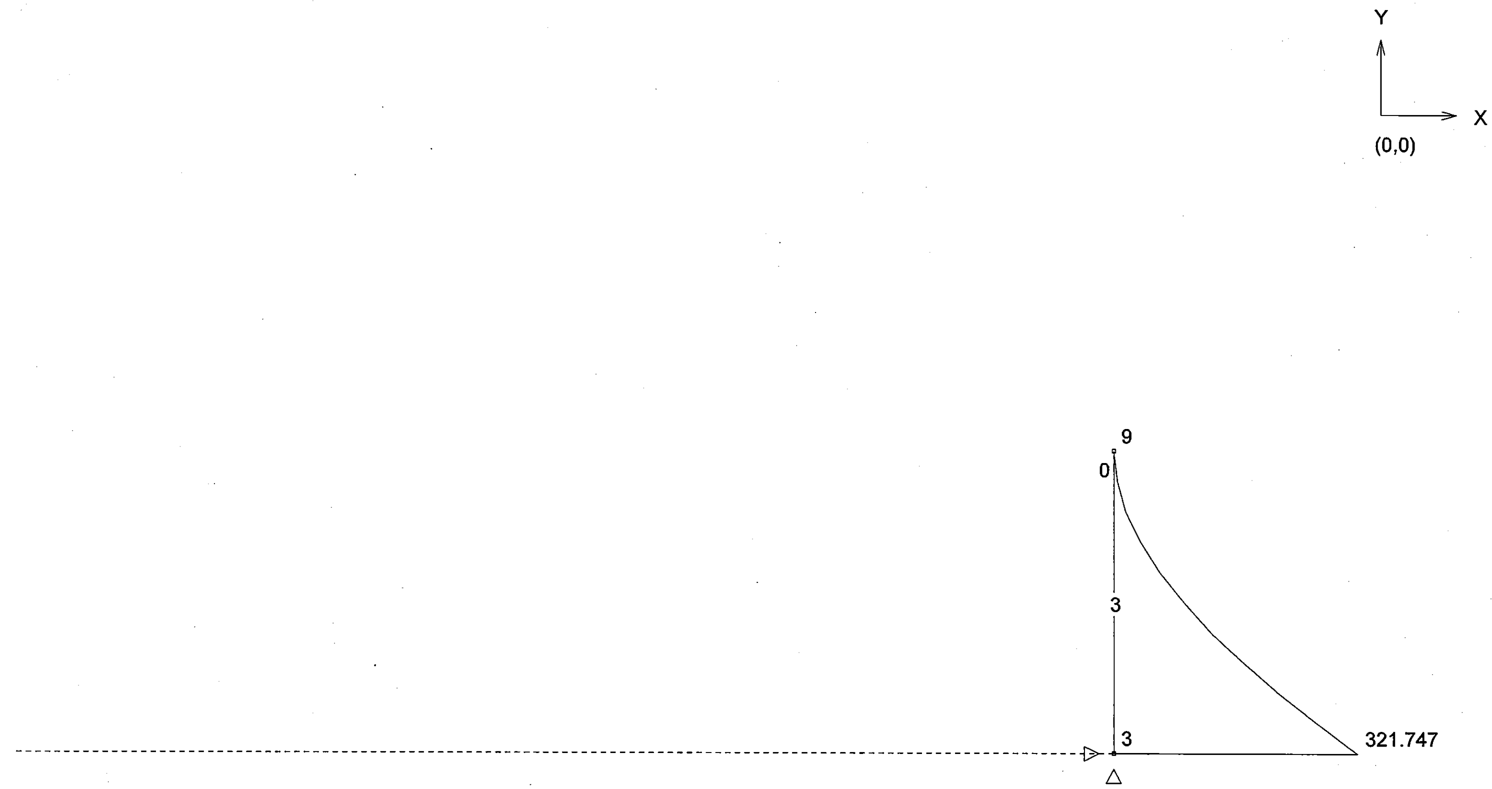
BASEMENT WALL FOR SLUDGE PUMP ROOM 17 Oct 2011, 4:56 pm

5 ——— 0.9D+E



Sections:
2 Basewall

BASEMENT WALL ANALYSIS MODEL

Job: BWALL1, Designer: FY, Units: m,kN,MPa, Scale: 1:110, Axes: XY
Load: None Disp: None Moment: 10 Shear: None Axial: None
POMSSUP
BASEMENT WALL FOR SLUDGE PUMP ROOM 17 Oct 2011, 4:57 pm

Basement Wall Design: Fixed Top By Slab

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	28.32	kN/m	(plus super)
Self wt	:	Wsw =	10.00	kN/m	
Blockwall	:	Wb =	16.74	kN/m	
TOTAL		=	55.06	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	1000	mm	D =	400	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	440	mm	Overall depth of Sect. D =	400	mm
Effective Length,Lef =	4000	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 =	900.00	mm	Moment of Inertia, Iy-y =	3.33E+10	mm^4
Effective width, beff =	1800.00	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 <u>INITIAL CHOICE OF SECTION</u>

Effective width =	1800 mm		clause 8.8.2	
Is the section RECT or T, L (ans : 1 or 2)		=	1	
k_1 =	0.045		(RECT sections)	1
=	0.02929333		(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 =	90.6374			
defln/Lef =	0.004	k_1/k_2 =	17.27977882	
d =	114 mm			
take d =	293 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	=	69.00	kNm	(left support)
Mmax +ve	=	236.00	kNm	(mid-span)
M -ve	=	0.00	kNm	(right support)
Vmax	=	104.00	kN	
V (other end)	=	84.00	kN	
N*	=	19.00	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 = **107** mm d = 293

Estimate Area of main steel Ast = 750.8205979 mm^2

6 **Y 20** Ast = **1884.955592** mm^2 OUTSIDE

p = 0.006433296

6 **Y 20** Asc = **1884.955592** mm^2 INSIDE

pc = 0.006433296 < pd = 0.025409

p-pc = 0 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 97 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)]

=	136	kNm	>	M* =	69.00 kNm	**OK**

Determine -ve steel at right support :

M -ve = 0.00 kNm

Number of reinforcement layers 1 or 2 : **1**

take do = **107** mm dst = 293 mm

Estimate Area of main steel Ast = 0 mm^2

6 **Y 20** Ast = **1884.955592** mm^2 OUTSIDE

p = 0.006433296

6 **Y 20** Asc = **1884.955592** mm^2 INSIDE

pc = 0.006433296 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 97 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)]

=	136	kNm	>	M* =	0.00 kNm	**OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 236 kNm

Steel in one layer then code = 1

" " two " " code = 2

what is the code ? 2

take dist between centroid of bar/s and soffit = **117** mm

d = 283 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 2658.76697 mm^2

6 **Y 20** Ast = 1884.955592 mm^2 OUTSIDE

6 **Y 20** Asc = 1884.955592 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1800 mm

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

= 0.05825301

hence dn = 16 < 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)]

=	192 kNm	<	M*+ve =	236.00 kNm	**NO GOOD**

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 20
	Asc	=	6	Y 20
Right support :	Ast	=	6	Y 20
	Asc	=	6	Y 20
midspan	Ast	=	6	Y 20
	Asc	=	6	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 = 1200 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 = 1200 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 = 400 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 1

a_b = 826 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 = 846 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 826 mm (dist between bars)

factor = 56006.24135

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

= 329.4485 mm

take fs/Fsy = M*/Phi Mu = 0.506134021

therefore

Lsy =	167 mm
say =	500 mm

≥ 25k1db 625

For Bottom Steel :

Cover, c = 75 mm d_b = 20 mm

# of lines of bars = 1

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

if ab < 2c C = a_b + d_b = 1194 mm

k1 = 1 A_b = 314.1592654 mm^2

k2 = 2.2 a_b = 1174 mm (dist between bars)

factor = 44804.99308

Lsyt = 263.5588 mm take fs/Fsy = M*/Phi Mu = 1.22630163

therefore

Lsy =	323 mm
say =	400 mm

≥ 25k1db 500

9 SHEAR CHECK

V* = 104.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 293 mm dq = 93 mm

do = 307 mm

V* = 88 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

= 167.83 kN

phi Vumax = 1719.2 kN > V* No web crushing

phi Vumin = 297 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 = 731.707317

Asv.min = 256.097561 mm^2

Asv.max = 5282 mm^2

Asv = 2 x area of stirrups = 400 mm^2 2 Ties

theta = 30.42946 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 525.48 kN

Phi Vu = 368 kN > V* = 87.79417 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**300**	**centres**

Other Support :

V* = 84.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 293 mm dq = 93 mm

do = 307 mm

V* = 67.79 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

= 167.8313 kN

phi Vumax = 1719.2 kN > V* No web crushing

phi Vumin = 297 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 = 731.707317

Asv.min = 256.097561 mm^2

Asv.max = 5282.21431 mm^2

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

theta = 31.02635 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 658.2886 kN

Phi Vu = 460.802 kN > V* = 67.79 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**300 centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	69.00 kNm	Es	=	200000 MPa
Ml	=	0.00 kNm	Ec	=	31975.35051 MPa
Mo	=	270.5 kNm			
Ms	=	236.00 kNm			
Ig	=	5.33E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	**b_{eff} =**	1800 mm	**A_{sc} =**		1884.955592 mm^2
webwidth	**b_w =**	1000 mm	**A_{st} =**		1884.955592 mm^2
Flange thickness	**t_{ff} =**	0 mm	**d_{sc} =**		117 mm
			d_{st} =		283 mm
			d =		293 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 + 21695.153 dn - 4613382.455) = 0

dn = 76.780451 mm

d = 283 mm

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

= 6.52E+08 mm4

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

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

= 1.02E+09 mm^4

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

b_w =	1000 mm	**A_{st} =**	1884.955592 mm^2
d_{sc} =	97 mm	**A_{sc} =**	1884.955592 mm^2
d =	293 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 + 21695.153 dn - 4415280.481) = 0

dn = 74.75 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.01E+08 mm^4

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

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.53E+10 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **97** mm

A_{st} = **1884.956** mm^2

A_{sc} = **1884.956** mm^2

d = **293** mm

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

500 dn^2 + 21695.153 dn - 4415280.481) = 0

dn = 74.7477867 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.01E+08 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 = 4.34E+09 mm^4

Midspan Deflection :

Lef = 4000 mm

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

=	2.75217054 mm

Total Deflection :

Long term factor	=	0.2		
Short term factor	=	0.5		
w - long term	=	55.11 kN/m		
w - short term	=	55.185 kN/m		
Defln,s,sus	=	2.748430163 mm		
At midspan :			Ast =	$1884.95559\ mm^2$
			Asc =	$1884.95559\ mm^2$

Asc/Ast =		1	
kcs	=	2 - (1.2 x Asc/Ast)	>= 0.8
	=	0.8	

therefore

Total Defln	=	Dfln,s + (kcs x Defln,s,sus)
	=	4.95091 mm

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

ALLOWABLE DEFLN	=	Lef / 500 =	8 mm
			> 0.9 x Total Defln

Basement Wall Design: Free Top

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	**28.32** kN/m	(plus super)
Self wt	:	Wsw =	**10.00** kN/m	
Blockwall	:	Wb =	**16.74** kN/m	
TOTAL		=	55.06 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	**1000** mm	D =	**400** mm
tff	**0** mm	Cover =	**75** mm
Actual length, L =	**440** mm	Overall depth of Sect. D =	400 mm
Effective Length,Lef =	**4000** 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 =	900.00 mm	Moment of Inertia, Iy-y =	3.33E+10 mm^4
Effective width, beff =	1800.00 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 =		1800 mm	clause 8.8.2	
Is the section RECT or T , L (ans : 1 or 2)	=			**1**
k_1	=	0.045	(RECT sections)	**1**
	=	0.02929333	(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	=	90.6374		
defln/Lef	=	**0.004** k_1/k_2 =	17.27977882	
d =		114 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	=	**321.70** kNm	(left support)
Mmax +ve	=	**144.00** kNm	(mid-span)
M -ve	=	**0.00** kNm	(right support)
Vmax	=	**125.00** kN	
V (other end)	=	**0.00** kN	
N*	=	**38.00** 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 =		**111** mm	d =	289		
Estimate Area of main steel		Ast =	3549.015747 mm^2			
10	Y 24	Ast =	**4523.893421** mm^2	OUTSIDE		
		p =	0.01565361			
9	Y 24	Asc =	**4071.504079** mm^2	INSIDE		
		pc =	0.014088249	<	pd =	0.025409
		p-pc =	0.001565361	pd=0.4*0.85*μ*f'c/fsy		

Check M* <= phiMu

ku =	0.024643	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)]

=	333	kNm	>	M* =	321.70 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

10 Y 24 Ast = 4523.893421 mm^2 OUTSIDE

p = 0.01565361

9 Y 24 Asc = 4071.504079 mm^2 INSIDE

pc = 0.014088249 < pd = 0.025409

p-pc = 0.001565361

Check M* <= phiMu

ku = 0.024643 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)]

=	333	kNm	>	M* =	0.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 144 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) = 1657.43853 mm^2

10 Y 24 Ast = 4523.893421 mm^2 OUTSIDE

9 Y 24 Asc = 4071.504079 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1800 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.14283553

hence dn = 40 < 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)]

=	437 kNm	>	M*+ve =	144.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	10	Y 24
	Asc	=	9	Y 24
Right support :	Ast	=	10	Y 24
	Asc	=	9	Y 24
midspan	Ast	=	10	Y 24
	Asc	=	9	Y 24

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 = 1200 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 = 1200 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 = 400 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 = 0.965511864

therefore

Lsy =	448 mm
say =	500 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 = 0.329440581

therefore

Lsy =	122 mm
say =	400 mm

≥ 25k1db 600

9 SHEAR CHECK

V* = 125.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 289 mm dq = 93 mm

do = 307 mm

V* = 109 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

= 224.64 kN

phi Vumax = 1719.2 kN > V* No web crushing

phi Vumin = 354 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 = 731.707317

Asv.min = 256.097561 mm^2

Asv.max = 5089 mm^2

Asv = 2 x area of stirrups = 400 mm^2 2 Ties

theta = 30.44665 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 606.43 kN

Phi Vu = 425 kN > V* = 109.01541 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**300**	**centres**

Other Support :

V* = 0.00 kN

Dist to centreline of bottom steel from soffit

critical location :	d	=	289	mm	dq	=	93	mm
					do	=	307	mm

V* = -15.98 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

= 224.638 kN

phi Vumax	=	1719.2 kN	>	V*		No web crushing
phi Vumin	=	354 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 = 731.707317

Asv.min = 256.097561 mm^2

Asv.max = 5088.79458 mm^2

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

theta = 31.06742 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 738.7625 kN

Phi Vu = 517.1338 kN > V* = -15.98 kN

hence

PROVIDE : 1 Y12 LIGS @ 300 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	321.70 kNm	Es	=	200000 MPa
Ml	=	0.00 kNm	Ec	=	31975.35051 MPa
Mo	=	304.85 kNm			
Ms	=	144.00 kNm			
Ig	=	5.33E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	$\mathbf{b_{eff}}$ =	1800 mm	$\mathbf{A_{sc}}$ =	4071.504079 mm^2
webwidth	$\mathbf{b_w}$ =	1000 mm	$\mathbf{A_{st}}$ =	4523.893421 mm^2
Flange thickness	$\mathbf{t_{ff}}$ =	0 mm	$\mathbf{d_{sc}}$ =	123 mm
			$\mathbf{d_{st}}$ =	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 + 49691.1435 dn - 10809168.28) = 0

dn = 105.510488 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

= 1.22E+09 mm4

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

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

= 2.65E+09 mm^4

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

$\mathbf{b_w}$ =	1000 mm	$\mathbf{A_{st}}$ =	4523.893421 mm^2
$\mathbf{d_{sc}}$ =	99 mm	$\mathbf{A_{sc}}$ =	4071.504079 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 + 49691.1435 dn - 10295687.96) = 0

dn = 102.17 mm

d = 289 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.34E+09 mm^4

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

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.47E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **99** mm

A_{st} = **4523.8934** mm^2

A_{sc} = **4071.5041** mm^2

d = **289** mm

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

500 dn^2 + 49691.1435 dn - 10295687.96) = 0

dn = 102.165979 mm

d = 289 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.34E+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 = 1.69E+09 mm^4

Midspan Deflection :

Lef = 4000 mm

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

= 3.44549335 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 55.11 kN/m

w - short term = 55.185 kN/m

Defln,s,sus = 3.440810701 mm

At midspan : Ast = 4523.89342 mm^2

Asc = 4071.50408 mm^2

Asc/Ast = 0.9

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

= 0.92

therefore

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

= 6.61104 mm

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

ALLOWABLE DEFLN = Lef / 500 = 8 mm

> 0.9 x Total Defln

SUMMARY

A. Roof

Purlin: 50 WD x 100 DP HWD @ 600 cts
Rafter: 75 WD x 150 DP HWD @ 900 cts

B. Roof Beam: B1

350 DP x 350 WD RC Beam

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

C. Block Wall

200 thk masonry blocks

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

D. Ground Floor Slab

200 thk RC two-way slab

Reinforcements:	Continuous Edge:	Y16 - 200 cts	Top & Bottom Each Way
	Discontinuous Edge:	Y16 - 200 cts	Top & Bottom Each Way
	Mid-span:	Y16 - 200 cts	Top & Bottom Each Way
Temp. & shrinkage (T&S) Reinforcement:		Y12 - 200 cts	Top & Bottom Each Way

E. Ground Floor Beam:

500 DP x 400 WD RC Beam

Reinforcements:	5 - Y24	Tensile Steel
	3 - Y20	Compressive Steel
	Y12 - 200 cts	Ligatures

F. Base Slab

600 thk RC Slab

Reinforcement: Y20 bars @ 200 cts Top & Bottom Each Way
T & S Y20 bars @ 200 cts Top & Bottom Each Way

G. RC Column:

400 x 400 SQ RC Col:

Main Vert. Bars: 8 - Y20
Y12 - 200 cts Ligatures

H. Basement Wall 1.

400 thk RC Wall

Reinforcements:	Vertical:	Y20 - 200 cts	Outside
		Y20 - 200 cts	Inside
	Horizontal	Y16 - 200 cts	

I : UV Disinfection Room (incl. Treated Water Tank)

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	Basement Loads		
A	Dead Loads		
1	Wall wt	10	kPa
2	Slab wt =	10	kPa
B	Mechanical Loads		
	UV Weir	4	kN
	UV Module	5	kN
	Utility TE Water supply Unit	2	kN
C	FLUID PRESSURE LOADS		
1	Hydrostatic Force	21.582	kN/m^2/m
D	Live Loads		
1	Surcharge Load	10	kPa
2	Ground Level Loads		
A	Dead Loads		
1	Slab wt	5	kPa
2	Services	1	kPa
3	Superimposed DL	1	kPa
		7.0	kPa
B	Mechanical Loads		
	Control Unit for UV	12.0	kN
		1.0	kN
	Hoist Block	5.0	kN
	Utility TE Water Supply Unit	15.0	kN
	Auto Strainer	15.0	kN
	Disinfection Inlet Gate	30.0	kN
	Chlorine Solution Tank	90.0	kN
	Chlorine Solution Dosing Charger	2.0	kN
	Chlorine Solution Storage Tank	2.0	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 =	5.9	m
	Width of Bldg =	13.8	m
	Length of Bldg =	7	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		
	=	29.08	m/s
3	Dynamic Pressure, qz		
	q_z = $0.6V_z$^2 x10^-3 ... Dynamic Pressure ...	0.51	kPa
4	Design Pressure, P_z		
	p_z = Cpq_z ... Design pressure...		
	h/d = Table B2.2 **α < 10° h/d ≥ 0.5**	0.43	
	h/b =	0.84	
	C_p = ... Slope D ...	-0.9	
	C_p = ... Slope E ...	0	
	p_z = ... Slope D ...	-0.46	kPa
	= ... Slope E ...	0.00	kPa
5	UDL on roof truss		

Rafter Spacing No.	Rafter 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	-1.553	**-1.550**	**-0.088**	0.000	**0.000**	**0.000**
3	5	2	5	-2.283	**-2.280**	**-0.130**	0.000	**0.000**	**0.000**
4	5	3	5	-2.283	**-2.280**	**-0.130**	0.000	**0.000**	**0.000**
5	5	4	5	-2.283	**-2.280**	**-0.130**	0.000	**0.000**	**0.000**
6	0.9	5	3.4	-1.553	**-1.550**	**-0.088**	0.000	**0.000**	**0.000**

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 =		5.5	m
Width of Bldg =		13.8	m
Length of Bldg =		7	m
Roof Pitch, α =		3	Degrees
Wind Angle, θ =		0	Degrees

2 Pressure Coefficients, Cp

h/d	=	Table B1.1		**0.40**
h/b	=			**0.79**
d/b	=			**1.97**
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)
5.9	1.0326	28.913	0.502
		0	0.000

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	13.8	1	6.9	2.769	**-1.730**

Vertical Face of Wall at 90° to Wind Direction	**Spacing, (m)**	**Column No.**	**Tributary Area (m)**	**Side, (kN/m)**
				GFL
1	13.8	1	6.9	**-2.076**

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.
Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.40	m
Height of Wall (H) =	3.70	m
Embedded Height of Wall (He) =	3.70	m
(Heel Depth) Thickness of Base (tb) =	0.40	m
Width of Base (wb) =	1.00	m
(Heel)/ Length of base (Lb) =	15.90	m
Toe Depth (td) =	0.40	m
Toe length (tl) =	0.00	m
Height of Ground Water (Hw) =	0.00	m
Slope of ground surface behind the wall (β) =	0	degrees
Slope of back face of the wall (α) =	90	degrees
Angle of wall friction (δ') =	0	degrees

MATERIAL PROPERTIES

f'c =	40	MPa
fsy =	410	MPa
Concrete Density =	25	kN/m^3
Cover (c) =	75	mm
Max. allowable crack width (W) =	0.30	mm

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (ϕ') =	42	degrees	
Design cohesion of retained mat'l (C) =	0	kN/m^3	**(Granular backfill)**
Density of retained mat'l (γ) =	17	kN/m^3	
Submerged Density of retained mat'l (γs) =	17	kN/m^3	
Design angle of int'l friction of base mat'l (ϕb) =	45	degree	
Design cohesion of base mat'l (Cb) =	17	kN/m^3	**(Cohesionless soil)**
Density of base mat'l (γb) =	17	kN/m^3	
Allowable gross ground bearing pressure (GBP) =	450	kN/m^2	

LOADINGS (Unfactored)

Surcharge load – live (Q) =	10	kN/m^2
Surcharge load – dead (G) =		kN/m^2

LATERAL FORCES

K_O = 0.33 K_A = (1-SIN ϕ) =

Earth Pressure coefficients, Ka & Kp;

	Active, Ka	Passive, Kp
Rankines	0.20	5.04
Coulomb	0.20	5.04

PE = 0.5γKaH^2 (B=1.0 m)

Force (kN/m)		Lever arm (m)	
Pa (Fe) =	23.07	LE =	1.233
PS(G) =	7.33	LS =	1.85
PS(Q) =	0.00	LS =	1.85
Total, Pa =	30.40	La =	1.38

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	23.07	1.23	28.45
PS =	7.33	1.85	13.57
Total	30.40	1.38	42.02

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	37	15.90	588.3
Wall wt =	33.00	15.90	524.7
Base wt =	159.00	7.95	1264.1
Total =	229.00	7.81	2377.1

CHECK GROUND BEARING FAILURE

∑MO =	42.02 kNm/m	
∑MR =	2377.1 kNm/m	
∑F =	229.00 kN/m	
ECCENTRICITY FROM BASE CENTRE, e =		
e = B/2 - [∑MR - ∑MO]/∑F	2.25 m	**OK, e < B/6**

Thus; **Pressure on Soil at Toe & Heel**

qtoe = ∑F/B [1+6e/B] =	0.91 kPa
qheel = ∑F/B [1-6e/B] =	0.07 kPa
Maximum Bearing Pressure =	0.91 kPa

< Allowable Bearing Pressure, OK

Earthquake Earth Pressure

REFERENCE:

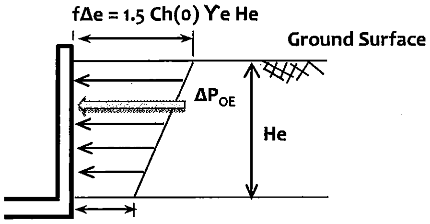
NZS 3106 : 2009 - DESIGN OF CONCRETE STRUCTURES FOR THE STORAGE OF LIQUIDS
SEISMIC DESIGN OF STORAGE TANKS "Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering" : 1986

Seismic Design Data

Location = Port Moresby
Category (pg 15) = C
Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15
p = 0.005
Tank Contents = Water: Portable Supply
probability factor; Fig. C2.7; pg 51 (Ap) = 1.25
geographical coefficient representing regional seismicity; Fig. C2.1; pg 47
β = 0.30
peak horizontal acceleration coefficient; pg 51 (α) = 0.35
horizontal force coefficient Ch(T) = α β Ah(T) Ap = 0.13 $A_h(T)$
normalized horizontal acceleration response = Ah(T)

4 Foundation Rigid/Fixed Foundation on Weathered Mudstone
Unit weight of engineering backfill soil (ϒe) = 17.00 kN/m^3

5 Earthquake Earth Pressure: NZS 3106, Appendix A, Cl. A2.6



Increment in at-rest pressure due to earthquake
He = embedded depth = 3.7 m
Ah(o) = 1
$C_h(o) = \alpha\ \beta\ A_h(o)\ A_p$, horizontal force coefficient = 0.131
fΔe = 1.5 Ch(o) ϒe He 12 kPa
fΔe = 0.5 Ch(o) ϒe He 4 kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	4	kN/m	UDL
Eu2 =	8	kN/m	Triangular

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
$\mu =$ 0.4 MPa
$Es =$ 180 MPa
$\pi =$ 3.141593

$D =$ 0.4 m
$F'c =$ 40 MPa
$\rho =$ 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	0.30	1.60E-03	31.9754	0.18	0.4	0.10356	20.00	0.8434	16.8686	No	Yes	Use Winkler	345201
2	0.40	2.13E-03	31.9754	0.18	0.4	0.11128	20.00	0.7991	15.9827	No	Yes	Use Winkler	278199
3	3.00	1.60E-02	31.9754	0.18	0.4	0.18412	15.00	0.5477	8.2152	No	Yes	Use Winkler	61372
4	1.50	8.00E-03	31.9754	0.18	0.4	0.15483	15.00	0.6237	9.3555	No	Yes	Use Winkler	103223

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 = 1200 mm Purlin Spacing = 600 mm CTS

75 100 HWD

From Analysis

M*max	=	0.05	kNm
V*max	=	0.18	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 0.432 MPa

Working Shear Stress, fs = 3V/2BD

= 0.036 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.032809 < 1 OK

fs = 0.036 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 = 0.061714 mm Def. by DL

Allowable Defln = L/300 4 mm > 0.061714 = Defln.DL OK

Defln.LL = 0.061714 mm Def. by LL

Allowable Defln = L/360 3.33333333 mm > 0.061714 = 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)

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

Rafter Design

Loads

DL = 0.36 kN/m LL = 0.3 kN/m

Combination: DL + LL = 0.66 kN/m

Size B (mm) D (mm) Rafter Span = 7 m

100 150 HWD

From Space Gass Analysis

M*max	=	4.04	kNm
V*max	=	2.31	kN

Permissible Stresses

a. In Bending

Fb = 13.167 MPa

b. Fs = k1k4k5k6Fs'

= 0.945 MPa

Working Bending Stress, fb = Mx/Zx

= 10.78 MPa

Working Shear Stress, fs = 3V/2BD

= 0.231 MPa

CHECK!

From equation 3.10; fbx/Fbx <= 1

0.818713 < 1 OK

fs = 0.231 MPa < Fs = 0.945 MPa OK

Deflection Check!

DL = 0.36 kN/m

LL = 0.3 kN/m

Defln = $5WL^4/384EI$

Defln.DL = 3.81E-11 mm Def. by DL

Allowable Defln = L/300 0.023333 mm > 3.81E-11 = Defln.DL OK

Defln.LL = 3.18E-11 mm Def. by LL

Allowable Defln = L/360 0.019444 mm > 3.18E-11 = Defln.LL 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 =	**4.00** kN/m	
Blockwall	:	Wb =	**0.00** kN/m	
TOTAL		=	4.60 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 =	**400** mm
tff	**0** mm	Cover =	**65** mm
Actual length, L =	**7000** mm	Overall depth of Sect. D =	400 mm
Effective Length,Lef =	**6600** mm	Area, A =	1.60E+05 mm^2
Neutral Axis, yc =	200.00 mm	Moment of Inertia, Ix-x =	2.13E+09 mm^4
Neutral Axis, xc =	860.00 mm	Moment of Inertia, Iy-y =	2.13E+09 mm^4
Effective width, beff =	1720.00 mm	Gross Area, Ag =	1.60E+05 mm^2

3 Design Parameters

F'c =	**32** MPa	D =	400 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 = 1720 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	**1**
	=	0.02052538	(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	=	7.883		
defln/Lef	=	**0.004**	k_1/k_2 = 7.881643621	
d =		114 mm		
take d =		**293** mm	Distance from d to extreme fibre of beam (do) =	**107** mm

5 Forces from an Elastic Analysis by Space Gass :

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 = 293

Estimate Area of main steel		Ast =	435.2583177 mm^2		
4	**Y 20**	Ast =	**1256.637061** mm^2		
	16	p =	0.010722159		
3	**Y 16**	Asc =	**603.1857895** mm^2		
		pc =	0.005146636	< pd =	0.021813

p-pc = 0.005575523 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0.102242 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)]

=	114	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 = 293 mm

Estimate Area of main steel Ast = 435.2583177 mm^2

4 Y 20 Ast = 1256.637061 mm^2

p = 0.010722159

3 Y 16 Asc = 603.1857895 mm^2

pc = 0.005146636 < pd = 0.021813

p-pc = 0.005575523

Check M* <= phiMu

ku = 0.102242 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)]

=	114	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 take dist between centroid of

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

what is the code ? 2 d = 293 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 282.917906 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 = 1720 mm

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

= 0.04572534

hence dn = 13 < 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)]

=	133 kNm	>	M*+ve =	26.00 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 = 1980 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 = 1980 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 = 660 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 = 0.351262341

therefore $\boxed{\text{Lsy = 96 mm; say = 500 mm}}$ ≥ 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 = 0.195033363

therefore $\boxed{\text{Lsy = 65 mm; say = 400 mm}}$ ≥ 25k1db 400

9 SHEAR CHECK

V* = 29.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 293 mm dq = 85 mm

do = 315 mm

V* = 28 kN beta 1 = 1.2425 (but not less than 1.1)

hence = 1.2425

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

= 74.92 kN

phi Vumax = 564.48 kN > V* No web crushing

phi Vumin = 128 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 = 292.682927

Asv.min = 102.439024 mm^2

Asv.max = 1625 mm^2

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

theta = 31.15853 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 263.67 kN

Phi Vu = 185 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 = 293 mm — dq = 85 mm

do = 315 mm

V* = 27.58 kN — beta 1 = 1.2425 (but not less than 1.1)

hence = 1.2425

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

= 74.91925 kN

phi Vumax = 564.48 kN > V* No web crushing

phi Vumin = 128 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 = 292.682927

Asv.min = 102.439024 mm^2

Asv.max = 1624.55865 mm^2

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

theta = 32.24254 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 332.2519 kN

Phi Vu = 232.5763 kN > V* = 27.58 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 = 2.13E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	$\mathbf{b_{eff}}$ =	1720 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
			$\mathbf{d_{st}}$ =	293 mm
			d =	293 mm
			D =	400 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 - 2961621.664) = 0

dn = 94.5699517 mm

d = 293 mm

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

= 4.59E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 36.2038672 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	4.98E+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 = 293 mm

Ig = 2.13E+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 - 2882092.715) = 0

dn = 92.98 mm
d = 293 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.59E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 36.2038672 kNm
Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.70E+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 = **293** mm

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

200 dn^2 + 12402.7399 dn - 2882092.715) = 0

dn = 92.9765698 mm
d = 293 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.59E+08 mm^4
Mcr = 0.6sqrtF'c x bD^2/6 = 36.2038672 kNm
Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.70E+09 mm^4

Iav	=	[Im + (IL + Ir)/2]/2	=	3.34E+09 mm^4

Midspan Deflection :

Lef = 6600 mm
Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

=	0.85500021 mm

Total Deflection :

Long term factor	=	0.2		
Short term factor	=	0.5		
w - long term	=	4.65 kN/m		
w - short term	=	4.725 kN/m		
Defln,s,sus	=	0.841428777 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)
	=	1.93338 mm

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

ALLOWABLE DEFLN	=	Lef / 250 =	26.4 mm
			> 0.9 x Total Defln
		Lef / 500 =	13.2 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° ≤ 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)		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		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) (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	0.0017	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

7 Cover Slab

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

Lx / Ds =< 105 [(Lx/Ly)^2 /wk]$^{0.33}$

= $\frac{5000}{Ds}$ =< 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 = 220 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 :

		Ly/Lx =	1.00	Design moments in central regions					
				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 = 182 mm f = 0.8
Long span d = 166 mm

Short span
f bd^2 Fsy p = -10.9E+9
f bd^2 Fsy p (- 0.6pFsy / F'c) = 83.5E+9
Min. As = 0.8/fsy *b * d
= 355.122 mm^2/m

Long span
f bd^2 Fsy p = -9.0E+9
f bd^2 Fsy p (- 0.6pFsy / F'c) = 9.2E+9
Min. As = 0.8/fsy *b * d
= 323.9024 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	0.0016	355.12	16	200.96	565.890	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	0.0021	384.44	16	200.96	522.737	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	0.0008	355.12	16	200.96	565.890	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	0.0019	355.12	16	200.96	565.890	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	0.0025	455.78	16	200.96	440.916	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	0.0009	355.12	16	200.96	565.890	300

Shrinkage and Temperature:

pmin = 0.0018 Table 16.1

As = pmin b Ds reinf. y 16

= 327.600 mm^2/m As = 200.96 mm^2

spacing = 613.431 mm

say 200 mm

USE	Y	16	at	200	CTS

Check for Shear:

b1 = 1.4 - d/2000 f = 0.7

= 1.309 d = 182 mm

p = 0.001951

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

= 94.601 kN/m

f Vuc = 66.221 kN/m > Ok

Hence shear failure is not critical

Basement Wall Design

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	**28.32**	kN/m	(plus super)
Self wt	:	Wsw =	**10.00**	kN/m	
Blockwall	:	Wb =	**16.74**	kN/m	
TOTAL		=	55.06	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	**1000**	mm	D =	**400**	mm
tff	**0**	mm	Cover =	**75**	mm
Actual length, L =	**440**	mm	Overall depth of Sect. D =	400	mm
Effective Length,Lef =	**4000**	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 =	900.00	mm	Moment of Inertia, Iy-y =	3.33E+10	mm^4
Effective width, beff =	1800.00	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 <u>INITIAL CHOICE OF SECTION</u>

Effective width = 1800 mm clause 8.8.2
Is the section RECT or T , L (ans : 1 or 2) = **1**

k_1	=	0.045	(RECT sections)	**1**
	=	0.02929333	(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	=	90.6374		
defln/Lef	=	**0.004**	k_1/k_2 = 17.27977882	

d = 114 mm
take d = **293** 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	=	**135.20**	kNm	(left support)
Mmax +ve	=	**18.30**	kNm	(mid-span)
M -ve	=	**0.00**	kNm	(right support)
Vmax	=	**81.30**	kN	
V (other end)	=	**0.00**	kN	
N*	=	**62.30**	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine -ve steel required :</u> (At arbitrary left support) Spacing 300 mm
Number of reinforcement layers 1 or 2 : 1
take do = **107** mm d = 293
Estimate Area of main steel Ast = 1471.173114 mm^2
6 **Y 20** Ast = **1884.955592** mm^2 OUTSIDE

		p =	0.006433296				
6	Y 20	Asc =	1884.955592 mm^2		INSIDE		
		pc =	0.006433296	<	pd	=	0.025409
		p-pc =	0	pd=0.4*0.85*µ*f'c/fsy			

Check M* <= phiMu

ku = 0 dsc = 97 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)]

=	136	kNm	>	M* =	135.20 kNm	OK

Determine -ve steel at right support :

M -ve = 0.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 107 mm dst = 293 mm

Estimate Area of main steel		Ast =	0 mm^2				
6	Y 20	Ast =	1884.955592 mm^2		OUTSIDE		
		p =	0.006433296				
6	Y 20	Asc =	1884.955592 mm^2		INSIDE		
		pc =	0.006433296	<	pd =		0.025409
		p-pc =	0				

Check M* <= phiMu

ku = 0 dsc = 97 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)]

=	136	kNm	>	M* =	0.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 18.3 kNm

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

" " two " " code = 2 bar/s and soffit = 117 mm

what is the code ? 2 d = 283 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 206.1671 mm^2

6	Y 20	Ast =	1884.955592 mm^2	OUTSIDE
6	Y 20	Asc =	1884.955592 mm^2	INSIDE

Assume N - A in flange

Effective width : beff = 1800 mm

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

= 0.05825301

hence dn = 16 < 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)]

=	192 kNm	>	M*+ve =	18.30 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 20
	Asc	=	6	Y 20
Right support :	Ast	=	6	Y 20
	Asc	=	6	Y 20
midspan	Ast	=	6	Y 20
	Asc	=	6	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 = 1200 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 = 1200 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 = 400 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm
d_b = 20 mm   # of lines of bars = 1
a_b = 826 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 $C = a_b + d_b$ = 846 mm
coaxial with and surrounding a bar
k1 = 1.25 A_b = 314.1592654 mm^2
k2 = 2.2 a_b = 826 mm   (dist between bars)
factor = 56006.24135
Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 329.4485 mm
take fs/Fsy = M*/Phi Mu = 0.991729269
therefore

Lsy =	327	mm
say =	500	mm

≥ 25k1db 625

For Bottom Steel :

Cover, c = 75 mm d_b = 20 mm
# of lines of bars = 1
If ab > 2c $C = 2c + d_b$ = 170 mm
if ab < 2c $C = a_b + d_b$ = 1194 mm
k1 = 1 A_b = 314.1592654 mm^2
k2 = 2.2 a_b = 1174 mm   (dist between bars)
factor = 44804.99308
Lsyt = 263.5588 mm   take fs/Fsy = M*/Phi Mu = 0.095090338
therefore

Lsy =	25	mm
say =	400	mm

≥ 25k1db 500

9 SHEAR CHECK

V* = 81.30 kN   Dist to centreline of bottom steel from soffit
critical location : d = 293 mm   dq = 93 mm
do = 307 mm
V* = 65 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
= 167.83 kN
phi Vumax = 1719.2 kN > V* No web crushing

phi Vumin = 297 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 = 731.707317
Asv.min = 256.097561 mm^2
Asv.max = 5282 mm^2

Asv = 2 x area of stirrups = 400 mm^2 2 Ties
theta = 30.42946 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 525.48 kN
Phi Vu = 368 kN > V* = 65.09417 kN
hence

PROVIDE :	**1 Y12 LIGS**	**@**	**300**	**centres**

Other Support :
V* = 0.00 kN

Dist to centreline of bottom steel from soffit
critical location : d = 293 mm dq = 93 mm
do = 307 mm
V* = -16.21 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
= 167.8313 kN
phi Vumax = 1719.2 kN > V* No web crushing
phi Vumin = 297 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 = 731.707317
Asv.min = 256.097561 mm^2
Asv.max = 5282.21431 mm^2

Asv = 2 x area of stirrups = 600 mm2 2 Ties
theta = 31.02635 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 658.2886 kN
Phi Vu = 460.802 kN > V* = -16.21 kN
hence

PROVIDE :	**1 Y12 LIGS**	**@**	**300 centres**

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 135.20 kNm Es = 200000 MPa
Ml = 0.00 kNm Ec = 31975.35051 MPa
Mo = 85.9 kNm
Ms = 18.30 kNm
Ig = 5.33E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width b_{eff} = 1800 mm A_{sc} = 1884.955592 mm^2

webwidth	b_w =	1000 mm	A_{st} =	1884.955592 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	283 mm
			d =	293 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 + 21695.153 dn - 4613382.455) = 0

dn = 76.780451 mm

d = 283 mm

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

= 6.52E+08 mm4

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

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

= 7.92E+11 mm^4

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

b_w =	1000 mm	A_{st} =	1884.955592 mm^2
d_{sc} =	97 mm	A_{sc} =	1884.955592 mm^2
d =	293 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 + 21695.153 dn - 4415280.481) = 0

dn = 74.75 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.01E+08 mm^4

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

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 2.64E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **97** mm

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

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

d = **293** mm

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

500 dn^2 + 21695.153 dn - 4415280.481) = 0

dn = 74.7477867 mm

d = 293 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.01E+08 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 = 3.97E+11 mm^4

Midspan Deflection :

Lef = 4000 mm

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

= 0.00062801 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 55.11 kN/m

w - short term	=	55.185 kN/m		
Defln,s,sus	=	0.000627159 mm		
At midspan :			Ast =	1884.95559 mm^2
			Asc =	1884.95559 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)
	=	0.00113 mm

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

ALLOWABLE DEFLN	=	Lef / 500 =	8 mm
			> 0.9 x Total Defln

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	20	3
2	250	20	2
3	336	20	3

OUTPUT

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

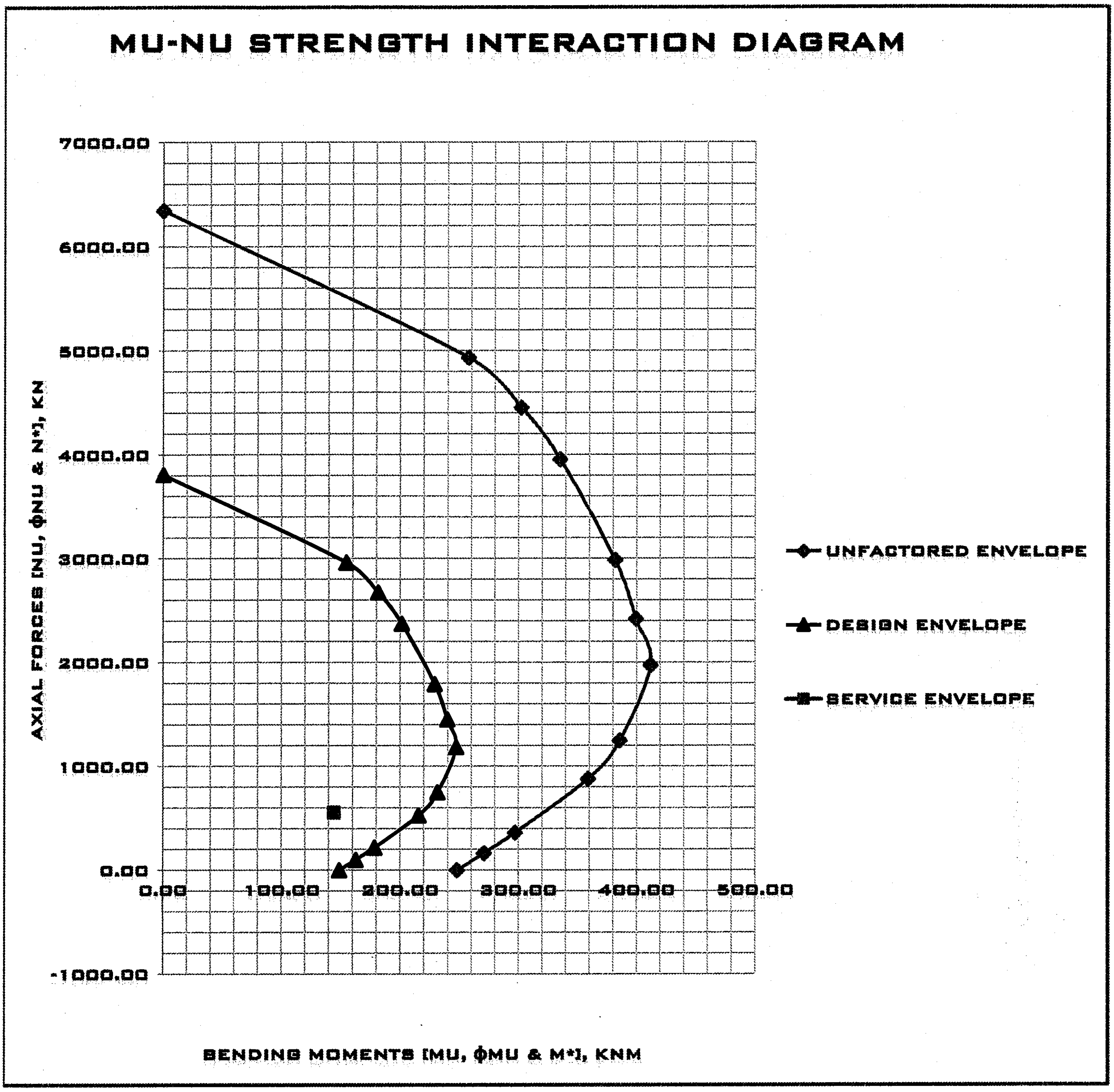
UNFACTORED ENVELOPE

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		



400 x 400 RC COLUMN IS OK!

ISOLATED COLUMN FOOTING DESIGN

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

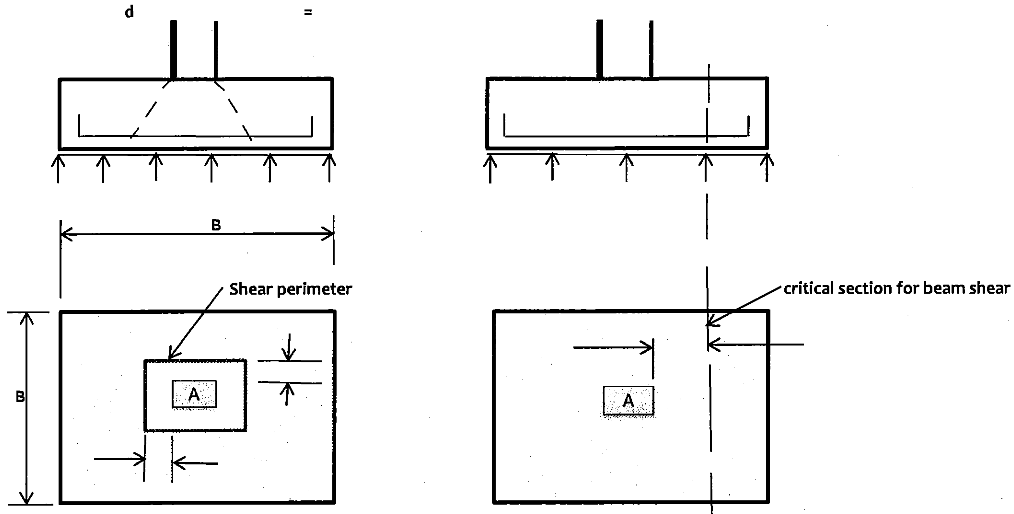
Calculations & Description						Result	Unit
1 Data							
DL	=	... DL from Column 1 ...				329.3	kN
LL	=	... LL from Column 1 ...				271.3	kN
P	=	DL + LL				600.6	kN
P*	=	1.4 DL	+	1.7 LL		922.23	kN
qa	=				0.175	450	kPa
f'c	=				3.85	32	MPa
fsy	=	... reinforcement...			0.525	410	MPa
col. Width A	=				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				1.4	m
L	=	Long side				1.4	m
						1.96	m^2
qult	=	P^*_{total}/A				0.47052551	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*	=	qult x B (C-Col.w/2-d)					
V*	=	312899.4643	-d	658.735714			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}	=	736.2389222 d					N
Equating V* & φVuc							
515.3672 d	=	312899.4643	-d	658.7357143			
1174.103 d	=	312899.4643					
d	=					266.5008734	mm



Punching shear

Beam shear

						Result	Unit
Cover	=					75	mm
Assume bar dia		Y20				20	mm
D	=	d + cover + bar dia/2				351.5008734	mm
Take D	=					400	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* - qult x punching shear area				654.018696	kN
V*	=	qult(A-punching shear area)				654.018696	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)	0.475	m
M*	=	qult x B x l^2/2	74.31362277	kNm
Ast.reqd.	=	M*/ϕ0.9dfsy	799.17	mm^2
Ast.min.	=	1.4/fsy bd	1,505.85	mm^2

Ast.min. Governs, Therefore, use Ast.min.

Hence provide 5 Y20 bars in longitudinal direction

Astprovide = 1,550.00 mm^2

6 **Tensile steel for bending in the transverse direction.**

Considering a transverse strip 1 m wide:

l	=	0.5(L-C1)	0.475	m
M*	=	qult x 1m x l^2/2	53.08115912	kNm
Ast.reqd.	=	M*/ϕ0.9dfsy	570.84	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 1,505.85 mm^2

Hence provide 5 Y20 bars in transverse direction

Astprovide = 1,550.00 mm^2

7 **Spacing between the bars**

Ast/bar	=			310	mm^2
S	=	[Ast/bar X beff]/Ast.prov.	... centre - to - centre spacing ...	280	mm

8 **Now check** 5 bars @ 280 CTS

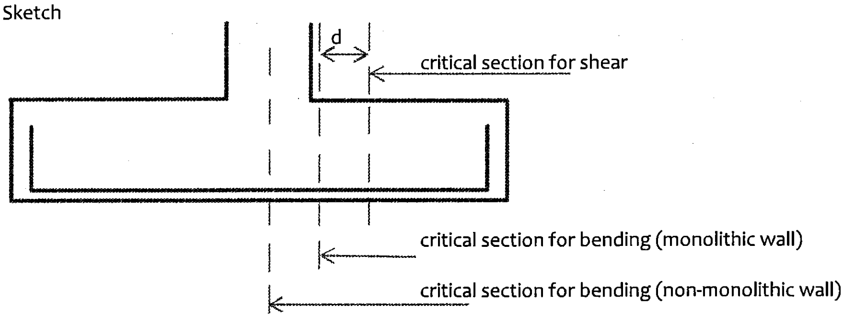
T = fsyAst	=						635.5	kN	
C = T	=						635.5	kN	
A	=	C/0.85f'c					23364.0	mm^2	
γkud	=	A/b					16.7	mm	
a	=	γkud/2					8.3	mm	
l	=	d-a					306.7	mm	
Mu(prov)	=	Txl					194.9	kN	
ϕMu	=	155.9 kNm	>	M*	=	74.3 kNm			OK
γ	=	0.85-0.007(f'c-28)					0.822		
ku	=	γkud/γd	0.06	<	0.4	UNDER-REINFORCED.Therefore, OK			

THEREFORE, USE 5 BARS @ 280 AT TOP & BOTTOM AND IN BOTH DIRECTIONS, Ast PROVIDED 1550 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					



	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)		3.75922	kNm
	qult	=	P^*_{total}/A		0.1253073	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*	=	15036.88 - 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}	=	339.8954302 d			N
	Equating V* & φVuc					
	237.9268 d		=	15036.88 - 75.1844 d		
	313.1112 d		=	15036.88		
	d		=		48.024088	mm
	Cover	=			75	mm
	Assume bar dia		Y16		16	mm
	D	=	d + cover + bar dia/2		131.02409	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		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
5	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:	75 WD x 100 DP HWD @ 600 cts
Rafter:	100 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. Cover Slab @ Ground FloorLevel

200 thk RC two-way slab

Reinforcements:	Continuous Edge:	Y16 - 300 cts	Top & Bottom Each Way
	Discontinuous Edge:	Y16 - 300 cts	Top & Bottom Each Way
	Mid-span:	Y16 - 300 cts	Top & Bottom Each Way
Temp. & shrinkage (T&S) Reinforcement:		Y12 - 300 cts	Top & Bottom Each Way

D. Roof Beam:

	B1		B2	
	400 DP x 400 WD RC Beam		300 DP x 300 WD RC Beam	
Reinforcements:	5 - Y20	Tensile Steel	3 - Y20	Tensile Steel
	3 - Y16	Compressive Steel	3 - Y16	Compressive Steel
	Y12 - 200 cts	Ligatures	Y12 - 200 cts	Ligatures

E. Ground Slab: 200 thk / 150 thk RC slab

Reinforcements: F92 Mesh/ F72 Mesh wire Top & Bottom

F. Base Slab

400 thk RC Slab

Reinforcement:	Y20 bars @ 300 cts Top & Bottom Each Way
T & S	Y20 bars @ 300 cts Top & Bottom Each Way

G. RC Column:

400 x 400 SQ RC Col:

Main Vert. Bars:	8 - Y20
	Y12 - 200 cts Ligatures

H. Basement Wall 1.

400 thk RC Wall

Reinforcements:	Vertical:	Y20 - 300 cts	Outside
		Y20 - 300 cts	Inside
	Horizontal	Y12 - 200 cts	

I. Basement Wall 2.

300 thk RC Wall

Reinforcements:	Vertical:	Y16 - 300 cts	Outside
		Y16 - 300 cts	Inside
	Horizontal	Y12 - 300 cts	

J. Pad Footing for Column

400 DP x 1400 SQR

Reinforcements:	Vertical:	5 - Y20 T & B - Length wise
	Horizontal:	5 - Y20 T & B - Widthth wise

K. Strip Footing

400 DP x 600 WD RC Footing

Reinforcements:	4 - Y16 Top & Bottom: Width Wise
	R10 - 300 CTS: Length Wise