No.

Independent Public Business Corporation

The Independent State of Papua New Guinea

DETAILED DESIGN (PHASE 2) ON PORT MORESBY SEWERAGE SYSTEM UPGRADING PROJECT IN THE INDEPENDENT STATE OF PAPUA NEW GUINEA

FINAL REPORT

PART I: Design Report

Volume II – Design Calculations

December 2011

JAPAN INTERNATIONAL COOPERATION AGENCY

NJS CONSULTANTS CO., LTD.

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DETAIL DESIGN ON
PORT MORESBY SEWERAGE SYSTEM UPGRADING PROJECT

FINAL REPORT

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Section A1: Flow Calculation of Sewer Network

Main Trunk Sewer

Table 1 Flow Calculation for Main Trunk Sewer

Span No	Lower Span	Upper point	Lower point	service population					Peak sewage flow (m^3/d)			Peak Flow (m^3/min)	Peak Flow (m^3/sec)
				lpcd 240		lpcd 400		Total	(240lpcd×2.0)	(400lpcd× 2.0)	Total		
				each span	cummulative	each span	cummulative						
1	**2**	Kanudi P.S	Idubada P.S	765	765	0	0	765	367	0	367	0.25	0.004
2	**3-1**	Idubada P.S	Hagara P.S	818	1,583	0	0	1,583	760	0	760	0.53	0.009
3-1	**3-2**	Hagara P.S	Receiving MH	3719	5,302	0	0	5,302	2,545	0	2,545	1.77	0.029
3-2	**4**	Receiving MH	Hanuabada P.S	3043	8,345	0	0	8,345	4,006	0	4,006	2.78	0.046
4	**5-1**	Hanuabada P.S	Konedobu P.S	5411	13,756	0	0	13,756	6,603	0	6,603	4.59	0.076
5-1	**5-2**	Konedobu P.S	Receiving Well (2)	0	13,756	7,142	7,142	20,898	6,603	5,714	12,317	8.55	0.143
5-2	**9**	Receiving Well (2)	Ela Beach Road	0	13,756	0	7,142	20,898	6,603	5,714	12,317	8.55	0.143
6	**8**	Stanley Esp|anade	Ela Beach Road	0	0	2,487	2,487	2,487	0	1,990	1,990	1.38	0.023
7	**8**	Sea Park P.S	Ela Beach Road	0	0	1,061	1,061	1,061	0	849	849	0.59	0.01
8	**9**	Ela Beach Road	Ela Beach Road	0	0	0	3,548	3,548	0	2,838	2,838	1.97	0.033
9	**10**	Ela Beach Road	Davara P.S	0	13,756	0	10,690	24,446	6,603	8,552	15,155	10.52	0.175
10	**11**	Davara P.S	Lawes Road P.S	0	13,756	607	11,297	25,053	6,603	9,038	15,641	10.86	0.181
11	**17**	Lawes Road P.S	Receiving Well (1)	0	13,756	1897	13,194	26,950	6,603	10,555	17,158	11.92	0.199
12-1	**12-2**	Koki P.S	Peak Point (3)	7,647	7,647	0	0	7,647	3,671	0	3,671	2.55	0.042
12-2	**13**	Peak Point (3)	Badili P.S	0	7,647	0	0	7,647	3,671	0	3,671	2.55	0.042
13	**17**	Badili P.S	Receiving Well (1)	12,644	20,291	0	0	20,291	9,740	0	9,740	6.76	0.113
14-1	**14-2**	Kila Police	Receiving MH	369	369	0	0	369	177	0	177	0.12	0.002
14.2	**15-1**	Receiving MH	Konebada P.S	369	738	0	0	738	354	0	354	0.25	0.004
15-1	**15-2**	Konebada P.S	Receiving MH	218	956	0	0	956	459	0	459	0.32	0.005
15-2	**16**	Receiving MH	Gabutu P.S	524	1480	0	0	1,480	710	0	710	0.49	0.008
16	**17**	Gabutu P.S	Receiving Well (1)	0	1480	0	0	1,480	710	0	710	0.49	0.008
17	**18**	Receiving Well (1)	Kaugere P.S	0	35,527	0	13,194	48,721	17,053	10,555	27,608	19.17	0.320
				15,000	15,000	0	0	15,000	7,200	0	7,200	5.00	0.083
18	**STP**	Kaugere P.S	STP	0	50,527	0	13,194	63,721	24,253	10,555	34,808	24.17	0.403
19	**STP**	Horsecamp P.S	STP	4,455	4,455	0	0	4,455	2,138	0	2,138	1.48	0.025
(Branch)		Old Yacht Club P.S		0	0	500	500	500	0	400	400	0.28	0.005

Table 2 Peak Flow and Pipe Dimension

(white)	Force Main
(blue)	Gravity Main
(yellow)	Existing Facility

Span No	Lower Span	Upper point	Lower point	Peak Flow (m^3/min)	Peak Flow (m^3/sec)	Pipe Facility: Diameter (mm)	Slope (1/1000)	Full capacity (m^3/sec)	Full Velocity (m/sec)	Length (m)	Remarks
1	**2**	Kanudi P.S	Idubada P.S	0.36	0.006	HDPE DN 110	-	0.006	0.82	1,100	
2	**3-1**	Idubada P.S	Hagara P.S	0.65	0.011	HDPE DN 110	-	0.011	1.51	897	
3-1	**3-2**	Hagara P.S	Receiving MH	1.90	0.032	HDPE DN 225	-	0.032	1.04	790	
3-2	**4**	Receiving MH	Hanuabada P.S	2.91	0.049	HDPE DN 355	3.0	0.109	1.43	530	
4	**5-1**	Hanuabada P.S	Konedobu P.S	4.80	0.08	HDPE DN 355	-	0.08	1.05	1,311	
5-1	**5-2**	Konedobu P.S	Receiving Well (2)	8.80	0.147	HDPE DN 450	-	0.147	1.2	1,362	
5-2	**9**	Receiving Well (2)	Ela Beach Road	8.80	0.147	HDPE DN 560	2.0	0.301	1.58	203	
6	**8**	Stanley Esplanade P.S	Ela Beach Road	1.40	0.023	HDPE DN 160	-	0.023	1.48	446	
7	**8**	Sea Park P.S	Ela Beach Road	0.60	0.01	HDPE DN 110	-	0.01	1.37	631	
8	**9**	Ela Beach Road	Ela Beach Road	2.00	0.033	HDPE DN 355	3.0	0.109	1.43	241	
9	**10**	Ela Beach Road	Davara P.S	10.8	0.180	HDPE DN 630	2.0	0.412	1.71	184	
10	**11**	Davara P.S	Lawes Road P.S	11.2	0.187	HDPE DN 450	-	0.187	1.52	746	
11	**17**	Lawes Road P.S	Receiving Well (1)	12.3	0.205	HDPE DN 500	-	0.205	1.35	2,708	
12-1	**12-2**	Koki P.S	Peak Point (3)	2.6	0.043	HDPE DN 225	-	0.043	1.4	71	
						φ250	-	0	0	253	existing
12-2	**13**	Peak Point (3)	Badili P.S	2.55	0.043	○300 (Gravity Flow)	-	0.043	0.61	273	existing
13	**17**	Badili P.S	Receiving Well (1)	6.90	0.115	HDPE DN 400	-	0.115	1.18	1,162	
14-1	**14-2**	Kila Police	Receiving MH	0.36	0.006	HDPE DN 110	-	0.006	0.82	221	
14.2	**15-1**	Receiving MH	Konebada P.S	0.45	0.008	HDPE DN 225	3.0	0.032	1.05	432	
15-1	**15-2**	Konebada P.S	Receiving MH	0.45	0.008	HDPE DN 110	-	0.008	1.1	214	
15-2	**16**	Receiving MH	Gabutu P.S	0.75	0.013	HDPE DN 225	3.0	0.032	1.05	331	
16	**17**	Gabutu P.S	Receiving Well (1)	0.75	0.013	HDPE DN 125	-	0.013	1.37	736	
17	**18**	Receiving Well (1)	Kaugere P.S	20.0	0.333	HDPE DN 710	2.0	0.567	1.85	627	
				5.00	0.083						
18	**STP**	Kaugere P.S	STP	25.0	0.417	HDPE DN 710	-	0.417	1.36	1,934	
19	**STP**	Horsecamp P.S	STP	1.50	0.025	HDPE DN 225	-	0.025	0.82	843	
(Branch)		Old Yacht Club P.S		0.65	0.011	φ150	-	0.011	0.62	131	existing

Branch Sewer

1. Idubada

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor: Accumulated Population (PE)	2.5: Unit flow Rate (m^3/day)	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level Start	IL: Invert Level End	EC: Earth Cover Start	EC: Earth Cover End	Remarks
ID01		1	400	400	0.24	0.00278	uPVC	50.75	150	0.0100	1.11		0.0196	706%	2.53	1.58	1.08	0.79	0.52	
ID01		2	32	432	0.24	0.00300	uPVC	26.35	150	0.0067	0.91		0.016	533%	1.75	0.85	0.67	0.75	0.42	
ID01		3	16	448	0.24	0.00311	uPVC	60.29	150	0.0067	0.91		0.016	514%	1.24	0.65	0.24	0.44	1.22	
ID01		4	16	464	0.24	0.00322	uPVC	21.75	150	0.0067	0.91		0.016	497%	1.62	0.22	0.08	1.24	1.37	
ID01		5	16	480	0.24	0.00333	uPVC	21.07	150	0.0067	0.91		0.016	480%	1.60	0.06	-0.08	1.39	1.48	
ID01		6	0	480	0.24	0.00333	uPVC	29.85	150	0.0067	0.91		0.016	480%	1.55	-0.10	-0.30	1.50	1.60	
ID01		7	0	480	0.24	0.00333	uPVC	22.21	150	0.0067	0.91		0.016	480%	1.45	-0.32	-0.47	1.62	1.64	
ID01		8	32	512	0.24	0.00356	uPVC	53.76	150	0.0067	0.91		0.016	450%	1.32	-0.49	-0.85	1.66	2.14	
ID01		9	32	544	0.24	0.00378	uPVC	31.76	150	0.0067	0.91		0.016	424%	1.44	-0.87	-1.09	2.16	2.18	
ID01		10	0	544	0.24	0.00378	uPVC	66.84	150	0.0067	0.91		0.016	424%	1.25	-1.11	-1.55	2.20	3.02	
ID01		11	200	744	0.24	0.00517	uPVC	20.43	150	0.0067	0.91		0.016	310%	1.62	-1.57	-1.71	3.04	3.12	
ID01		12	0	744	0.24	0.00517	uPVC	16.62	150	0.0067	0.91		0.016	310%	1.56	-1.73	-1.84	3.14	2.96	
ID01		13	74	818	0.24	0.00568	uPVC	11.74	150	0.0067	0.91		0.016	282%	1.27	-1.86	-1.94	2.98	3.03	
ID01		14	0	818	0.24	0.00568	uPVC	7.55	150	0.0067	0.91		0.016	282%	1.24	-1.96	-2.01	3.05	3.17	
END	Idubada PS														1.31					
						Total Length		440.97 m												

2. Hanuabada

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
H01		1	16	16	0.24	0.000111	uPVC	54.86	150	0.0634	2.79		0.0493	44414%	61.50	60.42	56.95	0.92	0.90	
H01		2	16	32	0.24	0.000222	uPVC	49.62	150	0.0279	1.85		0.0327	14730%	58.00	56.93	55.54	0.92	0.90	
H01		3	16	48	0.24	0.000333	uPVC	59.98	150	0.0180	1.49		0.0263	7898%	56.60	55.53	54.45	0.92	0.90	
H01		4	16	64	0.24	0.000444	uPVC	76.75	150	0.0268	1.81		0.0321	7230%	55.50	54.42	52.37	0.92	0.90	
H01		5	16	80	0.24	0.000556	uPVC	71.41	150	0.0595	2.70		0.0478	8597%	53.42	52.35	48.10	0.92	0.90	
H01		6	0	96	0.24	0.000667	uPVC	60.73	150	0.1018	3.54		0.0625	9370%	49.15	48.078	41.90	0.92	0.90	
H01		7	32	128	0.24	0.000889	uPVC	43.97	150	0.0554	2.61		0.0461	5186%	42.95	41.88	39.45	0.92	0.90	
END	Existing		80	208	0.24	0.001444	uPVC		150	0.0100	1.11		0.0196	1357%	40.50					Existing assuming minimum slope
H02		1	16	16	0.24	0.000111	uPVC	23.80	150	0.1261	3.94		0.0695	62613%	50.00	48.946	45.945	0.90	0.90	
H02		2	16	32	0.24	0.000222	uPVC	22.68	150	0.1312	4.01		0.0709	31937%	47.00	45.922	42.946	0.92	0.90	
H02		3	8	40	0.24	0.000278	uPVC	25.68	150	0.0382	2.17		0.0383	13777%	44.00	42.926	41.945	0.92	0.90	
H02		4	8	48	0.24	0.000333	uPVC	28.34	150	0.1950	4.89		0.0865	25976%	43.00	41.472	35.946	1.37	0.90	
H04		1	8	88	0.24	0.000611	uPVC	68.54	150	0.1750	4.64		0.0819	13404%	37.00	35.926	23.932	0.92	0.91	
H04		2	24	112	0.24	0.000778	uPVC	38.14	150	0.2300	5.32	3.49	0.0616	7918%	25.00	23.618	14.846	1.23	0.90	
H04		3	32	144	0.24	0.001	uPVC	35.52	150	0.1247	3.91		0.0691	6910%	15.90	14.679	10.25	1.07	1.05	Energy dissipation FRP form installatio
H04		4	32	176	0.24	0.001222	uPVC	35.00	150	0.0980	3.47		0.0613	5016%	11.45	10.23	6.8	1.07	1.05	
H04		5	32	208	0.24	0.001444	uPVC	29.87	150	0.0262	1.79		0.0317	2195%	8.00	6.78	5.997	1.07	1.05	
H04	H06	6	32	240	0.24	0.001667	uPVC	35.91	150	0.0260	1.79		0.0316	1896%	7.20	5.735	4.801	1.31	1.30	
H06		1	16	448	0.24	0.003111	uPVC	35.77	150	0.0067	0.91		0.016	514%	6.25	3.476	3.236	2.62	3.91	
H06		2	16	464	0.24	0.003222	uPVC	49.32	150	0.0067	0.91		0.016	497%	7.30	3.216	2.886	3.93	3.16	
H06		3	16	480	0.24	0.003333	uPVC	42.59	150	0.0067	0.91		0.016	480%	6.20	2.866	2.581	3.18	1.80	
H06	H07	4	16	496	0.24	0.003444	uPVC	57.30	150	0.0067	0.91		0.016	465%	4.53	1.93	1.546	2.45	0.90	
H07		5	160	656	0.24	0.004556	uPVC	17.00	150	0.0067	0.91		0.016	351%	2.60	-1.324	-1.438	3.77	3.78	
END		Hanuabada PS		656	0.24	0.004556	uPVC								2.50					
H03		1	32	32	0.24	0.000222	uPVC	53.93	150	0.0278	1.85		0.0327	14730%	38.50	37.245	35.746	1.10	1.10	
END	H04			32	0.24	0.000222	uPVC		150						37.00					
H05		1	160	160	0.24	0.001111	uPVC	14.54	150	0.0067	0.91		0.016	1440%	6.34	4.79	4.693	1.40	0.40	
H05		2	0	160	0.24	0.001111	uPVC	49.29	150	0.0067	0.91		0.016	1440%	5.25	4.176	3.846	0.92	1.60	
H05		3	32	192	0.24	0.001333	uPVC	49.13	150	0.0067	0.91		0.016	1200%	5.60	3.826	3.497	1.62	2.60	
END	H06														6.25					
MG01		1	5869	5869	0.24	0.040757	**HDPE**	40	312	0.0030	1.42		0.1083	266%	1.66	-1.184	-1.304	2.51	3.47	
END	Hanuabada. F	5													2.50					
							Total Lengh	1169.67 m												

3. Ela Makana

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor: Accumulated Population (PE)	2.5: Unit flow Rate (m^3/day)	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level Start	IL: Invert Level End	EC: Earth Cover Start	EC: Earth Cover End	Remarks
E01		MH01	136	136	**0.40**	0.001574	uPVC	27.49	150	0.0415	2.26		0.0399	2535%	17.64	16.586	15.445	0.90	0.90	
E01		MH02	24	160	**0.40**	0.001852	uPVC	34.38	150	0.0227	1.67		0.0295	1593%	16.50	15.425	14.645	0.92	0.90	
E01		MH03	0	160	**0.40**	0.001852	uPVC	20.55	150	0.0487	2.45		0.0432	2333%	15.70	14.625	13.624	0.92	0.90	
END	Existing														14.68					
E02		1	32	32	**0.40**	0.00037	uPVC	52.24	150	0.1330	4.04		0.0714	19297%	115.40	114.344	107.396	0.90	0.90	
E02		2	16	48	**0.40**	0.000556	uPVC	45.82	150	0.1412	4.16		0.0736	13237%	108.45	107.376	100.906	0.92	0.90	
E02		3	24	72	**0.40**	0.000833	uPVC	53.54	150	0.1156	3.77		0.0666	7995%	101.96	100.885	94.696	0.92	0.90	
E02		4	16	88	**0.40**	0.001019	uPVC	32.79	150	0.0527	2.54		0.045	4416%	95.75	94.674	92.946	0.92	0.90	
E02		5	16	104	**0.40**	0.001204	uPVC	29.53	150	0.0508	2.50		0.0441	3663%	94.00	92.926	91.426	0.92	0.90	
E02		6	8	112	**0.40**	0.001296	uPVC	23.85	150	0.0463	2.38		0.0421	3248%	92.48	91.406	90.302	0.92	0.90	
E02		7	8	120	**0.40**	0.001389	uPVC	11.29	150	0.0545	2.59		0.0457	3290%	91.36	90.282	89.667	0.92	0.91	
E02		8	8	128	**0.40**	0.001481	uPVC	48.29	150	0.0380	2.16		0.0382	2579%	90.73	89.647	87.812	0.93	1.23	
END	Existing	9													89.20					
							Total Length	379.77	m											

4. Touaguba

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor: Accumulated Population (PE)	2.5: Unit flow Rate (m^3/day)	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level Start	IL: Invert Level End	EC: Earth Cover Start	EC: Earth Cover End	Remarks
T01		1	24	24	**0.40**	0.000278	uPVC	24.80	150	0.0181	1.49		0.0263	9460%	126.00	124.945	124.496	0.90	0.90	
T01		2	16	40	**0.40**	0.000463	uPVC	56.46	150	0.0830	3.19		0.0564	12181%	125.55	124.472	119.786	0.92	0.90	
T01		3	16	56	**0.40**	0.000648	uPVC	55.16	150	0.0959	3.43		0.0606	9352%	120.84	119.766	114.476	0.92	0.90	
T01		4	8	64	**0.40**	0.000741	uPVC	29.44	150	0.0548	2.59		0.0458	6181%	115.53	114.456	112.843	0.92	0.90	
T01		5	8	72	**0.40**	0.000833	uPVC	29.34	150	0.0392	2.19		0.0388	4658%	113.90	112.823	111.673	0.92	0.90	
T01		6	8	80	**0.40**	0.000926	uPVC	39.07	150	0.0635	2.79		0.0493	5324%	112.73	111.653	109.172	0.92	0.90	
END	Existing	7	96	176	**0.40**	0.002037	uPVC								110.23					
T02		1	16	16	**0.40**	0.000185	uPVC	24.61	150	0.1990	4.94		0.0873	47189%	104.20	103.143	98.246	0.90	0.90	
T02		2	40	56	**0.40**	0.000648	uPVC	25.11	150	0.1990	4.94		0.0873	13472%	99.30	98.123	93.126	1.02	0.90	
T02		3	32	88	**0.40**	0.001019	uPVC	31.20	150	0.1077	3.64		0.0643	6310%	94.18	93.106	89.746	0.92	0.90	
T02		4	32	120	**0.40**	0.001389	uPVC	49.51	150	0.0297	1.91		0.0338	2433%	90.80	89.726	88.256	0.92	0.90	
T02	T04	5	32	152	**0.40**	0.001759	uPVC	31.96	150	0.0450	2.35		0.0415	2359%	89.31	88.234	86.796	0.92	0.90	
T04		1	16	296	**0.40**	0.003426	uPVC	15.55	150	0.6700	9.07	5.95	0.1052	3071%	87.85	86.365	75.946	1.33	0.90	T03 inlet
T04		2	80	376	**0.40**	0.004352	uPVC	40.37	150	0.2472	5.51	3.62	0.0639	1468%	77.00	75.925	65.946	0.92	0.90	Energy dissipation FRP form installatio
T04		3	72	448	**0.40**	0.005185	uPVC	38.82	150	0.2570	5.62	3.69	0.0652	1257%	67.00	65.923	55.946	0.92	0.90	
T04	T07	4	32	480	**0.40**	0.005556	uPVC	36.35	150	0.2125	5.11	3.35	0.0593	1067%	57.00	55.926	48.202	0.92	0.90	Energy dissipation FRP form installatio
T07		1	32	768	**0.40**	0.008889	uPVC	33.32	150	0.2173	5.17	3.39	0.0599	674%	49.26	47.988	40.748	1.12	1.10	T05, T06 inlet
T07		2	80	848	**0.40**	0.009815	uPVC	35.72	150	0.1954	4.90		0.0866	882%	42.00	40.728	33.748	1.12	1.10	
T07		3	72	920	**0.40**	0.010648	uPVC	27.67	150	0.1799	4.70		0.0831	780%	35.00	33.728	28.75	1.12	1.10	
T07		4	32	952	**0.40**	0.011019	uPVC	29.07	150	0.1990	4.94		0.0873	792%	30.00	28.73	22.945	1.12	0.90	
T07		5	16	968	**0.40**	0.011204	uPVC	48.86	150	0.2248	5.25	3.45	0.0609	544%	24.00	22.926	11.942	0.92	0.90	Energy dissipation FRP form installatio
T07		6	16	984	**0.40**	0.011389	uPVC	23.00	150	0.0783	3.10		0.0548	481%	13.00	11.922	10.121	0.92	0.91	
END		Existing													11.18					
T03		1	128	128	**0.40**	0.001481	uPVC	34.09	150	0.0992	3.49		0.0617	4166%	91.25	90.178	86.796	0.92	0.90	
END	T04														87.85					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
T05		1	32	32	**0.40**	0.00037	uPVC	29.13	150	0.0406	2.23		0.0395	10676%	71.50	70.429	69.246	0.92	0.90	
T05		2	16	48	**0.40**	0.000556	uPVC	34.49	150	0.0705	2.94		0.052	9353%	70.30	69.226	66.794	0.92	0.90	
T05		3	16	64	**0.40**	0.000741	uPVC	37.79	150	0.0225	1.66		0.0294	3968%	67.85	66.774	65.924	0.92	0.90	
T05		4	16	80	**0.40**	0.000926	uPVC	18.80	150	0.1790	4.69		0.0828	8942%	66.98	65.904	62.539	0.92	0.91	
T05		5	16	96	**0.40**	0.001111	uPVC	28.52	150	0.0643	2.81		0.0497	4473%	63.60	62.519	60.685	0.93	0.91	
T05		6	16	112	**0.40**	0.001296	uPVC	53.31	150	0.0494	2.46		0.0435	3356%	61.75	60.665	58.031	0.93	0.92	
T05		7	16	128	**0.40**	0.001481	uPVC	49.76	150	0.0770	3.08		0.0543	3666%	59.10	58.011	54.179	0.94	0.92	
T05		8	16	144	**0.40**	0.001667	uPVC	50.37	150	0.0780	3.10		0.0547	3281%	55.25	54.159	50.23	0.94	0.92	
T05		9	16	160	**0.40**	0.001852	uPVC	27.56	150	0.0248	1.75		0.0308	1663%	51.30	50.21	49.527	0.94	0.92	
T05		10	16	176	**0.40**	0.002037	uPVC	18.34	150	0.0301	1.92		0.034	1669%	50.60	49.507	48.955	0.94	0.92	
T05		11	16	192	**0.40**	0.002222	uPVC	12.83	150	0.0125	1.24		0.0219	986%	50.03	48.935	48.775	0.94	0.92	
T05		12	16	208	**0.40**	0.002407	uPVC	30.12	150	0.0190	1.53		0.027	1122%	49.85	48.58	48.008	1.12	1.10	
END	T07														49.26					
T06		1	32	32	**0.40**	0.00037	uPVC	43.80	150	0.0224	1.66		0.0293	7919%	50.50	49.427	48.446	0.92	0.90	
T06		2	16	48	**0.40**	0.000556	uPVC	20.32	150	0.0110	1.16		0.0205	3687%	49.50	48.426	48.202	0.92	0.90	
END	T07														49.26					
						Total Length		1214.62 m												

5. Koki

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
KK01		MH01	32	32	0.24	0.000222	uPVC	36.84	150	0.4072	7.07	4.64	0.082	36937%	67.50	66.446	51.445	0.90	0.90	
KK01		MH02	32	64	0.24	0.000444	uPVC	46.79	150	0.3138	6.21	4.07	0.072	16216%	52.50	51.425	36.742	0.92	0.90	Energy dissipation FRP form installation
KK01		MH03	32	96	0.24	0.000667	uPVC	49.50	150	0.2884	5.95	3.91	0.069	10345%	37.80	36.722	22.446	0.92	0.90	Energy dissipation FRP form installation
KK01		MH04	32	128	0.24	0.000889	uPVC	47.85	150	0.1793	4.69		0.0829	9325%	23.50	22.426	13.846	0.92	0.90	
KK01		MH05	48	176	0.24	0.001222	uPVC	27.44	150	0.1608	4.44	2.92	0.0515	4214%	14.90	13.828	9.416	0.92	0.90	
END	Existing	MH06													10.47					
		Future	1892	1892																Future Development (Estimation)
Existing01		MH00	224	2116	0.24	0.014694	uPVC	100.00	150	0.2920	5.99	3.93	0.0694	472%	70.00	68.946	39.746	0.90	0.90	Energy dissipation FRP form installation
KK02	Existing02	MH01	32	2148	0.24	0.014917	uPVC	69.75	150	0.2836	5.90	3.87	0.0684	459%	40.80	39.726	19.945	0.92	0.90	
Existing02		MH02	48	2196	0.24	0.01525	uPVC	13.20	150	0.2714	5.77	3.79	0.067	439%	21.00	19.925	16.343	0.92	0.90	
Existing02	KK03	MH03	16	2212	0.24	0.015361	uPVC	46.52	150	0.1158	3.77		0.0666	434%	17.40	16.267	10.88	0.98	0.94	
KK03		Existing	16	2228	0.24	0.015472	uPVC	6.90	150	0.0400	2.22		0.0392	253%	11.97	10.860	10.584	0.96	1.06	Better to replace or put more slope
KK03		MH01	16	2244	0.24	0.015583	uPVC	4.73	150	0.2930	6.00		0.106	680%	11.80	10.478	9.281	1.09	0.90	
KK03		MH02	400	2644	0.24	0.018361	uPVC	14.09	225	0.0102	1.47		0.0584	318%	10.41	9.261	9.117	0.92	0.90	
KK03		MH03	64	2708	0.24	0.018806	uPVC	45.62	225	0.0307	2.55		0.1013	539%	10.25	9.097	7.701	0.92	0.90	
KK03		MH04	16	2724	0.24	0.018917	uPVC	32.22	225	0.0142	1.73		0.0689	364%	8.83	7.681	6.834	0.92	1.29	
KK03		MH05	8	2732	0.24	0.018972	uPVC	57.23	225	0.0145	1.75		0.0696	367%	8.35	6.809	5.979	1.31	1.29	
KK03		MH06	32	2764	0.24	0.019194	uPVC	65.00	225	0.0030	0.80		0.0316	165%	7.50	5.946	5.433	1.33	1.74	
KK03		MH07	0	2764	0.24	0.019194	uPVC	22.58	225	0.0110	1.53		0.0606	316%	7.40	5.413	5.263	1.76	2.33	Drop pipe
END	Existing	MH08													7.10					
KK04		MH01	24	24	0.24	0.000167	uPVC	59.30	150	0.0386	2.18		0.0385	23054%	15.75	14.695	12.406	0.90	0.90	
KK04		MH02	0	24	0.24	0.000167	uPVC	59.42	150	0.0540	2.58		0.0455	27246%	13.46	12.385	9.176	0.92	0.90	
KK04		MH03	0	24	0.24	0.000167	uPVC	40.00	150	0.0603	2.72		0.0481	28802%	10.23	9.156	6.764	0.92	0.90	
END	Existing														7.82					
KK05		MH01	16	16	0.24	0.000111	uPVC	46.40	150	0.0595	2.70		0.0478	43063%	30.50	29.277	26.516	1.07	1.07	
KK05		MH02	16	32	0.24	0.000222	uPVC	45.98	150	0.1546	4.36		0.077	34685%	27.74	26.211	19.682	1.38	1.16	
KK05		MH03	16	48	0.24	0.000333	uPVC	31.85	150	0.0426	2.29		0.0404	12132%	21.00	19.643	18.305	1.20	1.23	
KK05		MH04	8	56	0.24	0.000389	uPVC	10.32	150	0.0042	0.72		0.0127	3265%	19.69	18.285	18.216	1.25	1.25	
KK05		MH05	8	64	0.24	0.000444	uPVC	34.77	150	0.0447	2.34		0.0414	9324%	19.62	18.196	16.989	1.27	0.91	
KK05		MH06	8	72	0.24	0.0005	uPVC	38.80	150	0.0780	3.10		0.0547	10940%	18.05	16.969	13.943	0.93	0.90	
KK05		MH07	0	72	0.24	0.0005	uPVC	57.75	150	0.0170	1.45		0.0255	5100%	15.00	13.923	12.595	0.92	1.25	
KK05	KK07	MH08	8	80	0.24	0.000556	uPVC	47.18	150	0.0759	3.05		0.054	9712%	14.00	12.572	8.991	1.27	1.26	
KK07		MH01	24	232	0.24	0.001611	uPVC	22.16	150	0.0398	2.21		0.0391	2427%	10.40	8.971	8.091	1.28	1.26	
END	Existing	MH02													9.50					
KK06		MH01	80	80	0.24	0.000556	uPVC	34.48	150	0.0174	1.46		0.0258	4640%	14.40	13.346	12.746	0.90	0.90	
KK06		MH02	48	128	0.24	0.000889	uPVC	70.07	150	0.0482	2.43		0.043	4837%	13.80	12.723	9.346	0.92	0.90	
END	KK07														10.40					
							Total Length	1284.74 m												

6. Badili

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
B01		1	240	240	0.24	0.001667	uPVC	14.64	150	0.0100	1.11		0.0196	1176%	15.40	14.346	14.200	0.90	0.95	
B01		2	16	256	0.24	0.001778	uPVC	48.62	150	0.0100	1.11		0.0196	1102%	15.30	14.180	13.694	0.97	1.45	
B01		3	16	272	0.24	0.001889	uPVC	29.57	150	0.0100	1.11		0.0196	1038%	15.30	13.642	13.346	1.50	0.90	
END	Existing	4													14.40					
B02-1		1	64	64	0.24	0.000444	uPVC	67.69	150	0.1182	3.81		0.0673	15158%	101.00	99.946	91.945	0.90	0.90	
B02-1		2	64	128	0.24	0.000889	uPVC	77.59	150	0.2575	5.62	3.69	0.0652	7334%	93.00	91.925	71.946	0.92	0.90	Energy dissipation FRP form installation
B02-1		3	64	192	0.24	0.001333	uPVC	70.55	150	0.1344	4.06		0.0718	5386%	73.00	71.928	62.446	0.92	0.90	
B02-1		4	64	256	0.24	0.001778	uPVC	52.72	150	0.1324	4.03		0.0713	4010%	63.50	62.426	55.446	0.92	0.90	
B02-1		5	64	320	0.24	0.002222	uPVC	51.87	150	0.1297	3.99		0.0705	3173%	56.50	55.424	48.696	0.92	0.90	
B02-1		6	64	384	0.24	0.002667	uPVC	53.37	150	0.0418	2.27		0.04	1500%	49.75	48.676	46.445	0.92	0.90	
B02-1		7	40	424	0.24	0.002944	uPVC	56.91	150	0.0110	1.16		0.0205	696%	47.50	46.422	45.796	0.92	0.90	
B02-1		8	40	464	0.24	0.003222	uPVC	39.44	150	0.0107	1.15		0.0203	630%	46.85	45.778	45.356	0.92	0.90	
B02-1		9	40	504	0.24	0.0035	uPVC	54.32	150	0.0808	3.15		0.0557	1591%	46.41	45.334	40.945	0.92	0.90	
B02-1	B03	10	16	520	0.24	0.003611	uPVC	44.15	150	0.0630	2.78		0.0491	1360%	42.00	40.927	38.146	0.92	0.90	
B03		1	32	632	0.24	0.004389	uPVC	51.26	150	0.1820	4.73		0.0836	1905%	39.20	38.070	28.741	0.98	1.56	
B03		2	32	664	0.24	0.004611	uPVC	56.1	150	0.1216	3.86		0.0683	1481%	30.45	28.721	21.899	1.58	0.95	
END	Existing		40	704	0.24	0.004889	uPVC		150		0.00		0	0%	23.00					
B02-2		1	80	80	0.24	0.000556	uPVC	79.36	150	0.1802	4.70		0.0831	14946%	53.50	52.390	38.089	0.90	0.90	
END	B03														39.20					
B04		1	16	16	0.24	0.000111	uPVC	42.56	150	0.0395	2.20		0.0389	35045%	73.78	72.726	71.045	0.90	0.90	
B04		2	16	32	0.24	0.000222	uPVC	40.50	150	0.0144	1.33		0.0235	10586%	72.10	71.025	70.442	0.92	0.90	
B04		3	16	48	0.24	0.000333	uPVC	33.95	150	0.0258	1.78		0.0315	9459%	71.50	70.422	69.546	0.92	0.90	
B04		4	16	64	0.24	0.000444	uPVC	52.72	150	0.0868	3.27		0.0577	12995%	70.60	69.522	64.946	0.92	0.90	
B04		5	16	80	0.24	0.000556	uPVC	76.97	150	0.0322	1.99		0.0351	6313%	66.00	64.924	62.446	0.92	0.90	
B04		6	16	96	0.24	0.000667	uPVC	45.61	150	0.0763	3.06		0.0541	8111%	63.50	62.426	58.946	0.92	0.90	
B04		7	16	112	0.24	0.000778	uPVC	30.35	150	0.0488	2.45		0.0433	5566%	60.00	58.926	57.445	0.92	0.90	
B04		8	16	128	0.24	0.000889	uPVC	67.09	150	0.0601	2.72		0.048	5399%	58.50	57.425	53.393	0.92	0.90	
B04		9	16	144	0.24	0.001	**HDPE**	41.98	**140**	0.0460	3.24		0.0499	4990%	54.45	53.373	51.442	0.92	0.90	HDPE DN160 for Heavy Trafic Highwa
B04		10	16	160	0.24	0.001111	**HDPE**	42.24	**140**	0.0397	3.01		0.0464	4176%	52.50	51.422	49.745	0.92	0.90	ditto
B04		11	16	176	0.24	0.001222	**HDPE**	43.77	**140**	0.0585	3.66		0.0563	4607%	50.80	49.725	47.164	0.92	0.90	ditto
B04		12	16	192	0.24	0.001333	**HDPE**	22.00	**140**	0.0760	4.17		0.0641	4809%	48.22	47.144	45.472	0.92	0.90	ditto
B04		13	16	208	0.24	0.001444	**HDPE**	18.00	**140**	0.0533	3.49		0.0537	3719%	46.53	45.452	44.493	0.92	0.90	ditto
B04		14	16	224	0.24	0.001556	**HDPE**	55.00	**140**	0.0469	3.27		0.0504	3239%	45.55	44.473	41.894	0.92	0.90	ditto
B04		15	16	240	0.24	0.001667	**HDPE**	61.87	**140**	0.0574	3.62		0.0557	3341%	42.95	41.874	38.323	0.92	0.90	ditto
B04		16	16	256	0.24	0.001778	**HDPE**	48.28	**140**	0.0623	3.77		0.0581	3268%	39.38	38.303	35.295	0.92	0.90	ditto

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
B04		17	16	272	0.24	0.001889	**HDPE**	40.65	**140**	0.0697	3.99		0.0614	3250%	36.35	35.275	32.442	0.92	0.90	ditto
B04		18	32	304	0.24	0.002111	**HDPE**	25.48	**140**	0.0973	4.72		0.0726	3439%	33.50	32.422	29.943	0.92	0.90	ditto
B04		19	32	336	0.24	0.002333	**HDPE**	29.84	**140**	0.0663	3.89		0.0599	2568%	31.00	29.923	27.945	0.92	0.90	ditto
B04		20	24	360	0.24	0.0025	**HDPE**	36.02	**140**	0.0713	4.04		0.0621	2484%	29.00	27.925	25.357	0.92	1.19	ditto
B04	B10	21	16	376	0.24	0.002611	**HDPE**	40.30	**140**	0.0620	3.76		0.0579	2218%	26.70	25.337	22.838	1.21	1.67	ditto
B10		1	32	3720	0.24	0.025833	**HDPE**	26.55	**200**	0.0395	3.82		0.1199	464%	24.66	22.727	21.678	1.70	0.90	ditto, Highway Crossing
B10		2		3720	0.24	0.025833	uPVC	47.04	**225**	0.0476	3.17		0.1261	488%	22.81	21.657	19.418	0.92	0.90	
B10		3	80	3800	0.24	0.026389	uPVC	51.04	**225**	0.0386	2.86		0.1136	430%	20.55	19.398	17.428	0.92	0.90	
B10		4		3800	0.24	0.026389	uPVC	37.57	**225**	0.0436	3.04		0.1207	457%	18.56	17.406	15.768	0.92	0.90	
B10		5	80	3880	0.24	0.026944	uPVC	41.09	**225**	0.0555	3.43		0.1362	505%	16.90	15.748	13.468	0.92	0.90	
B10		6		3880	0.24	0.026944	uPVC	37.74	**225**	0.0560	3.44		0.1368	508%	14.60	13.448	11.335	0.92	1.67	
B10		7	80	3960	0.24	0.0275	uPVC	22.82	**225**	0.0653	3.72		0.1478	537%	13.24	11.308	9.818	1.70	1.70	
END	Existing			3960	0.24	0.0275	uPVC								11.75					
B05		1	24	24	0.24	0.000167	uPVC	25.13	150	0.0358	2.10		0.0371	22216%	97.30	96.246	95.346	0.90	0.90	
B05		2	24	48	0.24	0.000333	uPVC	25.42	150	0.0650	2.83		0.0499	14985%	96.40	95.326	93.674	0.92	0.90	
B05		3	16	64	0.24	0.000444	uPVC	39.37	150	0.1005	3.51		0.0621	13986%	94.73	93.653	89.696	0.92	0.90	
B05		4	16	80	0.24	0.000556	uPVC	27.29	150	0.1202	3.84		0.0679	12212%	90.75	89.676	86.396	0.92	0.90	
B05		5	16	96	0.24	0.000667	uPVC	36.57	150	0.1280	3.97		0.0701	10510%	87.45	86.376	81.695	0.92	1.15	
B05		6	16	112	0.24	0.000778	uPVC	50.70	150	0.1220	3.87		0.0684	8792%	83.00	81.517	75.332	1.33	1.01	
B05		7	16	128	0.24	0.000889	uPVC	38.34	150	0.1535	4.34		0.0767	8628%	76.50	75.312	69.427	1.03	0.92	
B05		8	16	144	0.24	0.001	uPVC	47.96	150	0.0680	2.89		0.0511	5110%	70.50	69.407	66.146	0.94	1.20	
B05		9	16	160	0.24	0.001111	uPVC	50.44	150	0.0952	3.42		0.0604	5437%	67.50	66.119	61.317	1.23	1.03	
B05		10	16	176	0.24	0.001222	uPVC	38.42	150	0.1200	3.84		0.0678	5548%	62.50	61.256	56.646	1.09	0.90	
B05		11	16	192	0.24	0.001333	uPVC	29.13	150	0.1250	3.92		0.0692	5191%	57.70	56.487	52.846	1.06	0.90	
B05		12	16	208	0.24	0.001444	uPVC	29.88	150	0.0965	3.44		0.0608	4211%	53.90	52.826	49.943	0.92	0.90	
B05		13	16	224	0.24	0.001556	uPVC	35.32	150	0.1148	3.76		0.0664	4267%	51.00	49.923	45.868	0.92	1.18	
B05	B07	14	16	240	0.24	0.001667	uPVC	67.56	150	0.0873	3.27		0.0579	3473%	47.20	45.845	39.947	1.20	1.20	
B07		1	32	652	0.24	0.004528	uPVC	24.12	150	0.0705	2.94		0.052	1148%	41.30	39.446	37.746	1.70	1.70	
B07		2	16	668	0.24	0.004639	uPVC	27.37	150	0.0657	2.84		0.0502	1082%	39.60	37.724	35.926	1.72	0.90	B6 join into B7
B07		3	16	684	0.24	0.00475	uPVC	35.77	150	0.0716	2.97		0.0524	1103%	36.98	35.906	33.345	0.92	0.90	
B07	B09	4	16	700	0.24	0.004861	uPVC	32.13	150	0.0607	2.73		0.0482	992%	34.40	33.325	31.375	0.92	0.92	
B09		1	2524	3280	0.24	0.022778	uPVC	38.37	**225**	0.1029	4.66		0.1854	814%	32.45	31.018	27.070	1.20	1.20	For Future Development, Bigger Pipe
B09		2	16	3296	0.24	0.022889	uPVC	36.50	**225**	0.0548	3.40		0.1353	591%	28.50	27.050	25.050	1.22	1.22	
B09		3	16	3312	0.24	0.023	uPVC	74.65	**225**	0.0246	2.28		0.0907	394%	26.50	24.665	22.829	1.60	1.60	
END	B10														24.66					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
B06		1	140	140	0.24	0.000972	uPVC	53.22	150	0.2020	4.98		0.088	9053%	68.15	66.651	56.046	1.35	1.20	
B06		2	160	300	0.24	0.002083	uPVC	38.79	150	0.1620	4.46		0.0788	3783%	57.40	56.026	49.742	1.22	1.20	
B06		3	32	332	0.24	0.002306	uPVC	38.49	150	0.0865	3.26		0.0576	2498%	51.10	49.722	46.393	1.22	0.90	
B06		4	24	356	0.24	0.002472	uPVC	25.87	150	0.0823	3.18		0.0562	2273%	47.45	46.373	44.244	0.92	0.90	
B06		5	24	380	0.24	0.002639	uPVC	25.58	150	0.1556	4.37		0.0772	2925%	45.30	44.224	40.244	0.92	0.90	
END	B07														41.30					
B08		1	32	32	0.24	0.000222	uPVC	74.53	150	0.0808	3.15		0.0557	25090%	41.10	40.046	34.024	0.90	0.90	
B08		2	24	56	0.24	0.000389	uPVC	55.37	150	0.0475	2.42		0.0427	10977%	35.08	33.726	31.096	1.20	1.20	
END	B09														32.45					
B11		1	1240	1240	0.24	0.008611	uPVC	17.13	**225**	0.0040	0.92		0.0366	425%	2.06	-0.170	-0.239	2.00	2.03	For Future Development, Bigger Pipe
B11		2	80	1320	0.24	0.009167	uPVC	50.73	**225**	0.0040	0.92		0.0366	399%	2.02	-0.259	-0.462	2.05	2.39	
B11		3	40	1360	0.24	0.009444	uPVC	57.64	**225**	0.0040	0.92		0.0366	388%	2.16	-0.482	-0.713	2.41	2.38	
B11		4	40	1400	0.24	0.009722	uPVC	52.7	**225**	0.0040	0.92		0.0366	376%	1.90	-0.733	-0.921	2.40	2.81	
B11		5	40	1440	0.24	0.01	uPVC	10.96	**225**	0.0040	0.92		0.0366	366%	2.12	-0.941	-0.985	2.83	2.86	
END	Badili PS														2.10					
B12		1	32	32	0.24	0.000222	uPVC	33.16	150	0.0286	1.87		0.0331	14910%	28.00	26.944	25.996	0.90	0.90	
B12		2	8	40	0.24	0.000278	uPVC	34.5	150	0.0270	1.82		0.0322	11583%	27.05	25.976	25.045	0.92	0.90	
B12		3	8	48	0.24	0.000333	uPVC	42	150	0.0067	0.91		0.016	4805%	26.10	25.046	24.765	0.90	1.13	
B12		4	8	56	0.24	0.000389	uPVC	60	150	0.0383	2.17		0.0383	9846%	26.05	24.744	22.446	1.15	0.90	
END	Existing	5	24	80	0.24	0.000556	uPVC		150						23.50					
B13		1	32	32	0.24	0.000222	uPVC	48.59	150	0.0268	1.81		0.0321	14459%	87.80	86.746	85.444	0.90	0.90	
B13		2	0	32	0.24	0.000222	uPVC	18.35	150	0.0261	1.79		0.0316	14234%	86.50	85.424	84.945	0.92	0.90	
B13		3	8	40	0.24	0.000278	uPVC	59.39	150	0.0286	1.87		0.0331	11906%	86.00	84.925	83.226	0.92	0.90	
B13		4	0	40	0.24	0.000278	uPVC	41.15	150	0.0476	2.42		0.0427	15360%	84.28	83.205	81.246	0.92	0.60	
B13		5	0	40	0.24	0.000278	uPVC	30.22	150	0.0497	2.47		0.0436	15683%	82.00	81.228	79.726	0.62	0.60	
END	Existing	6													80.48					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
B14		1	160	160	0.24	0.001111	uPVC	29.12	150	0.0490	2.45		0.0433	3897%	35.63	34.141	32.714	1.34	0.90	
B14		2	40	200	0.24	0.001389	uPVC	34.36	150	0.0524	2.54		0.0448	3225%	33.77	32.694	30.894	0.92	0.90	
B14		3	40	240	0.24	0.001667	uPVC	35.3	150	0.0306	1.94		0.0342	2052%	31.95	30.874	29.794	0.92	0.90	
B14		4	40	280	0.24	0.001944	uPVC	16.94	150	0.0254	1.77		0.0312	1605%	30.85	29.774	29.344	0.92	0.90	
B14		5	32	312	0.24	0.002167	uPVC	50.93	150	0.0369	2.13		0.0376	1735%	30.40	29.324	27.445	0.92	0.90	
B14		6	32	344	0.24	0.002389	uPVC	38.5	150	0.0280	1.85		0.0328	1373%	28.50	27.424	26.346	0.92	0.90	
B14		7	32	376	0.24	0.002611	uPVC	68.03	150	0.0497	2.47		0.0436	1670%	27.40	26.326	22.945	0.92	0.90	
B14		8	24	400	0.24	0.002778	uPVC	21.96	150	0.0447	2.34		0.0414	1490%	24.00	22.925	21.943	0.92	0.90	
END	Existing														23.00					
B15		1	160	160	0.24	0.001111	uPVC	21.99	150	0.1023	3.54		0.0626	5635%	50.00	48.946	46.696	0.90	0.90	
B15		2	24	184	0.24	0.001278	uPVC	17.11	150	0.1276	3.96		0.0699	5469%	47.75	46.676	44.493	0.92	0.90	
B15		3	24	208	0.24	0.001444	uPVC	17.54	150	0.1875	4.80		0.0848	5873%	45.55	44.473	41.184	0.92	1.22	
B15		4	24	232	0.24	0.001611	uPVC	22.94	150	0.1007	3.52		0.0622	3861%	42.56	41.164	38.854	1.24	1.24	
B15		5	16	248	0.24	0.001722	uPVC	43.47	150	0.0633	2.79		0.0493	2863%	40.25	38.834	36.082	1.26	1.26	
B15		6	16	264	0.24	0.001833	uPVC	35.89	150	0.0200	1.57		0.0277	1511%	37.50	35.964	35.246	1.38	0.90	
B15		7	16	280	0.24	0.001944	uPVC	13.31	150	0.0067	0.91		0.016	823%	36.30	35.226	35.137	0.92	0.91	
END	Existing														36.20					
			682																	
			7586																	
						Total Length		4153.39	m											

7. Gabutu

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
G01		1	16	16	0.24	0.000111	uPVC	45.43	150	0.0100	1.11		0.0196	17658%	6.50	5.45	4.99	0.90	1.85	
G01		2	16	32	0.24	0.000222	uPVC	42.55	150	0.0240	1.72		0.0303	13649%	7.00	4.97	3.95	1.88	0.90	
G01		3	0	32	0.24	0.000222	uPVC	51.53	150	0.0240	1.72		0.0303	13649%	5.00	3.92	2.69	0.92	0.90	
G01		4	849	881	0.24	0.006118	uPVC	7.00	150	0.0100	1.11		0.0196	320%	3.74	-0.02	-0.09	3.61	3.24	From Kila Police 738PE
Konebada PS	Konebada PS		75	956	0.24	0.006639	uPVC		150						3.30					To Receiving Well
G02		Existing																		
END																				
G03		MH01	8	8	0.24	0.000056	uPVC	55.65	150	0.1078	3.64		0.0643	114821%	39.00	38.245	32.246	0.600	0.600	
G03		MH02	8	16	0.24	0.000111	uPVC	22.80	150	0.1087	3.65		0.0646	58198%	33.00	32.223	29.745	0.620	0.600	
G03		MH03	8	24	0.24	0.000167	uPVC	16.67	150	0.1488	4.28		0.0755	45210%	30.50	29.725	27.245	0.620	0.600	
G03		MH04	8	32	0.24	0.000222	uPVC	24.24	150	0.1600	4.43		0.0783	35270%	28.00	27.225	23.347	0.620	0.750	
G03	MG01	MH05	8	40	0.24	0.000278	uPVC	56.85	150	0.1434	4.20		0.0741	26655%	24.25	22.778	14.626	1.320	1.320	To Receiving Well
END															16.10					
G04		MH01	8	8	0.24	0.000056	uPVC	50.21	150	0.0807	3.15		0.0556	99286%	36.30	35.553	31.500	0.590	0.870	
G04		MH02	8	16	0.24	0.000111	uPVC	16.48	150	0.0578	2.66		0.0471	42432%	32.52	31.145	30.192	1.220	1.200	
G04		MH03	8	24	0.24	0.000167	uPVC	30.16	150	0.0746	3.03		0.0535	32036%	31.55	30.172	27.922	1.220	1.220	
G04		MH04	0	24	0.24	0.000167	uPVC	27.20	150	0.1161	3.78		0.0667	39940%	29.30	27.902	24.744	1.240	0.600	
G04		MH05	8	32	0.24	0.000222	uPVC	40.59	150	0.1473	4.25		0.0752	33874%	25.50	24.724	18.745	0.620	0.600	
G04		MH06	8	40	0.24	0.000278	uPVC	24.96	150	0.0500	2.48		0.0438	15755%	19.50	18.725	17.477	0.620	0.870	
G04	MG01	MH07	8	48	0.24	0.000333	uPVC	14.15	150	0.1687	4.55		0.0804	24144%	18.50	17.121	14.734	1.230	1.210	To Receiving Well
MG01	MG02	Receiving W	1044	1044	0.24	0.0058	**HDPE**	54.29	**200**	0.0795	5.41	2.05	0.17	2931%	16.10	14.606	10.290	1.210	1.180	Peak Factor 2.0, HDPE DN225, n=0.00
MG02		MH01	80	1124	0.24	0.006244	**HDPE**	52.31	**200**	0.0975	5.99	2.27	0.1883	3016%	11.75	10.270	5.170	1.200	1.150	Peak Factor 2.0, HDPE DN225, n=0.00
MG02	MG03	MH02	8	1132	0.24	0.006289	**HDPE**	45.97	**200**	0.0370	3.69		0.116	1844%	6.60	5.001	3.300	1.320	1.320	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH03	0	1132	0.24	0.006289	**HDPE**	28.03	**200**	0.0200	2.72		0.0853	1356%	4.90	3.280	2.719	1.340	1.300	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH04	0	1132	0.24	0.006289	**HDPE**	32.06	**200**	0.0628	4.81		0.1511	2403%	4.30	2.699	0.686	1.320	1.030	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH05	80	1212	0.24	0.006733	**HDPE**	23.70	**200**	0.0100	1.92		0.0603	896%	2.00	-0.459	-0.696	2.180	2.790	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH06	16	1228	0.24	0.006822	**HDPE**	34.37	**200**	0.0067	1.57		0.0494	724%	2.37	-0.716	-0.946	2.810	2.360	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH07	16	1244	0.24	0.006911	**HDPE**	51.33	**200**	0.0067	1.57		0.0494	715%	1.69	-0.966	-1.310	2.380	2.730	Peak Factor 2.0, HDPE DN225, n=0.00
MG03		MH08	236	1480	0.24	0.008222	**HDPE**	11.28	**200**	0.0089	1.81		0.0569	692%	1.70	-1.330	-1.430	2.750	3.150	Peak Factor 2.0, HDPE DN225, n=0.00
END	Gabutu P.S.														2.00					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
G05		MH01	8	8	0.24	0.000056	uPVC	26.34	150	0.0216	1.63		0.0288	51429%	24.87	24.059	23.490	0.600	0.600	
G05		MH02	8	16	0.24	0.000111	uPVC	34.51	150	0.0718	2.97		0.0525	47297%	24.3	23.468	20.990	0.620	0.600	
G05		MH03	8	24	0.24	0.000167	uPVC	38.26	150	0.0662	2.85		0.0504	30180%	21.8	20.970	18.437	0.620	0.600	
G05	G07	MH04	8	32	0.24	0.000222	uPVC	32.77	150	0.0450	2.35		0.0415	18694%	19.25	18.417	16.942	0.620	1.030	
G07		MH01	24	56	0.24	0.000389	uPVC	30.74	150	0.0817	3.17		0.056	14396%	18.18	16.630	14.119	1.340	1.340	
G07		MH02	8	64	0.24	0.000444	uPVC	33.76	150	0.0250	1.75		0.031	6982%	15.67	14.078	13.234	1.380	0.860	
G07		MH03	8	72	0.24	0.0005	uPVC	41.54	150	0.0470	2.40		0.0424	8480%	14.3	13.214	11.262	0.880	0.630	
G07	MG02	MH04	8	80	0.24	0.000556	uPVC	42.59	150	0.0082	1.00		0.0177	3183%	12.1	11.240	10.891	0.650	0.650	
END															11.75					
G06		MH01	8	8	0.24	0.000056	uPVC	48.66	150	0.0400	2.22		0.0392	70000%	22.5	21.690	19.744	0.600	1.050	
G06	G07	MH02	8	16	0.24	0.000111	uPVC	37.67	150	0.0749	3.03		0	0%	21	19.471	16.650	1.320	1.320	
END															18.18					
G08		MH01	32	32	0.24	0.000222	uPVC	29.09	150	0.0067	0.91		0.016	7207%	1.74	0.930	0.735	0.600	0.980	
G08		MH02	16	48	0.24	0.000333	uPVC	29.32	150	0.0067	0.91		0.016	4805%	1.92	0.715	0.519	1.000	1.450	
G08		MH03	8	56	0.24	0.000389	uPVC	22.56	150	0.0067	0.91		0.016	4113%	2.18	0.499	0.348	1.470	1.970	
G08		MH04	8	64	0.24	0.000444	uPVC	27.31	150	0.0067	0.91		0.016	3604%	2.53	0.328	0.145	1.990	2.500	
G08		MH05	8	72	0.24	0.0005	uPVC	40.34	150	0.0067	0.91		0.016	3200%	2.85	0.125	-0.145	2.520	2.940	
G08	MG03	MH06	8	80	0.24	0.000556	uPVC	40.91	150	0.0067	0.91		0.016	2878%	3	-0.165	-0.439	2.960	2.530	
END															2.3					
						Toal Length		1416.18	m											

8. Kaugere

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
KG01		FT01	24	24	0.24	0.00017	uPVC	10.00	100	0.5100	6.03		0.0474	28383%	58.40	57.946	52.846	0.30	0.30	
KG01		FT02	20	44	0.24	0.00031	uPVC	12.00	100	0.3070	4.68		0.0367	11993%	53.30	52.826	49.142	0.32	0.30	
KG01		FT03	16	60	0.24	0.00042	uPVC	13.00	100	0.5445	6.23		0.0489	11727%	49.60	49.122	42.044	0.32	0.30	
KG01		FT04	16	76	0.24	0.00053	uPVC	12.50	100	0.4965	5.95		0.0467	8845%	42.50	42.024	35.818	0.32	0.33	
KG01		FT05	16	92	0.24	0.00064	uPVC	23.12	150	0.1462	4.24		0.0749	11721%	36.30	35.798	32.418	0.35	0.93	
KG01		MH01	16	252	0.24	0.00175	uPVC	11.04	150	0.1050	3.59		0.0635	3629%	33.50	32.085	30.926	1.26	1.37	
KG01		MH02	36	288	0.24	0.00200	uPVC	29.10	150	0.1200	3.84		0.0678	3390%	32.45	30.906	27.414	1.39	1.23	
KG01		MH03	36	324	0.24	0.00225	uPVC	54.31	150	0.0785	3.11		0.0549	2440%	28.80	27.394	23.131	1.25	0.92	
KG01		MH04	36	360	0.24	0.00250	uPVC	54.79	150	0.0909	3.34		0.059	2360%	24.20	23.111	18.131	0.94	0.92	
END	Existing	MH05													19.20					
KG01-1		MH01-1	144	144	0.24	0.00100	uPVC	13.81	150	0.1050	3.59		0.0635	6350%	33.90	32.846	31.396	0.90	0.90	
END	KG01	MH01													32.45					
KG02		MH01	64	64	0.24	0.00044	uPVC	44.45	150	0.1136	3.74		0.066	14865%	60.75	59.696	54.646	0.90	0.90	
KG02		MH02	64	128	0.24	0.00089	uPVC	32.37	150	0.0067	0.91		0.016	1800%	55.70	54.422	54.205	1.12	1.74	
KG02	KG04	MH03	56	184	0.24	0.00128	uPVC	38.39	150	0.0140	1.31		0.0232	1815%	56.10	54.185	53.648	1.76	1.10	
KG04		MH01	40	280	0.24	0.00194	uPVC	26.68	150	0.0532	2.56		0.0452	2325%	54.90	53.516	52.097	1.23	1.20	
KG04		MH02	32	312	0.24	0.00217	uPVC	30.94	150	0.1077	3.64		0.0643	2967%	53.45	52.077	48.745	1.22	1.20	
KG04		MH03	32	344	0.24	0.00239	uPVC	24.65	150	0.1534	4.34		0.0767	3211%	50.10	48.725	44.944	1.22	1.20	
KG04		MH04	48	392	0.24	0.00272	uPVC	56.13	150	0.0940	3.40		0.06	2204%	46.30	44.924	39.648	1.22	1.20	
KG04	KG06	MH05	48	440	0.24	0.00306	uPVC	32.94	150	0.0146	1.34		0.0237	776%	41.00	39.628	39.147	1.22	1.20	
KG06		MH01	0	520	0.24	0.00361	uPVC	19.57	150	0.1270	3.95		0.0698	1933%	40.50	39.127	36.642	1.22	1.20	
KG06		MH02	8	528	0.24	0.00367	uPVC	26.22	150	0.0184	1.50		0.0266	725%	38.00	36.622	36.14	1.22	1.21	
END	Existing	Existing		528	0.24	0.00367	uPVC		150						37.50					
KG03		MH01	56	56	0.24	0.00039	uPVC	20.00	150	0.1550	4.36		0.0771	19820%	58.00	56.944	53.844	0.90	0.90	
END	KG04	MH01													54.90					
KG05		MH01	40	40	0.24	0.00028	uPVC	37.61	150	0.0678	2.89		0.051	18345%	44.55	43.494	40.944	0.90	0.90	
KG05		MH02	40	80	0.24	0.00056	uPVC	37.86	150	0.0396	2.21		0.039	7014%	42.00	40.924	39.425	0.92	0.92	
END	KG06	MH01													40.50					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor Accumulated Population (PE)	2.5 Unit flow Rate (m^3/day)	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level Start	IL: Invert Level End	EC: Earth Cover Start	EC: Earth Cover End	Remarks
KG07		MH01	48	48	0.24	0.00033	uPVC	53.660	150	0.0235	1.70		0.03	9009%	97.60	96.546	95.285	0.90	0.90	
KG07		MH02	32	80	0.24	0.00056	uPVC	33.630	150	0.0255	1.77		0.0313	5629%	96.34	95.264	94.406	0.92	0.90	
KG07		MH03	32	112	0.24	0.00078	uPVC	42.870	150	0.0500	2.48		0.0438	5630%	95.46	94.386	92.243	0.92	0.90	
KG07		MH04	48	160	0.24	0.00111	uPVC	33.500	150	0.0934	3.39		0.0599	5392%	93.30	92.223	89.094	0.92	1.00	
KG07		MH05	32	192	0.24	0.00133	uPVC	23.260	150	0.1070	3.63		0.0641	4809%	90.25	89.072	86.583	1.02	1.09	
KG07		MH06	48	240	0.24	0.00167	uPVC	11.490	150	0.1845	4.76		0.0841	5045%	87.83	86.563	84.443	1.11	0.90	
KG07		MH07	32	272	0.24	0.00189	uPVC	37.050	150	0.0720	2.97		0.0525	2779%	85.50	84.423	81.755	0.92	1.34	
KG07		MH08	48	320	0.24	0.00222	uPVC	38.150	150	0.1016	3.53		0.0624	2808%	83.25	81.418	77.542	1.68	1.65	
KG07		MH09	32	352	0.24	0.00244	uPVC	25.720	150	0.0805	3.14		0.0556	2275%	79.35	77.516	75.446	1.68	0.90	
KG07		MH10	32	384	0.24	0.00267	uPVC	23.480	150	0.1333	4.05		0.0715	2681%	76.50	75.426	72.296	0.92	0.90	
KG07		MH11	32	416	0.24	0.00289	uPVC	54.720	150	0.1248	3.92		0.0692	2395%	73.35	72.275	65.446	0.92	0.90	
KG07		MH12	32	448	0.24	0.00311	uPVC	30.000	150	0.1326	4.04		0.0713	2292%	66.50	65.424	61.446	0.92	0.90	
KG07	KG09	MH13	32	480	0.24	0.00333	uPVC	40.000	150	0.1870	4.79		0.0847	2541%	62.50	61.107	53.627	1.24	1.22	
KG09	KG11-1	MH01	32	648	0.24	0.00450	uPVC	48.000	150	0.1771	4.66		0.0824	1831%	55.00	53.087	44.586	1.76	1.76	KG08
KG11-1		MH01	16	664	0.24	0.00461	uPVC	18.170	150	0.1651	4.50		0.0796	1726%	46.50	44.537	41.537	1.81	1.81	KG10
KG11-1		MH02	24	688	0.24	0.00478	uPVC	16.710	150	0.0339	2.04		0.036	753%	43.50	41.512	40.946	1.83	0.90	
KG11-1		MH03	16	704	0.24	0.00489	uPVC	28.310	150	0.1937	4.88		0.0862	1763%	42.00	40.926	35.442	0.92	0.90	
KG11-1	KG11-3	MH04	16	720	0.24	0.00500	uPVC	41.810	150	0.0645	2.81		0.0497	994%	36.50	35.124	32.427	1.22	1.22	
KG11-3		MH01	16	752	0.24	0.00522	uPVC	36.060	150	0.0182	1.50		0.0264	506%	33.80	32.402	31.746	1.24	0.90	KG11-2
END		Existing		752	0.24	0.00522	uPVC		150						32.80					
KG08		MH01	72	72	0.24	0.00050	uPVC	48.790	150	0.0530	2.55		0.0451	9020%	56.75	55.693	53.107	0.90	1.74	
END	KG09	MH01	64	136	0.24	0.00094	uPVC		150						55.00					
KG11-2		MH01	16	16	0.24	0.00011	uPVC	20.890	150	0.0933	3.39		0.0598	53874%	35.75	34.695	32.746	0.90	0.90	
END	KG11-3	MH01		16	0.24	0.00011	uPVC		150						33.80					
KG10		MH01	64	64	0.24	0.00044	uPVC	70.970	150	0.0719	2.97		0.0525	11824%	59.60	58.546	53.443	0.90	0.90	
KG10		MH02	48	112	0.24	0.00078	uPVC	54.440	150	0.0415	2.26		0.0399	5129%	54.50	53.423	51.164	0.92	0.90	
KG10		MH03	32	144	0.24	0.00100	uPVC	56.090	150	0.0456	2.37		0.0418	4180%	52.22	51.144	48.586	0.92	1.16	
KG10		MH04	32	176	0.24	0.00122	uPVC	35.000	150	0.0152	1.37		0.0241	1972%	49.90	48.564	48.032	1.18	1.31	
KG10		MH05	32	208	0.24	0.00144	uPVC	41.960	150	0.0082	1.00		0.0177	1226%	49.50	47.957	47.613	1.39	0.93	
KG10		MH06	24	232	0.24	0.00161	uPVC	38.680	150	0.0330	2.01		0.0356	2210%	48.70	47.593	46.317	0.95	0.92	
KG10		MH07	32	264	0.24	0.00183	uPVC	35.450	150	0.0312	1.96		0.0346	1888%	47.39	45.911	44.805	1.33	1.59	
END	KG11-1	MH01					uPVC	34.000	150						46.55	44.785	44.557	1.61	1.89	
															46.60					
						Total Length		1734.340 m												

9. Kila Kila

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
KL01-1		1	124	124	0.24	0.000861	uPVC	65.72	150	0.0067	0.91		0.016	1858%	9.29	8.236	7.796	0.90	1.50	
KL01-1	KL01-3	2	140	264	0.24	0.001833	uPVC	70.87	150	0.0067	0.91		0.016	873%	9.45	7.776	7.301	1.52	1.84	
KL01-3		1	80	872	0.24	0.006056	uPVC	24.79	150	0.0327	2.00		0.0354	585%	9.29	7.281	6.470	1.86	1.88	
KL01-3		2	80	952	0.24	0.006611	uPVC	60.00	150	0.0135	1.29		0.0228	345%	8.50	6.450	5.640	1.90	1.88	
KL01-3		3	80	1032	0.24	0.007167	uPVC	50.73	150	0.0095	1.08		0.0191	266%	7.67	5.620	5.138	1.90	2.38	
END		4													7.67					
KL01-2		1	132	132	0.24	0.000917	uPVC	75.00	150	0.0067	0.91		0.016	1745%	12.50	11.446	10.944	0.90	1.40	
KL01-2		2	132	264	0.24	0.001833	uPVC	75.00	150	0.0067	0.91		0.016	873%	12.50	10.924	10.422	1.42	1.22	
KL01-2		3	132	396	0.24	0.00275	uPVC	62.46	150	0.0136	1.29		0.0228	829%	11.80	10.402	9.553	1.24	1.24	
KL01-2		4	132	528	0.24	0.003667	uPVC	60.92	150	0.0269	1.82		0.0321	875%	10.95	9.487	7.848	1.31	1.29	
END	KL01-3														9.29					
KL02		1	224	752	0.24	0.005222	uPVC	17.00	150	0.0071	0.93		0.0165	316%	13.50	12.446	12.325	0.90	0.90	
KL02		2	24	776	0.24	0.005389	uPVC	47.38	150	0.0067	0.91		0.016	297%	13.38	12.305	11.988	0.92	2.25	
KL02		3	16	792	0.24	0.0055	uPVC	62.50	150	0.0224	1.66		0.0293	533%	14.39	11.966	10.566	2.27	0.90	
KL02		4	24	816	0.24	0.005667	uPVC	47.39	150	0.0380	2.16		0.0382	674%	11.62	10.288	8.487	1.18	1.16	
KL02		5	56	872	0.24	0.006056	uPVC	57.79	150	0.0350	2.07		0.0366	604%	9.80	8.467	6.444	1.18	0.90	
KL02		6	24	896	0.24	0.006222	uPVC	54.11	150	0.0279	1.85		0.0327	526%	7.50	6.424	4.914	0.92	0.90	
KL02		7	125	1021	0.24	0.00709	uPVC	56.27	**225**	0.0284	2.45		0.0974	1374%	5.97	4.894	3.296	0.92	0.90	
KL02	KL06	8	40	1061	0.24	0.007368	uPVC	56.37	**225**	0.0187	1.99		0.079	1072%	4.35	3.276	2.222	0.92	0.90	
KL06		1	40	1597	0.24	0.01109	uPVC	45.09	**225**	0.0258	2.34		0.0929	838%	3.28	1.512	0.349	1.61	1.60	Inlet KL03 to KL05
KL06	KL08	2	40	1637	0.24	0.011368	uPVC	39.99	**225**	0.0229	2.20		0.0875	770%	2.10	0.329	-0.587	1.62	1.61	
KL08		1	24	1981	0.24	0.013757	uPVC	26.39	**225**	0.0067	1.19		0.0473	344%	1.18	-0.873	-1.050	1.90	2.20	Inlet KL07
KL08		2	20	2001	0.24	0.013896	uPVC	35.59	**225**	0.0067	1.19		0.0473	340%	1.30	-1.070	-1.308	2.22	2.58	
KL08		3	40	2041	0.24	0.014174	uPVC	68.30	**225**	0.0067	1.19		0.0473	334%	1.43	-1.328	-1.730	2.60	2.76	
KL08		4	24	2065	0.24	0.01434	uPVC	27.18	**225**	0.0067	1.19		0.0473	330%	1.18	-1.556	-1.724	2.58	2.37	
KL08		5	20	2085	0.24	0.014479	uPVC	29.23	**225**	0.0067	1.19		0.0473	327%	0.80	-1.744	-1.940	2.39	2.07	
KL08		6	20	2105	0.24	0.014618	uPVC	43.82	**225**	0.0067	1.19		0.0473	324%	0.28	-1.960	-2.170	2.09	2.31	
KL08		7	1198	3847	0.24	0.026715	**HDPE**	15.40	**312**	0.0097	2.55		0.1947	729%	0.29	-2.190	-2.419	2.33	2.33	Existing dia. 300 mm, KL09 to KL12
END	Horsecamp PS								(DN355)						0.06					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor: Accumulated Population (PE)	2.5: Unit flow Rate (m^3/day)	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level Start	IL: Invert Level End	EC: Earth Cover Start	EC: Earth Cover End	Remarks
KL03		1	32	32	0.24	0.000222	uPVC	37.19	150	0.1234	3.89		0.0688	30991%	18.09	17.035	12.446	0.90	0.90	
KL03		2	32	64	0.24	0.000444	uPVC	49.32	150	0.0655	2.84		0.0501	11284%	13.50	12.126	8.896	1.22	0.90	
KL03		3	32	96	0.24	0.000667	uPVC	35.53	150	0.0542	2.58		0.0456	6837%	9.95	8.872	6.946	0.92	0.90	
KL03		4	40	136	0.24	0.000944	uPVC	43.51	150	0.0260	1.79		0.0316	3347%	8.00	6.926	5.795	0.92	0.90	
KL03		5	48	184	0.24	0.001278	uPVC	43.77	150	0.0315	1.97		0.0348	2723%	6.85	5.775	4.396	0.92	0.90	
KL03		6	40	224	0.24	0.001556	uPVC	37.61	150	0.0353	2.08		0.0368	2365%	5.45	4.374	3.046	0.92	0.90	
END	KL05														4.10					
KL04		1	80	80	0.24	0.000556	uPVC	44.53	150	0.0900	3.32		0.0587	10558%	12.84	11.433	7.425	1.25	1.02	
KL04		2	24	104	0.24	0.000722	uPVC	45.34	150	0.0380	2.16		0.0382	5291%	8.60	7.399	5.676	1.05	0.90	
KL04		3	24	128	0.24	0.000889	uPVC	39.11	150	0.0500	2.48		0.0438	4927%	6.73	5.656	3.701	0.92	1.60	
KL04		4	24	152	0.24	0.001056	uPVC	5.59	150	0.1790	4.69		0.0828	7841%	5.45	3.681	2.680	1.62	0.92	
KL04	KL05	5	20	172	0.24	0.001194	uPVC	18.33	150	0.0067	0.91		0.016	1340%	3.75	2.660	2.537	0.94	1.41	
KL05		1	20	192	0.24	0.001333	uPVC	46.46	150	0.0067	0.91		0.016	1200%	4.10	2.533	2.222	1.41	1.25	
KL05		2	40	232	0.24	0.001611	uPVC	60.07	150	0.0067	0.91		0.016	993%	3.63	2.202	1.800	1.27	1.10	
KL05		3	40	272	0.24	0.001889	uPVC	37.00	150	0.0067	0.91		0.016	847%	3.05	1.780	1.532	1.12	1.59	
END	KL06														3.28					
KL07		1	24	24	0.24	0.000167	uPVC	46.48	150	0.0624	2.77		0.0489	29281%	14.05	12.996	10.096	0.90	0.90	
KL07		2	24	48	0.24	0.000333	uPVC	33.46	150	0.0622	2.76		0.0488	14655%	11.15	10.076	7.995	0.92	0.90	
KL07		3	20	68	0.24	0.000472	uPVC	50.54	150	0.0390	2.19		0.0387	8199%	9.05	7.975	6.004	0.92	0.90	
KL07		4	24	92	0.24	0.000639	uPVC	66.78	150	0.0377	2.15		0.038	5947%	7.06	5.984	3.466	0.92	0.90	
KL07		5	80	172	0.24	0.001194	uPVC	52.86	150	0.0394	2.20		0.0389	3258%	4.52	3.449	1.366	0.92	0.30	
KL07		6	20	192	0.24	0.001333	uPVC	43.21	150	0.0067	0.91		0.016	1200%	1.82	1.346	1.056	0.32	0.71	
KL07		7	24	216	0.24	0.0015	uPVC	48.27	150	0.0067	0.91		0.016	1067%	1.92	1.036	0.713	0.73	2.43	
KL07		8	24	240	0.24	0.001667	uPVC	52.30	150	0.0067	0.91		0.016	960%	3.30	0.693	0.343	2.45	1.75	
KL07		9	24	264	0.24	0.001833	uPVC	41.19	150	0.0067	0.91		0.016	873%	2.25	0.323	0.047	1.77	1.04	
KL07		10	20	284	0.24	0.001972	uPVC	45.28	150	0.0067	0.91		0.016	811%	1.24	0.027	-0.276	1.06	1.04	
KL07		11	20	304	0.24	0.002111	uPVC	27.05	150	0.0067	0.91		0.016	758%	0.92	-0.300	-0.481	1.07	1.08	
KL07		12	16	320	0.24	0.002222	uPVC	52.58	150	0.0067	0.91		0.016	720%	0.75	-0.501	-0.853	1.10	1.88	
END	KL08														1.18					

Sewer No.	Flows to	Manhole No.	Population (PE)	Peak Factor	2.5	Sewage Amount (m^3/sec)	Pipe Material	Pipe Length (m)	Diameter (mm)	Slope	Velocity (m/sec)	After Energy Dissipation (m/sec)	Pipe Capacity (m^3/sec)	Allowance (%)	GL: Ground Level	IL: Invert Level		EC: Earth Cover		Remarks
				Accumulated Population (PE)	Unit flow Rate (m^3/day)											Start	End	Start	End	
KL09		1	32	32	0.24	0.000222	uPVC	70.80	150	0.1232	3.89		0.0687	30946%	18.87	17.816	9.093	0.9	0.9	
KL09		2	32	64	0.24	0.000444	uPVC	39.00	150	0.0687	2.91		0.0513	11554%	10.15	9.073	6.394	0.92	0.9	
KL09		3	16	80	0.24	0.000556	uPVC	40.24	150	0.0539	2.57		0.0455	8183%	7.45	6.374	4.205	0.92	0.9	
KL09		4	32	112	0.24	0.000778	uPVC	40.00	150	0.0160	1.40		0.0248	3188%	5.26	4.185	3.545	0.92	0.92	
KL09		5	32	144	0.24	0.001	uPVC	30.00	150	0.0067	0.91		0.016	1600%	4.62	3.525	3.324	0.94	0.92	
KL09		6	24	168	0.24	0.001167	uPVC	43.88	150	0.0067	0.91		0.016	1371%	4.40	3.237	2.943	1.01	1.17	
KL09		7	48	216	0.24	0.0015	uPVC	31.75	150	0.0067	0.91		0.016	1067%	4.27	2.687	2.474	1.43	1.37	
END	Existing	8	8	224											4.00					
KL10		1	16	16	0.24	0.000111	uPVC	33.96	150	0.0333	2.02		0.0357	32162%	18.45	17.355	16.224	0.90	0.90	
KL10		2	16	32	0.24	0.000222	uPVC	28.20	150	0.0318	1.98		0.0349	15721%	17.32	16.202	15.305	0.92	0.90	
KL10		3	24	56	0.24	0.000389	uPVC	33.70	150	0.0410	2.24		0.0397	10206%	16.40	15.285	13.903	0.92	0.90	
KL10		4	32	88	0.24	0.000611	uPVC	48.98	150	0.1364	4.09		0.0723	11833%	15.00	13.728	7.047	1.08	1.06	
KL10		5	24	112	0.24	0.000778	uPVC	38.42	150	0.0891	3.31		0.0585	7519%	8.30	7.02	3.597	1.09	1.07	
KL10	KL12	6	24	136	0.24	0.000944	uPVC	30.98	150	0.0547	2.59		0.0458	4852%	4.86	3.577	1.882	1.09	1.07	
KL12		1	64	248	0.24	0.001722	uPVC	23.51	150	0.0174	1.46		0.0258	1498%	3.15	1.795	1.386	1.16	1.16	Inlet KL11
KL12		2	32	280	0.24	0.001944	uPVC	62.08	150	0.0085	1.02		0.0181	931%	2.74	1.366	0.838	1.18	1.18	
KL12		3	48	328	0.24	0.002278	uPVC	63.70	150	0.0067	0.91		0.016	702%	2.21	0.107	-0.32	1.91	2.37	Connect to Existing Sewer
END		4													2.24					
KL11		1	8	8	0.24	0.000056	uPVC	46.86	150	0.0010	0.35		0.0062	11071%	4.00	2.905	2.858	0.90	1.05	
KL11		2	8	16	0.24	0.000111	uPVC	50.86	150	0.0065	0.89		0.0158	14234%	4.10	2.836	2.505	1.07	0.90	
KL11		3	16	32	0.24	0.000222	uPVC	70.23	150	0.0083	1.01		0.0178	8018%	3.60	2.485	1.902	0.92	0.90	
KL11		4	16	48	0.24	0.000333	uPVC	69.86	150	0.0010	0.35		0.0062	1862%	3.00	1.882	1.812	0.92	1.14	
END	KL12														3.15					
KL13		1	16	16	0.24	0.000111	uPVC	29.03	150	0.0100	1.11		0.0196	17658%	0.95	-0.294	-0.584	1.09	0.90	
KL13		2	16	32	1.24	0.001148	uPVC	43.45	150	0.0100	1.11		0.0196	1707%	0.47	-0.604	-1.039	0.92	1.11	
KL13		3	16	48	2.24	0.003111	uPVC	38.3	150	0.0100	1.11		0.0196	630%	0.22	-1.059	-1.442	1.13	1.81	
KL13		4	16	64	3.24	0.006	uPVC	30.32	150	0.0067	0.91		0.016	267%	0.52	-1.462	-1.665	1.83	1.69	
KL13		5	16	80	4.24	0.009815	uPVC	43.71	150	0.0067	0.91		0.016	163%	0.18	-1.685	-1.978	1.71	1.92	
KL13		6	16	96	5.24	0.014556	uPVC	8.48	150	0.0067	0.91		0.016	110%	0.10	-1.998	-2.055	1.94	1.90	
END	Horsecamp PS.																			
							Total Length	3468.95 m												

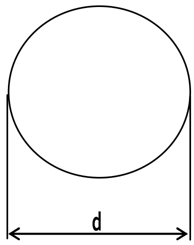
SECTION A2: Flow Calculation of Inlet Pipe for Pumping Stations

Aontributory Area	Peak Flow (m^3/min)	Peak Flow (m^3/sec)	Pipe Faciility Diameter (mm)	Slope (1/1000)	Full capacity (m^3/sec)	Full Velocity (m/sec)	Remarks
Kanudi Area	0.24	0.004	225 DIA-uPVC	4.0	0.037	0.93	KN
Idubada Area	0.30	0.005	225 DIA-uPVC	4.0	0.037	0.93	IB
Hagara Area	1.20	0.020	300 DIA-uPVC	4.0	0.080	1.12	HG
Konedobu Area	4.02	0.067	HDPE 400	4.0	0.175	1.79	KD
Lawes Road Area	1.08	0.018	300 DIA-uPVC	4.0	0.080	1.12	LR
Koki Area	2.52	0.042	HDPE 355	4.0	0.127	1.65	KK
Kila Police Area	0.12	0.002	225 DIA-uPVC	4.0	0.037	0.93	KP
Kaugere Area	4.98	0.083	HDPE 450	4.0	0.240	1.94	KG
Horsecamp Area	1.50	0.025	300 DIA-uPVC	4.0	0.080	1.12	HC

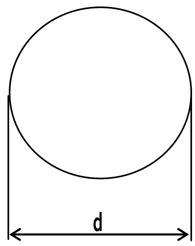
minimum diameter for sewer is 225mm, Dia225-300 uPVC , Dia355-450 HDPE

SECTION A3: Flow Calculation of Overflow Pipe for Pumping Stations

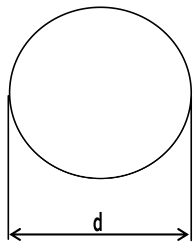
Facility	PS-01(Kanudi) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.004				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.004				
Dimensions						
Pipe diameter	d	0.198 m				
Cross-section area	AW	0.031 m^2				
Pipe length	L	38.70 m				
Friction loss coef.	f	0.023				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.130				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.001				
Straight pipe loss (m)	h_{f1}	0.004				
Inlet loss (m)	h_i	0.000				f = 0.50 n = 1
Outlet loss (m)	h_o	0.001				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.005				
Head loss (m)	h	0.010		Pump Well Top		
Upstream water level (m)	H	1.010	OK	1.400		H=H０＋h



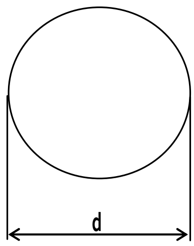
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-02 (Idubada) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.009				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.009				
Dimensions						
Pipe diameter	d	0.198	m			
Cross-section area	AW	0.031	m^2			
Pipe length	L	31.00	m			
Friction loss coef.	f	0.023				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.292				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.004				
Straight pipe loss (m)	h_{f1}	0.016				
Inlet loss (m)	h_i	0.002				f = 0.50 n = 1
Outlet loss (m)	h_o	0.004				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.022				
Head loss (m)	h	0.030		Pump Well Top		
Upstream water level (m)	H	1.030	OK	1.680		H=H0＋h

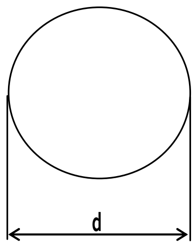
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-03 (Hagara) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.029				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.029				
Dimensions						
Pipe diameter	d	0.247 m				
Cross-section area	AW	0.048 m^2				
Pipe length	L	73.00 m				
Friction loss coef.	f	0.022				
Downstream water level (m)	Ho	1.290				
Flow velocity (m/s)	V	0.605				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.019				
Straight pipe loss (m)	h_{f1}	0.122				
Inlet loss (m)	h_i	0.009				f = 0.50 n = 1
Outlet loss (m)	h_o	0.019				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.150				
Head loss (m)	h	0.150		Pump Well Top		
Upstream water level (m)	H	1.440	OK	3.400		H=H0＋h

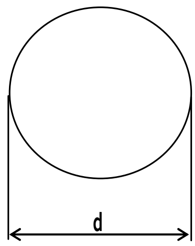
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-04 (Hanuabada) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.076				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.076				
Dimensions Pipe diameter Cross-section area Pipe length Friction loss coef.	 d AW L f	 0.352 m 0.097 m^2 73.00 m 0.021		 d		
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.781				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.031				
Straight pipe loss (m)	h_{f1}	0.136				
Inlet loss (m)	h_i	0.016				f = 0.50 n = 1
Outlet loss (m)	h_o	0.031				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.183				
Head loss (m)	h	0.190		Pump Well Top		
Upstream water level (m)	H	1.190	OK	3.450		H=H0＋h

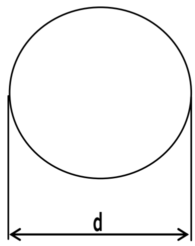
⇒ Upstream water Level is under the level of Pump well top slab.

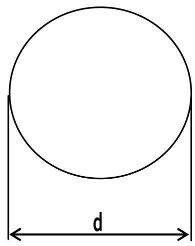
Facility	PS-05 (Konedobu) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.143				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.143				
Dimensions						
Pipe diameter	d	0.494	m			
Cross-section area	AW	0.192	m^2			
Pipe length	L	58.00	m			
Friction loss coef.	f	0.021				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.746				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.028				
Straight pipe loss (m)	h_{f1}	0.070				
Inlet loss (m)	h_i	0.014				f = 0.50 n = 1
Outlet loss (m)	h_o	0.028				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.112				
Head loss (m)	h	0.120		Pump Well Top		
Upstream water level (m)	H	1.120	OK	2.700		H=H0＋h

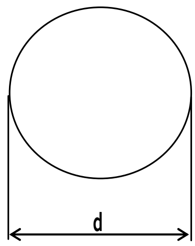
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-06 (Old Yacht Club) Over flow Pipe (Existing)					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.005				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.005				
Dimensions						
Pipe diameter	d	0.100 m				
Cross-section area	AW	0.008 m^2				
Pipe length	L	5.00 m				
Friction loss coef.	f	0.025				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.637				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.021				
Straight pipe loss (m)	h_{f1}	0.026				
Inlet loss (m)	h_i	0.010				f = 0.50 n = 1
Outlet loss (m)	h_o	0.021				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.057				
Head loss (m)	h	0.060		Pump Well Top		
Upstream water level (m)	H	1.060	OK	1.700		H=H0＋h

⇒ Upstream water Level is under the level of Pump well top slab.

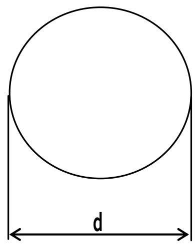
Facility	PS-07 (Stanley Esplanada) (Existing)					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	***This Pumping Station has an over flow weir to existing outfall***				
Number of units	N					
Unit Flow (m^3/s)	q					
Dimensions						
Pipe diameter	d		m			
Cross-section area	AW		m^2			
Pipe length	L		m			
Friction loss coef.	f					
Downstream water level (m)	Ho					
Flow velocity (m/s)	V					V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$					
Straight pipe loss (m)	h_{f1}					
Inlet loss (m)	h_i					f = n =
Outlet loss (m)	h_o					f = n =
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$					
Head loss (m)	h					
Upstream water level (m)	H					

Facility	PS-08 (Sea Park) (Existing)					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	***This Pumping Station has an over flow weir to existing outfall***				
Number of units	N					
Unit Flow (m^3/s)	q					
Dimensions Pipe diameter Cross-section area Pipe length Friction loss coef.	 d AW L f	 m m^2 m				
Downstream water level (m)	Ho					
Flow velocity (m/s)	V					V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$					
Straight pipe loss (m)	h_{f1}					
Inlet loss (m)	h_i					f = n =
Outlet loss (m)	h_o					f = n =
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$					
Head loss (m)	h					
Upstream water level (m)	H					

Facility	PS-09 (Davara) Over flow Pipe (Existing)					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.181				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.181				
Dimensions						
Pipe diameter	d	0.350 m				
Cross-section area	AW	0.096 m^2				
Pipe length	L	17.30 m				
Friction loss coef.	f	0.021				
Downstream water level (m)	Ho	0.250				
Flow velocity (m/s)	V	1.881				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.181				
Straight pipe loss (m)	h_{f1}	0.187				
Inlet loss (m)	h_i	0.090				f = 0.50 n = 1
Outlet loss (m)	h_o	0.181				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.458				
Head loss (m)	h	0.460		Pump Well Top		
Upstream water level (m)	H	0.710	OK	1.250		H=H0＋h

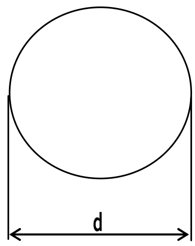
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-10 (Lawes Road) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.199				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.199				
Dimensions						
Pipe diameter	d	0.494 m				
Cross-section area	AW	0.192 m^2				
Pipe length	L	17.10 m				
Friction loss coef.	f	0.021				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	1.038				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.055				
Straight pipe loss (m)	h_{f1}	0.040				
Inlet loss (m)	h_i	0.027				f = 0.50 n = 1
Outlet loss (m)	h_o	0.055				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.122				
Head loss (m)	h	0.130		Pump Well Top		
Upstream water level (m)	H	1.130	OK	2.300		H=H0＋h

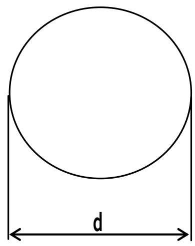


⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-11 (Koki) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.042				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.042				
Dimensions						
Pipe diameter	d	0.313 m				
Cross-section area	AW	0.077 m^2				
Pipe length	L	22.60 m				
Friction loss coef.	f	0.022				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.546				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.015				
Straight pipe loss (m)	h_{f1}	0.024				
Inlet loss (m)	h_i	0.008				f = 0.50 n = 1
Outlet loss (m)	h_o	0.015				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.047				
Head loss (m)	h	0.050		Pump Well Top		
Upstream water level (m)	H	1.050	OK	2.400		H=H0＋h

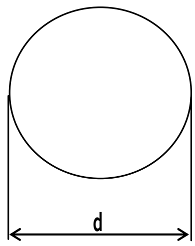


⇒ Upstream water Level is under the level of Pump well top slab.

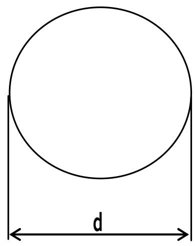
Facility	PS-12 (Badili) Over flow Pipe (Existing)					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.113				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.113				
Dimensions						
Pipe diameter	d	0.600 m				
Cross-section area	AW	0.283 m²				
Pipe length	L	30.00 m				
Friction loss coef.	f	0.021				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.400				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.008				
Straight pipe loss (m)	h_{f1}	0.009				
Inlet loss (m)	h_i	0.004				f = 0.50 n = 1
Outlet loss (m)	h_o	0.008				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.021				
Head loss (m)	h	0.030		Pump Well Top		
Upstream water level (m)	H	1.030	OK	2.300		H=H0＋h

⇒ Upstream water Level is under the level of Pump well top slab.

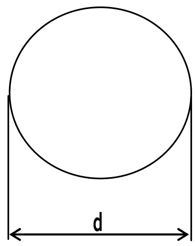
Facility	PS-13 (Kila Police) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.002				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.002				
Dimensions						
Pipe diameter	d	0.198 m				
Cross-section area	AW	0.031 m^2				
Pipe length	L	5.80 m				
Friction loss coef.	f	0.023				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.065				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.000				
Straight pipe loss (m)	h_{f1}	0.000				
Inlet loss (m)	h_i	0.000				f = 0.50 n = 1
Outlet loss (m)	h_o	0.000				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.000				
Head loss (m)	h	0.000		Pump Well Top		
Upstream water level (m)	H	1.000	OK	2.700		H=H0＋h



⇒ Upstream water Level is under the level of Pump well top slab.

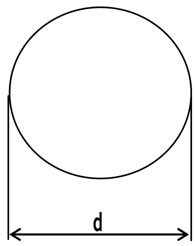
Facility	PS-14 (Konebada) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.006				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.006				
Dimensions						
Pipe diameter	d	0.198 m				
Cross-section area	AW	0.031 m^2				
Pipe length	L	47.00 m				
Friction loss coef.	f	0.023				
Downstream water level (m)	Ho	1.810				
Flow velocity (m/s)	V	0.195				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.002				
Straight pipe loss (m)	h_{f1}	0.011				
Inlet loss (m)	h_i	0.001				f = 0.50 n = 1
Outlet loss (m)	h_o	0.002				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.014				
Head loss (m)	h	0.020		Pump Well Top		
Upstream water level (m)	H	1.830	OK	3.000		H=H0＋h

⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-15 (Gabutu) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.008				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.008				
Dimensions						
Pipe diameter	d	0.198 m				
Cross-section area	AW	0.031 m^2				
Pipe length	L	78.50 m				
Friction loss coef.	f	0.023				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.260				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.003				
Straight pipe loss (m)	h_{f1}	0.031				
Inlet loss (m)	h_i	0.002				f = 0.50 n = 1
Outlet loss (m)	h_o	0.003				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	∑h	0.036				
Head loss (m)	h	0.040		Pump Well Top		
Upstream water level (m)	H	1.040	OK	2.200		H=H0＋h

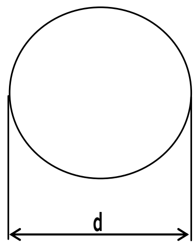
⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-16 (Horsecamp) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.025				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.025				
Dimensions						
Pipe diameter	d	0.313 m				
Cross-section area	AW	0.077 m^2				
Pipe length	L	6.00 m				
Friction loss coef.	f	0.022				
Downstream water level (m)	Ho	1.000				
Flow velocity (m/s)	V	0.325				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.005				
Straight pipe loss (m)	h_{f1}	0.002				
Inlet loss (m)	h_i	0.003				f = 0.50 n = 1
Outlet loss (m)	h_o	0.005				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.010				
Head loss (m)	h	0.010		Pump Well Top		
Upstream water level (m)	H	1.010	OK	1.200		H=H0＋h



⇒ Upstream water Level is under the level of Pump well top slab.

Facility	PS-17 (Kaugere) Over flow Pipe					
Item	Sign	Hourly max. Q(HM)				Remarks
Flow (m^3/s)	Q	0.403				
Number of units	N	1				
Unit Flow (m^3/s)	q	0.403				
Dimensions						
Pipe diameter	d	0.626 m				
Cross-section area	AW	0.308 m²				
Pipe length	L	5.30 m				
Friction loss coef.	f	0.021				
Downstream water level (m)	Ho	14.000				
Flow velocity (m/s)	V	1.309				V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.087				
Straight pipe loss (m)	h_{f1}	0.016				
Inlet loss (m)	h_i	0.044				f = 0.50 n = 1
Outlet loss (m)	h_o	0.087				f = 1.00 n = 1
Bend loss (m)	h_b					f = n =
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.147				
Head loss (m)	h	0.150		Pump Well Top		
Upstream water level (m)	H	14.150	OK	16.100		H=H０＋h



⇒ Upstream water Level is under the level of Pump well top slab.

SECTION A4: Process Calculation of STP

1 BASIC CONDITIONS

1.1 Basic Items

(1) Name : Kila Kila Sewage Treatment Plant

(2) Land Area : Approximately 7.05 ha (L470m*W150m)

(3) Ground Level : -1.0 ~ +30.0 m

(4) Inlet Pipe Diameter : Dia 600 mm

(5) Land Use : Bush

(6) Collection System : Separate Sewer System

(7) Treatment Method : [Sewage Treatment] Oxidation Ditch
[Sludge Treatment] Dewatering

(8) Effluent Discharge Point : Papuan Lagoon

(9) Discharge Point Water Level : +0.25 m (High Tide Level)
+1.00 m (High-high Water Level)

(10) Design Target Year : 2020 (Ultimate)

1.2 Service Area and Design Population

Service Area : Coastal Area

Design Population : 68,200 persons (year 2020)

1.3 Sewage

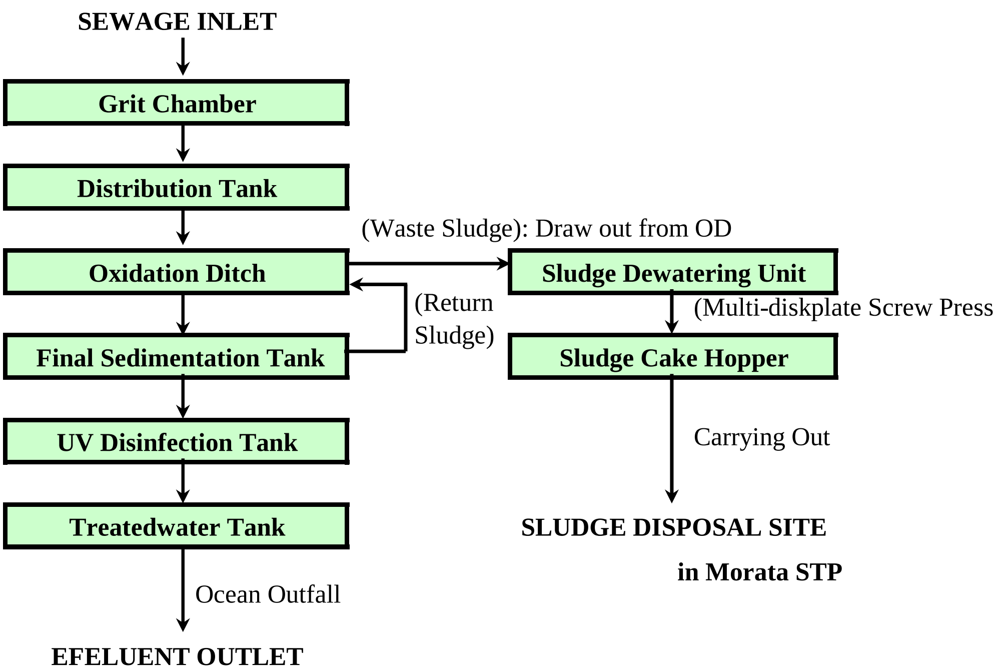
1.3.1 Design Sewage Flow

	ITEM	m^3/day	m^3/hr	m^3/min	m^3/sec
Ultimate (2020)	Daily Average (Q_{AV})	18,400 (Q_{AV-d})	766.7 (Q_{AV-h})	12.78 (Q_{AV-m})	0.213 (Q_{AV-s})
	Hourly Maximum (Q_{HM})	36,800 (Q_{HM-d})	1,533.3 (Q_{HM-h})	25.56 (Q_{HM-m})	0.426 (Q_{HM-s})
Proposed Project (Inflow Rate=75%)	Daily Average (Q'_{AV})	13,800 (Q'_{AV-d})	575.0 (Q'_{AV-h})	9.58 (Q'_{AV-m})	0.160 (Q'_{AV-s})
	Hourly Maximum (Q'_{HM})	27,600 (Q'_{HM-d})	1,150.0 (Q'_{HM-h})	19.17 (Q'_{HM-m})	0.319 (Q'_{HM-s})

1.3.2 Design Sewage Quality

ITEM	ITEM				
	BOD (mg/L)	SS (mg/L)	T-N (mg/L)	Coliform group (MPN/cm^3)	Oil&Greese (mg/L)
INFLUENT	190	180	45	-	-
EFFLUENT	20	20	20	3,000	10

1.4 System Flow Chart



1.6 Design Criteria

ITEMS	UNIT	Formula or Value	Application
1.6.1 Grit Chamber (For Hourly Maximum Flow)			
(1) Hydraulic Load	$m^3/m^2/day$	-	5,000
(2) Retention Time	sec	-	15
1.6.2 Distribution Chamber (For Hourly Maximum Flow)			
(1) Retention Time	min		2
1.6.3 Oxidation Ditch (For Daily Average Flow)			
(1) Hydraulic Retention Time (HRT)	hour	24.0 - 48.0	24.0
(2) MLSS Concentration	mg/l	3,000 -4,000	3,000
(3) BOD-SS Load (Reference only)	kg/kg/day	0.03 - 0.05	-
(4) Aerobic Sludge Retention Time(ASRT)	day	Calculation	6.0
* Necessary ASRT for nitrification on the condition of water temperature 20 degree.			
ASRT > 40.7*e(-0.101T) [T=Water Temp.]			
1.6.4 Final Sedimentation Tank (For Daily Average Flow)			
(1) Hydraulic Load	$m^3/m^2/day$	8.0-12.0	10
(2) Minimum Settling Time	hour	6.0-12.0	6.0-12.0
(3) Water Depth	m	3.0-4.0	3.5
(4) Weir Loading	$m^3/m/day$	150	150
1.6.5 Chlorine Disinfection (for Daily Average Flow)			
(1) Retention (Chlorination) Time	min	15	15
1.6.6 Treated Water Tank			
(1) Retention Time	min		10
1.6.7 Utility Water Tank			
(1) Retention Time	day		1
1.6.8 Wastewater Tank (For Daily Average Flow)			
(1) Retention Time	hour	-	2

2. CAPACITY CALCULATION

2.1 Grit Chamber

2.1.1 Design Condition

(1) Design Flow (For Hourly Maximum) $Q_{HM\text{-}d}$ = 36,800 m^3/day

$Q_{HM\text{-}s}$ = 0.426 m^3/sec

2.1.2 Design Criteria

(1) Hydraulic Load WSL ≤ 5,000 m^3/m^2/day

(2) Retention Time T≧ 15 sec

2.1.3 Capacity Calculation

ITEM	SYMBOL	DESIGN			
Structure					
Type		Vortex Circle Radiation-Flow Type			
Required Surface Area	RSA1	$Q_{HM\text{-}d}$ / WSL ≥	7.4 m^2		
Channel Number	CN		2 chamber		
Diameter	D	$(RSA1/CN \times 4/\pi)^{1/2}$=	2.16 m2 adopt	3.7 m	
Depth	H	$Q_{HM\text{-}s} \times T / (D^2 \times \pi / 4)$=	0.59 m adopt	0.6 m	
Check					
Hydraulic Load		$Q_{HM\text{-}d} / (D^2 \times \pi/4) / CN$ =	1,711 m^3/m^2/day		
			Less than	5,000	…OK
Retention Time		$(D^2 \times \pi / 4 \times H) \times CN / Q_{HM\text{-}s}$ =	15 sec		
			More than	15	…OK

2.1.4 Result

Dimension **Diameter 3.7m x Depth 0.6m x 2 chamber**

*Construction in the Proposed project **Ditto**

Note : The grit chamber dimensions shall be reviewed by the contractor based on the equipment specification.

2.2 Distribution Tank

2.2.1 Design Condition

(1) Design Flow (For Hourly Maximum) $Q_{HM\text{-}d}$ = 36,800 m^3/day

$Q_{HM\text{-}m}$ = 25.56 m^3/min

2.2.2 Design Criteria

(1) Retention Time T ≥ 2 min

2.2.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Structure		
Type	-	Rectangular Type
Required Volume	V	$Q_{HM\text{-}m} \times T \geq$ 51.1 m^3
Basin	BN	1 basin
Width	W	5.0 m
Length	L	3.0 m
Water Depth	H	3.5 m
Check		
Tank Volume	V	$W \times L \times H =$ 53 m^3
Retention Time		$V \times BN / Q_{HM\text{-}m}$ = 2.1 min
		More than 2 …OK

2.2.4 Result

Dimension **Width 5m x Length 3m x Depth 3.5m x 1 tank**

*Construction in the Proposed project **Ditto**

2.3 Oxidation Ditch

2.3.1 Design Condition

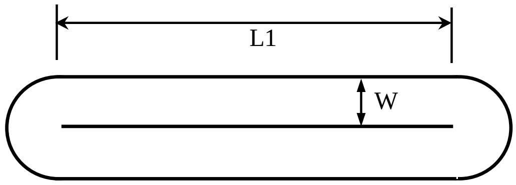
(1) Design Flow (for Daily Average) $Q_{AV\text{-}d}$ = 18,400 m^3/day

2.3.2 Design Criteria

(1) Hydraulic Retention Time (HRT) HRT ≥ 24 hours

2.3.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Structure		
Type	-	Oval Type
Tank Number	TN	4 tanks
Required Volume par Tank	V	$Q_{AV\text{-}D}$ × HRT / 24 = 18,400 m^3
Water Depth	H	6.0 m
Width	W	6.0 m
Section Area	A	W × H = 36.0 m^2
Required Length	L	V / A / TN = 127.8 m
		adopt 128 m
	L1	(L - W × π) / 2 = 54.6 m
		adopt ⌊* 1.1⌋ (Considering Haunches etc) 60.0 m



ITEM	SYMBOL	DESIGN
Check		
Hydraulic Retention Time	HRT	(A × L × TN) / $Q_{AV\text{-}d}$ ×24 = 24.0 hr
		More than 24 …OK
BOD-SS loading	BSS_L	($Q_{AV\text{-}d}$ × BODin) / (A × L × TN × Xa) = 0.0632 kg/kg/day
		*Ref. only 0.05 …OK
	BOD_{in}	Inflow BOD Concentration 190 mg/l
	Xa	MLSS Concentration 3000 mg/l
Sludge Retention Time	SRT	HRT/24×Xa / (a×S-BODin + b×SSin + c×HRT/24×Xa) =
		20.80 day
Aerobic Sludge Retention Time	ASRT	SRT× Aerobic Time/24 = 10.40 day
		(Aerobic Time : 12hr / day) More than 6.0 …OK
	$S\text{-}BOD_{in}$	Inflow S-BOD (: Solved BOD) Concentration
		BODin×0.67 = 127.3 mg/l
	SS_{in}	Inflow SS Concentration 180 mg/l
	a	Sludge converting ratio of solved BOD
		0.5 mgMLSS/mgBOD
	b	Sludge converting ratio of SS
		0.95 mgMLSS/mgSS
	c	Sludge reduction ratio caused by endogenous respiration
		0.03 (1/day)

Nitrogen Removal		
Denitrification Rate Coefficient	K_{DN}	$7.7 \times BSS_L + 0.6 =$ 1.087 mgN/gMLSS/hr
MLSS Quantity	Mq	$(Xa \times A \times L \times TN) \times 10^{-3} \times 0.6=$ 33,178 kgMLSS
		0.6 : Anoxic zone rate (assumption)
Denitrification Rate	K_D	$K_{DN} \times Mq \times 10^{-3} =$ 36.06 kgN/hr
Removal Denitrification Quantity Biologically		
	$L_{TN,DN}$	$K_D \times 12hr =$ 433 kgN/day
Removal Denitrification Quantity as Waste Sludge		
	$L_{TN,w}$	$Q_{AV\text{-}d} \times (SSin - SSout) \times 10^{-3} \times 7$ 9 206 kgN/day
		SS in : Inlet SS concentration 180 mg/l
		SS out : Outlet SS concentration 20 mg/l
Total Removal Denitrification Quantity		
	$L_{TN,T}$	$L_{TN,DN} + L_{TN,w} =$ 639 kgN/day
Inlet T-N Quantity	$L_{TN,in}$	$Q_{AV\text{-}d} \times C_{TN,in} \times 10^{-3} =$ 828 kgN/day
		$C_{TN,in}$: Inlet T-N concentration 45 mg/l
Effluent T-N		$(L_{TN,in} - L_{TN,T}) / Q_{AV\text{-}d} \times 10^{3} =$ 10.3 mg/L
		Less than 20.0 …OK

2.3.4 Result

Dimension **Width 6m x Length(L1) 60m x Depth 6m x 4 tanks**

*Construction in the Proposed project **3 tanks of 4 tanks**

2.4 Final Sedimentation Tank

2.4.1 Design Condition

(1) Design Flow (For Daily Average) $Q_{AV\text{-}d}$ = 18,400 m^3/day

2.4.2 Design Criteria

(1) Hydraulic Load L ≤ 10 m^3/m^2/day

(3) Water Depth h = 3.5 m

2.4.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Structure		
Type	-	Circular Radiation-Flow Type
Tank number	TN	4 basins
Required Surface Area	A	$Q_{AV\text{-}d}$ / L ≥ 1,840 m^2
Diameter	D	$(A/TN \times 4/\pi)^{1/2}$= 24.2 m adopt 25.0 m
Water Depth	H	3.5 m
Weir Length	L1	(D-1.0) × π ×TN = 301.6 m
Check		
Hydraulic Load		$Q_{AV\text{-}d}$ / ($D^2 \times \pi$ / 4) / TN = 9.4 m^3/m^2/day
		Less than 10 …OK
Settling Time		$(D^2 \times \pi / 4) \times H \times TN \times 24/ Q_{AV\text{-}d}$ = 8.96 hr
		Approx. 6.0-12.0 …OK
Weir Loading		$Q_{AV\text{-}d}$ / L1 = 61 m^3/m/day
		Less than 150 …OK

2.4.4 Result

Dimension **Diameter 25m x Depth 3.5m x 4 basins**

*Construction in the Proposed project **3 basins of 4 basins**

2.5 UV Disinfiction Tank

2.5.1 Design Condition

(1) Design Flow (For Hourly Maximum) $Q_{HM\text{-}d}$ = 36,800 m^3/day

2.5.2 Capacity Calculation

ITEM	SYMBOL	DESIGN
See Mechanical Equipment calculation.		
Width	D	1.0 m
Length	L	10 m
Line	N	2 Lines

2.5.3 Result

Dimension **Width 1m x Length 10m x2Lines**

*Construction in the Proposed project **Ditto**

2.6 Chlorine Disinfiction Tank for Backup disinfection

2.6.1 Design Condition

(1) Design Flow (For Daily Average) $Q_{AV\text{-}d}$ = 18,400 m^3/day

$Q_{AV\text{-}m}$ = 12.78 m^3/min

2.4.2 Design Criteria

(1) Retention Time T1 $\geq$ 15 min

2.5.2 Capacity Calculation

ITEM	SYMBOL	DESIGN
Check the capacity of Ocean Outfall as a Disinfection Tank		
Diameter	D	0.7 m
Length	L	1,600 m
Capacity	Ca	$(D^2 \times \pi / 4) \times L$ = 615.8 m^3
Check		
Clorination Time	CT	Ca / $Q_{AV\text{-}m}$ = 48.2 min
		More than 15.0 …OK

2.5.3 Result

Dimension **Diameter 0.7m x Length 1600m**

(Utilize the Ocean Outfall as a disinfection Tank)

*Construction in the Proposed project **Ditto**

2.7 Utility Water

2.7.1 Treated Water Tank

2.7.1.1 Design Condition

(1) Deforming Water Supply Amount Q_T = 0.5 m^3/min See Mechanical Equipment Calculation)

2.7.1.2 Design Criteria

(1) Retention Time T1 ≥ 10 min

2.7.1.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Treated Water Tank		
Type		RC Rectangular Tank
Unit Number	UN1	1 unit
Required Tank Volume	Vr	Q_T × T1 / UN1 = 5.0 m^3
Width	W1	3.0 m
Length	L1	6.0 m
Depth	H1	1.0 m
Tank Volume	V_T	18.0 m^3
Check		
Retention Time		V_T × UN1 / Q1 = 36.0 min
		More than 10 …OK

2.7.1.4 Result

Dimension **Width 3m x Length 6m x Depth 1m (18m3) x 1 unit**

*Construction in the Proposed project **Ditto**

2.7.2 Utility Water Tank

2.7.2.1 Design Condition

(1) Utility Water Q4 = 45.0 m^3/day (See Mechanical Equipment Calculation)

2.7.2.2 Design Criteria

(1) Retention Time T2 ≥ 1 day

2.7.2.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Utility Water Tank		
Type		RC Rectangular Tank
Unit Number	UN4	1 unit
Required Tank Volume	Vo	Q4 x T3 / UN4 = 45.0 m^3
Width	W4	4.6 m
Length	L4	2.5 m
Depth	H4	4.5 m
		adopt 51.8 m^3
Check		
Retention Time		(W4 x H4 x L4 x UN4) / Q4 = 1.2 min
		More than 1 …OK

2.7.2.4 Result

Dimension **Width 4.6m x Length 2.5m x Depth 4.5m (52m3) x 1 unit**

*Construction in the Proposed project **Ditto**

2.8 Wastewater Tank

2.8.1 Design Condition

(1) Wastewater from Dewatering Unit Q_D = 774.0 m^3/day (From Material Balance Calculation)

Q_H = 32.3 m^3/hour

2.8.2 Design Criteria

(1) Retention Time T1 $\geq$ 2 hour

2.8.3 Capacity Calculation

ITEM	SYMBOL	DESIGN
Treated Water Tank		
Type		RC Rectangular Tank
Unit Number	UN1	1 unit
Required Tank Volume	Vr	Q_H × T1 / UN1 = 64.5 m^3
Width	W1	5.0 m
Length	L1	5.0 m
Depth	H1	4.5 m
Tank Volume	V_T	112.5 m^3
Check		
Retention Time		V_T × UN1 / Q1 = 3.5 hour
		More than 2 …OK

2.8.4 Result

Dimension **Width 5m x Length 5m x Depth 4.5m (113m3) x 1 unit**

*Construction in the Proposed project **Ditto**

Material Balance Calculation (Secondary Sedimentation Tank + Direct Dewatering [Multi-diskplate Screw Pres

Table-1 Input Data

1. Calculation Manner		1	1:Premise that the quality of supernatants are same level removed with inlet sewage 2:Premise that the entire supernatants are removed at treatment process
2. Selection of Treatment Efficiency		1	1:Total Removal Ratio 2:Outlet Water Quality (input 1or2)
In case of 1 : input data		90	(%)
In case of 2 : input data			(mg/l)
3. Excess Sludge Generation		1	1:Consideration of Solid Matter Only 2:Consideration of Converting of Solved BOD (input 1or2)
In case of 1:Input data (Sludge generatio		75	Sludge generation ratio per removal SS(%)
In case of 2:Input data	a		T2=Q2・S2=(a・S_{BOD}+b・S1-c・θ・XA)・Q1/10^6・・(Excess sludge generation formula)
	b		a:Converting ratio of solved BOD(mgMLSS/mgBOD)
	c		b:Converting ratio of SS(mgMLSS/mgSS)
	SBOD		c:Sludge reduction ratio caused by endogenous respiration of activated sludge(1/day)
	XA		S_{BOD}:Solved BOD quality at inlet to reactor
	θ		XA:MLSS concentration(mg/l)
			θ:Hydraulic retention time(day)

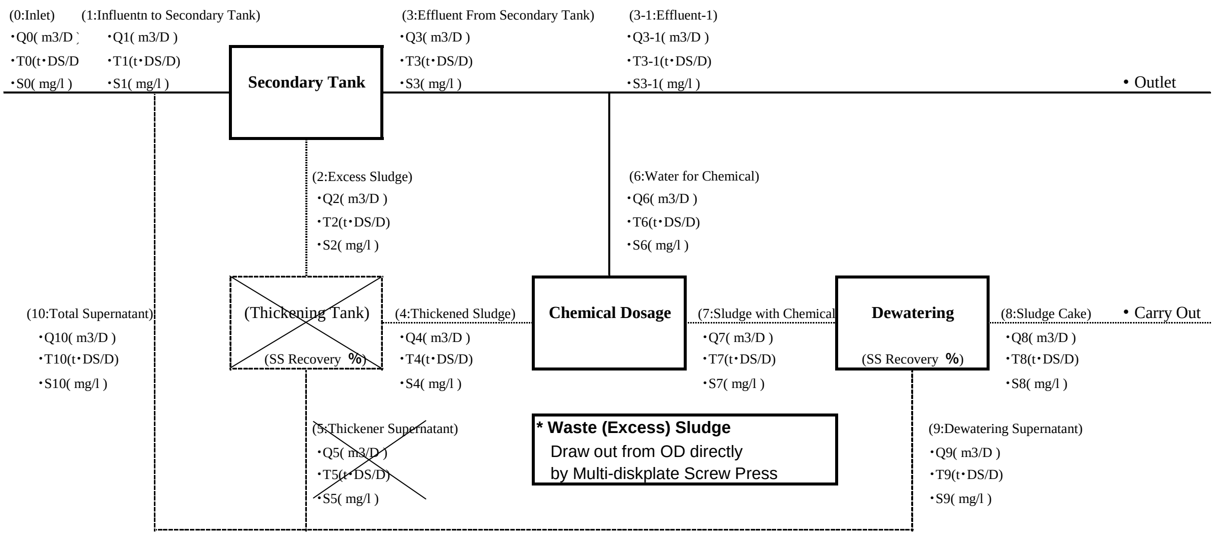
Table-2 Basic Conditions

Water Flow and Quality		Sludge Moisture and Recovery Ratio		Chemical Conditions for Dewatering	
・Inlet flow : Q0(m3/D)	18400	・Excess sludge moisture ratio : W1(%)	99.7	・Chemical dosage : A4(%)	1.50
・Inlet quality : S0(mg/l)	180	・Thickened sludge moisture ratio : W2(	99.7	・Chemical dissolve concentration : A5(	0.20
・Total removal ratio : A1(%)	90.0	・Dewatered sludge moisture ratio : W3(	85.0		
・Effluent quality : St(mg/l)	-	・Recovery ratio in sludge thickener : A2	100.0		
・Sludge generation ratio per removal S	75	・Recovery ratio in dewatering : A3(%)	95.0		

Table-3 Material Balance Calculation

	0	1	2	3	4	~~5~~	6	7	8	9	10	3-1
Q(m3/day)	18,400	19,174	772	18,595	772	0	17	789	15	774	774	18,578
T(t・DS/day)	3.312	3.429	2.315	0.343	2.315	0.000	0.035	2.350	2.232	0.117	0.117	0.308
S(mg/l)	180	179	3,000	18	3,000	#DIV/0!	2,000	2,978	150,000	152	152	18
X(Ti/T0*100)	100	103.5	69.9	10.4	69.9	0.0	1.0	70.9	67.4	3.5	3.5	9.3

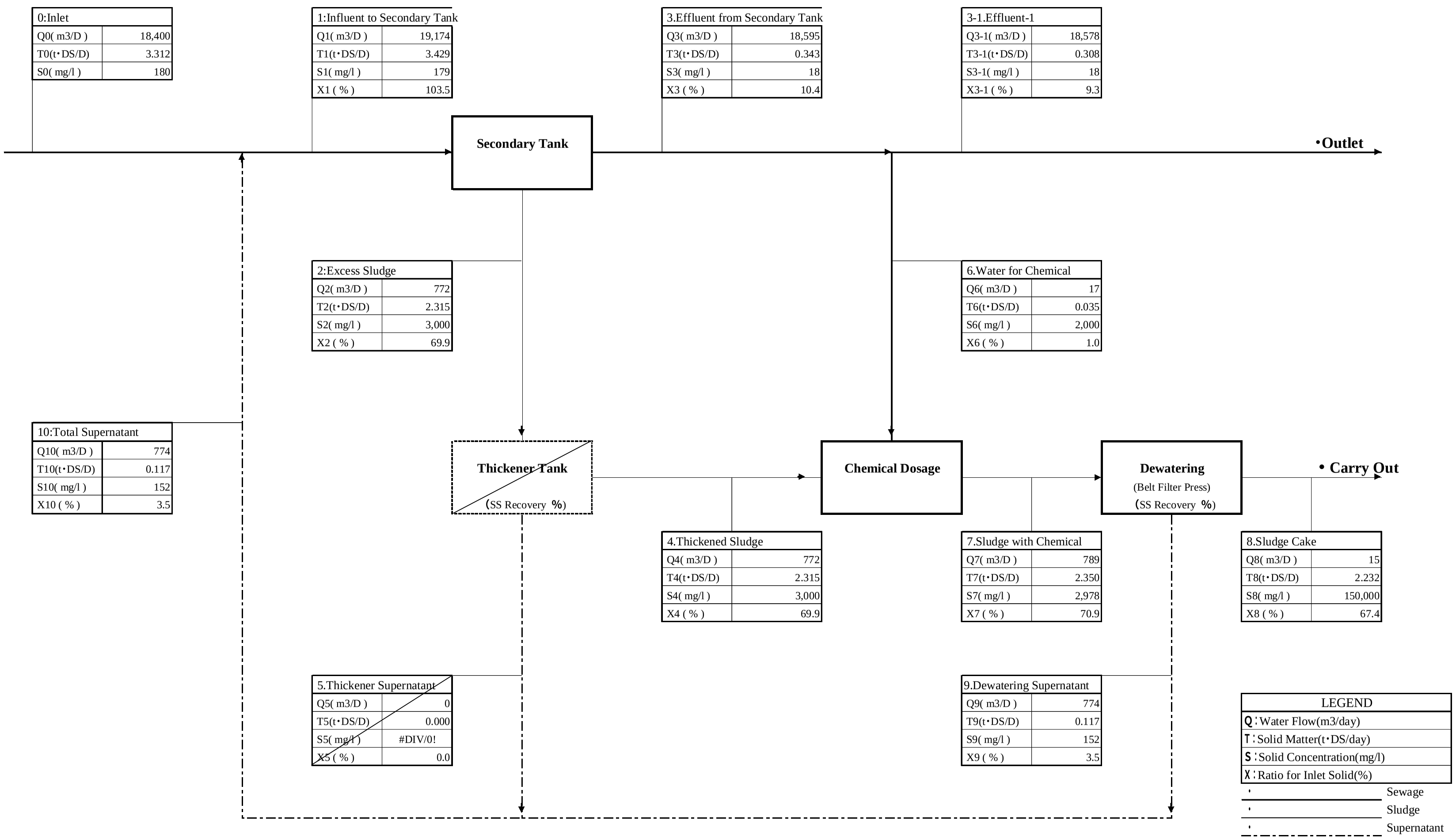
Figure-1 Material Balance Model



Calculation Formula

・Q0=input	・Q3=Q1-Q2*(T1-T3)/T2	・Q6=T4*A4/A5	・Q9=Q7-Q8	・Q3-1=Q3-Q6
・T0=Q0*S0*10^(-6)	・T3=T1*(100-A1)/100	・T6=Q6*S6/10^6	・T9=T7-T8	・T3-1=T3-T6
・S0=input	・S3=T3*10^6/Q3	・S6=10*4*A5	・S9=T9*10^6/Q9	・S3-1=S3
・Q1=Q0+Q10	・Q4=T4*100/(100-W2)	・Q7=Q4+Q6	・Q10=Q5+Q9	
・T1=T0+T10	・T4=T2*A2/100	・T7=T4+T6	・T10=T5+T9	
・S1=T1*10^6/Q1	・S4=10^6*(100-W2)/100	・S7=T7*10^6/Q7	・S10=T10*10^6/Q10	
・Q2=T2*100/(100-W1)	・Q5=Q2-Q4	・Q8 =T8*100/(100-W3)		
・T2=(T1-T3)*Si/100	・T5=T2-T4	・T8 =T7*A3/100		
・S2=10^6*(100-W2)/100	・S5=T5*10^6/Q5	・S8 =10^6*(100-W3)/100		

Material Balance Sheet



SECTION A5: Hydraulic Calculation of STP

Basic Design Conditions

(1) Name of STP Kila Kila STP

(2) Sewage collectioi Separate sewer system

(3) Design sewage flow

Classification		Design flow	
		m^3/d	m^3/s
Design daily average flow	Q（DA）	18,400	0.213
Design daily maximum flow	Q（DM）	18,400	0.213
Design hourly maximum flow	Q（HM）	36,800	0.426
Emergency flow	Q（WW）		

(4) Incoming pipe

Pipe diameter (Force Main)	Dia 600
Slope	-
Invert Level of pipe	+10.000m

(5) Effluent discharg

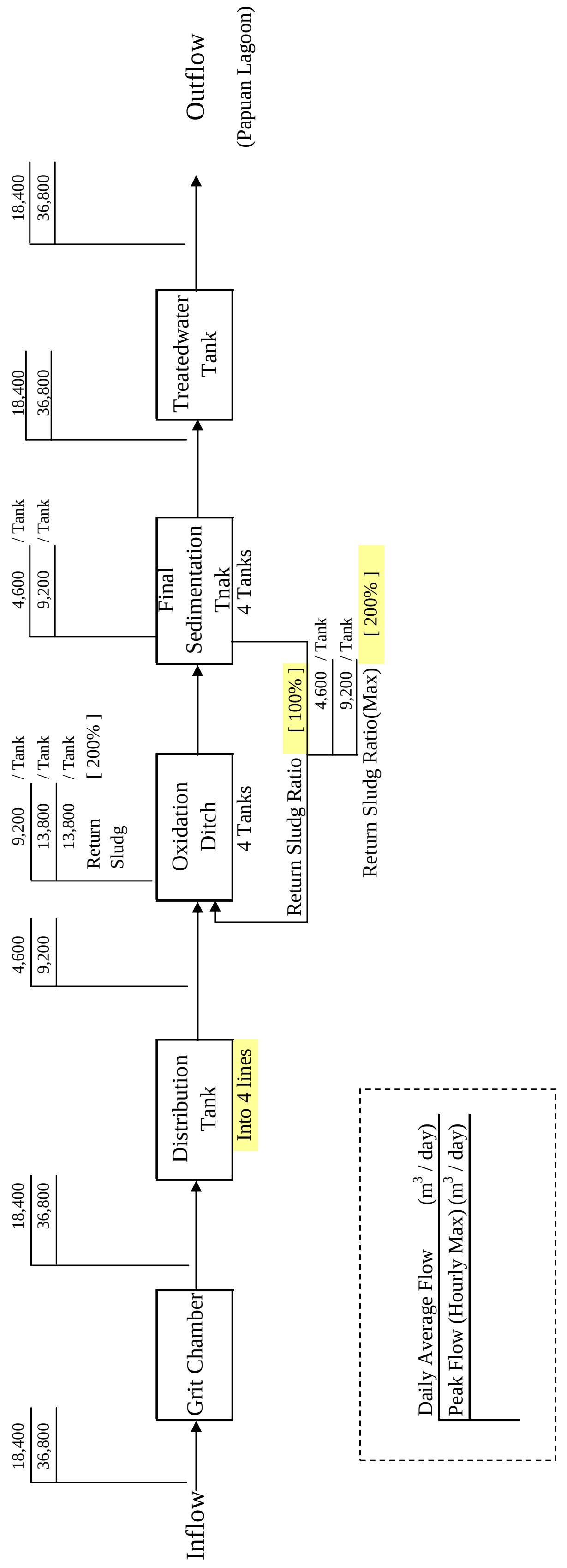
Discharge point/river		Papuan Lagoon
High water level (HWL) [High Tide Level]		+0.250m
Maximum high water level (HHWL) [Design water lev		+1.000m
Sea bed level (Current)	Approx.	-25.000m

(6) Design ground level of STP

+7.000m

+10.000m

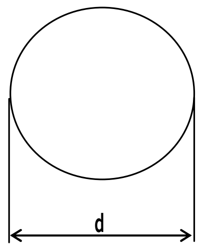
Balance Sheet of Sewage Flow



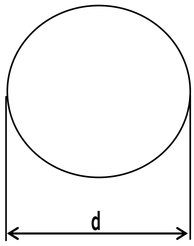
Summary Sheet for Hydraulic Calculation						
Sheet No.	Facility	Upstream water level (m)				Bottom level of facility (m)
		Daily ave. H(DA)	Daily max. H(DM)	Hourly max. H(HM)		
01	Effluent Discharge Pipe (Ocean Outfall-1) : FRPM φ800	2.100	2.100	2.800		
02	Effluent Discharge Pipe (Ocean Outfall-2) : HDPE φ700	2.500	2.500	4.100		
03	Treated water Tank : Measuring Weir	5.090	5.090	5.190		4.900
04	UV Disinfection : Inlet Gate Opening	6.060	6.060	6.160		5.600
05	Clarifier : Outlet pipe (2/2) HDPE φ(	6.220	6.220	6.770		
06	Clarifier : Outlet pipe (1/2) HDPE φ3	6.260	6.260	6.900		
07	Clarifier : Outlet trough 600W	7.410	7.410	7.470		7.300
08	Clarifier : Weir (V-notch)	8.000	8.000	8.010		7.980
09	Clarifier : Inlet pipe φ450	8.130	8.290	8.300		
10	Oxidation Ditch : Outlet weir 2000w	10.150	10.180	10.180		10.050
11	Oxidation Ditch : Inlet Pipe φ350	10.200	10.230	10.370		
12	Distribution Tank : Distribution Weir	10.910	10.910	10.980		10.800
13	Distribution Tank : Inlet Pipe φ600	10.990	10.990	11.300		
14	Grit Chamber : Measuring Weir	11.690	11.690	11.790		11.500
15	Grit Chamber : Grit Chamber Weir	11.900	11.900	11.950		11.800
16	Grit Chamber : Inlet Channel	12.060	12.060	12.120		11.300
17	Grit Chamber : Grit Chamber Gate Opening	12.130	12.130	12.270		11.800

NO.1

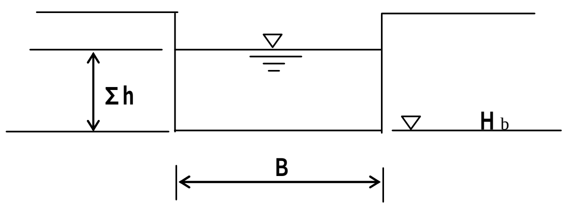
Facility	Effluent Discharge Pipe (Ocean Outfall-1) : **FRPM φ800**					Formula : Hazen Williams
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions						
Velocity coefficie	c	110				
Pipe diameter	d	0.800 m				
Pipe length	L	888.0 m				
Cross-section area	AW	0.503 m^2				
Downstream water level (m)	Ho	1.000	1.000	1.000		
Friction loss (m)	h_f	0.269	0.269	0.969		$h_f=10.666\times(Q/C)^{1.85}\times D^{-4.87}\times L$
Flow velocity (m/s)	V	0.424	0.424	0.848		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.009	0.009	0.037		
Inlet loss (m)	h_i	0.000	0.000	0.000		f = n =
Outlet loss (m)	h_o	0.009	0.009	0.037		f = 1.00 n = 1
Bend loss (m)	h_b	0.000	0.000	0.000		f = n =
Other losses	h_e	0.000	0.000	0.000		f = n =
Water head by specific gravity of seawater (1.03) (m)	h_d	0.750	0.750	0.750		h_d = D×(1.03-1.00) D: Depth at discharge point 25m
Total (m)	∑h	1.028	1.028	1.755		
Head loss (m)	h	1.100	1.100	1.800		Round up to 0.1m
Upstream water level (m)	H	2.100	2.100	2.800		$H=H_0+h$



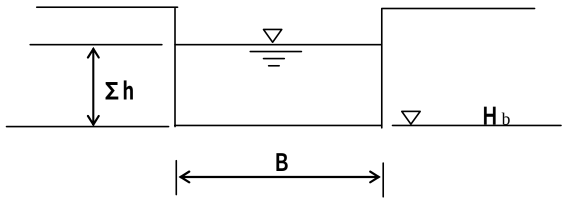
NO.2

Facility		**Effluent Discharge Pipe (Ocean Outfall-2) : HDPE φ700**				Formula : Hazen Williams
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions						
Velocity coefficie	c	130				
Pipe diameter	d	0.700 m				
Pipe length	L	715.0 m				
Cross-section area	AW	0.385 m^2				
Downstream water level (m)	Ho	2.100	2.100	2.800		
Friction loss (m)	h_f	0.304	0.304	1.097		$h_f=10.666\times(Q/C)^{1.85}\times D^{-4.87}\times L$
Flow velocity (m/s)	V	0.553	0.553	1.107		$V=q/AW$
Velocity head (m)	$\frac{V^2}{2g}$	0.016	0.016	0.063		
Inlet loss (m)	h_i	0.000	0.000	0.000		f = n =
Outlet loss (m)	h_o	0.016	0.016	0.063		f = 1.00 n = 1
Bend loss (m)	h_b	0.024	0.024	0.098		f = 0.13 n = 12
Other losses	h_e	0.000	0.000	0.000		f = n =
Total (m)	$\sum h$	0.344	0.344	1.257		
Head loss (m)	h	0.400	0.400	1.300		Round up 0.1m
Upstream water level (m)	H	2.500	2.500	4.100		$H=H_0+h$

NO.3

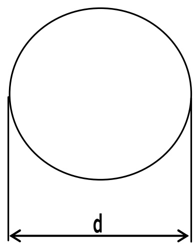
Facility	**Treated water Tank : Measuring Weir**					Formula : Francis / Full Width weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of openings	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions (for opening) Width Height Cross-section area	 B H_b	 1.50 m +4.900 m	 			
Downstream water level (m)	H_0	2.500	2.500	4.100		
Weir height (m)	H_b	4.900	4.900	4.900		
Downstream depth above weir	h_2					$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.181	0.181	0.288		$h_1=(q / (B \times 1.84))^{2/3}$
Total (m)	$\sum h$	0.181	0.181	0.288		
Head loss (m)	h	0.190	0.190	0.290		Round up to cm
Upstream water level (m)	H	5.090	5.090	5.190		$H=H_0+h$

NO.4

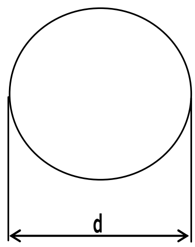
Facility		UV Disinfection : Inlet Gate Opening				Formula : Francis / Submerged weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	2	2	2		
Unit flow (m^3/s)	q	0.107	0.107	0.213		
Dimensions Weir width Weir height	 B H_b	 0.40 m +5.600 m				
Downstream water level (m)	H_0	6.000	6.000	6.000		From UV Disinfection Equipmen
Weir height (m)	H_b	5.600	5.600	5.600		
Downstream depth above weir	h_2	0.400	0.400	0.400		$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.055	0.055	0.160		$B=q / (1.84(h_1+1.4h_2)\sqrt{h_1})$
Total (m)	$\sum h$	0.455	0.455	0.560		
Head loss (m)	h	0.460	0.460	0.560		Round up to cm
Upstream water level (m)	H	6.060	6.060	6.160		$H=H_0+h$

NO.5

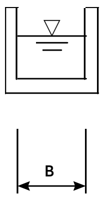
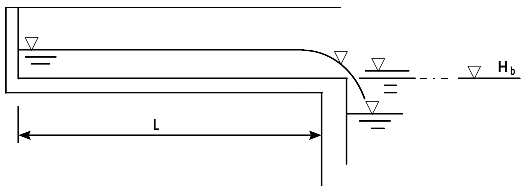
Facility	**Clarifier : Outlet pipe (2/2) HDPE φ600**					Formula : Darcy Weisbach
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions						
Pipe diameter	d	0.600 m				
Cross-section area	AW	0.283 m²				
Pipe length	L	125.00 m				
Friction loss coef.	f	0.019				
Downstream water level (m)	Ho	6.060	6.060	6.160		
Flow velocity (m/s)	V	0.753	0.753	1.507		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.029	0.029	0.116		
Straight pipe loss (m)	h_{f1}	0.115	0.115	0.458		h_{f1}=f×(L/d)×(v^2/2g)
Inlet loss (m)	h_i					f = n =
Outlet loss (m)	h_o	0.029	0.029	0.116		f = 1.00 n = 1
Bend loss (m)	h_b	0.008	0.008	0.030		f = 0.13 n = 2
Other losses	h_e					f = n =
Total (m)	∑h	0.152	0.152	0.604		
Head loss (m)	h	0.160	0.160	0.610		Round up to cm
Upstream water level (m)	H	6.220	6.220	6.770		H=Ho+h



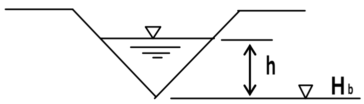
NO.6

Facility	**Clarifier : Outlet pipe (1/2) HDPE φ350**					Formula : Darcy Weisbach
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	4	4	4		
Unit flow (m^3/s)	q	0.053	0.053	0.107		
Dimensions						
Pipe diameter	d	0.355 m				
Cross-section area	AW	0.099 m^2				
Pipe length	L	10.00 m				
Friction loss coef.	f	0.019				
Downstream water level (m)	Ho	6.220	6.220	6.770		
Flow velocity (m/s)	V	0.535	0.535	1.081		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.015	0.015	0.060		
Straight pipe loss (m)	h_{f1}	0.008	0.008	0.032		$h_{f1}=f\times(L/d)\times(v^2/2g)$
Inlet loss (m)	h_i	0.007	0.007	0.030		f = 0.50 n = 1
Outlet loss (m)	h_o	0.015	0.015	0.060		f = 1.00 n = 1
Bend loss (m)	h_b	0.002	0.002	0.008		f = 0.13 n = 1
Other losses	h_e					f = n =
Total (m)	$\sum h$	0.032	0.032	0.130		
Head loss (m)	h	0.040	0.040	0.130		Round up to cm
Upstream water level (m)	H	6.260	6.260	6.900		H=Ho+h

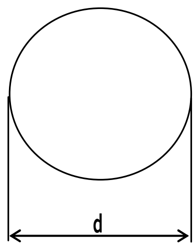
NO.7

Facility		**Clarifier : Outlet trough 600W**				Formula : Thomas Camp
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	8	8	8		4 Clarifiers x 2 Troughs (2 Way)
Unit flow (m^3/s)	q	0.027	0.027	0.053		
Dimensions Trough width Trough length Trough slope Trough bottom level	 B L i H_b	 0.60 m 40.00 m 0.00 ‰ +7.300 m				
Downstream water level (m)	H_0	6.260	6.260	6.900		
Trough bottom level (m)	H_b	7.300	7.300	7.300		
Water depth at end of trough	h_1					
Critical depth (m)	h_{cl}	0.059	0.059	0.093		
Water depth at start of trough	h_0	0.102	0.102	0.161		
Friction loss (m)	h_f	0.102	0.102	0.161		$i \times L + h_o$
Total (m)	$\sum h$	0.102	0.102	0.161		
Head loss (m)	h	0.110	0.110	0.170		Round up to cm
Upstream water level (m)	H	7.410	7.410	7.470		$H=H_0+h$

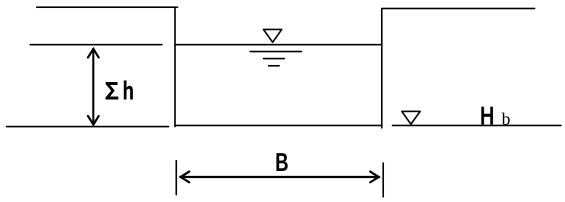
NO.8

Facility	**Clarifier : Weir (V-notch)**					Formula : Thomson Right-angle triangular weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	2512	2512	2512		N = φ25m×π×4×8unit/m
Unit flow (m^3/s)	q	$\times 10^{-4}$ 0.848	$\times 10^{-4}$ 0.848	$\times 10^{-4}$ 1.696		
Dimensions						
Notch level	H_b	+7.980 m				
Downstream water level (m)	H_0	7.410	7.410	7.470		
Notch level (m)	H_b	7.980	7.980	7.980		
Overflow depth (m)	h	0.020	0.020	0.027		$h=(q/1.42)^{2/5}$
Total (m)	∑h	0.020	0.020	0.027		
Head loss (m)	h	0.020	0.020	0.030		Round up to cm
Upstream water level (m)	H	8.000	8.000	8.010		$H=H_0+h$

NO.9

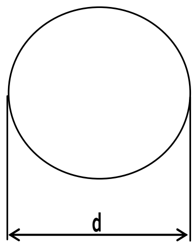
Facility		**Clarifier : Inlet pipe φ450**				Formula : Darcy Weisbach
Item	Sign	Daily ave. Q(DA)	Hourly max. Q(HM)	Return Sludge max. Q(RS200)		Remarks
Flow (m^3/s)	Q	0.426	0.639	0.639		Q(DA) =0.213+0.213×100% = 0.42 Q(HM) =0.426+0.213×100% = 0.63
Number of units	N	4	4	4		Q(RS200) =0.213+0.213×200% = 0.639
Unit flow (m^3/s)	q	0.107	0.160	0.160		
Dimensions						
Pipe diameter	d	0.450 m				
Cross-section area	AW	0.159 m²				
Pipe length	L	75.00 m				
Friction loss coef.	f	0.019				
Downstream water level (m)	Ho	8.000	8.000	8.010		
Flow velocity (m/s)	V	0.673	1.006	1.006		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.023	0.052	0.052		
Straight pipe loss (m)	h_{f1}	0.073	0.164	0.164		h_{f1}=f×(L/d)×(v^2/2g)
Inlet loss (m)	h_i	0.012	0.026	0.026		f = 0.50 n = 1
Outlet loss (m)	h_o	0.023	0.052	0.052		f = 1.00 n = 1
Bend loss (m)	h_b	0.018	0.041	0.041		f = 0.20 n = 4
Other losses	h_e					f = n =
Total (m)	∑h	0.126	0.283	0.283		
Head loss (m)	h	0.130	0.290	0.290		Round up to cm
Upstream water level (m)	H	8.130	8.290	8.300		H=Ho+h

NO.10

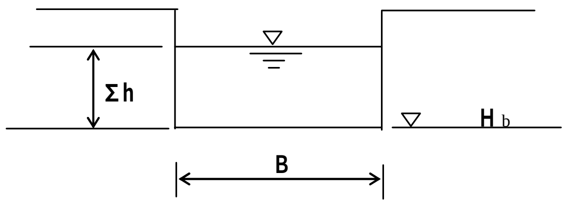
Facility		**Oxidation Ditch : Outlet weir 2000w**				Formula : Francis / Full Width weir
Item	Sign	Daily ave. Q(DA)	Hourly max. Q(HM)	Return Sludge max. Q(RS200)		Remarks
Flow (m^3/s)	Q	0.426	0.639	0.639		Ditto
Number of units	N	4	4	4		
Unit flow (m^3/s)	q	0.107	0.160	0.160		
Dimensions Weir width Weir height	 B H_b	 2.00 m +10.050 m				
Downstream water level (m)	H_0	8.130	8.290	8.300		
Weir height (m)	H_b	10.050	10.050	10.050		
Downstream depth above weir	h_2					$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.095	0.124	0.124		$h_1=(q / (B \times 1.84))^{2/3}$
Total (m)	$\sum h$	0.095	0.124	0.124		
Head loss (m)	h	0.100	0.130	0.130		Round up to cm
Upstream water level (m)	H	10.150	10.180	10.180		$H=H_0+h$

NO.11

Facility		Oxidation Ditch : Inlet Pipe φ350			Formula : Darcy Weisbach	
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	4	4	4		
Unit flow (m^3/s)	q	0.053	0.053	0.107		
Dimensions						
Pipe diameter	d	0.355 m				
Cross-section area	AW	0.099 m^2				
Pipe length	L	20.00 m				
Friction loss coef.	f	0.019				
Downstream water level (m)	Ho	10.150	10.180	10.180		
Flow velocity (m/s)	V	0.535	0.535	1.081		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.015	0.015	0.060		
Straight pipe loss (m)	h_{f1}	0.016	0.016	0.064		
Inlet loss (m)	h_i	0.007	0.007	0.030		f = 0.50 n = 1
Outlet loss (m)	h_o	0.015	0.015	0.060		f = 1.00 n = 1
Bend loss (m)	h_b	0.009	0.009	0.036		f = 0.20 n = 3
Other losses	h_e					f = n =
Total (m)	∑h	0.047	0.047	0.190		
Head loss (m)	h	0.050	0.050	0.190		Round up to cm
Upstream water level (m)	H	10.200	10.230	10.370		H=Ho+h

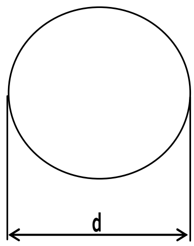


NO.12

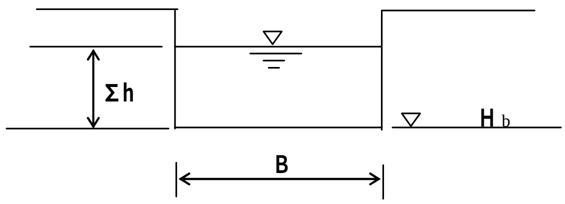
Facility		**Distribution Tank : Distribution Weir**			Formula : Francis / Full Width weir	
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	4	4	4		
Unit flow (m^3/s)	q	0.053	0.053	0.107		
Dimensions Weir width Weir height	 B H_b	 0.80 m +10.800 m				
Downstream water level (m)	H_0	10.200	10.230	10.370		
Weir height (m)	H_b	10.800	10.800	10.800		
Downstream depth above weir	h_2					$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.109	0.109	0.174		$h_1=(q / (B \times 1.84))^{2/3}$
Total (m)	$\sum h$	0.109	0.109	0.174		
Head loss (m)	h	0.110	0.110	0.180		Round up to cm
Upstream water level (m)	H	10.910	10.910	10.980		$H=H_0+h$

NO.13

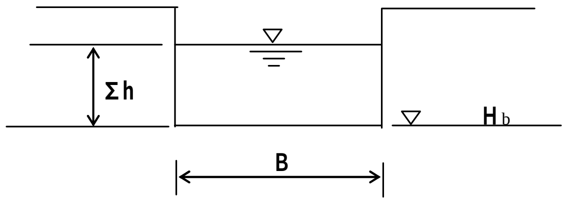
Facility		**Distribution Tank : Inlet Pipe φ600**				Formula : Darcy Weisbach
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions						
Pipe diameter	d	0.600 m				
Cross-section area	AW	0.283 m²				
Pipe length	L	20.00 m				
Friction loss coef.	f	0.019				
Downstream water level (m)	Ho	10.910	10.910	10.980		
Flow velocity (m/s)	V	0.753	0.753	1.507		V=q／AW
Velocity head (m)	$\frac{V^2}{2g}$	0.029	0.029	0.116		
Straight pipe loss (m)	h_{f1}	0.018	0.018	0.073		
Inlet loss (m)	h_i	0.014	0.014	0.058		f = 0.50 n = 1
Outlet loss (m)	h_o	0.029	0.029	0.116		f = 1.00 n = 1
Bend loss (m)	h_b	0.017	0.017	0.069		f = 0.20 n = 3
Other losses	h_e					f = n =
Total (m)	∑h	0.078	0.078	0.316		
Head loss (m)	h	0.080	0.080	0.320		Round up to cm
Upstream water level (m)	H	10.990	10.990	11.300		H=Ho+h



NO.14

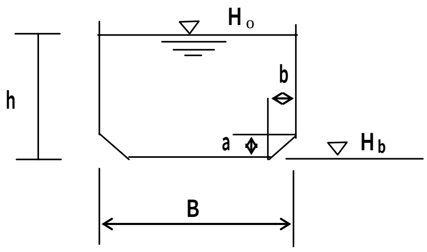
Facility		Grit Chamber : Measuring Weir				Formula : Francis / Full Width weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	1	1	1		
Unit flow (m^3/s)	q	0.213	0.213	0.426		
Dimensions						
Weir width	B	1.50 m				
Weir height	H_b	+11.500 m				
Downstream water level (m)	H_0	10.990	10.990	11.300		
Weir height (m)	H_b	11.500	11.500	11.500		
Downstream depth above weir	h_2					$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.181	0.181	0.288		$h_1=(q / (B \times 1.84))^{2/3}$
Total (m)	$\sum h$	0.181	0.181	0.288		
Head loss (m)	h	0.190	0.190	0.290		Round up to cm
Upstream water level (m)	H	11.690	11.690	11.790		$H=H_0+h$

NO.15

Facility		**Grit Chamber : Grit Chamber Weir**				Formula : Francis / Full Width weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	2	2	2		
Unit flow (m^3/s)	q	0.107	0.107	0.213		
Dimensions Weir width Weir height	 B H_b	 2.00 m +11.800 m				
Downstream water level (m)	H_0	11.690	11.690	11.790		
Weir height (m)	H_b	11.800	11.800	11.800		
Downstream depth above weir	h_2					$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.095	0.095	0.150		$h_1=(q / (B \times 1.84))^{2/3}$
Total (m)	$\sum h$	0.095	0.095	0.150		
Head loss (m)	h	0.100	0.100	0.150		Round up to cm
Upstream water level (m)	H	11.900	11.900	11.950		$H=H_0+h$

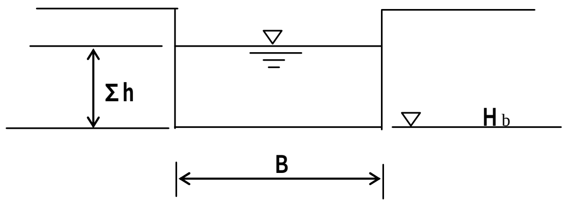
NO.16

Facility		**Grit Chamber : Inlet Channel**				Formula : Manning's formula / Rectangular open channel
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	2	2	2		
Unit flow (m^3/s)	q	0.107	0.107	0.213		
Dimensions						
Roughness coef.	n	0.013				
Channel width	B	0.800 m				
Channel length	L	11.00 m				
Haunch height	a	0.00 m				
Haunch width	b	0.00 m				
Bottom level	H_b	11.300 m				
Downstream water level (m)	H_0	11.900	11.900	11.950		
Channel bottom level (m)	H_b	11.300	11.300	11.300		
Effective depth (m)	h	0.600	0.600	0.650		$h = H_0 - H_b$
Cross-section area (m²)	AW	0.480	0.480	0.520		AW=B×h − (a×b)
Hydraulic radius (m)	R	0.240	0.240	0.248		$R = \frac{AW}{\left(2h + B - 2(a+b) + 2\sqrt{(a^2+b^2)}\right)}$
Flow velocity (m/s)	V	0.223	0.223	0.410		$V = \frac{q}{AW}$
Velocity head (m)	$\frac{V^2}{2g}$	0.003	0.003	0.009		
Hydraulic gradient (‰)	I	0.056	0.056	0.182		$I = (n \cdot V / R^{2/3})^2$
Friction loss (m)	h_f	0.001	0.001	0.002		$h_f = I \times L$
Bend loss (m)	hse	0.001	0.001	0.002		f = 0.13 n = 2
Outlet loss (m)	h_o	0.003	0.003	0.009		f = 1.00 n = 1
Other losses (m)	h_e	0.150	0.150	0.150		Screen Loss
Total (m)	$\sum h$	0.155	0.155	0.163		
Head loss (m)	h	0.160	0.160	0.170		Round up to cm
Upstream water level (m)	H	12.060	12.060	12.120		



NO.17

Facility		**Grit Chamber : Grit Chamber Gate Opening**				Formula : Francis / Submerged weir
Item	Sign	Daily ave. Q(DA)	Daily max. Q(DM)	Hourly max. Q(HM)		Remarks
Flow (m^3/s)	Q	0.213	0.213	0.426		
Number of units	N	2	2	2		
Unit flow (m^3/s)	q	0.107	0.107	0.213		
Dimensions						
Weir width	B	0.50 m				
Weir height	H_b	+11.800 m				
Downstream water level (m)	H_0	12.060	12.060	12.120		
Weir height (m)	H_b	11.800	11.800	11.800		
Downstream depth above weir	h_2	0.260	0.260	0.320		$h_2=H_o-H_b$
Upstream depth (m)	h_1	0.070	0.070	0.150		$B=q / (1.84(h_1+1.4h_2)\sqrt{h_1})$
Total (m)	$\sum h$	0.330	0.330	0.470		
Head loss (m)	h	0.330	0.330	0.470		Round up to cm
Upstream water level (m)	H	12.130	12.130	12.270		$H=H_0+h$



Section B1: Pipe Foundation Calculation

Main Trunk Sewer

Pipe Bed Structural Analysis HDPE Pipe DN = 110 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	96.4	mm	
Outer Pipe Diameter	Bc	:	11.0	cm	
Pipe Thickness (minimum)	t	:	0.68	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	5.160	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2
H : Earth Cover = 0.9 m
γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2
H : Earth Cover = 0.9 m
P : Unit rear tire load = 100.000 kN
a : Tire ground contact width = 0.2 m
C : Width between two rear tires = 2.75 m
θ : Load spreading angle = 45°
i : Shock load coefficient = 0.5
"i" varies depend on earth cover:

	H≦1.5	1.5<H<6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 5.16^2$

$= 0.015$ kN・cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 5.16^2$

$= 0.007$ kN・cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$$= \frac{0.015 \times 10^3}{0.08 \times 10^2}$$

$= 1.946$ N/mm² ≦ **6.4** **N/mm²** OK

《Pipe Bottom》

$$= \frac{0.007 \times 10^3}{0.08 \times 10^2}$$

$= 0.908$ N/mm² ≦ **6.4** **kN/mm²** OK

where:

M : Bending Moment kN・cm/cm

σ : Bending Load kN/mm²

Z : Section Modulus = 0.08 cm³/cm $=t^2/6$

R : Radius in pipe thickness center 5.160 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m²

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$$

$R^4 = 709$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 709}{1000 \times 10^2 \times 0.0262}$$

$= 0.071$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.071}{2 \times 5.16} \times 100$$

$= 0.7$ % $\leqq$ **5** **%** OK

where:

δx : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 5.16 cm

E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.0262 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.028	3.633	0.278	2.7	OK
0.6	70.13	10.80	0.018	2.336	0.181	1.8	OK
0.8	57.75	13.50	0.016	2.076	0.163	1.6	OK
0.9	49.09	16.20	0.015	1.946	0.152	1.5	OK
1.0	44.63	18.00	0.015	1.946	0.147	1.4	OK
1.2	37.76	21.60	0.014	1.817	0.143	1.4	OK
1.5	30.68	27.00	0.014	1.817	0.144	1.4	OK
2.0	23.38	36.00	0.015	1.946	0.154	1.5	OK
2.5	18.88	45.00	0.017	2.206	0.171	1.7	OK
3.0	15.84	54.00	0.019	2.465	0.19	1.8	OK
3.5	13.64	63.00	0.021	2.725	0.212	2.1	OK
4.0	11.97	72.00	0.024	3.114	0.239	2.3	OK
4.5	10.67	81.00	0.026	3.374	0.268	2.6	OK
5.0	9.63	90.00	0.029	3.763	0.297	2.9	OK
5.5	8.77	99.00	0.032	4.152	0.327	3.2	OK
6.0	8.05	108.00	0.035	4.542	0.356	3.4	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 125 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	109.8	mm	
Outer Pipe Diameter	Bc	:	12.5	cm	
Pipe Thickness (minimum)	t	:	0.76	cm	PE-100 PN-10 SDR 17 Poliplex C:
Radius in pipe thickness center	R	:	5.870	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H = 18 \times 0.9 = 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2
H : Earth Cover = 0.9 m
γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2\times P\times(1+i)\times\beta}{C\times(a + 2H\times\tan\theta)} = \frac{2\times100.000\times(1+0.5)\times0.900}{2.75\times(0.2 + 2\times0.900\times\tan 45°)} = 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2
H : Earth Cover = 0.9 m
P : Unit rear tire load = 100.000 kN
a : Tire ground contact width = 0.2 m
C : Width between two rear tires = 2.75 m
θ : Load spreading angle = 45°
i : Shock load coefficient = 0.5
"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca:
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 5.87^2$

$= 0.019$ kN·cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 5.87^2$

$= 0.009$ kN·cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.019 \times 10^3}{0.096 \times 10^2}$

$= 1.974$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$= \dfrac{0.007 \times 10^3}{0.096 \times 10^2}$

$= 0.935$ N/mm^2 ≦ **6.4** **kN/mm^2** OK

where:

M : Bending Moment kN·cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.096 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 5.870 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$ $R^4 =$ 1187

$= \dfrac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 1187}{1000 \times 10^2 \times 0.0366}$

$= 0.085$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.085}{2 \times 5.16} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 5.87 cm

E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.0366 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm², Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.036	3.74	0.333	2.8	OK
0.6	70.13	10.80	0.023	2.389	0.217	1.8	OK
0.8	57.75	13.50	0.021	2.181	0.195	1.7	OK
0.9	49.09	16.20	0.019	1.974	0.182	1.6	OK
1.0	44.63	18.00	0.019	1.974	0.177	1.5	OK
1.2	37.76	21.60	0.018	1.87	0.172	1.5	OK
1.5	30.68	27.00	0.018	1.87	0.172	1.5	OK
2.0	23.38	36.00	0.02	2.078	0.185	1.6	OK
2.5	18.88	45.00	0.022	2.285	0.205	1.7	OK
3.0	15.84	54.00	0.024	2.493	0.228	1.9	OK
3.5	13.64	63.00	0.027	2.805	0.254	2.2	OK
4.0	11.97	72.00	0.03	3.116	0.287	2.4	OK
4.5	10.67	81.00	0.034	3.532	0.322	2.7	OK
5.0	9.63	90.00	0.038	3.947	0.357	3	OK
5.5	8.77	99.00	0.042	4.363	0.392	3.3	OK
6.0	8.05	108.00	0.045	4.675	0.427	3.6	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 160 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	140.6	mm	
Outer Pipe Diameter	Bc	:	16.0	cm	
Pipe Thickness (minimum)	t	:	0.97	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	7.515	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1\times q + k2\times p)\times R^2$

《Pipe Top》

$= (0.107\times 16.200\times 10^{-4} + 0.0799\times 49.091\times 10^{-4})\times 7.515^2$

$= 0.032$ kN･cm/cm

《Pipe Bottom》

$= (0.121\times 16.200\times 10^{-4} + 0.011\times 49.091\times 10^{-4})\times 7.515^2$

$= 0.014$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.032\times 10^3}{0.157\times 10^2}$

$= 2.041$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$= \dfrac{0.014\times 10^3}{0.157\times 10^2}$

$= 0.893$ N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.157 cm^3/cm =$t^2/6$

R : Radius in pipe thickness center 7.515 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k3\times q + k4\times p)\times R^4}{E\times I}$ $R^4 = 3189$

$= \dfrac{(0.070\times 16.200\times 10^{-1}+0.030\times 49.091\times 10^{-1})\times 3189}{1000\times 10^2\times 0.0761}$

$= 0.109$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.109}{2 \times 7.515} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

- δx : Total flexure caused by backfilling soil load and active load cm
- V : Flexure Ratio
- R : Radius in pipe thickness center = 7.515 cm
- E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)
- I : Geometric Moment of Inertia per 1 cm pipe length = 0.0761 cm^4/cm $=t^3/12$
- q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2
- p : Active Load = 49.091 kN/m^2
- k1 : Flexure factor for backfilling soil
- k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.059	3.762	0.431	2.9	OK
0.6	70.13	10.80	0.038	2.423	0.281	1.9	OK
0.8	57.75	13.50	0.034	2.168	0.252	1.7	OK
0.9	49.09	16.20	0.032	2.041	0.235	1.6	OK
1.0	44.63	18.00	0.031	1.977	0.229	1.5	OK
1.2	37.76	21.60	0.03	1.913	0.222	1.5	OK
1.5	30.68	27.00	0.03	1.913	0.223	1.5	OK
2.0	23.38	36.00	0.032	2.041	0.239	1.6	OK
2.5	18.88	45.00	0.036	2.296	0.264	1.8	OK
3.0	15.84	54.00	0.04	2.551	0.295	2	OK
3.5	13.64	63.00	0.044	2.806	0.328	2.2	OK
4.0	11.97	72.00	0.05	3.188	0.371	2.5	OK
4.5	10.67	81.00	0.056	3.571	0.416	2.8	OK
5.0	9.63	90.00	0.062	3.954	0.461	3.1	OK
5.5	8.77	99.00	0.068	4.336	0.506	3.4	OK
6.0	8.05	108.00	0.074	4.719	0.552	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 225 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	197.6	mm	
Outer Pipe Diameter	Bc	:	22.5	cm	
Pipe Thickness (minimum)	t	:	1.37	cm	PE-100 PN-10 SDR 17 Poliplex C.
Radius in pipe thickness center	R	:	10.565	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H = 18\times0.9 = 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2\times P\times(1+i)\times\beta}{C\times(a + 2H\times\tan\theta)} = \frac{2\times100.000\times(1+0.5)\times0.900}{2.75\times(0.2 + 2\times0.900\times\tan 45°)} = 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5<H<6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca:
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

M = $(k1\times q + k2\times p)\times R^2$

《Pipe Top》

$= (0.107\times16.200\times10^{-4} + 0.0799\times49.091\times10^{-4})\times10.565^2$

= 0.063 kN･cm/cm

《Pipe Bottom》

$= (0.121\times16.200\times10^{-4} + 0.011\times49.091\times10^{-4})\times10.565^2$

= 0.028 kN･cm/cm

Bending load is:

σ = M/Z

《Pipe Top》

$$= \frac{0.063\times10^3}{0.313\times10^2}$$

= 2.014 N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$$= \frac{0.028\times10^3}{0.313\times10^2}$$

= 0.895 N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.313 cm^3/cm =$t^2/6$

R : Radius in pipe thickness center 10.565 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3\times q + k4\times p)\times R^4}{E\times I}$$ R^4= 12459

$$= \frac{(0.070\times16.200\times10^{-1}+0.030\times49.091\times10^{-1})\times12459}{1000\times10^2\times0.2143}$$

= 0.152 cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.152}{2 \times 10.565} \times 100$$

$= 0.7$ % ≦ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 10.565 cm

E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.2143 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.116	3.708	0.597	2.8	OK
0.6	70.13	10.80	0.075	2.398	0.389	1.8	OK
0.8	57.75	13.50	0.068	2.174	0.349	1.7	OK
0.9	49.09	16.20	0.063	2.014	0.326	1.5	OK
1.0	44.63	18.00	0.061	1.95	0.317	1.5	OK
1.2	37.76	21.60	0.059	1.886	0.308	1.5	OK
1.5	30.68	27.00	0.06	1.918	0.309	1.5	OK
2.0	23.38	36.00	0.064	2.046	0.331	1.6	OK
2.5	18.88	45.00	0.071	2.27	0.367	1.7	OK
3.0	15.84	54.00	0.079	2.525	0.409	1.9	OK
3.5	13.64	63.00	0.087	2.781	0.455	2.2	OK
4.0	11.97	72.00	0.099	3.165	0.514	2.4	OK
4.5	10.67	81.00	0.111	3.548	0.577	2.7	OK
5.0	9.63	90.00	0.123	3.932	0.639	3	OK
5.5	8.77	99.00	0.135	4.316	0.702	3.3	OK
6.0	8.05	108.00	0.147	4.699	0.765	3.6	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 355 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	312.0	mm	
Outer Pipe Diameter	Bc	:	35.5	cm	
Pipe Thickness (minimum)	t	:	2.15	cm	PE-100 PN-10 SDR 17 Poliplex C.
Radius in pipe thickness center	R	:	16.675	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma\times H$$
$$= 18\times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2
H : Earth Cover = 0.9 m
γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2\times P\times(1+i)\times\beta}{C\times(a + 2H\times\tan\theta)}$$
$$= \frac{2\times 100.000\times(1+0.5)\times 0.900}{2.75\times(0.2 + 2\times 0.900\times\tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2
H : Earth Cover = 0.9 m
P : Unit rear tire load = 100.000 kN
a : Tire ground contact width = 0.2 m
C : Width between two rear tires = 2.75 m
θ : Load spreading angle = 45°
i : Shock load coefficient = 0.5
"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 16.675^2$

$= 0.157$ kN·cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 16.675^2$

$= 0.070$ kN·cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$$= \frac{0.157 \times 10^3}{0.77 \times 10^2}$$

$= 2.038$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$$= \frac{0.070 \times 10^3}{0.77 \times 10^2}$$

$= 0.909$ N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN·cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.770 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 16.675 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3 \times q + k4 \times p) \times R^4}{E \times I} \qquad R^4 = 77315$$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 77315}{1000 \times 10^2 \times 0.8282}$$

$= 0.243$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.243}{2 \times 16.675} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

δx : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 16.675 cm

E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.8282 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σmax<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.289	3.751	0.959	2.9	OK
0.6	70.13	10.80	0.188	2.44	0.625	1.9	OK
0.8	57.75	13.50	0.168	2.181	0.561	1.7	OK
0.9	49.09	16.20	0.157	2.038	0.524	1.6	OK
1.0	44.63	18.00	0.153	1.986	0.509	1.5	OK
1.2	37.76	21.60	0.148	1.921	0.494	1.5	OK
1.5	30.68	27.00	0.148	1.921	0.496	1.5	OK
2.0	23.38	36.00	0.159	2.064	0.532	1.6	OK
2.5	18.88	45.00	0.176	2.284	0.589	1.8	OK
3.0	15.84	54.00	0.196	2.544	0.656	2	OK
3.5	13.64	63.00	0.218	2.83	0.73	2.2	OK
4.0	11.97	72.00	0.246	3.193	0.826	2.5	OK
4.5	10.67	81.00	0.276	3.582	0.926	2.8	OK
5.0	9.63	90.00	0.306	3.972	1.027	3.1	OK
5.5	8.77	99.00	0.336	4.361	1.127	3.4	OK
6.0	8.05	108.00	0.366	4.751	1.228	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 400 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	351.7	mm	
Outer Pipe Diameter	Bc	:	40.0	cm	
Pipe Thickness (minimum)	t	:	2.42	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	18.790	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1\times q + k2\times p)\times R^2$

《Pipe Top》

$= (0.107\times 16.200\times 10^{-4} + 0.0799\times 49.091\times 10^{-4})\times 18.790^2$

$= 0.200$ kN·cm/cm

《Pipe Bottom》

$= (0.121\times 16.200\times 10^{-4} + 0.011\times 49.091\times 10^{-4})\times 18.790^2$

$= 0.088$ kN·cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \frac{0.20\times 10^3}{0.976\times 10^2}$

$= 2.049$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$= \frac{0.088\times 10^3}{0.976\times 10^2}$

$= 0.902$ N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN·cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.976 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 18.790 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \frac{(k3\times q + k4\times p)\times R^4}{E\times I}$ $R^4=$ 124654

$= \frac{(0.070\times 16.200\times 10^{-1}+0.030\times 49.091\times 10^{-1})\times 124654}{1000\times 10^2\times 1.181}$

$= 0.275$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.275}{2 \times 18.790} \times 100$$

$= 0.7$ % ≦ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 18.79 cm

E : Coefficient of Erasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 1.1810 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: ($\sigma max<6.4kN/mm^2$, $Vmax<4\%$)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.367	3.76	1.084	2.9	OK
0.6	70.13	10.80	0.239	2.449	0.707	1.9	OK
0.8	57.75	13.50	0.214	2.192	0.634	1.7	OK
0.9	49.09	16.20	0.2	2.049	0.592	1.6	OK
1.0	44.63	18.00	0.194	1.988	0.575	1.5	OK
1.2	37.76	21.60	0.188	1.926	0.559	1.5	OK
1.5	30.68	27.00	0.189	1.936	0.561	1.5	OK
2.0	23.38	36.00	0.202	2.07	0.601	1.6	OK
2.5	18.88	45.00	0.223	2.285	0.666	1.8	OK
3.0	15.84	54.00	0.249	2.551	0.742	2	OK
3.5	13.64	63.00	0.276	2.828	0.825	2.2	OK
4.0	11.97	72.00	0.312	3.197	0.933	2.5	OK
4.5	10.67	81.00	0.35	3.586	1.047	2.8	OK
5.0	9.63	90.00	0.388	3.975	1.161	3.1	OK
5.5	8.77	99.00	0.426	4.364	1.275	3.4	OK
6.0	8.05	108.00	0.465	4.764	1.389	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 450 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	395.6	mm	
Outer Pipe Diameter	Bc	:	45.0	cm	
Pipe Thickness (minimum)	t	:	2.72	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	21.140	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5<H<6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 21.14^2$

$= 0.253$ kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 21.14^2$

$= 0.112$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.253 \times 10^3}{1.233 \times 10^2}$

$= 2.052$ N/mm² ≦ **6.4** **N/mm²** OK

《Pipe Bottom》

$= \dfrac{0.112 \times 10^3}{1.233 \times 10^2}$

$= 0.908$ N/mm² ≦ **6.4** **N/mm²** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm²

Z : Section Modulus = 1.233 cm³/cm $=t^2/6$

R : Radius in pipe thickness center 21.140 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m²

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$ $R^4 =$ 199719

$= \dfrac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 199719}{1000 \times 10^2 \times 1.677}$

$= 0.31$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.31}{2 \times 21.14} \times 100$$

$= 0.7$ % ≦ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 21.14 cm

E : Coefficient of Erasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 1.6770 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm², Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.464	3.763	1.223	2.9	OK
0.6	70.13	10.80	0.302	2.449	0.797	1.9	OK
0.8	57.75	13.50	0.271	2.198	0.715	1.7	OK
0.9	49.09	16.20	0.253	2.052	0.668	1.6	OK
1.0	44.63	18.00	0.245	1.987	0.649	1.5	OK
1.2	37.76	21.60	0.238	1.93	0.631	1.5	OK
1.5	30.68	27.00	0.239	1.938	0.633	1.5	OK
2.0	23.38	36.00	0.256	2.076	0.679	1.6	OK
2.5	18.88	45.00	0.283	2.295	0.751	1.8	OK
3.0	15.84	54.00	0.315	2.555	0.837	2	OK
3.5	13.64	63.00	0.35	2.838	0.931	2.2	OK
4.0	11.97	72.00	0.395	3.203	1.053	2.5	OK
4.5	10.67	81.00	0.443	3.593	1.181	2.8	OK
5.0	9.63	90.00	0.491	3.982	1.31	3.1	OK
5.5	8.77	99.00	0.54	4.379	1.438	3.4	OK
6.0	8.05	108.00	0.588	4.769	1.567	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 500 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	439.7	mm	
Outer Pipe Diameter	Bc	:	50.0	cm	
Pipe Thickness (minimum)	t	:	3.02	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	23.490	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 23.49^2$

$= 0.312$ kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 23.49^2$

$= 0.138$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.312 \times 10^3}{1.520 \times 10^2}$

$= 2.053$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$= \dfrac{0.138 \times 10^3}{1.520 \times 10^2}$

$= 0.908$ N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 1.520 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 23.490 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$ $R^4 =$ 304461

$= \dfrac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 304461}{1000 \times 10^2 \times 2.295}$

$= 0.346$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.346}{2 \times 23.49} \times 100$$

$$= 0.70 \quad \% \quad \leqq \quad \mathbf{4} \quad \mathbf{\%} \quad \text{OK}$$

where:

- δx : Total flexure caused by backfilling soil load and active load cm
- V : Flexure Ratio
- R : Radius in pipe thickness center = 23.49 cm
- E : Coefficient of Elasticity = 1,000 N/mm^2 (Japanese standard value)
- I : Geometric Moment of Inertia per 1 cm pipe length = 2.295 cm^4/cm $=t^3/12$
- q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2
- p : Active Load = 49.091 kN/m^2
- k1 : Flexure factor for backfilling soil
- k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.573	3.77	1.363	2.9	OK
0.6	70.13	10.80	0.373	2.454	0.888	1.9	OK
0.8	57.75	13.50	0.334	2.197	0.797	1.7	OK
0.9	49.09	16.20	0.312	2.053	0.744	1.6	OK
1.0	44.63	18.00	0.303	1.993	0.723	1.5	OK
1.2	37.76	21.60	0.294	1.934	0.702	1.5	OK
1.5	30.68	27.00	0.295	1.941	0.705	1.5	OK
2.0	23.38	36.00	0.316	2.079	0.756	1.6	OK
2.5	18.88	45.00	0.349	2.296	0.837	1.8	OK
3.0	15.84	54.00	0.389	2.559	0.932	2	OK
3.5	13.64	63.00	0.432	2.842	1.037	2.2	OK
4.0	11.97	72.00	0.488	3.21	1.173	2.5	OK
4.5	10.67	81.00	0.547	3.599	1.316	2.8	OK
5.0	9.63	90.00	0.607	3.993	1.459	3.1	OK
5.5	8.77	99.00	0.666	4.381	1.602	3.4	OK
6.0	8.05	108.00	0.726	4.776	1.745	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 560 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	492.4	mm	
Outer Pipe Diameter	Bc	:	56.0	cm	
Pipe Thickness (minimum)	t	:	3.38	cm	PE-100 PN-10 SDR 17 Poliplex C.
Radius in pipe thickness center	R	:	26.310	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca:
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 26.31^2$

$= 0.392$ kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 26.31^2$

$= 0.173$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$$= \frac{0.392 \times 10^3}{1.904 \times 10^2}$$

$= 2.059$ N/mm^2 $\leqq$ **6.4** **N/mm^2** OK

《Pipe Bottom》

$$= \frac{0.173 \times 10^3}{1.904 \times 10^2}$$

$= 0.909$ N/mm^2 $\leqq$ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 1.904 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 26.310 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$$

$R^4 = 479163$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 479163}{1000 \times 10^2 \times 3.2179}$$

$= 0.388$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.388}{2 \times 26.31} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

- δx : Total flexure caused by backfilling soil load and active load cm
- V : Flexure Ratio
- R : Radius in pipe thickness center = 26.31 cm
- E : Coefficient of Erasticity = 1,000 N/mm^2 (Japanese standard value)
- I : Geometric Moment of Inertia per 1 cm pipe length = 3.2179 cm^4/cm $=t^3/12$
- q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2
- p : Active Load = 49.091 kN/m^2
- k1 : Flexure factor for backfilling soil
- k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.719	3.776	1.53	2.9	OK
0.6	70.13	10.80	0.468	2.458	0.997	1.9	OK
0.8	57.75	13.50	0.419	2.201	0.894	1.7	OK
0.9	49.09	16.20	0.392	2.059	0.836	1.6	OK
1.0	44.63	18.00	0.38	1.996	0.812	1.5	OK
1.2	37.76	21.60	0.369	1.938	0.788	1.5	OK
1.5	30.68	27.00	0.37	1.943	0.791	1.5	OK
2.0	23.38	36.00	0.396	2.08	0.849	1.6	OK
2.5	18.88	45.00	0.438	2.3	0.939	1.8	OK
3.0	15.84	54.00	0.488	2.563	1.047	2	OK
3.5	13.64	63.00	0.542	2.847	1.164	2.2	OK
4.0	11.97	72.00	0.612	3.214	1.317	2.5	OK
4.5	10.67	81.00	0.687	3.608	1.477	2.8	OK
5.0	9.63	90.00	0.761	3.997	1.637	3.1	OK
5.5	8.77	99.00	0.836	4.391	1.798	3.4	OK
6.0	8.05	108.00	0.911	4.784	1.959	3.7	OK

Pipe Bed Structural Analysis   HDPE Pipe DN = 710 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	624.3	mm	
Outer Pipe Diameter	Bc	:	71.0	cm	
Pipe Thickness (minimum)	t	:	4.29	cm	PE-100 PN-10 SDR 17 Poloplex C
Radius in pipe thickness center	R	:	33.355	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5<H<6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 33.355^2$

$= 0.629$ kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 33.355^2$

$= 0.278$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.629 \times 10^3}{3.067 \times 10^2}$

$= 2.051$ N/mm^2 $\leqq$ **6.4** **N/mm^2** OK

《Pipe Bottom》

$= \dfrac{0.278 \times 10^3}{3.067 \times 10^2}$

$= 0.906$ N/mm^2 $\leqq$ **6.4** **kN/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 3.067 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 33.355 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$ $R^4 =$ 1237781

$= \dfrac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 1237781}{1000 \times 10^2 \times 6.579}$

$= 0.49$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.49}{2 \times 33.355} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 33.355 cm

E : Coefficient of Erasticity = 1,000 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 6.579 cm^4/cm $=t^3/12$

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	1.155	3.765	1.933	2.9	OK
0.6	70.13	10.80	0.752	2.452	1.26	1.9	OK
0.8	57.75	13.50	0.674	2.197	1.13	1.7	OK
0.9	49.09	16.20	0.629	2.051	1.056	1.6	OK
1.0	44.63	18.00	0.611	1.992	1.026	1.5	OK
1.2	37.76	21.60	0.593	1.933	0.996	1.5	OK
1.5	30.68	27.00	0.594	1.937	1	1.5	OK
2.0	23.38	36.00	0.636	2.073	1.072	1.6	OK
2.5	18.88	45.00	0.704	2.295	1.186	1.8	OK
3.0	15.84	54.00	0.784	2.556	1.322	2	OK
3.5	13.64	63.00	0.871	2.84	1.471	2.2	OK
4.0	11.97	72.00	0.984	3.208	1.664	2.5	OK
4.5	10.67	81.00	1.103	3.596	1.866	2.8	OK
5.0	9.63	90.00	1.223	3.987	2.069	3.1	OK
5.5	8.77	99.00	1.343	4.378	2.272	3.4	OK
6.0	8.05	108.00	1.464	4.773	2.475	3.7	OK

Pipe Bed Structural Analysis HDPE Pipe DN = 800 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	HDPE Pipe for Sewer		
Nominal Pipe Diameter	D	:	703.2	mm	
Outer Pipe Diameter	Bc	:	80.0	cm	
Pipe Thickness (minimum)	t	:	4.84	cm	PE-100 PN-10 SDR 17 Poliplex C
Radius in pipe thickness center	R	:	37.580	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	4	%	
Allowable Bending Load		:	6.4	N/mm^2	ISO standard 9.6(Creep strength)/1.5
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H = 18 \times 0.9 = 16.200 \ kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)} = \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)} = 49.091 \ kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5<H<6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Ca
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 37.58^2$

$= 0.799$ kN・cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 49.091 \times 10^{-4}) \times 37.58^2$

$= 0.353$ kN・cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$$= \frac{0.799 \times 10^3}{3.904 \times 10^2}$$

$= 2.046$ N/mm^2 ≦ **6.4** **N/mm^2** OK

《Pipe Bottom》

$$= \frac{0.353 \times 10^3}{3.904 \times 10^2}$$

$= 0.904$ N/mm^2 ≦ **6.4** **N/mm^2** OK

where:

M : Bending Moment kN・cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 3.904 cm^3/cm $=t^2/6$

R : Radius in pipe thickness center 37.580 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3 \times q + k4 \times p) \times R^4}{E \times I} \qquad R^4 = 1994468$$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 1994468}{1000 \times 10^2 \times 9.448}$$

$= 0.55$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.55}{2 \times 37.58} \times 100$$

$= 0.7$ % $\leqq$ **4** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm
V : Flexure Ratio
R : Radius in pipe thickness center = 37.58 cm
E : Coefficient of Erasticity = 1,000 N/mm^2 (Japanese standard value)
I : Geometric Moment of Inertia per 1 cm pipe length = 9.448 cm^4/cm $=t^3/12$
q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2
p : Active Load = 49.091 kN/m^2
k1 : Flexure factor for backfilling soil
k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<6.4kN/mm^2, Vmax<4%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	1.466	3.755	2.169	2.9	OK
0.6	70.13	10.80	0.955	2.446	1.413	1.9	OK
0.8	57.75	13.50	0.856	2.192	1.268	1.7	OK
0.9	49.09	16.20	0.799	2.046	1.185	1.6	OK
1.0	44.63	18.00	0.776	1.988	1.151	1.5	OK
1.2	37.76	21.60	0.753	1.929	1.118	1.5	OK
1.5	30.68	27.00	0.754	1.931	1.122	1.5	OK
2.0	23.38	36.00	0.808	2.07	1.203	1.6	OK
2.5	18.88	45.00	0.893	2.287	1.331	1.8	OK
3.0	15.84	54.00	0.995	2.548	1.484	2	OK
3.5	13.64	63.00	1.106	2.833	1.65	2.2	OK
4.0	11.97	72.00	1.249	3.199	1.867	2.5	OK
4.5	10.67	81.00	1.401	3.588	2.094	2.8	OK
5.0	9.63	90.00	1.553	3.978	2.321	3.1	OK
5.5	8.77	99.00	1.705	4.367	2.549	3.4	OK
6.0	8.05	108.00	1.858	4.759	2.777	3.7	OK

Branch Sewer

Pipe Bed Structural Analysis PVC Pipe Diameter = 150 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	uPVC Pipe for Sewer		
Nominal Pipe Diameter	D	:	150	mm	
Outer Pipe Diameter	Bc	:	16.0	cm	
Pipe Thickness (minimum)	t	:	0.46	cm	Assumed from AS/NZS 1260:2009, P17
Radius in pipe thickness center	R	:	7.730	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	5	%	
Allowable Bending Load		:	17.7	N/mm^2	
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".

Rear tire load of vehicle specified by T-25 is adopted.

Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Case
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 7.998$

$= 0.034$ kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 44.628 \times 10^{-4}) \times 7.998$

$= 0.015$ kN･cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$$= \frac{0.034 \times 10^3}{0.154 \times 10^2}$$

$= 2.208$ kN/mm^2 ≦ **17.7** **kN/mm^2** OK

《Pipe Bottom》

$$= \frac{0.015 \times 10^3}{0.154 \times 10^2}$$

$= 0.974$ kN/mm^2 ≦ **17.7** **kN/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.154 cm^3/cm Reference#1: Code of Japanese water supply facility design, 2000, P460

R : Radius in pipe thickness center 7.73 cm

q : Vertical soil pressure by backfilling soil = 16.200 kN/m2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k3 \times q + k4 \times p) \times R^4}{E \times I}$$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 7.730}{2{,}942 \times 10^2 \times 0.07373}$$

$= 0.043$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.043}{2 \times 7.73} \times 100$$

$= 0.3$ % ≦ **5** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 7.73 cm

E : Coefficient of Elasticity = 2,942 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.07373 cm^4/cm

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<17.7kN/mm^2, Vmax<5%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.062	4.026	0.169	1.1	OK
0.6	70.13	10.80	0.04	2.597	0.11	0.7	OK
0.8	57.75	13.50	0.036	2.338	0.099	0.6	OK
0.9	49.09	16.20	0.034	2.208	0.092	0.6	OK
1.0	44.63	18.00	0.033	2.143	0.09	0.6	OK
1.2	37.76	21.60	0.032	2.078	0.087	0.6	OK
1.5	30.68	27.00	0.032	2.078	0.087	0.6	OK
2.0	23.38	36.00	0.034	2.208	0.094	0.6	OK
2.5	18.88	45.00	0.038	2.468	0.104	0.7	OK
3.0	15.84	54.00	0.042	2.727	0.116	0.8	OK
3.5	13.64	63.00	0.047	3.052	0.129	0.8	OK
4.0	11.97	72.00	0.053	3.442	0.146	0.9	OK
4.5	10.67	81.00	0.059	3.831	0.163	1.1	OK
5.0	9.63	90.00	0.066	4.286	0.181	1.2	OK
5.5	8.77	99.00	0.072	4.675	0.199	1.3	OK
6.0	8.05	108.00	0.079	5.13	0.217	1.4	OK

Pipe Bed Structural Analysis PVC Pipe Diameter = 225 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	uPVC Pipe for Sewer		
Nominal Pipe Diameter	D	:	213.8	mm	
Outer Pipe Diameter	Bc	:	225.3	cm	
Pipe Thickness (minimum)	t	:	0.70	cm	Assumed from AS/NZS 1477:2006, P18
Radius in pipe thickness center	R	:	11.040	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	5	%	
Allowable Bending Load		:	17.7	N/mm^2	
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2
H : Earth Cover = 0.9 m
γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2
H : Earth Cover = 0.9 m
P : Unit rear tire load = 100.000 kN
a : Tire ground contact width = 0.2 m
C : Width between two rear tires = 2.75 m
θ : Load spreading angle = 45°
i : Shock load coefficient = 0.5
"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Case
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

$M = (k1\times q + k2\times p)\times R^2$

《Pipe Top》

$= (0.107\times 16.200\times 10^{-4} + 0.0799\times 49.091\times 10^{-4})\times 10.96$

$= 0.069$ kN・cm/cm

《Pipe Bottom》

$= (0.121\times 16.200\times 10^{-4} + 0.011\times 44.628\times 10^{-4})\times 10.96$

$= 0.03$ kN・cm/cm

Bending load is:

$\sigma = M/Z$

《Pipe Top》

$= \dfrac{0.034\times 10^3}{0.154\times 10^2}$

$= 4.481$ kN/mm^2 ≦ **17.7** **kN/mm^2** OK

《Pipe Bottom》

$= \dfrac{0.015\times 10^3}{0.154\times 10^2}$

$= 1.948$ kN/mm^2 ≦ **17.7** **kN/mm^2** OK

where:

M : Bending Moment kN・cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.154 cm^3/cm Reference#1: Code of Japanese water supply facility design, 2000, P460

R : Radius in pipe thickness center 11.04 cm

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k1	k2
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$\delta x = \dfrac{(k1\times q + k2\times p)\times R^4}{E\times I}$

$= \dfrac{(0.070\times 16.200\times 10^{-1}+0.030\times 49.091\times 10^{-1})\times 11.04}{2{,}942\times 10^2\times 0.07373}$

$= 0.119$ cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.119}{2 \times 7.77} \times 100$$

$$= 0.5 \ \% \ \leqq \ \mathbf{5} \ \% \quad \text{OK}$$

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 11.04 cm

E : Coefficient of Erasticity = 2,942 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.111 cm^4/cm

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k3	k4
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<17.7kN/mm^2, Vmax<5%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.127	8.247	0.467	2.1	OK
0.6	70.13	10.80	0.082	5.325	0.305	1.4	OK
0.8	57.75	13.50	0.074	4.805	0.273	1.2	OK
0.9	49.09	16.20	0.069	4.481	0.255	1.2	OK
1.0	44.63	18.00	0.067	4.351	0.248	1.1	OK
1.2	37.76	21.60	0.065	4.221	0.241	1.1	OK
1.5	30.68	27.00	0.065	4.221	0.242	1.1	OK
2.0	23.38	36.00	0.07	4.545	0.259	1.2	OK
2.5	18.88	45.00	0.077	5	0.287	1.3	OK
3.0	15.84	54.00	0.086	5.584	0.32	1.4	OK
3.5	13.64	63.00	0.095	6.169	0.356	1.6	OK
4.0	11.97	72.00	0.108	7.013	0.402	1.8	OK
4.5	10.67	81.00	0.121	7.857	0.451	2	OK
5.0	9.63	90.00	0.134	8.701	0.5	2.3	OK
5.5	8.77	99.00	0.147	9.545	0.549	2.5	OK
6.0	8.05	108.00	0.16	10.39	0.598	2.7	OK

Pipe Bed Structural Analysis PVC Pipe Diameter = 300 mm

Vertical Soil Pressure Formula

1. Design Conditions

Pipe Material		:	uPVC Pipe for Sewer		
Nominal Pipe Diameter	D	:	299.5	mm	
Outer Pipe Diameter	Bc	:	315.5	cm	
Pipe Thickness (minimum)	t	:	0.75	cm	Assumed from AS/NZS 1477:2006, P18
Radius in pipe thickness center	R	:	15.350	cm	
Earth Cover	H	:	90.000	cm	
Unit weight of Backfilling Soil	γ	:	18	kN/m^3	
Active Load		:	T-25		
Allowable Flexure Ratio		:	5	%	
Allowable Bending Load		:	17.7	N/mm^2	
Design Supporting Angle		:	120	°	(360° Sand Bed)
Range of Earth Cover		:	0.5 to 6	m	

2. Vertical Soil Pressure

Vertical soil pressure generated by backfilling soil is calculated as follows:

$$q = \gamma \times H$$
$$= 18 \times 0.9$$
$$= 16.200 \quad kN/m^2$$

where:

q : Vertical soil pressure kN/m^2

H : Earth Cover = 0.9 m

γ: Unit weight of Backfilling Soil = 18 kN/m^3

3. Active Load

Effect of vehicle load is considered as "Active Load".
Rear tire load of vehicle specified by T-25 is adopted.
Provided that the load is spread in longitudinal direction by 45°with groung contact width of 0.2 m.

Unit vertical load (p) generated by active load is calculated as follows:

$$p = \frac{2 \times P \times (1+i) \times \beta}{C \times (a + 2H \times \tan\theta)}$$
$$= \frac{2 \times 100.000 \times (1+0.5) \times 0.900}{2.75 \times (0.2 + 2 \times 0.900 \times \tan 45°)}$$
$$= 49.091 \quad kN/m^2$$

where:

p : Active Load kN/m^2

H : Earth Cover = 0.9 m

P : Unit rear tire load = 100.000 kN

a : Tire ground contact width = 0.2 m

C : Width between two rear tires = 2.75 m

θ : Load spreading angle = 45°

i : Shock load coefficient = 0.5

"i" varies depend on earth cover:

	H≦1.5	1.5＜H＜6.5	H≧6.5
i	0.5	0.65 - 0.1H	0

β: Load reduction factor = 0.9

	H≦1.0 and inner diameter D≧4.0	Other Case
β	1.0	0.9

4. Bending Moment

If design supporting angle is 120°, bending moment to be applied on installed pipe is calculated as follows:

M = $(k1 \times q + k2 \times p) \times R^2$

《Pipe Top》

$= (0.107 \times 16.200 \times 10^{-4} + 0.0799 \times 49.091 \times 10^{-4}) \times 15.35$

= 0.133 kN･cm/cm

《Pipe Bottom》

$= (0.121 \times 16.200 \times 10^{-4} + 0.011 \times 44.628 \times 10^{-4}) \times 15.35$

= 0.059 kN･cm/cm

Bending load is:

σ = M/Z

《Pipe Top》

$$= \frac{0.034 \times 10^3}{0.154 \times 10^2}$$

= 8.636 kN/mm^2 ≦ **17.7** **kN/mm^2** OK

《Pipe Bottom》

$$= \frac{0.015 \times 10^3}{0.154 \times 10^2}$$

= 3.831 kN/mm^2 ≦ **17.7** **kN/mm^2** OK

where:

M : Bending Moment kN･cm/cm

σ : Bending Load kN/mm^2

Z : Section Modulus = 0.154 cm^3/cm Reference#1: Code of Japanese water supply facility design, 2000, P460

R : Radius in pipe thickness center 15.35 cm

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Moment factor for backfilling soil

k2 : Moment factor for active load

	k3	k4
Pipe Top	0.107	0.079
Pipe Bottom	0.121	0.011

Fundation: 360 degree sand bed (supporting angle=120 degree)

Bending ratio of installed pipe

$$\delta x = \frac{(k1 \times q + k2 \times p) \times R^4}{E \times I}$$

$$= \frac{(0.070 \times 16.200 \times 10^{-1} + 0.030 \times 49.091 \times 10^{-1}) \times 15.35}{2{,}942 \times 10^2 \times 0.07373}$$

= 0.443 cm

$$V = \frac{\delta x}{2R} \times 100$$

$$= \frac{0.443}{2 \times 7.77} \times 100$$

$= 1.4$ % ≦ **5** **%** OK

where:

δ x : Total flexure caused by backfilling soil load and active load cm

V : Flexure Ratio

R : Radius in pipe thickness center = 15.35 cm

E : Coefficient of Erasticity = 2,942 N/mm^2 (Japanese standard value)

I : Geometric Moment of Inertia per 1 cm pipe length = 0.111 cm^4/cm

q : Vertical soil pressure by backfilling soil = 18.000 kN/m^2

p : Active Load = 49.091 kN/m^2

k1 : Flexure factor for backfilling soil

k2 : Flexure factor for active load

	k1	k2
Factor	0.070	0.030

5. Results of Pipe Bed Structural Analysis

Abovementioned calculation procedure is conducted with earth cover ranging 0.3 m to 6.0 m and the results are shown below: (σ max<17.7kN/mm^2, Vmax<5%)

Earth Cover	p (kN/m^2)	q (kN/m^3)	M(max)	σ (max)	δ (max)	V(max)	Judgement
0.3	122.73	5.40	0.245	15.909	1.747	**5.7**	**NG**
0.6	70.13	10.80	0.159	10.325	1.138	3.7	OK
0.8	57.75	13.50	0.143	9.286	1.021	3.3	OK
0.9	49.09	16.20	0.133	8.636	0.954	3.1	OK
1.0	44.63	18.00	0.129	8.377	0.927	3	OK
1.2	37.76	21.60	0.126	8.182	0.9	2.9	OK
1.5	30.68	27.00	0.126	8.182	0.903	2.9	OK
2.0	23.38	36.00	0.135	8.766	0.969	3.2	OK
2.5	18.88	45.00	0.149	9.675	1.072	3.5	OK
3.0	15.84	54.00	0.166	10.779	1.195	3.9	OK
3.5	13.64	63.00	0.185	12.013	1.329	4.3	OK
4.0	11.97	72.00	0.208	13.506	1.503	4.9	OK
4.5	10.67	81.00	0.234	15.195	1.686	**5.5**	**NG**
5.0	9.63	90.00	0.259	16.818	1.869	**6.1**	**NG**
5.5	8.77	99.00	0.285	18.506	2.053	**6.7**	**NG**
6.0	8.05	108.00	0.31	20.13	2.237	**7.3**	**NG**

SECTION B2: Structural Calculation of PS and STP Facilities

TABLE OF CONTENTS

1 GENERAL

1.1. Scope

This document summarizes the requirements for the detailed design and engineering of building/structure including foundation design which are in Port Moresby Sewerage System Upgrading Project (POMSSUP). The succeeding sub-heading 1.2 provides a summary of the building structure type and the associated seismic design parameters adopted for the design of the buildings.

Design methods which shall be adopted for the different material type and use are;

- Limit State Design Method – Steel Structures
- Ultimate Strength Method – Concrete & Masonry Structures
- Limit State Method – Concrete Structures for Wastewater Containment
- Working Stress Method – Timber Structures

1.2. Structure Type for Each Building

The structural design philosophy adopted for the building structures and associated secondary elements of each building enables the designers to produce a design for the buildings that represents the structural system and its behavior in responding to seismic loading. Presented in Table-1 below is a summary of the structural seismic design parameters adopted for each building.

Table-1

Building	Structure Type	Base Seismic Coefficient $C_d(T_1)$	Structural Type Factor S_p
B-01 2 Storey Administration Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-02 Single Storey Generator Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-03 2 Storey Sludge Pump Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-04 2 Storey Clorination Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-05 Single Storey Electrical Substation Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-06 Single Storey Bowler/ Local Control Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0
B-07 2 Storey Clorination Building	Cantilever Shearwall of Limited Ductility to PNGS 1001 – 1982 Part 4 Clause 3.4.4.4.	0.05	2.0

1.3. Applicable Codes, Specification, Standards and Drawings

The following specifications, standards and drawings shall be applied in the design, in conjunction with these design criteria and in the priority stated below.

Local authority requirements, laws and regulations shall prevail, if these are more stringent than those listed below.

1. International Design Codes And Standards

- AS 3600-2009	Concrete Structures
- AS 3600-Supp1-1994	Concrete Structures - Commentary
- AS 4100-1998	Steel Structures
- AS 4100 Supp1-1999	Steel Structures-Commentary
- AS 3735 – 2001	Australian Standard, for Concrete Structures for Retaining Liquids
- NZS 3106:2009	New Zealand Standard, for Concrete Structures for Retaining Liquids
- BS 8007: 1987;	British Standard for Concrete Structures for Retaining Liquids
- PNGS 1292 – 1989	PNG Standard for Timber Structures

2. International Design Codes And Standards For Reference

- AS/NZS 1170.0	Structural Design Actions Part:0 General Principles
- AS/NZS 1170.0 Supp	Structural design actions Part::0 General principles – Commentary
- AS/NZS 1170.1	Structural design actions Part:1 Permanent, imposed and other actions
- AS/NZS 1170.1 Supp	Structural design actions Part:1 Permanent, imposed and other actions – Commentary
- AS/NZS 1170.2	Structural design actions Part:2 Wind actions
- AS/NZS 1170.2 Supp	Structural design actions Part:2 Wind actions – Commentary

- AS 1170.4	Structural design actions Part:4 Earthquake actions in Australia
- NZS 1170.5	Structural design actions Part 5 Earthquake actions – New Zealand

SI units will be used throughout the project.

1.4. Design Groundwater Levels

Groundwater table level shall be taken at ground level as per Geotechnical Report.

1.5. Structural Analysis and Computer Programs to be used in the design

For structural analysis SPACEGASS shall be utilized and Excel Design spreadsheets for Static Torsional analysis.

For other design items like steel connection, concrete pedestal/column and other concrete members, Excel Design spreadsheets shall be utilized.

For structure/building foundation design, Excel Design spreadsheets.

Seismic Analysis for Tank for Retaining Wastewater.

Analysis for earthquake loads for the final sedimentation tanks and oxidation ditches shall be done in accordance with 'Seismic Design of Storage Tanks' by Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering.

2 MATERIALS

The structural materials to be used for the work shall be in accordance with the following requirements unless noted otherwise on the drawings.

2.1 Concrete

Concrete strength will be in accordance with Table 2.

Table-2

CONCRETE MIX CLASS	Concrete Design Strength (fc)	Applications
Class 1B	40 MPa	Liquid Retaining Structures
Class 2	32 MPa	For above grade concrete (Beam, wall slab, column.. etc.)
Class 2A	32 MPa	Fireproofing
Class 3	40 MPa	For on or below grade structures (all foundations, Manhole, Catch basin, RC Pit etc.)
Class 4	25 MPa	For paving
Class 5	15 MPa	Lean (Blinding) Concrete & Fill (Mass) concrete

Design calculations of concrete structures and foundations will be based on the "Concrete Design Strength" indicated in Table-2 above.

2.2 Reinforcement

- Deformed High Strength Reinforcing Bar. (min grade 400 MPa)

  Deformed bar shall comply with one of the following:

  1. AS 4671, Grade 500N
  2. ASTM A615, Grade 420
  3. JIS G3112, SD 490
  4. BS 4449, Grade 500A/B

  Minimum size of rebar shall be 12mm in diameter except that for ties /stirrups where the minimum size is 10mm in diameter.

Bar Size	Nominal Diameter	Sectional Area
Y10	10 mm	78 mm^2
Y12	12 mm	113 mm^2
Y16	16 mm	201 mm^2
Y20	20 mm	314 mm^2
Y24	24 mm	452 mm^2
Y28	28 MM	616 mm^2
Y32	32 mm	804 mm^2

- Wire Fabric

  (AS/NZS 4671/ASTM A185)

  200(mm) x 200(mm) x 7(mm) for light-duty concrete paving

  200(mm) x 200(mm) x 10(mm) for heavy-duty concrete paving

2.3 Embedded Steel

All holding down bolts, embedded plates, cleats, trench angles, edges angles, sleeves, ladder rungs etc. which need to be embedded into concrete will be fabricated from steel with a minimum grade of 250 in accordance with AS3678 and AS3679, and/or approved equivalent i.e. ASTM A36 SS Grade 36, BS EN 10025 Grade S275 and JIS G3101 SS490. All items shall be hot dipped galvanized.

2.4 Structural Steel

Hot-rolled Section:

AS/NZS 3679 Gr.350 or BS EN 10025 Gr S 355 with hot dip galvanizing.

Hot-rolled Plate:

AS/NZS 3679 Gr.350 or BS EN 10025 Grade S 355 with hot dip galvanizing.

The minimum thickness of any part of a structural steel shape shall be 6 mm. Minimum thickness of plate material used for structural connection shall be 10 mm.

These minimum thickness provisions shall apply to non vertical steel structures other than Pre-Engineered Buildings (PEB).

2.5 Connection Bolt

Bolts M20 and larger shall be high strength bolts conforming to AS1252 Gr 8.8 or equal with hot dip galvanizing

Bolts M16 and smaller bolts shall be conforming to AS 1110 Grade 4.6 with hot dip galvanizing.

All bolted connections shall be designed based on bearing type connection with threads included.

For main and secondary members	:	Min. M20
For railing , ladders and ladder cages	:	Min. M16
For removable floor plates and gratings	:	As required
For purlins, girts and stair treads	:	Min. M12

The minimum number of 2 bolts shall be provided on either side of a connection or splice.

2.6 Hollow Section for Structural Use

Closed structural shapes, such as tubes, pipes and box members, are not permitted unless approved by the Contractor. When such shapes are approved for use, special precautions to prevent internal corrosion shall be taken.

2.7 Welding Electrode

E48XX with tensile strength 480 MPa in accordance with AS/NZS 1553.1 and shall comply to Standard Structural Notes drawing.

2.8 Coating System for Structural Steel and Connection Bolts

1. All onshore structural steel shall be galvanized unless otherwise noted.

2. All floor grating and floor plates (checkered plates) including stair treads and ladder rungs shall be galvanised.
3. For structures to be galvanized, the design shall consider criteria to mitigate the risk of LMAC (Liquid Metal Assisted Cracking).

Hot dip galvanizing for structural steel

Conforming to AS4791 or ASTM A 123/A 123M

Hot dip galvanizing for high strength bolt, ordinary strength bolts, nuts & washers

Conforming to AS1214 or ASTM a 153/A 153M.

2.9 Anchor Bolt & Nut

Refer to the Standard Drawing for Anchor Bolt .

2.10 Concrete Masonry Unit (CMU)

Concrete Masonry Unit (CMU) shall conform to AS-3700.

2.11 Timber (PNGS 1292 – 1989)

The hardwood timber shall be specified where timber is to be used as the construction material particularly for structural purposes.

3 DESIGN LOAD CASES

3.1 General

The loads and external forces which shall be taken into account in the design of the structures are described below. The various combinations of those loads and external forces to be used in the calculations are given in Section 4.

Loads and Forces	Expressed as:
• Dead load	- DL
• Equipment load	- EL
• Live load	- LL
• Impact load	- IL
• Wind load	- Wu,Ws (=Wu:Refer note 3 in Section 4)
• Earthquake load	- EuEs (=Eu: Refer note 4 in Section 4)

3.2 Dead Load (DL)

Dead loads will include self weight of the structures and the weight of fireproofing and soil on footings. 10% shall be added to the weight of the steel structure as an allowance for the base plates and connections. In the absence of specific values, the following unit weights of the major construction materials shall be used.

1. Steel 76.9 kN/m^3
2. Reinforced concrete 25.0 kN/m^3
3. Plain concrete 24.0 kN/m^3
4. Cement mortar and all grouts 21.0 kN/m^3
5. Soil
   - Well-compacted fill 17 kN/m^3
   - Above ground water level 16 kN/m^3
   - Below ground water level 6 kN/m^3
6. Ground water 10.0 kN/m^3

The other loads of building material, shall be in accordance with AS 1170.1.

3.3 Equipment Load (EL)

Equipment load shall be as per technical data to be made available by engineers.

3.4 Live Load (LL)

Except where it is specified below, minimum live loads shall be as per AS/NZS 1170.1:2002, Structural Design Actions, Part 1: Permanent, Imposed and Other Actions. The minimum live loading shall be as follows:

Table-3 MINIMUM LOADS

Components Design	**Loaded Area**	**Minimum Live Load (kN/m^2)**
(A) BUILDINGS - FLOOR	Self contained dwellings	1.5
	Stair and landing in self contained dwellings	2.0
(B) other areas not covered elsewhere	Electrical Room	20.0
	Laboratory	5.0
	UPS ./Battery room	10.0

	Control room	6.0
(C) Warehouse/ Workshop areas		25.0
(D) Floor Plate, Grating and Slabs	1. Walkways and Access Platforms	5.0
	2. Platforms for Operating Storage or Maintenance Storage Loads	6.0
(E) Floor Framing and Bracing	1. Walkways and Access Platforms	3.0 or a moving concentrated load of 4.5 kN
	2. Platform for Operating Storage or Maintenance Storage Loads	6.0
(F)Columns and Brackets	1. Walkways and Access Platforms	3.0 or a moving concentrated load of 4.5 kN
	2. Platform for Operating Storage or Maintenance Storage Loads.	6.0
(G) Roof (Sloped or Flat)	-	1.0

Note:

1. The minimum live loads specified in AS/NZS 11.70.1:2002 and above Table shall be reviewed for their adequacy in areas that may be subjected to heavy equipment and material loads during periodic maintenance operations.

3.5 Impact Load (IL)

Crane Load (Ic)$_2$ and Lifts

Following percentage increase shall be applied to vertical, lateral and longitudinal impact loads on the supports of moving bridge cranes, trolleys and monorail cranes shall increase in accordance with AS/NZS 1170.1:2002.

a. For Cranes:

1. Vertical Actions

   For Electric overhead cranes 25%

   For hand-operated cranes 10%

2. Transverse to Rails

   For Electric overhead cranes 20%

   For hand-operated cranes 5%

3. Parallel to Rails

   For Electric overhead cranes 10%

   For hand-operated cranes 5%

3.6 Wind Load (Wu,Ws)

For wind load calculations, refer to AS/NZS 1170.2

For importance level of 2, Structures shall be designed for basic wind speed V=54 m/s (ULS case) and V= 39 m/s (in SLS case)

For importance level of 3, structures shall be designed for wind speed V=57 m/s (in ULS case) and 39 m/s (in SLS case).

3.7 Earthquake/Seismic Load (Eu, Es)

(1) Earthquake Loadings (PNGS 1001 – 1982: Part 4)

PNGS 1001 – 1982: Part 4 sets down requirements for the general structural design and seismic design loadings for structures within any of the following categories:

i. All buildings having a floor area greater than 20 m^2.

ii. All masonry or concrete walls greater than 1.5 m in height.

iii. Elevated tanks of up to 200 m^3 capacity. Larger tanks than this should be subject of special studies.

iv. All buildings to which the general public have access.

The requirements are not intended to apply to:

a. Unusual buildings or other types of structures
b. Civil engineering works
c. Single-storey or two-storey residential buildings housing less than 10 people.

(2) Method of Analysis (Calculation) of Earthquake Load

Selection of method

For regular structures up to 40 m in height complying with the requirements of Clause 3.2, PNGS 1001 – 1982: Part 4; the horizontal seismic loads may be determined by the equivalent static load analysis specified in Clause 3.4. For all other structures a dynamic analysis shall be carried out in accordance with Clause 3.5.

i. Equivalent Static Seismic Load Analysis Method

All structures within the scope of this code shall be designed in accordance with this seismic design procedure. All structures not within the scope of this code shall be designed in accordance with relevant New Zealand standard Seismic Design Procedure.
The loads shall be calculated using the Equivalent Static Load Analysis Method given in this code, using the following equations:

a. Total Horizontal Seismic Base Shear, V.

Every building shall be designed and constructed to withstand a total horizontal seismic base shear (V) in the direction of consideration.
V = CIKWt
Where; Wt = combination of the vertical dead and design live load above the level of lateral restraint imposed by the ground.
C = Basic seismic coefficient; obtained from the function of building period, T, and nature of the soil, i.e. C (T, nature of soil),
T = 0.09H/√D’,
H = height of the structure above the level of lateral restraint (in meters),
D’ = overall length of the building at the base in the direction under consideration (in meters).
I = Importance Factor
K = structural type factor

b. Horizontal Torsional Moments – By Static Torsional Analysis

To allow for torsional motion the horizontal seismic load shall be applied at an eccentricity from the centre of rigidity at that level.

The centre of mass of the floor shall be calculated from the mass of all the gravity loads supported laterally at that level.

The centre of rigidity at a particular level in a building shall be calculated as that point at which the application of a transverse force will cause no rotation about a vertical axis of the floor level under consideration.

3.8 Earth Pressures (Q – PNGS 1001, Fe – NZS 3106)

(1) Earth Pressure Load

This includes: the weight of the retained soil, which causes horizontal pressures on the stem, thus tending to cause forward sliding, bearing failure or overturning, or the weight of the infill soil, which causes horizontal pressures on the facing, thus tending to cause stem rupture.

There are four main earth pressures considered in design of the earth retaining and tank wall/base structures.

a. Active Pressure

In response to soil pressure, the wall will move away from the soil, thus partially relieving the pressure. This reduced pressure is the active pressure.

b. Pressure at Rest

If the wall is unable to move away from the soil embankment, as may be the case for a propped cantilever basement wall, there will be no relief of the pressure and the soil will exert the full pressure at rest.

c. Passive Pressure

If the structure pushes into the soil, as is the case at the toe of a retaining wall, the resistance by the soil is greater than the pressure at rest. This is the passive pressure. If the soil in front of the toe is disturbed or loose, the full passive pressure may not be mobilized. It is strongly recommended that passive pressure in front of the wall be ignored in design.

d. Seismic Earth Pressure

The seismic behavior of retaining wall depends on the total lateral earth pressure that develops during the earth shaking. This total pressure includes both the static gravitational pressure that exist before earthquake occurs and the transient dynamic pressure induced by the earthquake. Therefore, the static pressure on the retaining wall is of significant in the seismic design of retaining wall.

The commonly used method adopted in the evaluation of dynamic seismic coefficient for earth pressure is Mononobe – Okabe Seismic Coefficient Analysis.

e. Methods for Earth Pressure calculation – Rankine & Coulomb's Lateral Earth Pressure Theories

For the determination of above earth pressures, we shall use two relatively simple classical thoeries that are widely used:

- Rankine's Earth Pressure Theory, or
- Coulomb's Earth Pressure Theory.

Rankines Theory assumes:

- There is no adhesion or friction between the wall and soil
- Lateral pressure is limited to vertical walls
- Failure (in the backfill) occurs as a sliding wedge along an assumed failure plane defined by ϕ.
- Lateral pressure varies linearly with depth and the resultant pressure is located one-third of the height (H) above the base of the wall.
- The resultant force is parallel to the backfill surface.

The Coulomb Theory is similar to Rankine except that:

- There is friction between the wall and soil and takes this into account by using a soil-wall friction angle of δ. Note that δ ranges from ϕ/2 to ϕ/3 and δ = 2ϕ/3 is commonly.
- Lateral pressure is not limited to vertical walls.
- The resultant force is not necessarily parallel to the backfill surface because of the soil-wall friction value δ.

The intensity of the active/passive horizontal pressure, which is a function of the applicable earth pressure coefficient, depends upon the degree of wall movement since movement controls the degree of shear strength mobilized in the surrounding soil.

(2) Lateral Earth Pressure Coefficients

Lateral earth pressure is related to the vertical earth pressure by a coefficient termed:

- At Rest Earth Pressure Coefficient (Ko)
- Active Earth Pressure Coefficient (Ka)
- Passive Earth Pressure Coefficient (Kp)
- Seismic Earth Pressure Coefficient (Kae)

(3) Other Forces Acting on the Wall

Aside from the earth pressure force acting on the wall, other forces might also act on the wall and these are superimposed onto the earth pressure force. For example, these forces might include:

- Surcharge load – Load results from that are applied along the surface of the backfill behind the wall. These forces apply an additional lateral force along the back of the wall. Surcharge pressures result from loads such as a line load, strip loa, embankment load, traffic (such as a parking lot), floor loads and temporary loads such as construction traffic and stockpiles of material.

- Earthquake load (as discussed in 3.7 and 3.8.1.d)

3.9 Water Pressures (F – PNGS 1001, Flp – NZS 3106)

Retaining walls are typically designed to not withstand hydrostatic pressure as these are prevented from developing behind the wall. Therefore the loads applied to most walls do not include water pressure. In cases where hydrostatic or water pressure might will develop behind an undrained wall, the additional force resulting from the water pressure must be applied as a superimposed on the lateral earth pressure.

<u>Hydrodynamic Force/Seismic Effect</u>

When a tank containing liquid with a free surface is subjected to horizontal earthquake ground motion, tank wall and liquid are subjected to horizontal acceleration. The liquid in the lower region of tank behaves like a mass that is rigidly connected to wall. This mass is termed as impulsive liquid mass and induces impulsive hydrodynamic pressure on tank wall and similarly on base. Liquid mass in the upper region of tank undergoes sloshing motion. This mass is termed as convective liquid mass and it exerts convective hydrodynamic pressure on tank wall and base. Thus, total liquid mass gets divided into two parts, i.e., impulsive mass and convective mass.

The method of analysis and design are outlined by "Seismic Design of Storage Tanks by Recommendations of a Study Group of the New Zealand National Society for Earthquake Engineering" and NZS 3106: 2009, respectively.

4 LOAD COMBINATIONS

4.1 General

Design should be based on the load combination causing the most unfavourable effect. When excluding loads other than dead loads results in a more critical loading condition, then such exclusion shall be considered. Load factors and combinations are subject to the design engineers approval.

For modifications or additions to existing structures, reduced load factors or higher allowable stresses may be used with the approval of the Contractor.

4.2 Load Combinations

(1) Load Combinations for Limit State Design (for Steel Structures)

Table-4.1: Load Combinations for Strength Limit State

PNGS 1001	
1.4DL + 1.7LL	Dead + Live Loads
1.4DL + 1.7 Ψl LL	
1.0DL + 1.3LL + 1.3W	With Wind Loads
1.0DL + ΨcLL + W	
0.9DL + 1.3W	
1.0DL + 1.3LL + 1.0E	With Earthquake Loads
DL + Ψc LL + E	
0.9DL + E	

Where, DL = Dead Loads; LL = Live Loads; W = wind loads for ultimate limit state
Ψc = combination factor

= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.
= 0 for all other roofs
= 0.6 for storage areas and others

Ψl = long term factor
= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.
= 0.6 for storage areas and others

Serviceability Limit State

The serviceability limit state design includes checking or designing for;

- Deflections
- Vibrations

Serviceability limit state is also employed to check for slips in bolted connections.
For Serviceability limit state, the following loading combinations, in Table 4.2, are often considered.

Table-4.2: Load Combinations for Serviceability Limit State

PNGS 1001 – 1982: Part 1	
DL + ΨsLL	Dead + Live Loads
DL + ΨlLL	Dead + Live Loads

Where, DL = Dead Loads; Q = LL = Live Loads; Wu = W = wind loads for ultimate limit state
Ψs = short term factor
= 0.7 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.
= 1.0 for storage areas and others

Ψl = long term factor
= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.
= 0.6 for storage areas and others

(2) Load Combinations for Ultimate Strength Method (for Concrete & Masonry Structures)

Table-4.3: Load Combinations for Ultimate Strength

PNGS 1001	
1.4DL + 1.7LL	Dead + Live Loads
1.4DL + 1.7ΨlLL	
1.0DL + 1.3LL + 1.3W	With Wind Loads
1.0DL + ΨcLL + Wu	
0.9DL + 1.3W	

1.0DL + 1.3LL + 1.0E	With Earthquake Loads
1.0DL + ΨcLL + 1.0E	
0.9DL + E	
1.4DL + 1.7LL + 1.4F	With Liquid Pressure & Earth Pressure
1.4DL + ΨcLL + F	
0.9DL + 1.4F	

Where, DL = Dead Loads; LL = Live Loads; W = wind loads for ultimate strength, F = Liquid or earth pressure (Also Refer to Clause 4.2.3, AS/NZS 1170.0: 2002)

Ψc = combination factor

= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.

= 0 for all other roofs

= 0.6 for storage areas and others

Ψl = long term factor

= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.

= 0.6 for storage areas and others

Serviceability & Serviceability Load Combinations

The serviceability design includes checking or designing for;

- Deflections
- Vibrations
- lateral drifts
- cracks

The Serviceability is also employed to check for slips in bolted connections.

For Serviceability, the following loading combinations, in Table 4.4, are often considered.

Table-4.4: Load Combinations for Serviceability

PNGS 1001	
DL + ΨsLL	Dead + Live Loads
DL + ΨlLL	Dead + Live Loads

Where, DL = Dead Loads; LL = Live Loads; W = wind loads for ultimate limit state

Ψs = short term factor

= 0.7 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.

= 1.0 for storage areas and others

Ψl = long term factor

= 0.4 for roofs used for floor type activities, floors of the residential and domestic buildings, offices, parking and retail area.

= 0.6 for storage areas and others

(3) Load Combinations for Working Stress Method (for Timber Structures)

Table 4.5: Load Combination for Working Stress Method

PNGS 1001 – 1982: Part 1	
DL + LL	Dead + Live Loads
DL + LL + W	With Wind Loads
DL + LL + 0.8E	With Earthquake Loads
0.7DL + 0.8E	
DL + LL + Q	With Earth Pressure
DL + Q	
DL + LL + F	With Liquid Pressure
DL + F	

F = Liquid Pressure, Q = Earth Pressure

(4) Load Combinations for Ultimate Limit State (for Water Containment Concrete Structures)

The following combinations for the ultimate limit states shall be considered:

Table 4.6: Load Combinations ultimate limit state

NZS 3106: 2009
1.2G + 1.2Flp
1.2G + 1.5Fe +1.5Fgw
1.2G + 1.2Flp + 1.5Fe + 1.5Fgw
G + Eu +Flp
G + Eu + Flp + Fe + Fgw

Where, G = Dead Loads; Q = Live Loads; Fe = earth pressure loads; Flp = liquid pressure loads; Fgw = ground water loads; Eu = Earthquake loadings.

The strain induced loads: temperature and shrinkage will not significantly affect the ultimate strength of the structure and hence do not need to be considered at the limit state.

Serviceability & Serviceability Load Combinations

The serviceability limit state design includes checking or designing for;

- Deflections
- Vibrations
- lateral drifts
- cracks

The Serviceability is also employed to check for slips in bolted connections.
For Serviceability, the following loading combinations, in Table 5.9, are often considered.

Table 4.7: Load Combinations for Serviceability

Group A (long term) loads	Group A (short term) loads	
	Tank full	Tank empty
G + Fe +Fgw + Fp + (Fsh or 0.5Fsw)	G + Flp + Fgw + Fp + FT + 0.7Fsw	G +Fe + Fgw + Fp + FT + (0.7Fsh or 0.35Fsw)
G + Flp + Fp + 0.5Fsw	G + Flp + Fe + Fgw +Fp + (Es1 or Es2)	
G + Flp + Fe + Fgw + Fp + 0.5Fsw		

Where; Fp = force resulting from prestress, Fsh = force resulting from shrinkage, Fsw = force resulting from swelling, Es1 & Es2 = Earthquake load for serviceability limit state 1 & 2, FT = force resulting from temperature variation

(5) Load Combinations for Foundation (Bearing Check)

Operation (Normal)

E = [G]

E = 1.0[DL + EL]

E = [G, ΨeQ]

E = 1.0 [DL + EL] + 1.0[LL + IL]

E = [G, ΨeQ]

E = 1.0[DL + EL] + 0.6[LL + IL]

E = [G, ΨeQ, Ws]

E = 1.0[DL +EL] + 0.6[LL] +[Ws]

E = [G, Ws]

E = 1.0[DL + EL] + [Ws]

E = [G, Eu, ΨeQ]

E = 1.0 [DL + EL] + 0.6[LL] + [Es]

E = [G, Ss, ΨeQ]

E = 1.0[DL + EL] + 0.6[LL + IL]

Notes for Section 4:

1. "DL" is dead load of structure without fireproofing.
2. The basis of load combination above, for example "E = [1.2G, 1.5Q]" were taken from AS/NZS 1170.0 and redefined as below:

    G = Permanent Action

    Q = Imposed Action

    Wu/Ws = Wind Action

    Eu/Es = Earthquake Action

    Su/Ss = Earth and/or Water Pressure

    E = Action Effect

    Ψc = combination factor for imposed action

    Ψs = short-term factor for imposed action

    Ψe = long-term factor for imposed action

3. W&E is calculated using Ultimate Limit State method.

Stability is not considered in combining loads as it is not considered to be a critical load condition for the structures designed.

5 MATERIAL STRENGTHS

5.1 Basic Design Strength

1. Concrete:

    Compressive Strength for Design

For Concrete Above Ground	: f'c = 32 N/mm²
For Concrete below ground	: f'c = 40 N/mm²
For Paving Concrete	: f'c = 25 N/mm²
For Lean/Plain Concrete	: f'c = 15 N/mm²

2. Reinforcing Steel Bar:

Material Grade	Yield Strength
For Design Use	fy = 420 MPa (ASTM A615)

3. Welded Wire Fabric (reinforcement) :

    For Design Use fy = 485 MPa (ASTM A185)

4. Structural Steel:

Material Grade	Yield Strength
For Design Use For all plate thickness	fy = 345 MPa

5. Embedded Steel :

Material Grade	Yield Strength
For Design Use For all plate thickness	fy = 250 MPa

6. High Strength Bolts:

Material Grade	Min. Tensile Strength
For Design Use	f_{uf} = 830 MPa

For standard round holes of bearing type connections, bolt threads shall be included in shear plane.

7. Anchor Bolts :

Allowable force for anchor bolt shall comply with Standard Drawing for Anchor Bolt.

8. Ordinary Strength Bolts :

Material Grade	Min. Tensile Strength
For Design Use	f_{uf} = 400 MPa

9. Weld Metal

The nominal tensile strength of weld metal (uw) of E7XT-1 and E7XT-5 for design use is equal to 480 MPa (70ksi).

10. Compressive Strength of Masonry:

Compressive strength of masonry shall be as per AS 3700.

6 SOIL BEARING CAPACITY

Refer to Appendix-A for Allowable bearing Capacity

7 STRUCTURAL STABILITY

When checked using unfactored loads as follows.

7.1 Overturning

Safety factor against overturning shall be not less than 1.5.

7.2 Sliding

Safety factor against sliding shall be not less than 1.5.
Friction coefficient between soil and concrete foundation shall be taken as 0.50.

7.3 Buoyancy

(1) Safety factor against buoyancy shall be not less than 1.2; For enclose (box) structures such as pits, manholes, etc.

(2) Safety factor against uplift due to buoyancy shall be not less than 1.1.

8 CONCRETE STRUCTURES AND FOUNDATIONS

8.1 Design Codes

All concrete design shall conform to AS 3600-2001, unless noted otherwise herein.

8.2 Details for Concrete and Reinforcement

1. Minimum Concrete Cover shall be as follows;

   -Element exposed to sewerage / spillage : 75mm
   (Grade 40 MPa Concrete)
   (Catch Basin / Manhole / Spill Basin etc.)

   -Other structures above ground : 65 mm
   (Grade 32 MPa Concrete)

   For concrete cast against ground (40 MPa Concrete), minimum concrete cover shall be 75mm.

2. Arrangement of Reinforcement Steel Bars
   Unless otherwise shown, the minimum clear distance between parallel bars shall be 1.5 times the diameter for round bars provided that in no case shall be spacing between the bars less than 25mm, or less than 1.33 times the maximum size of the coarse aggregate.

3. Standard Minimum top of pedestals

   The top of concrete shall be a minimum of 200mm above finished grade or high point of paving for foundations for the following:

   a. Steel column bases of steel structures & pre-engineered buildings

4. For Shallow rigid (block-type foundation), the horizontal eccentricity if any direction between the centroid of mass of the machine foundation system and the centroid of the base contact area shall not exceed 5 percent of the respective base dimensions.

5. The recommended minimum embedded depth of foundations:

a) Major foundations, such as Building, Pre-engineered building structure : 1400mm below FGL

b) Transformer foundations HVAC foundations : 600mm below FGL

c) Small Minor foundations such as Staircase foundation, ladder foundation : 300mm below FGL

8.3 Building wall Design:

External walls in buildings shall NOT be treated as non-loadbearing infill blockwalls except and unless they are specified as such. Otherwise all walls shall be treated as loadbearing walls offering the role of the shearwalls in the building.

The consequence of this is that the buildings will be rigid squat structures which will primarily become elastic structures not requiring the need to employ ductility. Hence appropriate structural factors will be adopted.

8.4 Grout:

A minimum 25mm thick of grout shall be used for steel building structures and equipments of buildings between steel base plate and concrete pedestal.

8.5 General Requirements:

Binding concrete when used shall be laid with a minimum thickness of 50mm with 100mm on either side of foundation and screeded to the correct level. Polythene sheet of 0.2mm thick shall be used between blinding concrete & foundation bottom for protection of concrete foundations.

9 STEEL STRUCTURES

9.1 Design Code

The design of structural steel, details, fabrication and erection shall be in accordance with AS 4100-1998. The load combinations are shown in Section 4 above.

9.2 Seismic Design

Seismic Design shall be done in compliance with NSZS 1170.5.

9.3 Allowable Deflection for Crane & Lifts

1. Crane Girder

Vertical Deflection (Without dynamic or impact factors applied)
1/500 of span or 60mm, whichever is lesser

1/300 of span for cantilever

Lateral Deflection
1/600 of span or 20mm, whichever is lesser

2. Beam./Trolley Beam

   1/250 of span
   1/125 of span for cantilever

9.4 Connections

1. For connection bolts material and use, refer to Section 2.5 above.
2. For welding electrode and use, refer to Section 2.7 above.
3. Moment resisting connections shall be designed for most unfavorable load combinations resulted from SPACEGASS.
4. Connections shall be field bolted and shop welded or bolted.
5. Bolted connection shall be designed based on bearing-type connections using minimum 2-M20 bolts each joint.
6. Bolts of 20mm diameter and larger shall be high strength bolts Gr 8.8.

10 OTHER STRUCTURAL ISSUES:

10.1 Allowable Deflection and Lateral / Drift for Buildings

a) For concrete buildings

Based on AS/NZS 1170 & NZS 1170.5

i. Vertical Deflections
For beams & slab - Span / 250
(simply supported)
For Cantilever beams/slabs - Span / 125

b) For steel buildings:

As per AS:4100, the following values shall be used.

i. Vertical Deflections
For beams/frames - Span / 250
For Cantilever beams - Span / 125

ii. Horizontal Deflections for columns
For buildings clad in steel or aluminium sheeting : Height / 150
For Buildings with masonry walls
supported by steel work : Height / 250

10.2 Design of RC Frames

The RC frame is designed and detailed as an Intermediate Moment Resisting Frame (IMRF) for which the ductility factor shall be taken 3 as per AS/NZS 1170.4. The walls in the building shall be treated as cantilever limited ductile shearwalls in accordance with the PNG Earthquake Loadings Code and shall be load bearing and supported on appropriately designed strip footings. The walls will not be required to support any out-of-plane loading from the building except for its own self weight. The building loads will be effectively distributed to and be supported by the boundary shear walls via the diaphragm action of the roof slab.

10.3 Provision of Expansion Joint

As some of the concrete structures like the oxidation ditches (L=70m), the issue of providing expansion joint needs to be looked at carefully. In the absence of any clear direction from Australian codes, the matter was referred to AC-224-1995 where for extensive civil structures and for temperature change of 20 ºC (around 68 º F), around 200 feet length can be provided without any expansion joint. In our case about 18 º C (31-13) temperature change (65ºF), it can be stated that up to 65m, there is no need of expansion joint. So civil structures having more than 65 m length i.e. Oxidation Ditch, expansion joint needs to be provided. Further more, Grit chamber will be also provided expansion joint since it is various in form.

11 REFERENCE: METHOD OF STRUCTURAL DESIGN

In the design of the structures and facilities, members shall be proportioned for adequate strength by one of the following design methods, Limit State Design, Ultimate Strength, Working Stress and Ultimate Limit State. The choice of method is governed by the type of material that is to be used for construction for the structures.

- For Structural steel works we shall use limit state design method,
- For Concrete structures we shall use ultimate strength method, and
- For Timber structures we shall use working stress method.

However, for the wastewater containment structures shall be looked at separately as provided in Clause 4 below.

11.1 Limit State Design – For Steel Structures

The limit state design philosophy, to put it simply is to ensure that the factored design (internal) actions (bending moments, M*, shear forces, V*, axial forces, N*) are less than or equal to the factored nominal capacities (ϕM, ϕV, ϕN) of the element/member of the structure. The factored nominal capacities can be given mathematically as;

- Bending/flexural strength, $\phi M > M^*$
- Shear strength, $\phi V > V^*$

- Axial compressive strength, ϕNc > Nc*
- Axial tensile strength, ϕNt > Nt*

Strength Limit State & Load Combinations for Strength Limit State

The strength limit state includes designing for bending moments (M*), shear forces (V*), axial forces (N*) in compression or tension and combined bending with axial compression or tension. These actions are obtained from structural analysis using simple statics, or one of the known methods of analysis such as the stiffness method of analysis or using structural analysis software such as SPACE GASS.

Often in design, the structure is designed for the combined loading condition that produces the most adverse effects. These combinations are in accordance the PNG Standards are listed in Table 4.1.

11.2 **Ultimate Strength Method** – For Concrete Structures

The limit state is the advanced and modified method of ultimate strength method. In the ultimate strength method, the capacities of the members of the structures are compared with the structural member actions from the loads acting on the structures.

i. Requirements

The design of a structure and its component members shall take into account, as appropriate, stability, strength, serviceability, durability, fire resistance and any other relevant design requirements, in accordance with the procedures given in this section of the notes.
For more detail design information, we shall consult AS 3600 – 2001 in respective sections given below.

1. Design for Stability (SECTION 3, AS 3600 – 2001)
2. Design for Strength (SECTION 3, AS 3600 – 2001)
3. Design for Serviceability (SECTION 3, AS 3600 – 2001)
4. Design for Durability (SECTION 4, AS 3600 – 2001)
5. Design for Fire Resistance (SECTION 5, AS 3600 – 2001)
6. Other Design Requirements (SECTION 3, AS 3600 – 2001)

We shall focus on strength design part of the design requirement as it governs very much the sizing of structural members. The factored design (internal) actions (bending moments, M*, shear forces, V*, axial forces, N*) are less than or equal to the factored nominal capacities (ϕM, ϕV, ϕN) of the element/member of the structure. The factored nominal capacities can be given mathematically as;

- Bending/flexural strength, ϕM > M*
- Shear strength, ϕV > V*

- Axial compressive strength, $\phi Nc > Nc^*$
- Axial tensile strength, $\phi Nt > Nt^*$

ii. Load Combinations for Strength

Often in design, the structure is designed for the combined loading condition that produces the most adverse effects. Some of these combinations in accordance to both the PNG and Australian Standards are listed in Table 4.3.

11.3 **Working Stress Method** – For Timber Structures

The working stress method shall be used to design timber structures .

Recommendation for the Code of Practice for Working Stress Method. For Load combinations refer to Table 4.5.

We will use the PNGS 1292 - 1989 for timber structural design of the structures. Thus we will use load factors for the load combinations and capacity reduction factors given in PNGS 1001 – 1982: Part 1.

11.4 Ultimate Limit State Method

– For Wastewater Containment Concrete Structures

For the wastewater containment structures, final sedimentation tanks, oxidation ditch, etc, we shall make provision in accordance with NZS 3106: 2009. NZS 3106: 2009 outlines the design criteria for the ultimate limit state method. It is the latest code which supersedes the former NZS 3106: 1986.

i. Requirements

The design of a structure and its component members shall take into account, as appropriate, stability, strength, serviceability, durability, fire resistance and any other relevant design requirements, in accordance with the procedures given in this section of the notes.
NZS 3106: 2009 outlines the following design criteria;

1. Design for Stability

   In addition mass of the empty structure, the design resistance against uplift and/or buoyancy may take account;

   a. Anchoring systems,
   b. Drainage systems,
   c. Pressure relief valves, or
   d. Any combination of items (a), (b), and (c).

2. Design for Strength

a. Flexural Strength

The bending moment due to the forces acting the wall and foundation slab shall be withstood with sufficient amount of flexural reinforcement.

b. Hoop Strength

Hoop tension forces and moments induced in the circular tank wall by hydrodynamic forces shall be withstood with sufficient amount of hoop/horizontal reinforcements around the wall.

Hoop reinforcements perform several functions such as;
Shear reinforcements and temperature and shrinkage reinforcements.

3. Design for Serviceability

Serviceability governs the design of the wastewater containment structures.
In serviceability design aspect, following are considered;

a. Liquid tightness

   Leakage to be minimal. Appearance not to be impaired by staining.

b. Construction Joints

   Special provision shall be made for construction joints to maintain the watertightness of the joints. The construction joints shall be located the correct locations such as at changes of section.

c. Control of Cracking

   A minimum crack width of 0.20 mm shall be allowed.

4. Design for Durability

Durability shall be allowed for in design by determining the exposure classification and, for that exposure classification, complying with the appropriate requirements for:

a. Concrete quality and curing,
b. Cementitious binder composition,
c. Cover,
d. Chemical silica reaction precautions,
e. Protection of fixings

For Load combinations refer to Table 4.7.

A-1: PUMP WELL - PS01 - 03 and 13 - 16

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

Load Calculation

Data

Density of Water (ρ) =	9.81	**kN/m^3**
Height of Water, h =	2.4	**m**
Wall Thickness, tw =	0.3	**m**
Height of Wall, hw =	7.5	**m**
Base Slab Thickness, tb =	0.5	**m**
Length of Base slab, Lb =	3.1	**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	23.54	**kN/m (=kPa/m)**

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

Water Pressure on base slab, Pw = ρgh 23.54 **kPa**

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

Flp = 23.54 **kN/m**

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.30	m
Height of Wall (H) =	7.50	m
Embedded Height of Wall (He) =	6.60	m
(Heel Depth) Thickness of Base (tb) =	0.50	m
Width of Base (wb) =	1.00	m
(Heel)/ Length of base (Lb) =	3.10	m
Toe Depth (td) =	0.50	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) =	45	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 $K_A = (1\text{-SIN } \phi)$ =

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) =	73.40	LE =	2.200
PS(G) =	13.08	LS =	3.30
PS(Q) =	0.00	LS =	3.30
Total, Pa =	86.48	La =	2.37

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	73.40	2.20	161.47
PS =	13.08	3.30	43.17
Total	86.48	**2.37**	204.65

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	56.25	3.10	174.4
Wall wt =	52.50	3.10	162.8
Base wt =	38.75	1.55	60.1
Total =	147.50	1.51	397.2

CHECK GROUND BEARING FAILURE

∑MO = 204.65 kNm/m

∑MR = 397.2 kNm/m

∑F = 147.50 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

e = B/2 - [∑MR - ∑MO]/∑F 0.24 m **OK, e < B/6**

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

qtoe = ∑F/B [1+6e/B] = 0.72 kPa

qheel = ∑F/B [1-6e/B] = 0.26 kPa

Maximum Bearing Pressure = 0.72 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

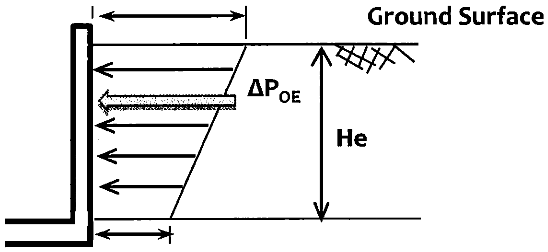
Location =	Port Moresby	
Category (pg 15) =	C	
Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15		
p =	0.002	
Tank Contents =	Non-volatile toxic chemicals	
probability factor; Fig. C2.7; pg 51 (Ap) =	1.85	
geographical coefficient representing regional seismicity; Fig. C2.1; 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 =	7.5	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	73	kPa
fΔe = 0.5 Ch(0) ϒe He	24	kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	24	kN/m	UDL
Eu2 =	49	kN/m	Triangular

Base Slab Analysis

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

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

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

$ks = k's\, B$

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

--> $\lambda L < \pi/4$ Rigid member - Bending not influenced much by ks   333.75

$\lambda L > \pi$ 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	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
2	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
3	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
4	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
5	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
6	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
7	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
8	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

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 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	0.50	5.21E-03	31.9754	0.18	0.4	0.11128	9.57	0.6393	6.1163	No	Yes	Use Winkler	222559
2	0.50	5.21E-03	31.9754	0.18	0.4	0.11128	9.57	0.6393	6.1163	No	Yes	Use Winkler	222559
3	0.50	5.21E-03	31.9754	0.18	0.4	0.11128	9.57	0.6393	6.1163	No	Yes	Use Winkler	222559
4	0.50	5.21E-03	31.9754	0.18	0.4	0.11128	9.57	0.6393	6.1163	No	Yes	Use Winkler	222559
5	2.25	2.34E-02	31.9754	0.18	0.4	0.16205	9.57	0.4822	4.6131	No	Yes	Use Winkler	72023
6	2.25	2.34E-02	31.9754	0.18	0.4	0.16205	9.57	0.4822	4.6131	No	Yes	Use Winkler	72023
7	2.25	2.34E-02	31.9754	0.18	0.4	0.16205	9.57	0.4822	4.6131	No	Yes	Use Winkler	72023
8	2.25	2.34E-02	31.9754	0.18	0.4	0.16205	9.57	0.4822	4.6131	No	Yes	Use Winkler	72023

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

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

Design of a Continuous Beam :

1 Loads

1 Serviceability Load Combination : Long term loads

Main Load Dead Load =	33.60	kN/m	Self weight =	28.13	kN/m
Total G =	61.73	kN/m	Flp =	23.54	kN/m
Fe =	86.48	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	148.20	kN/m	COMB. 2	G + Flp =	85.27	kN/m
COMB. 3	G + Flp + Fgw =	85.27	kN/m		Long term Load Factors =	1	

2 Beam Section Properties

Beam Size

bw =	2250	mm	D =	500	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	3100	mm	Overall depth of sect. D =	500	mm
Effective Length,Lef =	2900	mm	Area, A =	1.13E+06	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	2.34E+10	mm^4
Neutral Axis, xc =	1415.00	mm	Moment of Inertia, Iy-y =	4.75E+11	mm^4
Effective width, beff =	2830.00	mm	Gross Area, Ag =	1.13E+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 = 2830 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.0371995	(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	=	101.229		
defln/Lef	=	0.004	k_1/k_2 = 17.28	
d	=	74 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	=	20.00	kNm	(left support)
Mmax +ve	=	20.00	kNm	(mid-span)
M -ve	=	20.00	kNm	(right support)
Vmax	=	144.00	kN	
V (other end)	=	144.00	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 = 383

Estimate Area of main steel Ast = 166.49 mm^2

7 Y 20 Ast = 2199.11 mm^2 **SPACING = 300** mm

p = 0.0026

7 Y 20 Asc = 2199.11 mm^2

pc = 0.0026 < pd = 0.0254088

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)]

= 232 kNm > M* = 20.00 kNm **OK**

Determine -ve steel at right support :

M -ve = 20.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst : 383 mm

Estimate Area of main steel Ast = 166.49 mm^2

7 Y 20 Ast = 2199.11 mm^2

p = 0.0026

7 Y 20 Asc = 2199.11 mm^2

pc = 0.0026 < pd = 0.0254088

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)]

= 232 kNm > M* = 20.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 20 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) = 166.489148 mm^2

Bottom 7 Y 20 Ast = 2199.11486 mm^2

Top 7 Y 20 Asc = 2199.11486 mm^2

Assume N - A in flange

Effective width : beff = 2830 mm

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

= 0.0319403

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)]

= 307 kNm > M*+ve = 20.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	7	Y 20
	Asc	=	7	Y 20
Right support :	Ast	=	7	Y 20
	Asc	=	7	Y 20
midspan	Ast	=	7	Y 20
	Asc	=	7	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 = 870 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 = 870 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 = 290 mm

8 STRESS DEVELOPMENT

<u>For Top Steel :</u>

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 1038 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 = 1058 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1038 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 = 0.086177

therefore Lsy = 28 mm ≥ 25k1db 625

say = 650 mm

<u>For Bottom Steel :</u>

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 = 1232 mm

k1 = 1 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1212 mm (dist between bars)

factor = 44805

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.06514821

therefore Lsy = 17 mm ≥ 25k1db 500

say = 500 mm

9 PUNCHING SHEAR CHECK

Wall Dimension

V* = 144.00 kN b = 500 mm

Dist to centreline of bottom steel from soffit d = 500 mm

Critical shear perimeter, μ

dq = 85 mm μ = 2(b+dom) + 2(d+dom)

dom = 415 mm 3660 mm

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

βh = 1 longer side/shorter side

fcv =	$0.17(1+2/\beta h)\sqrt{f'c} \le 0.34\sqrt{f'c}$					
fcv =	13.60	MPa				
ϕVuo =	14,460	kN	>	V* =	144 kN	**OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	20.00 kNm		Es	=	200000 MPa
Ml	=	20.00 kNm		Ec	=	31975.3505 MPa
Mo	=	40 kNm				
Ms	=	20.00 kNm				
Ig	=	2.34E+10 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2830 mm	A_{sc} =	2199.11486 mm^2
webwidth	b_w =	2250 mm	A_{st} =	2199.11486 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)

1125 dn^2 + 25311.012 dn - 7E+06) = 0

dn = 66.282598 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

= 1.60E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

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

= 1.23E+14 mm^4

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

b_w =	2250 mm	A_{st} =	2199.1 mm^2
d_{sc} =	97 mm	A_{sc} =	2199.1 mm^2
d =	383 mm		

Ig = 2.34E+10 mm^4

At right end (arbitrary)

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

1125 dn^2 + 25311.012 dn - 6E+06) = 0

dn = 64.95 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.60E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 355.756237 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)′ = 1.23E+14 mm^4

At left end (arbitrary)

b_w =	**2250** mm
d_{sc} =	**97** mm
A_{st} =	**2199.11** mm^2
A_{sc} =	**2199.11** mm^2
d =	**383** mm

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

1125 dn^2 + 25311.012 dn - 6E+06) = 0

dn = 64.946213 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.60E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.23E+14 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.23E+14 mm^4

Midspan Deflection :

Lef = 2900 mm

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

= 3.566E-06 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 148.20 kN/m

w - short term = 148.20 kN/m

Defln,s,sus = 3.57E-06 mm

At midspan : Ast = 2199.11486 mm^2

Asc = 2199.11486 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)

= 0.00001 mm

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

ALLOWABLE DEFLN = Lef / 500 : 5.8 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = 0.3 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 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	I	Ec	Es	μ	k's	L	λ	λL	Is	Is	Analysis	ks = k'sB
	m	m^4	GPa	GPa	MPa		m			$\lambda L < \pi/4$	$\lambda L > \pi$	method ?	kNm
1	0.30	6.75E-04	31.9754	0.18	0.4	0.11128	7.50	1.0655	7.9914	No	Yes	Use Winkler	370932

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	28.32	kN/m	(plus super)
Self wt	:	Wsw =	7.50	kN/m	
Blockwall	:	Wb =	16.74	kN/m	
TOTAL		=	52.56	kN/m	
LL :		Wll =	10	kN/m	
LL factor	- long term		0.4		
	- short term		0.7		

2 Beam Section Properties

Beam Size

bw	1000	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	7500	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	7100	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	1210.00	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	2420.00	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	mm
Fsy =	410	MPa	Ec =	31975.3505	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

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

Effective width =	2420 mm		clause 8.8.2	
Is the section RECT or T , L (ans : 1 or 2)		=	1	
k_1 =	0.045		(RECT sections)	1
=	0.0251734		(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 =	99.7584			
defln/Lef =	0.004	k_1/k_2 =	17.2797788	
d =	189 mm			
take d =	193 mm	Distance from d to extreme fibre of beam (do) =	107 mm	

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

Mmax -ve	=	140.50	kNm	(left support)
Mmax +ve	=	67.60	kNm	(mid-span)
M -ve	=	4.90	kNm	(right support)
Vmax	=	105.00	kN	
V (other end)	=	105.00	kN	
N*	=	74.00	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine - ve steel required :</u> (At arbitrary left support) Bar Spacing [illegible] mm

Number of reinforcement layers 1 or 2 :			1			
take do =	107 mm		d =	193		
Estimate Area of main steel		Ast =	2320.99243 mm^2			
13	Y 20	Ast =	4084.07045 mm^2	OUTSIDE		
		p =	0.02116099			
13	Y 20	Asc =	4084.07045 mm^2	INSIDE		
		pc =	0.02116099	<	pd =	0.025409
		p-pc =	0	pd=0.4*0.85*μ*f'c/fsy		

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

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)]

=	145	kNm	>	M* =	140.50	kNm	OK

Determine -ve steel at right support :

M -ve = 4.90 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 107 mm dst = 193 mm

Estimate Area of main steel		Ast =	80.9456433 mm^2			
13	Y 20	Ast =	4084.07045 mm^2	OUTSIDE		
		p =	0.02116099			
13	Y 20	Asc =	4084.07045 mm^2	INSIDE		
		pc =	0.02116099	<	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)]

=	145	kNm	>	M* =	4.90	kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 67.6 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 = 183 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1177.74241 mm^2

13	Y 20	Ast =	4084.07045 mm^2	OUTSIDE
13	Y 20	Asc =	4084.07045 mm^2	INSIDE

Assume N - A in flange

Effective width : beff = 2420 mm

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

= 0.14517871

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)]

=	260 kNm	>	M*+ve =	67.60 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	13	Y 20
	Asc	=	13	Y 20
Right support :	Ast	=	13	Y 20
	Asc	=	13	Y 20
midspan	Ast	=	13	Y 20
	Asc	=	13	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 = 2130 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 = 2130 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 = 710 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 = 314.159265 mm^2

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

factor = 56006.2413

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

= 329.4485 mm
take fs/Fsy = M*/Phi Mu = 0.971148179
therefore

Lsy =	320 mm
say =	650 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 = 314.159265 mm^2
k2 = 2.2 a_b = 1174 mm (dist between bars)
factor = 44804.9931
Lsyt = 263.5588 mm take fs/Fsy = M*/Phi Mu = 0.259550352
therefore

Lsy =	68 mm
say =	500 mm

≥ 25k1db 500

9 **SHEAR CHECK**

V* = 105.00 kN Dist to centreline of bottom steel from soffit
critical location : d = 193 mm dq = 93 mm
do = 207 mm
V* = 93 kN beta 1 = 1.2965 (but not less than 1.1)
hence = 1.2965
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
= 173.62 kN
phi Vumax = 1159.2 kN > V* No web crushing
phi Vumin = 261 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 = 4977 mm^2
Asv = 2 x area of stirrups = 400 mm^2 2 Ties
theta = 30.45724 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 440.47 kN
Phi Vu = 308 kN > V* = 92.92592 kN
hence

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

Other Support :
V* = 105.00 kN
Dist to centreline of bottom steel from soffit
critical location : d = 193 mm dq = 93 mm
do = 207 mm
V* = 92.93 kN beta 1 = 1.2965 (but not less than 1.1)
hence = 1.2965
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
= 173.6212 kN
phi Vumax = 1159.2 kN > V* No web crushing
phi Vumin = 261 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 = 4976.91653 mm^2
Asv = 2 x area of stirrups = 600 mm2 2 Ties
theta = 31.09272 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 529.4924 kN
Phi Vu = 370.6447 kN > V* = 92.93 kN
hence

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

11 **SERVICEABILITY CHECK**

From an Elastic Linear Analysis :

Mr	=	140.50 kNm		Es	=	200000 MPa
Ml	=	4.90 kNm		Ec	=	31975.35051 MPa
Mo	=	140.3 kNm				
Ms	=	67.60 kNm				
Ig	=	2.25E+09 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2420 mm	A_{sc} =	4084.07045 mm^2
webwidth	b_w =	1000 mm	A_{st} =	4084.07045 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	183 mm
			d =	193 mm
			D =	300 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 + 47006.1648 dn - 7441150.22) = 0

dn = 83.7297521 mm

d = 183 mm

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

= 4.47E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	1.52E+09 mm^4

<u>Calculate Icr , Mcr, ILef and Iref at end supports :</u>

b_w =	1000 mm	A_{st} =	4084.07045 mm^2
d_{sc} =	97 mm	A_{sc} =	4084.07045 mm^2
d =	193 mm		

Ig = 2.25E+09 mm^4

<u>At right end (arbitrary)</u>

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

500 dn^2 + 47006.1648 dn - 7011929.28) = 0

dn = 80.40 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.97E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 6.14E+08 mm^4

<u>At left end (arbitrary)</u>

b_w =	**1000** mm
d_{sc} =	**97** mm
A_{st} =	**4084.07** mm^2
A_{sc} =	**4084.07** mm^2
d =	**193** mm

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

500 dn^2 + 47006.1648 dn - 7011929.28) = 0

dn = 80.4043453 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 4.97E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.75E+12 mm^4

Iav	=	[Im + (IL + Ir)/2]/2	=	6.88E+11 mm^4

Midspan Deflection :

Lef = 7100 mm

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

= 0.01266543 mm

<u>Total Deflection :</u>

Long term factor	=	0.2
Short term factor	=	0.5
w - long term	=	54.56 kN/m
w - short term	=	57.56 kN/m
Defln,s,sus	=	0.012005315 mm

At midspan : Ast = 4084.07045 mm^2

Asc = 4084.07045 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)
	=	0.02227 mm

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

ALLOWABLE DEFLN	=	Lef / 250 =	28.4 mm
			> 0.9 x Total Defln

Job name: PS01WALL Designer: FYP Date: 6 Nov 2011, 10:31 am Page: 1

POMSSUP

WALL FOR PS-11

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PS01WALL	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	4	(2500)
Nodes with spring restraints	3	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	0	(10000)
Member distributed forces	6	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	8	
Total degrees of freedom	12	
Primary load cases	2	(100)
Mass load cases	0	(100)

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

case 8: 1.4D+1.7L+1.7Q

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

MEMBER FORCES AND MOMENTS (kN,kNm)

(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	74.001*	0.000	0.000	0.000	0.000	0.000
1	3	0.000#	-34.457	0.000	0.000	0.000	32.660
5	3	0.000	33.931*	0.000	0.000	0.000	0.000
1	3	0.000	-34.457#	0.000	0.000	0.000	32.660
1	3	0.000	-34.457	0.000	0.000	0.000	32.660*
4	3	0.000	15.903	0.000	0.000	0.000	-7.938#

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
6	3	36.728*	0.000	0.000	0.000	0.000	0.000
1	1	0.000#	52.858	0.000	0.000	0.000	0.000
1	8	0.000	74.001*	0.000	0.000	0.000	0.000
1	3	34.457	0.000#	0.000	0.000	0.000	-32.660
1	1	0.000	52.858	0.000	0.000	0.000	0.000*
1	3	34.457	0.000	0.000	0.000	0.000	-32.660#

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	74.001*	0.000	0.000	0.000	0.000	0.000
5	7	0.000#	33.931	0.000	0.000	0.000	0.000
5	3	0.000	33.931*	0.000	0.000	0.000	0.000
1	3	0.000	-34.457#	0.000	0.000	0.000	32.660
1	3	0.000	-34.457	0.000	0.000	0.000	32.660*
5	3	0.000	-0.565	0.000	0.000	0.000	-14.320#

Job name: PS01WALL   Designer: FYP   Date: 9 Nov 2011, 1:29 pm   Page: 1

POMSSUP

Circular Pump Well

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PS01WALL	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	4	(2500)
Nodes with spring restraints	3	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	1	(10000)
Member distributed forces	6	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	8	
Total degrees of freedom	12	
Primary load cases	3	(100)
Mass load cases	0	(100)

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

```
1.400 * Load case  1: D
1.700 * Load case  2: L
1.000 * Load case  3: E
1.700 * Load case  4: Q
1.400 * Load case  5: F
```

LOAD CASE TITLES

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

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	74.001*	0.000	0.000	0.000	0.000	0.000
1	3	0.000#	-34.457	0.000	0.000	0.000	32.660
4	12	24.667	179.154*	0.000	0.000	0.000	-69.086
4	12	49.334	-166.833#	0.000	0.000	0.000	4.864
1	12	74.001	-105.035	0.000	0.000	0.000	140.512*
4	12	24.667	179.154	0.000	0.000	0.000	-69.086#

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

Envelope = All Load Cases
and All Nodes

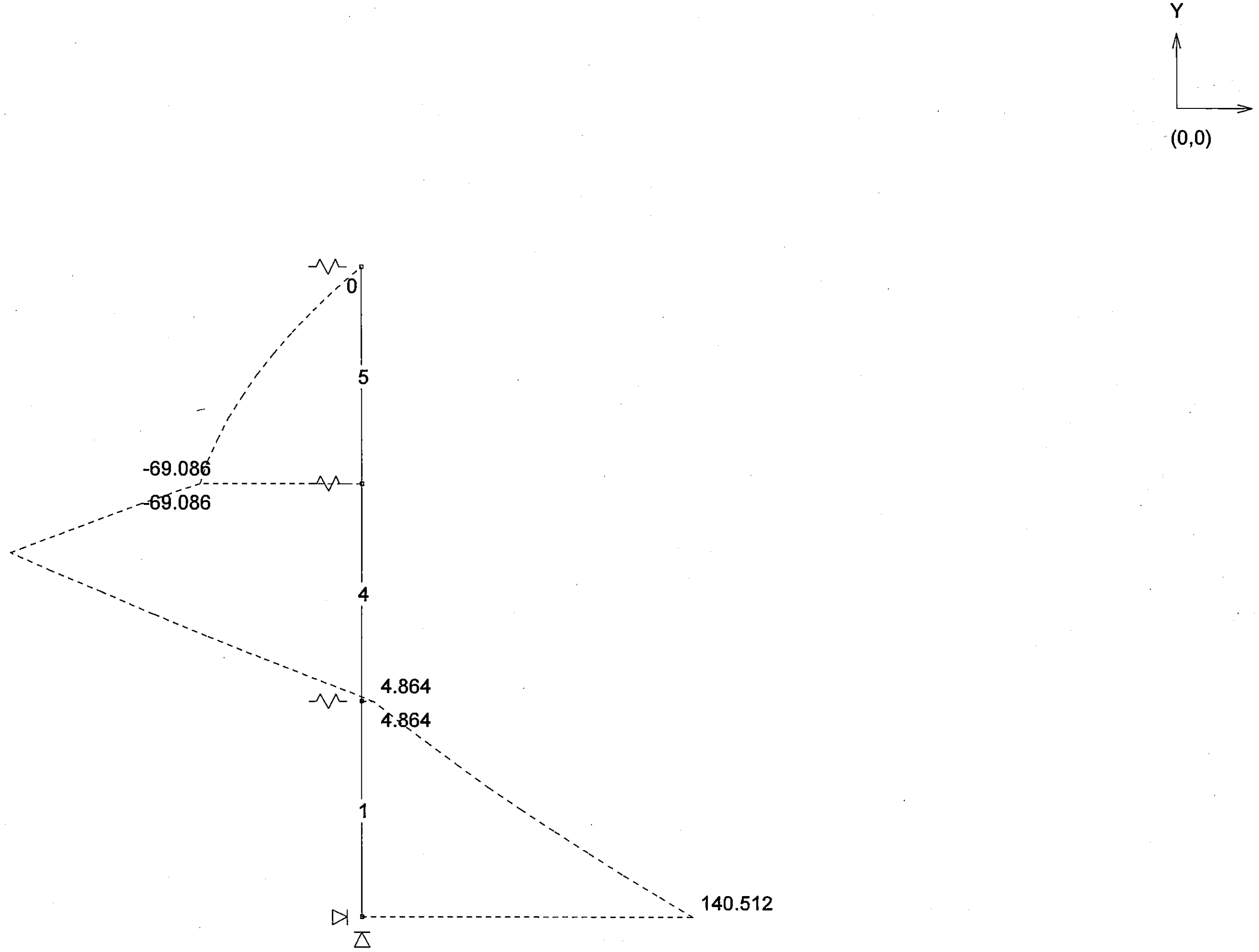
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
6	12	158.287*	0.000	0.000	0.000	0.000	0.000
1	1	0.000#	52.858	0.000	0.000	0.000	0.000
1	8	0.000	74.001*	0.000	0.000	0.000	0.000
1	3	34.457	0.000#	0.000	0.000	0.000	-32.660
1	1	0.000	52.858	0.000	0.000	0.000	0.000*
1	12	105.035	74.001	0.000	0.000	0.000	-140.512#

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	74.001*	0.000	0.000	0.000	0.000	0.000
5	7	0.000#	33.931	0.000	0.000	0.000	0.000
4	12	24.667	179.154*	0.000	0.000	0.000	-69.086
4	12	49.334	-166.833#	0.000	0.000	0.000	4.864
1	12	74.001	-105.035	0.000	0.000	0.000	140.512*
4	12	32.516	-140.318	0.000	0.000	0.000	-149.241#

12 1.7D+1.7L+E+1.7Q+1.4F

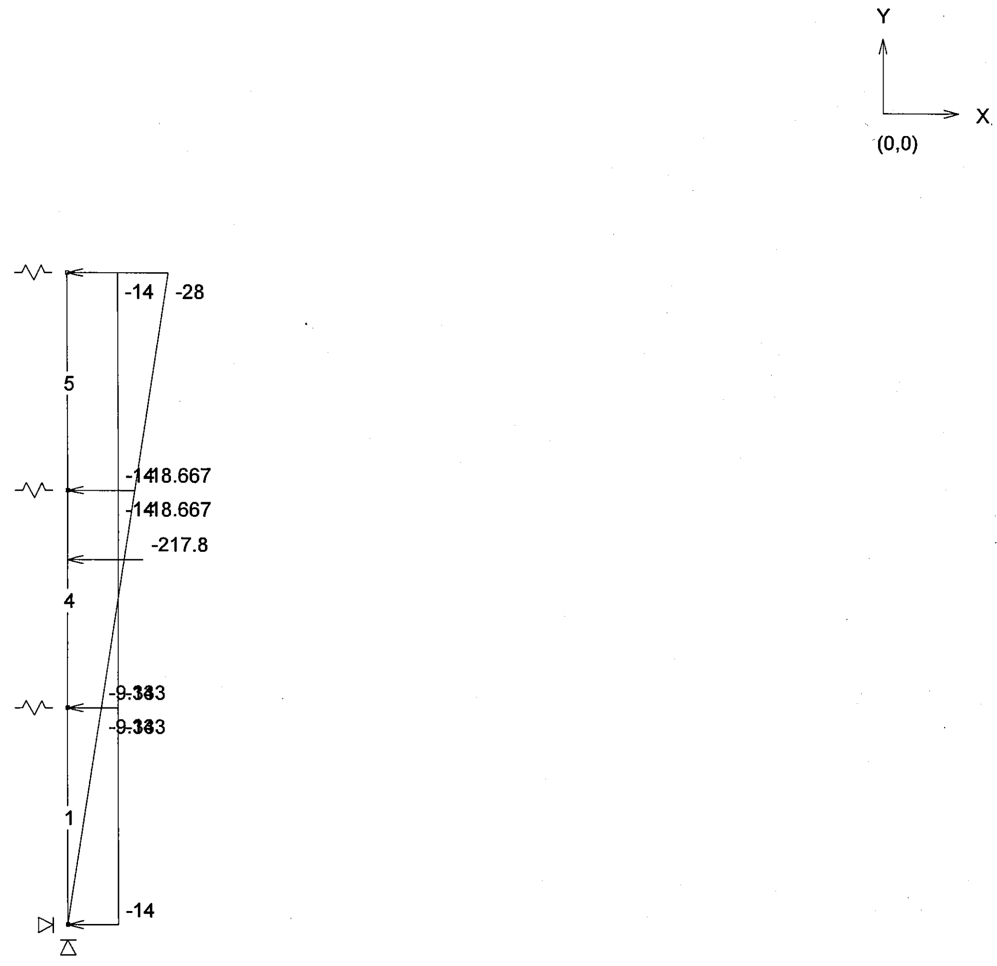


Sections:
1 500 x 1000

Bending Moment Diagram

Job: PS01WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:51, Axes: XY
Load: 2.1 Disp: None Moment: 3.2 Shear: None Axial: None
POMSSUP
Circular Pump Well 9 Nov 2011, 1:29 pm

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



Sections:
1 500 x 1000

Circular Pump Well Model

Job: PS01WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:51, Axes: XY
Load: 2.1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Circular Pump Well 9 Nov 2011, 1:28 pm

SUMMARY

A. Base Slab

500 thk RC Slab

Reinforcement:	Y20 bars @ 300 cts	Top & Bottom Each Way
T & S	Y20 bars @ 300 cts	Top & Bottom Each Way

B. Pit Wall

300 thk RC Wall

Reinforcements:	Vertical:	Y20 - 200 cts	Outside Face
		Y20 - 200 cts	Inside Face
	Horizontal	Y12 - 300 cts	

A-2: Pump Well - PS 04-05, 10 & 17

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

Load Calculation

Data

1	Density of Water (ρ) =	9.81	kN/m^3
	Height of Water, h =	3.5	m
	Wall Thickness, tw =	0.7	m
	Height of Wall, hw =	10.4	m
	Base Slab Thickness, tb =	0.7	m
	Length of Base slab, Lb =	5.9	m

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

Atmospheric Pressure, Patm =	0	kPa
Hydrostatic Pressure along 1 m width wall, Pw	34.34	kN/m (=kPa/m)

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

Water Pressure on base slab, Pw = ρgh 34.34 kPa

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

Flp = 34.34 kN/m

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	**0.70**	m
Height of Wall (H) =	**10.40**	m
Embedded Height of Wall (He) =	**10.20**	m
(Heel Depth) Thickness of Base (tb) =	**0.70**	m
Width of Base (wb) =	**1.00**	m
(Heel)/ Length of base (Lb) =	**4.90**	m
Toe Depth (td) =	0.70	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) =	175.30	LE =	3.400
PS(G) =	20.22	LS =	5.10
PS(Q) =	0.00	LS =	5.10
Total, Pa =	195.52	La =	3.58

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	175.30	3.40	596.02
PS =	20.22	5.10	103.12
Total	195.52	**3.58**	699.14

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	182	4.90	891.8
Wall wt =	169.75	4.90	831.8
Base wt =	85.75	2.45	210.1
Total =	437.50	2.38	1933.7

CHECK GROUND BEARING FAILURE

$\sum$MO = 699.14 kNm/m

$\sum$MR = 1933.7 kNm/m

$\sum$F = 437.50 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

e = B/2 - [$\sum$MR - $\sum$MO]/$\sum$F 0.37 m **OK, e < B/6**

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

qtoe = $\sum$F/B [1+6e/B] = 0.71 kPa

qheel = $\sum$F/B [1-6e/B] = 0.26 kPa

Maximum Bearing Pressure = 0.71 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

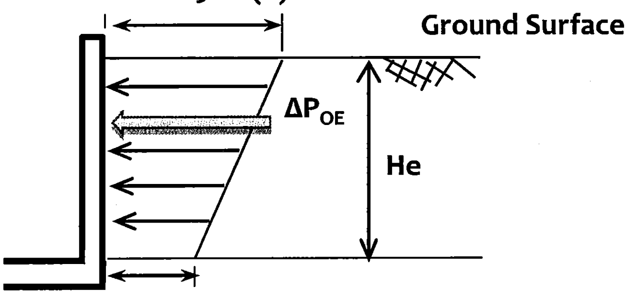
Seismic Design Data

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

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 = 10.2 m
Ah(0) = 1
$C_h(0) = \alpha\, \beta\, A_h(0)\, A_p$, horizontal force coefficient = 0.131
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 =	11 kN/m	UDL
Eu2 =	23 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

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

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

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

--> $\lambda L < \pi/4$ Rigid member - Bending not influenced much by ks   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	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
2	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
3	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
4	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
5	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
6	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
7	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
8	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

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

Design of a Continuous Beam :

1 **Loads**

1 **Serviceability Load Combination : Long term loads**

Main Load Dead Load =	33.60	kN/m	Self weight =	28.13	kN/m
Total G =	61.73	kN/m	Flp =	34.34	kN/m
Fe =	195.52	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	257.25	kN/m	COMB. 2	G + Flp =	96.06 kN/m
COMB. 3	G + Flp + Fgw =	96.06	kN/m		Long term Load Factors =	1

2 **Beam Section Properties**

Beam Size

bw =	2250	mm	D =	500	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	5000	mm	Overall depth of sect. D =	500	mm
Effective Length,Lef =	4600	mm	Area, A =	1.13E+06	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	2.34E+10	mm^4
Neutral Axis, xc =	1585.00	mm	Moment of Inertia, Iy-y =	4.75E+11	mm^4
Effective width, beff =	3170.00	mm	Gross Area, Ag =	1.13E+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 = 3170 mm clause 8.8.2

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

k_1 = 0.045 (RECT sections) 1

= 0.0342397 (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 = 101.229

defln/Lef = **0.004** k_1/k_2 = 17.28

d = 112 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	=	176.00	kNm	(left support)
Mmax +ve	=	190.00	kNm	(mid-span)
M -ve	=	176.00	kNm	(right support)
Vmax	=	218.00	kN	
V (other end)	=	218.00	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 = 383

Estimate Area of main steel Ast = 1465.1 mm^2

10 Y 20 Ast = 3141.59 mm^2 **SPACING = 210** mm

p = 0.0036

10 Y 20 Asc = 3141.59 mm^2

pc = 0.0036 < pd = 0.0254088

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)]

= 332 kNm > M* = 176.00 kNm	OK

Determine -ve steel at right support :

M -ve = 176.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst : 383 mm

Estimate Area of main steel Ast = 1465.1 mm^2

10 Y 20 Ast = 3141.59 mm^2

p = 0.0036

10 Y 20 Asc = 3141.59 mm^2

pc = 0.0036 < pd = 0.0254088

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)]

= 332 kNm > M* = 176.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 190 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) = 1581.6469 mm^2

Bottom 10 Y 20 Ast = 3141.59265 mm^2

Top 10 Y 20 Asc = 3141.59265 mm^2

Assume N - A in flange

Effective width : beff = 3170 mm

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

= 0.040735

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)]

= 437 kNm > M*+ve = 190.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	10	Y 20
	Asc	=	10	Y 20
Right support :	Ast	=	10	Y 20
	Asc	=	10	Y 20
midspan	Ast	=	10	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 = 1380 mm ?

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

For -ve steel = 2 bars

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

= 0.3 x le = 1380 mm ?

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

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 1038 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 = 1058 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1038 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 = 0.530848

therefore

Lsy =	175 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 = 1232 mm

k1 = 1 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1212 mm (dist between bars)

factor = 44805

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.43471803

therefore

Lsy =	115 mm
say =	500 mm

≥ 25k1db 500

9 PUNCHING SHEAR CHECK

Wall Dimension

V* = 218.00 kN b = 500 mm

Dist to centreline of bottom steel from soffit d = 500 mm

Critical shear perimeter, μ

dq = 85 mm μ = 2(b+dom) + 2(d+dom)

dom = 415 mm 3660 mm

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

βh = 1 longer side/shorter side

fcv =	$0.17(1+2/\beta h)\sqrt{f'c} \le 0.34\sqrt{f'c}$				
fcv =	13.60 MPa				
ϕVuo =	14,460 kN	>	V* =	218 kN	**OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	176.00 kNm	Es	=	200000 MPa
Ml	=	176.00 kNm	Ec	=	31975.3505 MPa
Mo	=	366 kNm			
Ms	=	190.00 kNm			
Ig	=	2.34E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	3170 mm	A_{sc} =	3141.59265 mm^2
webwidth	b_w =	2250 mm	A_{st} =	3141.59265 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)

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 77.015006 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

= 2.18E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

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

= 1.42E+11 mm^4

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

b_w =	2250 mm	A_{st} =	3141.6 mm^2
d_{sc} =	97 mm	A_{sc} =	3141.6 mm^2
d =	383 mm		

Ig = 2.34E+10 mm^4

At right end (arbitrary)

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

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 75.43 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.18E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 355.756237 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 1.78E+11 mm^4

At left end (arbitrary)

b_w =	**2250** mm
d_{sc} =	**97** mm
A_{st} =	**3141.59** mm^2
A_{sc} =	**3141.59** mm^2
d =	**383** mm

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

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 75.425002 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.18E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.78E+11 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.60E+11 mm^4

Midspan Deflection :

Lef = 4600 mm

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

= 0.0668084 mm

Total Deflection :

Long term factor	=	1
Short term factor	=	1
w - long term	=	257.25 kN/m
w - short term	=	257.25 kN/m
Defln,s,sus	=	0.066808 mm

At midspan :

Ast = 3141.59265 mm^2

Asc = 3141.59265 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)

= 0.12026 mm

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

ALLOWABLE DEFLN = Lef / 500 : 9.2 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw, eff =	1000	mm	D =	850	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	8600	mm	Overall depth of Sect. D =	850	mm
Effective Length,Lef =	8200	mm	Area, A =	8.50E+05	mm^2
Neutral Axis, yc =	425.00	mm	Moment of Inertia, Ix-x =	5.12E+10	mm^4
Neutral Axis, xc =	1320.00	mm	Moment of Inertia, Iy-y =	7.08E+10	mm^4
Effective width, beff =	2640.00	mm	Gross Area, Ag =	8.50E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	850	mm
Fsy =	410	MPa	Ec =	31975.3505	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

4 INITIAL CHOICE OF SECTION

Effective width = 2640 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.02423838	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	29.96		
defln/Lef	=	0.004	k_1/k_2 = 17.2797788	

d = 142 mm

take d = 731 mm Distance from d to extreme fibre of beam (do) = 119 mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	4300.60	kNm	(left support)	1.116
Mmax +ve	=	1847.60	kNm	(mid-span)	
M -ve	=	648.42	kNm	(right support)	
Vmax	=	1154.80	kN		
V (other end)	=	1154.80	kN		
N*	=	202.50	kN	Axial Load	

6 FLECTURAL STRENGTH STEEL

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

Number of reinforcement layers 1 or 2 : 1

take do = 119 mm d = 731

Estimate Area of main steel		Ast =	18757.1297	mm^2		
23	Y 32	Ast =	18497.69754	mm^2	OUTSIDE	
		p =	0.02530465			
24	Y 32	Asc =	19301.94526	mm^2	INSIDE	
		pc =	0.02640485	<	pd =	0.025409
		p-pc =	-0.0011002	pd=0.4*0.85*µ*f'c/fsy		

Check M* <= phiMu

ku = 0 dsc = 103 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)]

= 4473 kNm > M* = 4300.60 kNm OK

Determine - ve steel at right support :

M -ve = 648.42 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 119 mm dst = 731 mm

Estimate Area of main steel Ast = 2828.0933 mm^2

23	Y 32	Ast =	18497.69754 mm^2		OUTSIDE
		p =	0.02530465		
24	Y 32	Asc =	19301.94526 mm^2		INSIDE
		pc =	0.02640485	<	pd = 0.025409
		p-pc =	-0.0011002		

Check M* <= phiMu

ku = 0 dsc = 103 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)]

= 4473 kNm > M* = 648.42 kNm OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 1847.6 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 135 mm

what is the code ? 2 d = 715 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 8238.66075 mm^2

23	Y 32	Ast =	18497.6975 mm^2	OUTSIDE
24	Y 32	Asc =	19301.9453 mm^2	INSIDE

Assume N - A in flange

Effective width : beff = 2640 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.15427085

hence dn = 110 < 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)]

= 4592 kNm > M*+ve = 1847.60 kNm OK

Summary of reinforcement requirement :

left support :	Ast =	23	Y 32
	Asc =	24	Y 32
Right support :	Ast =	23	Y 32
	Asc =	24	Y 32
midspan	Ast =	23	Y 32
	Asc =	24	Y 32

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 = 2460 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 = 2460 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 = 820 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 32 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 182 mm
C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 858 mm
coaxial with and surrounding a bar

k1 = 1.25 A_b = 804.247719 mm^2

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

factor = 143375.978

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

= 787.7801 mm

take fs/Fsy = M*/Phi Mu = 0.961483777

therefore Lsy = 757 mm ≥ 25k1db 1000
say = 750 mm

For Bottom Steel :

Cover, c = 75 mm d_b = 32 mm
# of lines of bars = 1

If ab > 2c   C = 2c + d_b = 182 mm
if ab < 2c   C = a_b + d_b = 1206 mm
k1 = 1 A_b = 804.247719 mm^2
k2 = 2.2 a_b = 1174 mm   (dist between bars)
factor = 114700.782
Lsyt = 630.2241 mm   take fs/Fsy = M*/Phi Mu = 0.402353533
therefore | Lsy = 254 mm | ≥ 25k1db 800
| say = 900 mm |

9 **SHEAR CHECK**

V* = 1154.80 kN   Dist to centreline of bottom steel from soffit

critical location : d = 731 mm   dq = 93 mm
do = 757 mm
V* = 1140 kN   beta 1 = 1.0215 (but not less than 1.1)
hence = 1.1
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
= 578.48 kN
phi Vumax = 4239.2 kN > V* No web crushing
phi Vumin = 896 kN > V* SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 200 mm   Fsyf = 410 MPa
Determine theta :
bv x s/Fsy f = 487.804878
Asv.min = 170.731707 mm^2
Asv.max = 3370 mm^2
Asv = 2 x area of stirrups = 400 mm^2 2 Ties
theta = 31.07497 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 1856.43 kN
Phi Vu = 1300 kN > V* = 1140.18 kN
hence

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

Other Support :
V* = 1154.80 kN
Dist to centreline of bottom steel from soffit

critical location : d = 731 mm   dq = 93 mm
do = 757 mm
V* = 1140.18 kN   beta 1 = 1.0215 (but not less than 1.1)
hence = 1.1
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
= 578.474 kN
phi Vumax = 4239.2 kN > V* No web crushing
phi Vumin = 896 kN < V* SHEAR REO REQ'D
If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 200 mm   Fsyf = 410 MPa
Determine theta :
bv x s/Fsy f = 487.804878
Asv.min = 170.731707 mm^2
Asv.max = 3369.91886 mm^2
Asv = 2 x area of stirrups = 600 mm2   2 Ties
theta = 32.01271 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 2315.744 kN
Phi Vu = 1621.021 kN > V* = 1140.18 kN
hence

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

11 **SERVICEABILITY CHECK**

From an Elastic Linear Analysis :

Mr	=	4300.60 kNm		Es	=	200000 MPa
Ml	=	648.42 kNm		Ec	=	31975.35051 MPa
Mo	=	4322.11 kNm				
Ms	=	1847.60 kNm				
Ig	=	5.12E+10 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2640 mm	A_{sc} =	19301.94526 mm^2
webwidth	b_w =	1000 mm	A_{st} =	18497.69754 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	135 mm
			d_{st} =	715 mm
			d =	731 mm
			D =	850 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 + 217127.943 dn - 98269313.1) = 0

dn = 276.514812 mm

d = 715 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2
= 2.93E+10 mm4
Mcr = 0.6sqrtF'c x bD^2/6 = 456.949122 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]
= 2.96E+10 mm^4

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

b_w = 1000 mm A_{st} = 18497.6975 mm^2
d_{sc} = 103 mm A_{sc} = 19301.9453 mm^2
d = 731 mm
Ig = 5.12E+10 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
500 dn^2 + 217127.943 dn - 95023610.4) = 0
dn = 269.90 mm
d = 731 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.12E+10 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 456.949122 kNm
Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 3.12E+10 mm^4

At left end (arbitrary)

b_w = **1000** mm
d_{sc} = **103** mm
A_{st} = **18497.7** mm^2
A_{sc} = **19301.95** mm^2
d = **731** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
500 dn^2 + 217127.943 dn - 95023610.4) = 0
dn = 269.895428 mm
d = 731 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.12E+10 mm^4
Mcr = 0.6sqrtF'c x bD^2/6 = 456.949122 kNm
Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 3.82E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 3.21E+10 mm^4

Midspan Deflection :

Lef = 8200 mm
Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 9.21737896 mm

Total Deflection :

Long term factor = 0.2
Short term factor = 0.5
w - long term = 12 kN/m
w - short term = 15 kN/m
Defln,s,sus = 7.373903164 mm
At midspan : Ast = 18497.6975 mm^2
Asc = 19301.9453 mm^2
Asc/Ast = 1.043478261
kcs = 2 - (1.2 x Asc/Ast) >= 0.8
= 0.8

therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)
= 15.11650 mm

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

ALLOWABLE DEFLN = Lef / 250 = 32.8 mm
> 0.9 x Total Defln

Serviceability is OK!

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw, eff =	1000 mm	D =	500 mm
tff =	0 mm	Cover =	75 mm
Actual length, L =	2866.66667 mm	Overall depth of Sect. D =	500 mm
Effective Length,Lef =	2466.66667 mm	Area, A =	5.00E+05 mm^2
Neutral Axis, yc =	250.00 mm	Moment of Inertia, Ix-x =	1.04E+10 mm^4
Neutral Axis, xc =	746.67 mm	Moment of Inertia, Iy-y =	4.17E+10 mm^4
Effective width, beff =	1493.33 mm	Gross Area, Ag =	5.00E+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 <u>INITIAL CHOICE OF SECTION</u>

Effective width	=	1493.33333 mm	clause 8.8.2	
Is the section RECT or T , L (ans : 1 or 2)	=			1
k_1	=	0.045	(RECT sections)	1
	=	0.03290273	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	29.96		
defln/Lef	=	0.004	k_1/k_2 = 17.27977882	
d =	52 mm			
take d =	381 mm		Distance from d to extreme fibre of beam (do) =	119 mm

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

Mmax -ve	=	659.10 kNm	(left support)
Mmax +ve	=	362.50 kNm	(mid-span)
M -ve	=	278.90 kNm	(right support)
Vmax	=	526.00 kN	
V (other end)	=	526.00 kN	
N*	=	202.50 kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine - ve steel required :</u> (At arbitrary left support) Bar Spacing [illegible] mm

Number of reinforcement layers 1 or 2 :				1	
take do =	119 mm	d =		381	
Estimate Area of main steel		Ast =	5515.451171 mm^2		
8	Y 32	Ast =	6433.981755 mm^2	OUTSIDE	
		p =	0.016887091		
8	Y 32	Asc =	6433.981755 mm^2	INSIDE	
		pc =	0.016887091	< pd =	0.025409
		p-pc =	0	pd=0.4*0.85*μ*f'c/fsy	

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

ku =	0	dsc =	103 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)]

=	660	kNm	>	M* =	659.10 kNm	OK

Determine - ve steel at right support :

M -ve = 278.90 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 119 mm dst = 381 mm

Estimate Area of main steel Ast = 2333.878519 mm^2

8 Y 32 Ast = 6433.981755 mm^2 OUTSIDE

p = 0.016887091

8 Y 32 Asc = 6433.981755 mm^2 INSIDE

pc = 0.016887091 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 103 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)]

=	660	kNm	>	M* =	278.90 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 362.5 kNm

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

" " two " " code = 2 bar/s and soffit = 119 mm

what is the code ? 1 d = 381 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 3033.4563 mm^2

8 Y 32 Ast = 6433.981755 mm^2 OUTSIDE

8 Y 32 Asc = 6433.981755 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1493.333333 mm

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

= 0.17802224

hence dn = 68 < 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)]

=	843 kNm	>	M*+ve =	362.50 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 32
	Asc	=	8	Y 32
Right support :	Ast	=	8	Y 32
	Asc	=	8	Y 32
midspan	Ast	=	8	Y 32
	Asc	=	8	Y 32

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 = 740 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 = 740 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 = 246.6666667 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 32 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 182 mm
C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 858 mm
coaxial with and surrounding a bar

k1 = 1.25 A_b = 804.2477193 mm^2

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

factor = 143375.9779

Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 787.7801 mm

take fs/Fsy = M*/Phi Mu = 0.998620149

therefore	Lsy =	787 mm	≥	25k1db	1000
	say =	750 mm			

For Bottom Steel :

Cover, c = 75 mm d_b = 32 mm
# of lines of bars = 1

If ab > 2c C = 2c + d_b = 182 mm

if ab < 2c C = a_b + d_b = 1206 mm

k1 = 1 A_b = 804.2477193 mm^2

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

factor = 114700.7823

Lsyt = 630.2241 mm take fs/Fsy = M*/Phi Mu = 0.430076774

therefore	Lsy =	271 mm	≥	25k1db	800
	say =	600 mm			

9 SHEAR CHECK

V* = 526.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 381 mm dq = 93 mm
do = 407 mm

V* = 518 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
= 292.64 kN

phi Vumax = 2279.2 kN > V* No web crushing

phi Vumin = 464 kN > V* SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 487.804878

Asv.min = 170.731707 mm^2

Asv.max = 3401 mm^2

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

theta = 31.0645 degrees

Determine Phi Vu :

Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 972.08 kN

Phi Vu = 680 kN > V* = 518.38 kN

hence

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

Other Support :

V* = 526.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 381 mm dq = 93 mm

do = 407 mm

V* = 518.38 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

= 292.6308 kN

phi Vumax = 2279.2 kN > V* No web crushing

phi Vumin = 464 kN < V* SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 487.804878

Asv.min = 170.731707 mm^2

Asv.max = 3401.39757 mm^2

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

theta = 31.9931 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 1219.402 kN

Phi Vu = 853.5816 kN > V* = 518.38 kN

hence

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

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 659.10 kNm Es = 200000 MPa

Ml = 278.90 kNm Ec = 31975.35051 MPa

Mo = 831.5 kNm

Ms = 362.50 kNm

Ig = 1.04E+10 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width b_{eff} = 1493.333333 mm A_{sc} = 6433.981755 mm^2

webwidth b_w = 1000 mm A_{st} = 6433.981755 mm^2

Flange thickness t_{ff} = 0 mm d_{sc} = 119 mm

d_{st} = 381 mm

d = 381 mm

D = 500 mm

n = Es/Ec = 6.25481807

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

500 dn^2 + 74052.7889 dn - 19356048.83) = 0

dn = 136.175452 mm

d = 381 mm

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

= 3.25E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 158.113883 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	3.85E+09 mm^4

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

b_w = 1000 mm A_{st} = 6433.981755 mm^2

d_{sc} = 103 mm A_{sc} = 6433.981755 mm^2

d = 381 mm

Ig = 1.04E+10 mm^4

At right end (arbitrary)

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

500 dn^2 + 74052.7889 dn - 18815098.37) = 0

dn = 133.59 mm

d = 381 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.26E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 158.113883 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 3.36E+09 mm^4

At left end (arbitrary)

b_w = 1000 mm

d_{sc} = 103 mm

A_{st} = 6433.9818 mm^2

A_{sc} = 6433.9818 mm^2

d = 381 mm

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

500 dn^2 + 74052.7889 dn - 18815098.37) = 0

dn = 133.586351 mm

d = 381 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 3.26E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 158.113883 kNm

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

Iav = [Im + (IL + Ir)/2]/2 = 3.90E+09 mm^4

Midspan Deflection :

Lef = 2466.66667 mm

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

= 1.36425751 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 12 kN/m

w - short term = 15 kN/m

Defln,s,sus = 1.091406004 mm

At midspan : Ast = 6433.98175 mm^2

Asc = 6433.98175 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)

= 2.23738 mm

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

ALLOWABLE DEFLN = Lef / 250 = 9.86666667 mm

> 0.9 x Total Defln

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw, eff =	1000	mm	D =	300	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	2866.66667	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	2466.66667	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	746.67	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	1493.33	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	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 =	1493.33333 mm	clause 8.8.2		
Is the section RECT or T , L (ans : 1 or 2)	=		1	
k_1 =	0.045	(RECT sections)		1
=	0.03290273	(T and L sections)		2
what type of span?	5	interior span =		5
		end span =		6
k_2 =	0.0026042			
kcs =	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef =	29.96			
defln/Lef =	0.004	k_1/k_2 =	17.27977882	
d =	52 mm			
take d =	181 mm	Distance from d to extreme fibre of beam (do) =	119 mm	

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	278.90	kNm	(left support)
Mmax +ve	=	129.20	kNm	(mid-span)
M -ve	=	0.00	kNm	(right support)
Vmax	=	632.00	kN	
V (other end)	=	0.00	kN	
N*	=	266.00	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine -ve steel required : (At arbitrary left support) Bar Spacing = 124 mm

Number of reinforcement layers 1 or 2 :				1		
take do =	119 mm			d =	181	
Estimate Area of main steel		Ast =	4912.74981 mm^2			
13	Y 32	Ast =	10455.22035 mm^2	OUTSIDE		
		p =	0.057763648			
13	Y 32	Asc =	10455.22035 mm^2	INSIDE		
		pc =	0.057763648	<	pd =	0.025409
		p-pc =	0	pd=0.4*0.85*μ*f'c/fsy		

Check M* <= phiMu

ku =	0	dsc =	103 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)]

=	301	kNm	>	M* =	278.90 kNm	OK

Determine -ve steel at right support :

M -ve = 0.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 119 mm dst = 181 mm

Estimate Area of main steel Ast = 0 mm^2

13 Y 32 Ast = 10455.22035 mm^2 OUTSIDE

p = 0.057763648

13 Y 32 Asc = 10455.22035 mm^2 INSIDE

pc = 0.057763648 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 103 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)]

=	301	kNm	>	M* =	0.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 129.2 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 135 mm

what is the code ? 2 d = 165 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 2496.50981 mm^2

13 Y 32 Ast = 10455.22035 mm^2 OUTSIDE

13 Y 32 Asc = 10455.22035 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1493.333333 mm

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

= 0.66798799

hence dn = 110 < 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)]

=	474 kNm	>	M*+ve =	129.20 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	13	Y 32
	Asc	=	13	Y 32
Right support :	Ast	=	13	Y 32
	Asc	=	13	Y 32
midspan	Ast	=	13	Y 32
	Asc	=	13	Y 32

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 = 740 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 = 740 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 = 246.6666667 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 32 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 182 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 858 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 804.2477193 mm^2

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

factor = 143375.9779

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

= 787.7801 mm

take fs/Fsy = M*/Phi Mu = 0.926817777

therefore

Lsy =	730 mm	≥ 25k1db	1000
say =	750 mm		

For Bottom Steel :

Cover, c = 75 mm d_b = 32 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 182 mm

if ab < 2c C = a_b + d_b = 1206 mm

k1 = 1 A_b = 804.2477193 mm^2

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

factor = 114700.7823

Lsyt = 630.2241 mm take fs/Fsy = M*/Phi Mu = 0.272742201

therefore

Lsy =	172 mm	≥ 25k1db	800
say =	600 mm		

9 SHEAR CHECK

V* = 632.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 181 mm dq = 93 mm

do = 207 mm

V* = 628 kN beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 237.45 kN

phi Vumax = 1159.2 kN > V* No web crushing

phi Vumin = 324 kN > V* SHEAR REO REQ'D

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 487.804878

Asv.min = 170.731707 mm^2

Asv.max = 3103 mm^2

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

theta = 31.1728 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 619.79 kN

Phi Vu = 434 kN < V* = 628.38 kN

hence

PROVIDE :	1 Y16 LIGS	@	200	centres

Other Support :

V* = 0.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 181 mm   dq = 93 mm

do = 207 mm

V* = -3.62 kN   beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 237.4361 kN

phi Vumax = 1159.2 kN > V* No web crushing

phi Vumin = 324 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 200 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 487.804878

Asv.min = 170.731707 mm^2

Asv.max = 3103.11206 mm^2

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

theta = 32.19584 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 743.5734 kN

Phi Vu = 520.5014 kN > V* = -3.62 kN

hence

PROVIDE :	1 Y16 LIGS	@	200 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 278.90 kNm   Es = 200000 MPa

Ml = 0.00 kNm   Ec = 31975.35051 MPa

Mo = 268.65 kNm

Ms = 129.20 kNm

Ig = 2.25E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1493.333333 mm	A_{sc} =	10455.22035 mm^2
webwidth	b_w =	1000 mm	A_{st} =	10455.22035 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	135 mm
			d_{st} =	165 mm
			d =	181 mm
			D =	300 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 + 120335.782 dn - 19253523.61) = 0

dn = 109.854895 mm

d = 165 mm

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

= 6.41E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

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

= 7.78E+08 mm^4

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

b_w = 1000 mm A_{st} = 10455.22035 mm^2

d_{sc} = 103 mm A_{sc} = 10455.22035 mm^2

d = 181 mm

Ig = 2.25E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 120335.782 dn - 17495434.63) = 0

dn = 102.09 mm

d = 181 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.62E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 7.75E+08 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **103** mm

A_{st} = **10455.22** mm^2

A_{sc} = **10455.22** mm^2

d = **181** mm

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

500 dn^2 + 120335.782 dn - 17495434.63) = 0

dn = 102.086271 mm

d = 181 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 7.62E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

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

Iav = [Im + (IL + Ir)/2]/2 = 5.83E+08 mm^4

Midspan Deflection :

Lef = 2466.66667 mm

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

= 3.44546446 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 12 kN/m

w - short term = 15 kN/m

Defln,s,sus = 2.756371568 mm

At midspan : Ast = 10455.2204 mm^2

Asc = 10455.2204 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)

= 5.65056 mm

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

ALLOWABLE DEFLN = Lef / 250 = 9.86666667 mm

> 0.9 x Total Defln

7 **Ground Floor Slab Design**

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

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

DESIGN AS A TWO WAY SLAB

Design parameters

f'c	=	40 MPa	Short term live load factor, ψ_s=	1.00	Table 9.2
c	=	75 mm	Long term live load factor, ψ_l =	0.80	Table 9.2
fsy	=	410 MPa	K_1 =	1.70	Cl 16.1
Lx	=	4900 mm			
Ly	=	5900 mm			
Ec	=	32E+3 MPa			
ρc	=	2400 kg/m^3			

Design Loading

Dead load			23600	
			655.5556	
Slab thickness	=	700 mm	327.7778	800
Self weight, wt	=	16.80 kN/m^2	472.2222	423.0769
Ceiling & Services	=	66.16 kN/m^2		
Super	=	0.00 kN/m^2		
	Total =	82.96		
Total DL		82.96 kN/m^2		
Live load	=	4.00 kN/m^2		
Service Load	=	86.964 kN/m^2		

HENCE

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

$= \frac{4900}{Ds} =<$ 17.57652 Ds = 278.781 mm

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

No it is not a domestic building

-

-

Hence minimum thickness =	278.781 mm
Take Ds =	700 mm

Design moments and shears:

Design load, Fd = 1.4G + 1.7Q

= 122.95 kN/m^2

Load combination

DL = 1.40

LL = 1.70

Design moment :

	Ly/Lx =	1.20		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	1.20	0.0740	0.0560	218.45	290.54	109.22	165.31	219.87	82.66

Maximum design unit shear force in the slab = 301.2265 kN/m

Check Flexral steel:

Design parameters

gamma, g = 0.85 | pmin = 0.001951 | Reinf. Y 32

short span d = 609 mm | f = 0.8

Long span d = 577 mm

Short span

f bd^2 Fsy p = -121.6E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 748.1E+9

Min. As = 0.8/fsy *b * d

= 1188.293 mm^2/m

Long span

f bd^2 Fsy p = -109.2E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 111.4E+9

Min. As = 0.8/fsy *b * d

= 1125.854 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	218.4E+6	0.0018	1188.29	32	803.84	676.466	300

Continious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	290.5E+6	0.0024	1476.51	32	803.84	544.420	300

Discontinious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	109.2E+6	0.0009	1188.29	32	803.84	676.466	300

Midspan

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	165.3E+6	0.0015	1188.29	32	803.84	676.466	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	219.9E+6	0.0020	1228.70	32	803.84	654.221	270

Discontinous edge

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	82.7E+6	0.0008	1188.29	32	803.84	676.466	300

Shrinkage and Temperature:

pmin = 0.0018 Table 16.1

As = pmin b Ds reinf. y 16

= 1096.200 mm^2/m As = 200.96 mm^2

spacing = 183.324 mm

say 180 mm

USE	Y	16	at	180	CTS

Check for Shear:

$b_1 = 1.4 - d/2000$ f = 0.7

= 1.0955   d = 609 mm

p = 0.0039

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

= 359.369 kN/m

f Vuc = 251.558 kN/m   <   Not ok

Hence shear failure is not critical

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	**10.00**	kN/m	(plus super)
Self wt	:	Wsw =	**0.00**	kN/m	
Blockwall	:	Wb =	**0.00**	kN/m	
TOTAL		=	10.00	kN/m	
LL :		Wll =	**0.25**	kN/m	
LL factor	- long term		**0.4**		
	- short term		**0.7**		

2 Beam Section Properties

Beam Size

bw, eff =	**1000**	mm	D =	**500**	mm
tff =	**0**	mm	Cover =	**75**	mm
Actual length, L =	**5900**	mm	Overall depth of Sect. D =	500	mm
Effective Length,Lef =	**4900**	mm	Area, A =	5.00E+05	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	1.04E+10	mm^4
Neutral Axis, xc =	990.00	mm	Moment of Inertia, Iy-y =	4.17E+10	mm^4
Effective width, beff =	1980.00	mm	Gross Area, Ag =	5.00E+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 <u>INITIAL CHOICE OF SECTION</u>

Effective width =	1980 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)	=		1	
k_1 =	0.045	(RECT sections)		1
=	0.02778367	(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 =	16.739			
defln/Lef =	**0.004**	k_1/k_2 =	17.27977882	
d =	77 mm			
take d =	397 mm	Distance from d to extreme fibre of beam (do) =	103 mm	

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

Mmax -ve	=	**207.00**	kNm	(left support)
Mmax +ve	=	**232.00**	kNm	(mid-span)
M -ve	=	**207.00**	kNm	(right support)
Vmax	=	**299.00**	kN	
V (other end)	=	**299.00**	kN	
N*	=	63.00	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine -ve steel required :</u> (At arbitrary left support) Bar Spacing = 200 mm

Number of reinforcement layers		1 or 2 :		1		
take do =		103 mm	d =		397	
Estimate Area of main steel		Ast =	1662.396236	mm^2		
10	Y 16	Ast =	**2010.619298**	mm^2	OUTSIDE	
		p =	0.005064532			
10	Y 16	Asc =	**2010.619298**	mm^2	INSIDE	
		pc =	0.005064532	<	pd =	0.025409
		p-pc =	0	pd=0.4*0.85*µ*f'c/fsy		

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

ku =	0	dsc =	95 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)]

=	224	kNm	>	M* =	207.00	kNm	OK

Determine - ve steel at right support :

M -ve = 207.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 103 mm dst = 397 mm

Estimate Area of main steel Ast = 1662.396236 mm^2

10 Y 16 Ast = 2010.619298 mm^2 OUTSIDE

p = 0.005064532

10 Y 16 Asc = 2010.619298 mm^2 INSIDE

pc = 0.005064532 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 95 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)]

=	224	kNm	>	M* =	207.00	kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 232 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 = 111 mm

d = 389 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1901.48582 mm^2

10 Y 16 Ast = 2010.619298 mm^2 OUTSIDE

10 Y 16 Asc = 2010.619298 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1980 mm

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

= 0.04109521

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)]

=	284 kNm	>	M*+ve =	232.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	10	Y 16
	Asc	=	10	Y 16
Right support :	Ast	=	10	Y 16
	Asc	=	10	Y 16
midspan	Ast	=	10	Y 16
	Asc	=	10	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 = 1470 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 = 1470 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 = 490 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 16 mm   # of lines of bars = 1

a_b = 826 mm   distance between bars

If a_b > 2c   C = 2c + d_b = 166 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c   C = a_b + d_b = 842 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

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

factor = 35843.99446

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

= 215.9277 mm

take fs/Fsy = M*/Phi Mu = 0.923862182

therefore

Lsy =	199 mm
say =	750 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 75 mm d_b = 16 mm

# of lines of bars = 1

If ab > 2c   C = 2c + d_b = 166 mm

if ab < 2c   C = a_b + d_b = 1190 mm

k1 = 1 A_b = 201.0619298 mm^2

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

factor = 28675.19557

Lsyt = 172.7421 mm   take fs/Fsy = M*/Phi Mu = 0.816717947

therefore

Lsy =	141 mm
say =	600 mm

≥ 25k1db 400

9 SHEAR CHECK

V* = 299.00 kN   Dist to centreline of bottom steel from soffit

critical location : d = 397 mm   dq = 93 mm

do = 407 mm

V* = 295 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

= 198.66 kN

phi Vumax = 2279.2 kN > V* No web crushing

phi Vumin = 370 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 = 5343 mm^2

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

theta = 30.4243 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 662.66 kN

Phi Vu = 464 kN > V* = 294.93075 kN

hence

PROVIDE :	1 Y16 LIGS	@	300	centres

Other Support :

V* = 299.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 397 mm   dq = 93 mm

do = 407 mm

V* = 294.93 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

= 198.6575 kN

phi Vumax = 2279.2 kN > V* No web crushing

phi Vumin = 370 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 = 5343.44735 mm^2

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

theta = 31.01399 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 838.926 kN

Phi Vu = 587.2482 kN > V* = 294.93 kN

hence

PROVIDE :	1 Y16 LIGS	@	300 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 207.00 kNm   Es = 200000 MPa

Ml = 207.00 kNm   Ec = 31975.35051 MPa

Mo = 439 kNm

Ms = 232.00 kNm

Ig = 1.04E+10 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1980 mm	A_{sc} =	2010.619298 mm^2
webwidth	b_w =	1000 mm	A_{st} =	2010.619298 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	111 mm
			d_{st} =	389 mm
			d =	397 mm
			D =	500 mm

n = Es/Ec = 6.25481807

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

500 dn^2 + 23141.4965 dn - 6165458.676) = 0

dn = 90.2888619 mm

d = 389 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 = 158.113883 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	4.23E+09 mm^4

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

b_w = 1000 mm   **A_{st}** = 2010.619298 mm^2

d_{sc} = 95 mm   **A_{sc}** = 2010.619298 mm^2

d = 397 mm

Ig = 1.04E+10 mm^4

At right end (arbitrary)

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

500 dn^2 + 23141.4965 dn - 5996411.658) = 0

dn = 88.79 mm

d = 397 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.43E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 158.113883 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 5.43E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **95** mm

A_{st} = **2010.6193** mm^2

A_{sc} = **2010.6193** mm^2

d = **397** mm

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

500 dn^2 + 23141.4965 dn - 5996411.658) = 0

dn = 88.7886253 mm

d = 397 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.43E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 158.113883 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 5.43E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 4.83E+09 mm^4

Midspan Deflection :

Lef = 4900 mm

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

= 3.08473568 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 10.05 kN/m

w - short term = 10.125 kN/m

Defln,s,sus = 3.061885785 mm

At midspan : Ast = 2010.6193 mm^2

Asc = 2010.6193 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)

= 5.53424 mm

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

ALLOWABLE DEFLN = Lef / 250 = 19.6 mm

> 0.9 x Total Defln

SUMMARY

PS 04, PS 05 & PS 10

A. Base Slab

850 thk RC Slab

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

B. Pit Wall

Non-uniform Section RC Wall

a. 2 metre from base	850 thk with 500 mm Haunch
b. (L-2)/2	500 thk with 500 mm Haunch
c. (L-2)/2	300 thk

Reinforcements:	Vertical:	Y32 - 120 cts	Outside
		Y32 - 120 cts	Inside
	Horizontal	Y16 - 200 cts	

Job name: PS05WALL Designer: FYP Date: 9 Nov 2011, 1:43 pm Page: 1
POMSSUP
WALL FOR PS05

ANALYSIS STATUS REPORT

Job .... C:\SGWIN\SMIT\PS05WALL
Units system .... Kilonewtons, Metres

Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	4	(10000)
Member distributed forces	5	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)

Static analysis	Y
Dynamic analysis	N
Response analysis	N
Buckling analysis	N
Ill-conditioned	N
Non-linear convergence	Y
Frontwidth	6
Total degrees of freedom	12
Primary load cases	4 (100)
Mass load cases	0 (100)

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

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

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	7.0000E-01	6.4921E-02	5.8333E-02	2.8583E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.700	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

MEMBER CONCENTRATED LOADS (m,kN,kNm)

Load Case	Memb	Sub Load	Axes Sys	Load Position	X Force/ Moment	Y Force/ Moment	Z Force/ Moment
4	1	1	G	3.650	-137.090	0.000	0.000
					0.000	0.000	0.000
	3	1	G	3.650	137.090	0.000	0.000
					0.000	0.000	0.000
5	1	1	G	3.400	-520.200	0.000	0.000
					0.000	0.000	0.000
	3	1	G	3.400	520.200	0.000	0.000
					0.000	0.000	0.000

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

1.400 * Load case 1: D
1.700 * Load case 2: L
1.000 * Load case 3: E
1.700 * Load case 4: Q
1.400 * Load case 5: F

LOAD CASE TITLES

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

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	202.494*	-233.053	0.000	0.000	0.000	850.643
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
3	12	202.494	1154.833*	0.000	0.000	0.000	-4300.603
1	12	202.494	-1154.833#	0.000	0.000	0.000	4300.603
1	12	202.494	-1154.833	0.000	0.000	0.000	4300.603*
3	12	202.494	1154.833	0.000	0.000	0.000	-4300.603#

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

Envelope = All Load Cases
and All Nodes

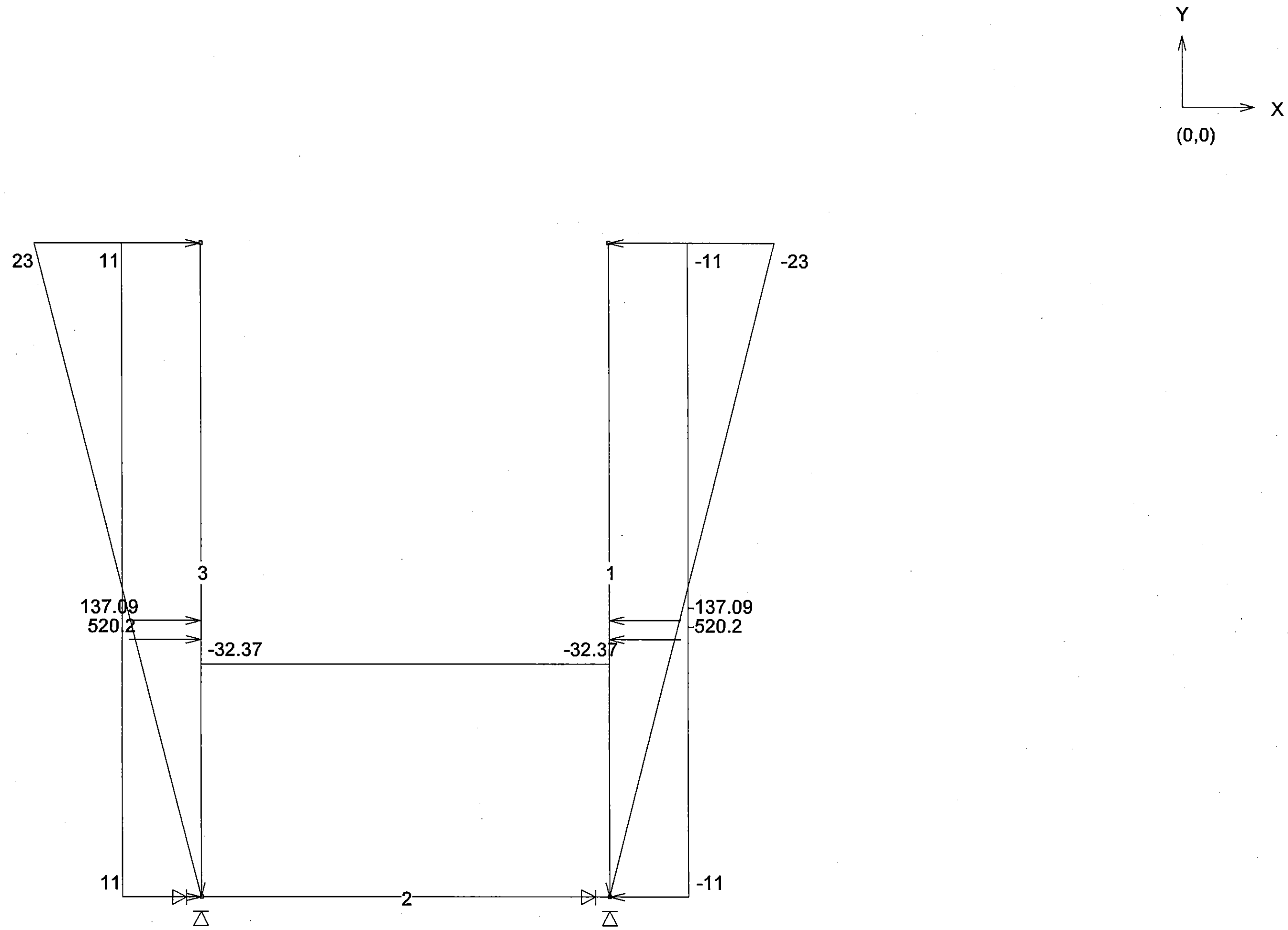
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	1154.833*	388.426	0.000	0.000	0.000	-4467.941
3	12	-1154.833#	388.426	0.000	0.000	0.000	4467.941
1	10	728.280	388.426*	0.000	0.000	0.000	-2643.491
1	3	193.500	0.000#	0.000	0.000	0.000	-973.807
3	12	-1154.833	388.426	0.000	0.000	0.000	4467.941*
1	12	1154.833	388.426	0.000	0.000	0.000	-4467.941#

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	202.494*	-233.053	0.000	0.000	0.000	850.643
1	3	0.000#	-193.500	0.000	0.000	0.000	973.807
3	12	202.494	1154.833*	0.000	0.000	0.000	-4300.603
1	12	202.494	-1154.833#	0.000	0.000	0.000	4300.603
1	12	202.494	-1154.833	0.000	0.000	0.000	4300.603*
3	12	202.494	1154.833	0.000	0.000	0.000	-4300.603#

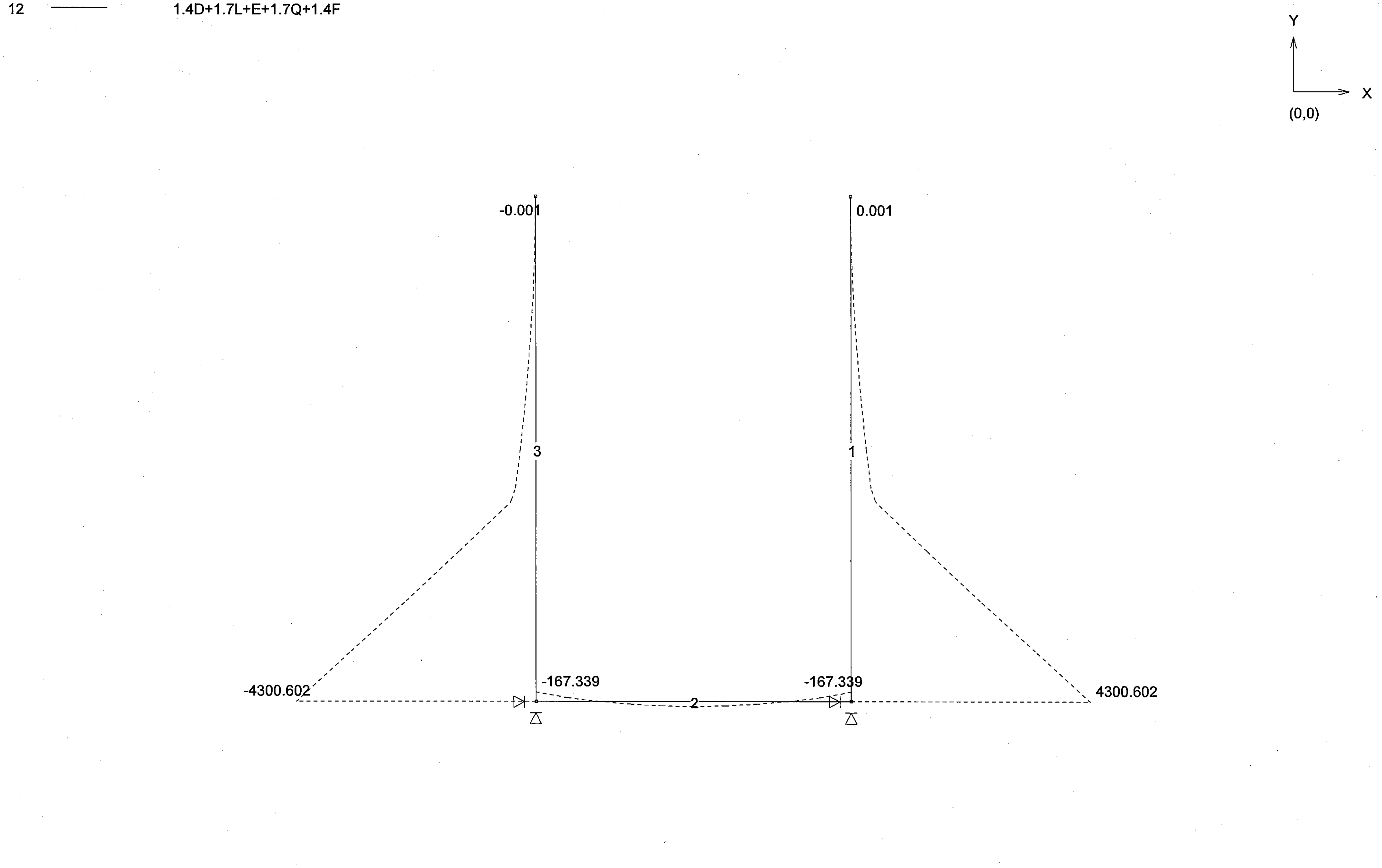
1 (SW) D
3 E
4 Q
5 F
6 D+1.3L+E
7 0.9D+E
8 1.4D+1.7L+1.7Q
9 0.9D+1.7Q
10 1.4D+1.7L+1.4F
11 0.9D+1.4F
12 1.4D+1.7L+E+1.7Q+1.4F



Sections:
1 500 x 1000

Rectangular Pump Well Model

Job: PS05WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:95, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
WALL FOR PS05 9 Nov 2011, 1:44 pm



Sections:
1 500 x 1000

Bending Moment Diagram

Job: PS05WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:95, Axes: XY
Load: 1 Disp: None Moment: 100 Shear: None Axial: None
POMSSUP
WALL FOR PS05 9 Nov 2011, 1:45 pm

Job name: PS05BWAL Designer: FYP Date: 6 Nov 2011, 10:59 am Page: 1
POMSSUP
WALL FOR PS-11

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PS05BWAL	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	4	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	0	(10000)
Member distributed forces	1	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	12	
Total degrees of freedom	12	
Primary load cases	2	(100)
Mass load cases	0	(100)

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	12	429.974*	-73.709	0.000	0.000	0.000	120.014
4	1	0.000#	35.439	0.000	0.000	0.000	-34.848
2	12	73.709	347.564*	0.000	0.000	0.000	-241.159
2	12	73.709	-347.564#	0.000	0.000	0.000	-241.159
1	12	429.974	-73.709	0.000	0.000	0.000	120.014*
1	12	347.564	-73.709	0.000	0.000	0.000	-241.159#

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

Envelope = All Load Cases
and All Nodes

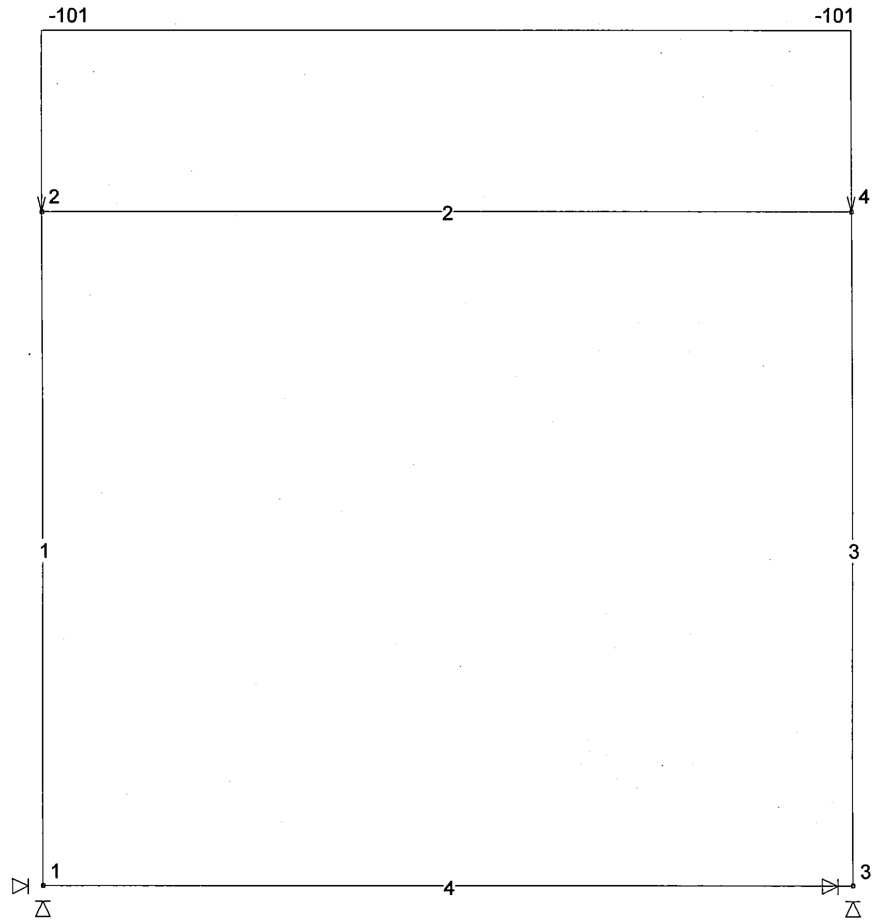
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	73.709*	479.589	0.000	0.000	0.000	-71.226
3	12	-73.709#	479.589	0.000	0.000	0.000	71.226
1	12	73.709	479.589*	0.000	0.000	0.000	-71.226
1	9	6.764	116.768#	0.000	0.000	0.000	20.350
3	3	-63.187	297.950	0.000	0.000	0.000	102.882*
1	3	63.187	297.950	0.000	0.000	0.000	-102.882#

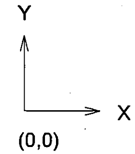
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

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	12	429.974*	-73.709	0.000	0.000	0.000	120.014
4	1	0.000#	35.439	0.000	0.000	0.000	-34.848
2	12	73.709	347.564*	0.000	0.000	0.000	-241.159
2	12	73.709	-347.564#	0.000	0.000	0.000	-241.159
2	12	73.709	0.000	0.000	0.000	0.000	271.498*
2	12	73.709	-347.564	0.000	0.000	0.000	-241.159#

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



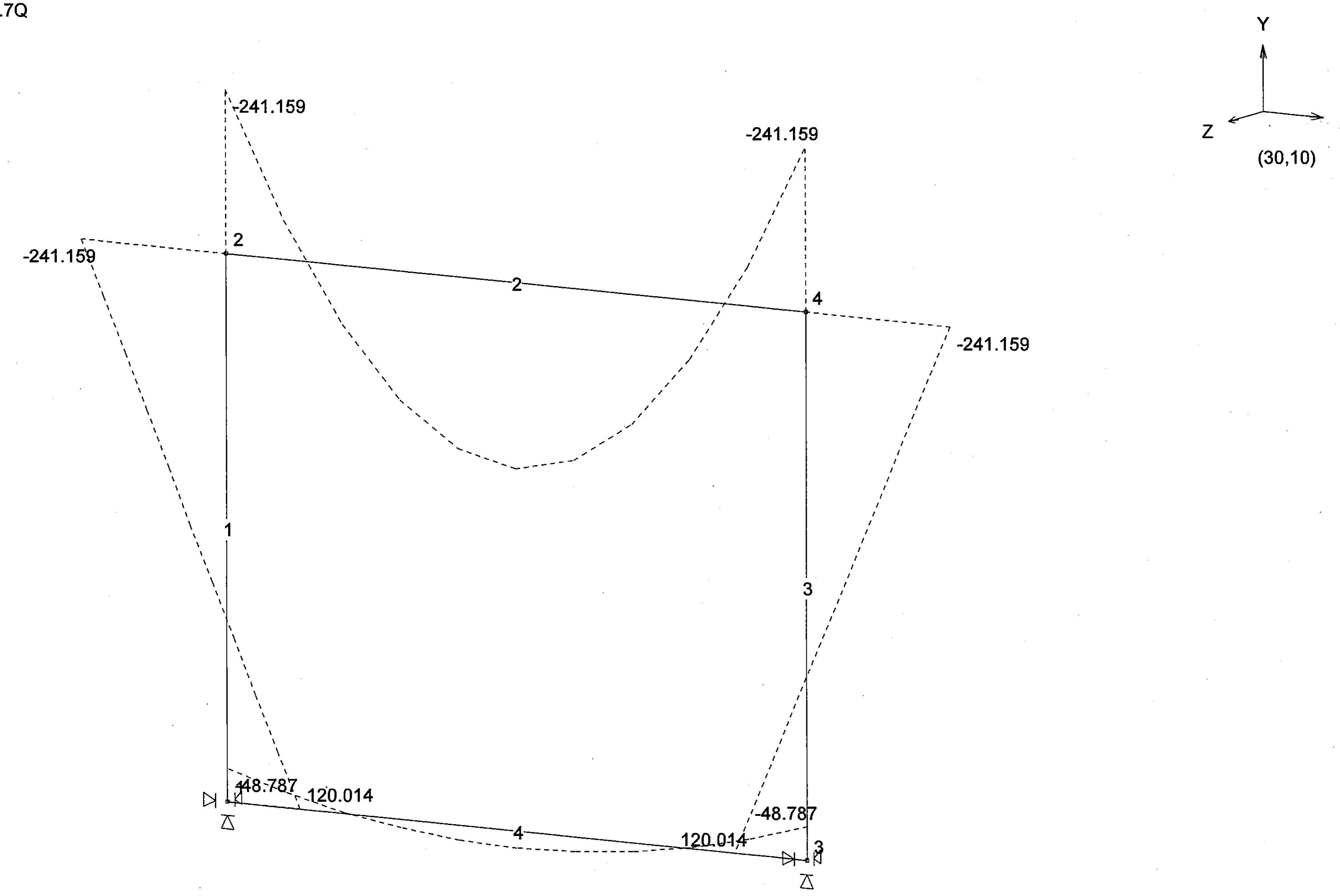


Sections:
1 500 x 1000

Rectangular Pump well Model 2

Job: PS05BWAL, Designer: FYP, Units: m,kN,MPa, Scale: 1:55, Axes: XY
Load: 4.2 Disp: None Moment: None Shear: None Axial: None
POMSSUP
WALL FOR PS-11 6 Nov 2011, 11:00 am

12 —— 1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

Model 2 with Bending Moment Diagram

Job: PS05BWAL, Designer: FYP, Units: m,kN,MPa, Scale: 1:61, Axes: XY
Load: 4.2 Disp: None Moment: 10 Shear: None Axial: None
POMSSUP
WALL FOR PS-11 6 Nov 2011, 10:58 am

A-3: Pump Well - PS 11

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

Load Calculation

Data

1	Density of Water (ρ) =	9.81	kN/m^3
	Height of Water, h =	3	m
	Wall Thickness, tw =	0.5	m
	Height of Wall, hw =	6.5	m
	Base Slab Thickness, tb =	0.7	m
	Length of Base slab, Lb =	5	m

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

Atmospheric Pressure, Patm =	0	kPa
Hydrostatic Pressure along 1 m width wall, Pw	29.43	kN/m (=kPa/m)

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

Water Pressure on base slab, Pw = ρgh 29.43 kPa

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

Flp = 29.43 kN/m

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.50	m
Height of Wall (H) =	6.50	m
Embedded Height of Wall (He) =	5.60	m
(Heel Depth) Thickness of Base (tb) =	0.70	m
Width of Base (wb) =	5.00	m
(Heel)/ Length of base (Lb) =	4.00	m
Toe Depth (td) =	0.70	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) =	45	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 $K_A = (1\text{-SIN } \phi)$ =

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) =	52.84	LE =	1.867
PS(G) =	11.10	LS =	2.80
PS(Q) =	0.00	LS =	2.80
Total, Pa =	63.94	La =	2.03

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	52.84	1.87	98.63
PS =	11.10	2.80	31.08
Total	63.94	**2.03**	129.72

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	81.25	4.00	325.0
Wall wt =	72.50	4.00	290.0
Base wt =	350.00	2.00	700.0
Total =	503.75	1.97	1315.0

CHECK GROUND BEARING FAILURE

∑MO = 129.72 kNm/m

∑MR = 1315.0 kNm/m

∑F = 503.75 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

e = B/2 - [∑MR - ∑MO]/∑F 0.35 m **OK, e < B/6**

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

qtoe = ∑F/B [1+6e/B] = 0.75 kPa

qheel = ∑F/B [1-6e/B] = 0.23 kPa

Maximum Bearing Pressure = 0.75 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

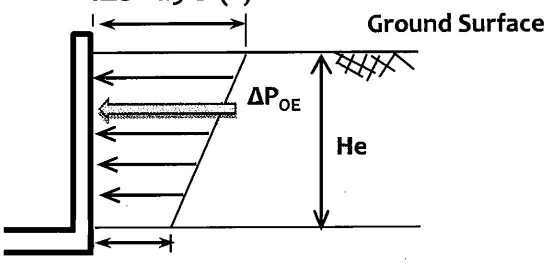
horizontal force coefficient $Ch(T) = \alpha\ \beta\ Ah(T)\ Ap$ = 0.13 $A_h(T)$

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 = 5.6 m

Ah(0) = 1

$C_h(0) = \alpha\ \beta\ A_h(0)\ A_p$, horizontal force coefficient = 0.131

$f\Delta e = 1.5\ Ch(0)\ \Upsilon e\ He$ = 19 kPa

$f\Delta e = 0.5\ Ch(0)\ \Upsilon e\ He$ = 6 kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	6	kN/m	UDL
Eu2 =	12	kN/m	Triangular

Base Slab Analysis

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = 0.7 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

333.75

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	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
2	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
3	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
4	0.50	1.43E-02	31.9754	0.18	0.4	0.10231	9.57	0.4864	4.6533	No	Yes	Use Winkler	204611
5	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
6	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
7	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215
8	2.25	6.43E-02	31.9754	0.18	0.4	0.14898	9.57	0.3669	3.5097	No	Yes	Use Winkler	66215

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

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

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

Design of a Continuous Beam :

1 **Loads**

1 **Serviceability Load Combination : Long term loads**

Main Load Dead Load =	33.60	kN/m	Self weight =	28.13	kN/m
Total G =	61.73	kN/m	Flp =	29.43	kN/m
Fe =	63.94	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	125.67 kN/m	COMB. 2	G + Flp =	91.16	kN/m
COMB. 3	G + Flp + Fgw =	91.16 kN/m		Long term Load Factors =	1	

2 **Beam Section Properties**

Beam Size

bw =	2250	mm	D =	500	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	5000	mm	Overall depth of sect. D =	500	mm
Effective Length,Lef =	4600	mm	Area, A =	1.13E+06	mm^2
Neutral Axis, yc =	250.00	mm	Moment of Inertia, Ix-x =	2.34E+10	mm^4
Neutral Axis, xc =	1585.00	mm	Moment of Inertia, Iy-y =	4.75E+11	mm^4
Effective width, beff =	3170.00	mm	Gross Area, Ag =	1.13E+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 = 3170 mm clause 8.8.2

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

k_1 = 0.045 (RECT sections) 1

= 0.0342397 (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 = 101.229

defln/Lef = **0.004** k_1/k_2 = 17.28

d = 112 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	=	176.00	kNm	(left support)
Mmax +ve	=	190.00	kNm	(mid-span)
M -ve	=	176.00	kNm	(right support)
Vmax	=	218.00	kN	
V (other end)	=	218.00	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 = 383

Estimate Area of main steel Ast = 1465.1 mm^2

10 Y 20 Ast = 3141.59 mm^2 **SPACING = 210** mm

p = 0.0036

10 Y 20 Asc = 3141.59 mm^2

pc = 0.0036 < pd = 0.0254088

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)]

= 332 kNm > M* = 176.00 kNm **OK**

Determine - ve steel at right support :

M -ve = 176.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst : 383 mm

Estimate Area of main steel Ast = 1465.1 mm^2

10 Y 20 Ast = 3141.59 mm^2

p = 0.0036

10 Y 20 Asc = 3141.59 mm^2

pc = 0.0036 < pd = 0.0254088

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)]

= 332 kNm > M* = 176.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 190 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) = 1581.6469 mm^2

Bottom 10 Y 20 Ast = 3141.59265 mm^2

Top 10 Y 20 Asc = 3141.59265 mm^2

Assume N - A in flange

Effective width : beff = 3170 mm

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

= 0.040735

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)]

= 437 kNm > M*+ve = 190.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	10	Y 20
	Asc	=	10	Y 20
Right support :	Ast	=	10	Y 20
	Asc	=	10	Y 20
midspan	Ast	=	10	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 = 1380 mm ?

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

For -ve steel = 2 bars

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

= 0.3 x le = 1380 mm ?

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

= 2 bars

ii) 1/4 Ast into interior support

= 2 bars

iii) Remainder curtailed at 0.1Ln from supports

= 0.1 x le = 460 mm

8 STRESS DEVELOPMENT

<u>For Top Steel :</u>

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 1038 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 = 1058 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1038 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 = 0.530848

therefore

Lsy =	175 mm
say =	650 mm

≥ 25k1db 625

<u>For Bottom Steel :</u>

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 = 1232 mm

k1 = 1 A_b = 314.16 mm^2

k2 = 2.2 a_b = 1212 mm (dist between bars)

factor = 44805

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.43471803

therefore

Lsy =	115 mm
say =	500 mm

≥ 25k1db 500

9 PUNCHING SHEAR CHECK

V* = 218.00 kN

Dist to centreline of bottom steel from soffit

dq = 85 mm

dom = 415 mm

Wall Dimension

b = 500 mm

d = 500 mm

Critical shear perimeter, μ

μ = 2(b+dom) + 2(d+dom)

3660 mm

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

βh = 1 longer side/shorter side

$f_{cv} = 0.17(1+2/\beta_h)\sqrt{f'_c} \le 0.34\sqrt{f'_c}$

$f_{cv} =$ 13.60 MPa

$\phi V_{uo} =$ 14,460 kN > $V^* =$ 218 kN **OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	176.00 kNm	Es	=	200000 MPa
Ml	=	176.00 kNm	Ec	=	31975.3505 MPa
Mo	=	366 kNm			
Ms	=	190.00 kNm			
Ig	=	2.34E+10 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	3170 mm	A_{sc} =	3141.59265 mm^2
webwidth	b_w =	2250 mm	A_{st} =	3141.59265 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)

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 77.015006 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

= 2.18E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

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

= 1.42E+11 mm^4

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

b_w =	2250 mm	A_{st} =	3141.6 mm^2
d_{sc} =	97 mm	A_{sc} =	3141.6 mm^2
d =	383 mm		

Ig = 2.34E+10 mm^4

At right end (arbitrary)

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

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 75.43 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.18E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 355.756237 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 1.78E+11 mm^4

At left end (arbitrary)

b_w =	**2250** mm
d_{sc} =	**97** mm
A_{st} =	**3141.59** mm^2
A_{sc} =	**3141.59** mm^2
d =	**383** mm

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

1125 dn^2 + 36158.588 dn - 9E+06) = 0

dn = 75.425002 mm

d = 383 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.18E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 355.756237 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.78E+11 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.60E+11 mm^4

Midspan Deflection :

Lef = 4600 mm

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

= 0.0668084 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 125.67 kN/m

w - short term = 125.67 kN/m

Defln,s,sus = 0.066808 mm

At midspan : Ast = 3141.59265 mm^2

Asc = 3141.59265 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)

= 0.12026 mm

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

ALLOWABLE DEFLN = Lef / 500 : 9.2 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw, eff =	1000 mm	D =	400 mm
tff =	0 mm	Cover =	75 mm
Actual length, L =	6500 mm	Overall depth of Sect. D =	400 mm
Effective Length,Lef =	6200 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 =	1120.00 mm	Moment of Inertia, Iy-y =	3.33E+10 mm^4
Effective width, beff =	2240.00 mm	Gross Area, Ag =	4.00E+05 mm^2

3 Design Parameters

F'c =	40 MPa	D =	400 mm
Fsy =	410 MPa	Ec =	31975.3505 MPa
m =	2400 kg/m^3	Es =	200000 MPa
		Gamma =	0.766

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

Effective width	=	2240 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)		=		1	
k_1	=	0.045	(RECT sections)		1
	=	0.02609751	(T and L sections)		2
what type of span?		5	interior span =		5
			end span =		6
k_2	=	0.0026042			
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef	=	29.96			
defln/Lef	=	0.004	k_1/k_2 =	17.2797788	
d =		113 mm			
take d =		289 mm	Distance from d to extreme fibre of beam (do) =	111 mm	

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

Mmax -ve	=	440.20 kNm	(left support)	1.116
Mmax +ve	=	62.70 kNm	(mid-span)	
M -ve	=	0.00 kNm	(right support)	
Vmax	=	176.00 kN		
V (other end)	=	176.00 kN		
N*	=	132.00 kN	Axial Load	

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine -ve steel required :</u> (At arbitrary left support) Bar Spacing [illegible]

Number of reinforcement layers	1 or 2 :		1		
take do =		111 mm	d =	289	
Estimate Area of main steel		Ast =	4856.31561 mm^2		
12	Y 24	Ast =	5428.672105 mm^2	OUTSIDE	
		p =	0.01878433		
15	Y 24	Asc =	6785.840132 mm^2	INSIDE	
		pc =	0.02348042	< pd =	0.025409
		p-pc =	-0.00469608	pd=0.4*0.85*μ*f'c/fsy	

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

ku = 0 dsc = 99 mm

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

= 476 kNm > M* = 440.20 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
12 Y 24 Ast = 5428.672105 mm^2 OUTSIDE
p = 0.01878433
15 Y 24 Asc = 6785.840132 mm^2 INSIDE
pc = 0.02348042 < pd = 0.025409
p-pc = -0.00469608

Check M* <= phiMu
ku = 0 dsc = 99 mm
phi Mu = phi[Fsy x Asc (d - dsc) + 0.85 x F'c x b x Gamma x ku x d x (d - 0.5 x 0.85 x ku x d)]
= 476 kNm > M* = 0.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI
Tensile Steel at Midspan :
M* = 62.7 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) = 721.676361 mm^2
12 Y 24 Ast = 5428.67211 mm^2 OUTSIDE
15 Y 24 Asc = 6785.84013 mm^2 INSIDE
Assume N - A in flange
Effective width : beff = 2240 mm
ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.13773426
hence dn = 38 < flange thickness
hence N -A is in flange
Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]
= 526 kNm > M*+ve = 62.70 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	12	Y 24
	Asc	=	15	Y 24
Right support :	Ast	=	12	Y 24
	Asc	=	15	Y 24
midspan	Ast	=	12	Y 24
	Asc	=	15	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 = 1860 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 = 1860 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 = 620 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 = 452.389342 mm^2
k2 = 2.2 a_b = 826 mm (dist between bars)
factor = 80648.9875
Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C
= 463.4999 mm
take fs/Fsy = M*/Phi Mu = 0.925265667
therefore Lsy = 429 mm ≥ 25k1db 750
say = 750 mm
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 =	452.389342 mm^2			
k2 =	2.2	a_b =	1174 mm	(dist between bars)		
		factor =	64519.19			
Lsyt =	370.7999 mm	take fs/Fsy = M*/Phi Mu =		0.119290044		
therefore	Lsy = 44 mm		≥	25k1db	600	
	say = 900 mm					

9 **SHEAR CHECK**

V* =	176.00 kN			Dist to centreline of bottom steel from soffit		
critical location :	d =	289 mm		dq =	93 mm	
				do =	307 mm	
V* =	170 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					
=	238.70 kN					
phi Vumax =	1719.2 kN	>	V*	No web crushing		
phi Vumin =	368 kN	>	V*	OK		

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s =	200 mm		Fsyf =	410 MPa	
Determine theta :					
	bv x s/Fsy f	=	487.804878		
	Asv.min	=	170.731707 mm^2		
	Asv.max	=	3361 mm^2		
Asv =	2 x area of stirrups	=	400 mm^2	2 Ties	
theta =	31.07811 degrees				

Determine Phi Vu :

Vu = Vuc + Vus

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

= 758.68 kN

Phi Vu = 531 kN > V* = 170.22 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 =	289 mm		dq =	93 mm	
				do =	307 mm	
V* =	170.22 kN	beta 1 =	1.2465	(but not less than 1.1)		
	hence	=	1.2465			
		beta 2 =	1			
		beta 3 =	1			
phi Vuc =	phi x beta 1 x beta 2 x beta 3 x bw x do[Ast x F'c/bw x do]^0.333					
=	238.699 kN					
phi Vumax =	1719.2 kN	>	V*	No web crushing		
phi Vumin =	368 kN	<	V*	OK		

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s =	200 mm		Fsyf =	410 MPa	
Determine theta :					
	bv x s/Fsy f	=	487.804878		
	Asv.min	=	170.731707 mm^2		
	Asv.max	=	3360.61246 mm^2		
Asv =	2 x area of stirrups	=	600 mm2	2 Ties	
theta =	32.01858 degrees				

Determine Phi Vu :

Vu = Vuc + Vus

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

= 944.8651 kN

Phi Vu = 661.4056 kN > V* = 170.22 kN

hence

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

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	440.20 kNm		Es	=	200000 MPa
Ml	=	0.00 kNm		Ec	=	31975.35051 MPa
Mo	=	282.8 kNm				
Ms	=	62.70 kNm				
Ig	=	5.33E+09 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2240 mm	A_{sc} =	6785.840132 mm^2
webwidth	b_w =	1000 mm	A_{st} =	5428.672105 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	123 mm
			d_{st} =	277 mm
			d =	289 mm
			D =	400 mm

n = Es/Ec = 6.25481807

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

500 dn^2 + 69613.7117 dn - 14199075.7) = 0

dn = 112.716265 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
= 1.39E+09 mm4
Mcr = 0.6sqrtF'c x bD^2/6 = 101.192885 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]
= 1.80E+10 mm^4

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

b_w = 1000 mm **A_{st} =** 5428.67211 mm^2
d_{sc} = 99 mm **A_{sc} =** 6785.84013 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 + 69613.7117 dn - 13343275.2) = 0
dn = 107.96 mm
d = 289 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.53E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 101.192885 kNm
Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.58E+09 mm^4

At left end (arbitrary)
b_w = **1000** mm
d_{sc} = **99** mm
A_{st} = **5428.672** mm^2
A_{sc} = **6785.84** mm^2
d = **289** mm
1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
500 dn^2 + 69613.7117 dn - 13343275.2) = 0
dn = 107.960552 mm
d = 289 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.53E+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 = 9.37E+09 mm^4

Midspan Deflection :
Lef = 6200 mm
Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.24959367 mm

Total Deflection :
Long term factor = 0.2
Short term factor = 0.5
w - long term = 12 kN/m
w - short term = 15 kN/m
Defln,s,sus = 0.199674933 mm
At midspan : Ast = 5428.67211 mm^2
Asc = 6785.84013 mm^2
Asc/Ast = 1.25
kcs = 2 - (1.2 x Asc/Ast) >= 0.8
= 0.8
therefore

Total Defln = Dfln,s + (kcs x Defln,s,sus)
= 0.40933 mm

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

ALLOWABLE DEFLN = Lef / 250 = 24.8 mm
> 0.9 x Total Defln

7 Ground Floor Slab Design

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

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

DESIGN AS A TWO WAY SLAB

Design parameters

f'c	=	40 MPa	Short term live load factor, y s=	1.00	Table 9.2
c	=	75 mm	Long term live load factor, y l =	0.80	Table 9.2
fsy	=	410 MPa	K_1 =	1.70	Cl 16.1
Lx	=	4900 mm			
Ly	=	5900 mm			
Ec	=	32E+3 MPa			
ρc	=	2400 kg/m^3			

Design Loading 23600

Dead load 655.5556

Slab thickness = 700 mm 327.7778 800

Self weight, wt = 16.80 kN/m^2 472.2222 423.0769

Ceiling & Services = 66.16 kN/m^2

Super = 0.00 kN/m^2

Total = 82.96

Total DL 82.96 kN/m^2

Live load = 4.00 kN/m^2

Service Load = 86.964 kN/m^2

HENCE

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

$= \frac{4900}{Ds} =< 17.57652$ Ds = 278.781 mm

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

No it is not a domestic building

-

-

Hence minimum thickness =	278.781 mm
Take Ds =	700 mm

Design moments and shears:

Design load, Fd = 1.4G + 1.7Q Load combination

= 122.95 kN/m^2 DL = 1.40

LL = 1.70

Design moment :

		Ly/Lx =	1.20	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	1.20	0.0740	0.0560	218.45	290.54	109.22	165.31	219.87	82.66

Maximum design unit shear force in the slab = 301.2265 kN/m

Check Flexral steel:

Design parameters

gamma, g = 0.85 pmin = 0.001951 Reinf. Y 32

short span d = 609 mm f = 0.8

Long span d = 577 mm

Short span

f bd^2 Fsy p = -121.6E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 748.1E+9

Min. As = 0.8/fsy *b * d

= 1188.293 mm^2/m

Long span

f bd^2 Fsy p = -109.2E+9

f bd^2 Fsy p (- 0.6pFsy / F'c) = 111.4E+9

Min. As = 0.8/fsy *b * d

= 1125.854 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	218.4E+6	0.0018	1188.29	32	803.84	676.466	300

Continious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	290.5E+6	0.0024	1476.51	32	803.84	544.420	300

Discontinious edge

Direction	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Short	109.2E+6	0.0009	1188.29	32	803.84	676.466	300

Midspan

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	165.3E+6	0.0015	1188.29	32	803.84	676.466	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	219.9E+6	0.0020	1228.70	32	803.84	654.221	270

Discontinous edge

Direction of span	Mu kN/m	p	Ast mm^2/m	Reinf. y	As mm^2/m	Spacing mm	say mm
Long	82.7E+6	0.0008	1188.29	32	803.84	676.466	300

Shrinkage and Temperature:

pmin = 0.0018 Table 16.1

As = pmin b Ds reinf. y 16

= 1096.200 mm^2/m As = 200.96 mm^2

spacing = 183.324 mm

say 180 mm

USE	Y	16	at	180	CTS

Check for Shear:

b1 =	1.4 - d/2000		f =	0.7
=	1.0955		d =	609 mm
			p =	0.0039
Vuc =	b b d (p x F'c)$^{1/3}$			
=	359.369 kN/m			
f Vuc =	251.558 kN/m	<		Not ok

Hence shear failure is not critical

Pump Pit Wall Design

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

Design of a Continuous Beam :

1 Loads

DL	:	Wdl =	10.00	kN/m	(plus super)
Self wt	:	Wsw =	0.00	kN/m	
Blockwall	:	Wb =	0.00	kN/m	
TOTAL		=	10.00	kN/m	
LL :		Wll =	0.25	kN/m	
LL factor	- long term		0.4		
	- short term		0.7		

2 Beam Section Properties

Beam Size

bw, eff =	1000	mm	D =	450	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	5900	mm	Overall depth of Sect. D =	450	mm
Effective Length,Lef =	4900	mm	Area, A =	4.50E+05	mm^2
Neutral Axis, yc =	225.00	mm	Moment of Inertia, Ix-x =	7.59E+09	mm^4
Neutral Axis, xc =	990.00	mm	Moment of Inertia, Iy-y =	3.75E+10	mm^4
Effective width, beff =	1980.00	mm	Gross Area, Ag =	4.50E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	450	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 = 1980 mm clause 8.8.2
Is the section RECT or T , L (ans : 1 or 2) = 1

k_1 = 0.045 (RECT sections) 1
= 0.02778367 (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 = 16.739
defln/Lef = 0.004 k_1/k_2 = 17.27977882
d = 77 mm
take d = 347 mm Distance from d to extreme fibre of beam (do) = 103 mm

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

Mmax -ve	=	207.00	kNm	(left support)
Mmax +ve	=	232.00	kNm	(mid-span)
M -ve	=	207.00	kNm	(right support)
Vmax	=	299.00	kN	
V (other end)	=	299.00	kN	
N*	=	63.00	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine -ve steel required :</u> (At arbitrary left support) Bar Spacing 198 mm
Number of reinforcement layers 1 or 2 : 1
take do = 103 mm d = 347
Estimate Area of main steel Ast = 1901.934598 mm^2
11 Y 16 Ast = 2211.681228 mm^2 OUTSIDE
p = 0.006373721
12 Y 16 Asc = 2412.743158 mm^2 INSIDE
pc = 0.00695315 < pd = 0.025409
p-pc = -0.00057943 pd=0.4*0.85*μ*f'c/fsy

<u>**Check M* <= phiMu**</u>
ku = 0 dsc = 95 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)]

=	224	kNm	>	M* =	207.00	kNm	OK

Determine -ve steel at right support :

M -ve = 207.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 103 mm dst = 347 mm

Estimate Area of main steel		Ast	=	1901.934598	mm^2		
11	Y 16	Ast	=	2211.681228	mm^2	OUTSIDE	
		p	=	0.006373721			
12	Y 16	Asc	=	2412.743158	mm^2	INSIDE	
		pc	=	0.00695315	<	pd =	0.025409
		p-pc	=	-0.00057943			

Check M* <= phiMu

ku = 0 dsc = 95 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)]

=	224	kNm	>	M* =	207.00	kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 232 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 103 mm

what is the code ? 1 d = 347 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 2131.63684 mm^2

11	Y 16	Ast	=	2211.681228	mm^2	OUTSIDE
12	Y 16	Asc	=	2412.743158	mm^2	INSIDE

Assume N - A in flange

Effective width : beff = 1980 mm

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

= 0.0506762

hence dn = 18 < 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)]

=	278	kNm	>	M*+ve =	232.00	kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	11	Y 16
	Asc	=	12	Y 16
Right support :	Ast	=	11	Y 16
	Asc	=	12	Y 16
midspan	Ast	=	11	Y 16
	Asc	=	12	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 = 1470 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 = 1470 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 = 490 mm

8 **STRESS DEVELOPMENT**

For Top Steel :

cover : c = 75 mm

d_b = 16 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 166 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 842 mm

coaxial with and surrounding a bar

k1 =	1.25	A_b =	201.0619298 mm^2	
k2 =	2.2	a_b =	826 mm	(dist between bars)
		factor =	35843.99446	

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

= 215.9277 mm

take fs/Fsy = M*/Phi Mu = 0.922640142

therefore

Lsy =	199 mm
say =	750 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 75 mm d_b = 16 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 166 mm

if ab < 2c C = a_b + d_b = 1190 mm

k1 =	1	A_b =	201.0619298 mm^2	
k2 =	2.2	a_b =	1174 mm	(dist between bars)
		factor =	28675.19557	

Lsyt = 172.7421 mm take fs/Fsy = M*/Phi Mu = 0.835452383

therefore

Lsy =	144 mm
say =	600 mm

≥ 25k1db 400

9 **SHEAR CHECK**

V* = 299.00 kN Dist to centreline of bottom steel from soffit

critical location : d = 347 mm dq = 93 mm

do = 357 mm

V* = 295 kN beta 1 = 1.2215 (but not less than 1.1)

hence = 1.2215

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

= 191.82 kN

phi Vumax = 1999.2 kN > V* No web crushing

phi Vumin = 342 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 = 5292 mm^2

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

theta = 30.42863 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 606.30 kN

Phi Vu = 424 kN > V* = 295.44325 kN

hence

PROVIDE :	1 Y16 LIGS	@	300	centres

Other Support :

V* = 299.00 kN

Dist to centreline of bottom steel from soffit

critical location : d = 347 mm   dq = 93 mm

do = 357 mm

V* = 295.44 kN   beta 1 = 1.2215 (but not less than 1.1)

hence = 1.2215

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

= 191.8226 kN

phi Vumax = 1999.2 kN > V* No web crushing

phi Vumin = 342 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 = 5292.00185 mm^2

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

theta = 31.02435 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 760.7648 kN

Phi Vu = 532.5353 kN > V* = 295.44 kN

hence

PROVIDE :	1 Y16 LIGS	@	300 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr = 207.00 kNm   Es = 200000 MPa

Ml = 207.00 kNm   Ec = 31975.35051 MPa

Mo = 439 kNm

Ms = 232.00 kNm

Ig = 7.59E+09 mm^4

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width b_{eff} = 1980 mm A_{sc} = 2412.743158 mm^2

webwidth b_w = 1000 mm A_{st} = 2211.681228 mm^2

Flange thickness t_{ff} = 0 mm d_{sc} = 103 mm

d_{st} = 347 mm

d = 347 mm

D = 450 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 + 26512.19 dn - 6106169.517) = 0

dn = 87.1330266 mm

d = 347 mm

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

= 1.15E+09 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 128.072245 kNm

$$\boxed{Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3] = 2.24E+09\ mm^4}$$

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

b_w = 1000 mm A_{st} = 2211.681228 mm^2

d_{sc} = 95 mm A_{sc} = 2412.743158 mm^2

d = 347 mm

Ig = 7.59E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 26512.19 dn - 6004741.306) = 0

dn = 86.24 mm

d = 347 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.15E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 128.072245 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 2.68E+09 mm^4

At left end (arbitrary)

b_w = 1000 mm

d_{sc} = 95 mm

A_{st} = 2211.6812 mm^2

A_{sc} = 2412.7432 mm^2

d = 347 mm

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

500 dn^2 + 26512.19 dn - 6004741.306) = 0

dn = 86.2369955 mm

d = 347 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.15E+09 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 128.072245 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.68E+09 mm^4

Iav	=	[Im + (IL + Ir)/2]/2	=	2.46E+09 mm^4

Midspan Deflection :

Lef = 4900 mm

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

= 6.0634245 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 10.05 kN/m

w - short term = 10.125 kN/m

Defln,s,sus = 6.018510241 mm

At midspan : Ast = 2211.68123 mm^2

Asc = 2412.74316 mm^2

Asc/Ast = 1.090909091

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

= 0.690909091

therefore

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

= 10.22167 mm

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

ALLOWABLE DEFLN = Lef / 250 = 19.6 mm

> 0.9 x Total Defln

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. Pit Wall

400 thk RC Wall

Reinforcements:	Vertical:	Y24 - 200 cts	Outside
		Y24 - 200 cts	Inside
	Horizontal	Y16 - 200 cts	

Job name: PS11WALL Designer: FYP Date: 6 Nov 2011, 11:04 am Page: 1
FOMSSUP
WALL FOR PS-11

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PS11WALL	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	2	(10000)
Member distributed forces	5	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	6	
Total degrees of freedom	12	
Primary load cases	4	(100)
Mass load cases	0	(100)

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

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

Sect	Area of Section	Torsion Constant	Y-Axis Mom of In	Z-Axis Mom of In	Y-Axis Shr Area	Z-Axis Shr Area	Princ Angle
1	7.0000E-01	6.4921E-02	5.8333E-02	2.8583E-02	INFINITE	INFINITE	0.00

Sect	Section Shape	D	B/Bt	Bb/Hf	Tw	Tf
1	Rectangle	0.700	1.000			

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

case 8: 1.4D+1.7L+1.7Q

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	131.856*	-108.698	0.000	0.000	0.000	220.657
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
3	12	131.856	175.898*	0.000	0.000	0.000	-440.177
1	12	131.856	-175.898#	0.000	0.000	0.000	440.177
1	12	131.856	-175.898	0.000	0.000	0.000	440.177*
3	12	131.856	175.898	0.000	0.000	0.000	-440.177#

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

Envelope = All Load Cases
and All Nodes

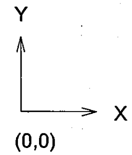
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	175.898*	195.430	0.000	0.000	0.000	-497.393
3	12	-175.898#	195.430	0.000	0.000	0.000	497.393
1	10	0.000	306.675*	0.000	0.000	0.000	-157.337
1	3	67.200	0.000#	0.000	0.000	0.000	-219.520
3	12	-175.898	195.430	0.000	0.000	0.000	497.393*
1	12	175.898	195.430	0.000	0.000	0.000	-497.393#

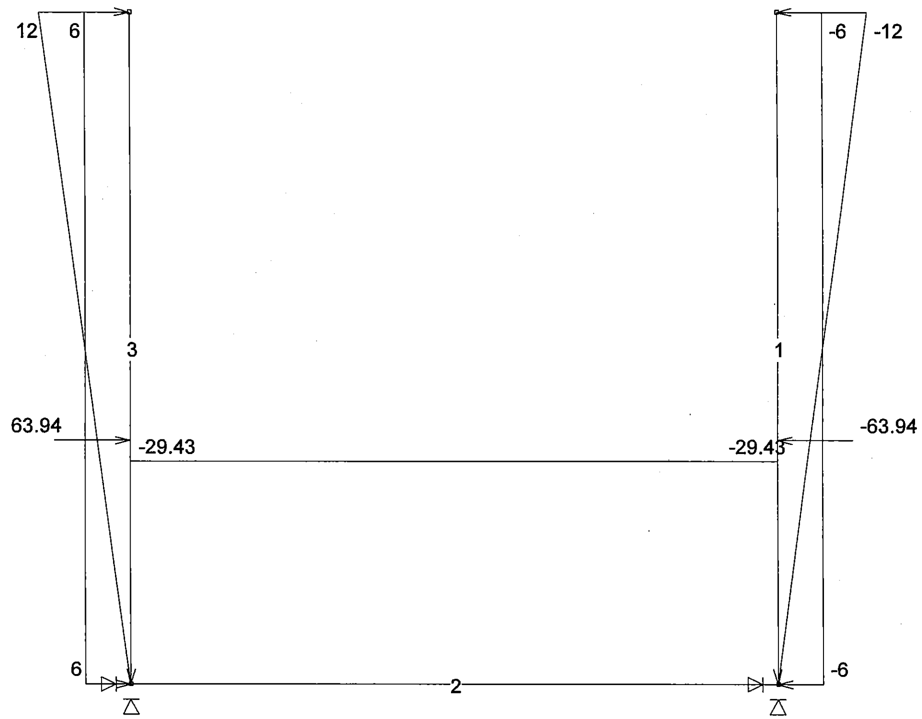
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
------------------------------- (*=Maximum, #=Minimum)

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	131.856*	-108.698	0.000	0.000	0.000	220.657
1	8	0.000#	0.000	0.000	0.000	0.000	0.000
3	12	131.856	175.898*	0.000	0.000	0.000	-440.177
1	12	131.856	-175.898#	0.000	0.000	0.000	440.177
1	12	131.856	-175.898	0.000	0.000	0.000	440.177*
3	12	131.856	175.898	0.000	0.000	0.000	-440.177#

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





Sections:
1 500 x 1000

PS 11 Model with All Load Cases

Job: PS11WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:63, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
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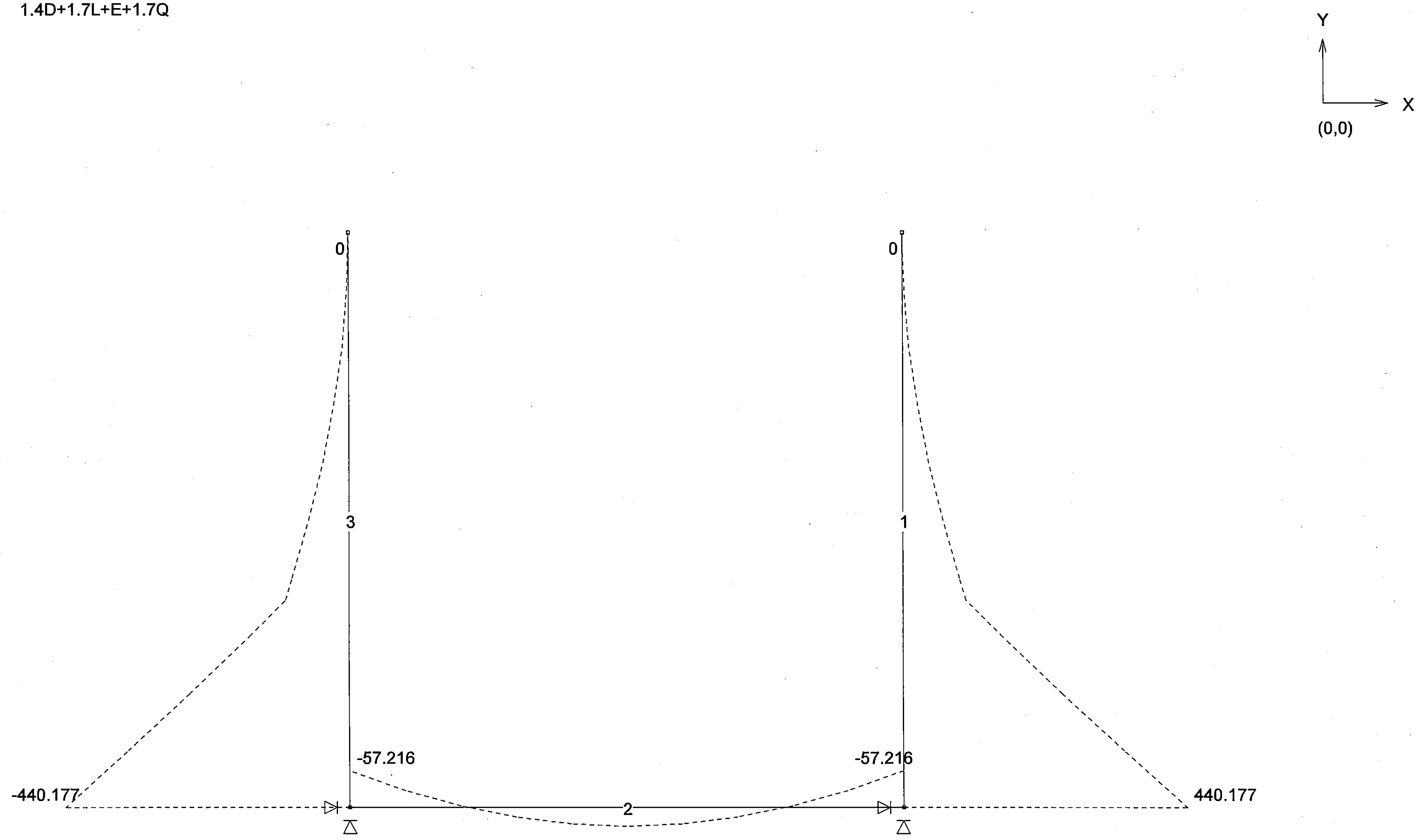
12 —— 1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

Bending Moment Diagram

Job: PS11WALL, Designer: FYP, Units: m,kN,MPa, Scale: 1:63, Axes: XY
Load: 1 Disp: None Moment: 10 Shear: None Axial: None
POMSSUP
WALL FOR PS-11 6 Nov 2011, 11:03 am

Job name: PS11BWAL Designer: FYP Date: 6 Nov 2011, 11:07 am Page: 1
POMSSUP
WALL FOR PS-11

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PS11BWAL	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	4	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	0	(10000)
Member distributed forces	1	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	12	
Total degrees of freedom	12	
Primary load cases	2	(100)
Mass load cases	0	(100)

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	12	266.820*	-44.340	0.000	0.000	0.000	58.832
4	1	0.000#	30.033	0.000	0.000	0.000	-25.027
2	12	44.340	199.546*	0.000	0.000	0.000	-118.530
2	12	44.340	-199.546#	0.000	0.000	0.000	-118.530
1	12	266.820	-44.340	0.000	0.000	0.000	58.832*
1	12	199.546	-44.340	0.000	0.000	0.000	-118.530#

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

Envelope = All Load Cases
and All Nodes

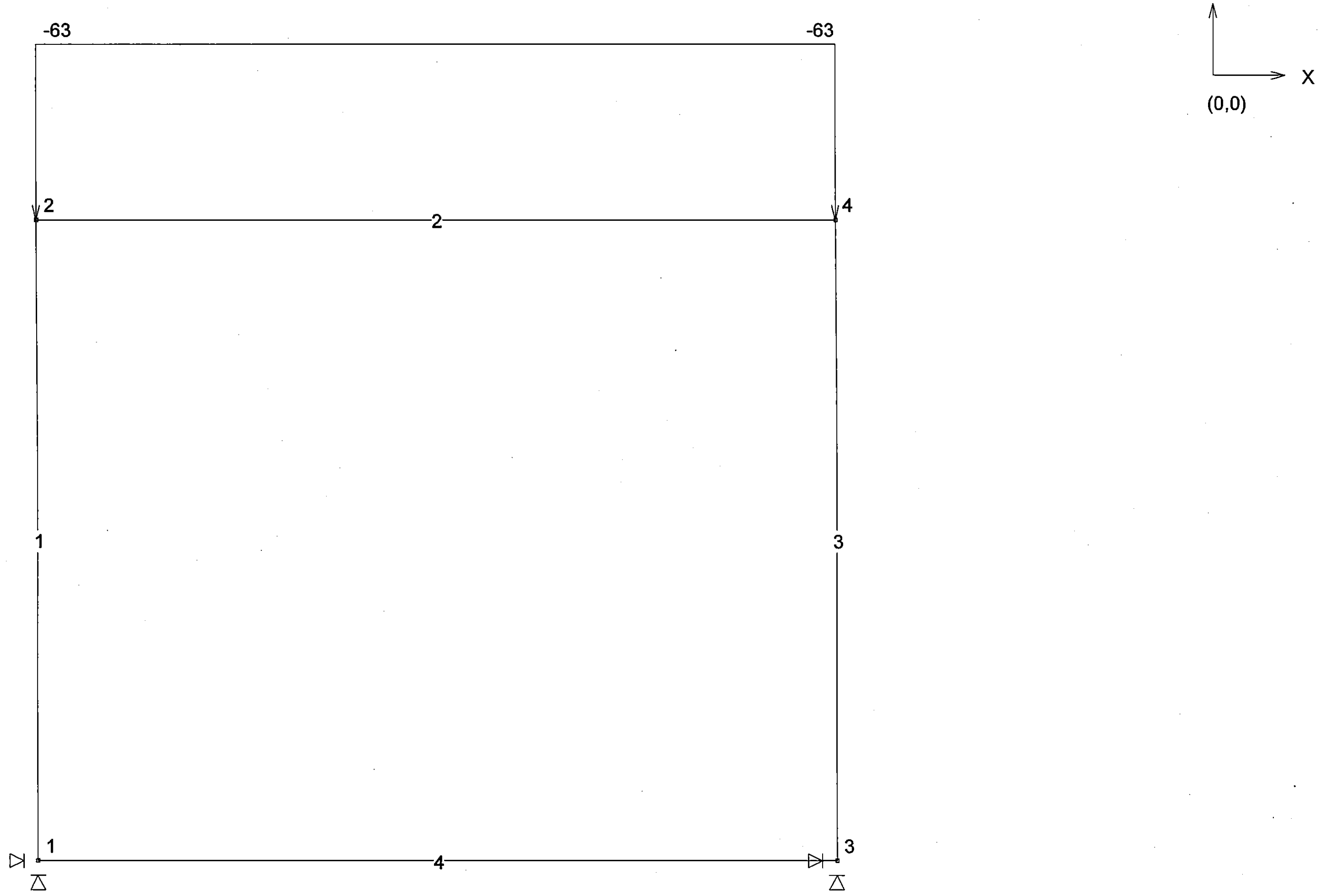
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	12	44.340*	308.866	0.000	0.000	0.000	-23.794
3	12	-44.340#	308.866	0.000	0.000	0.000	23.794
1	12	44.340	308.866*	0.000	0.000	0.000	-23.794
1	9	6.006	97.306#	0.000	0.000	0.000	14.556
3	3	-34.998	157.500	0.000	0.000	0.000	46.436*
1	3	34.998	157.500	0.000	0.000	0.000	-46.436#

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	12	266.820*	-44.340	0.000	0.000	0.000	58.832
4	1	0.000#	30.033	0.000	0.000	0.000	-25.027
2	12	44.340	199.546*	0.000	0.000	0.000	-118.530
2	12	44.340	-199.546#	0.000	0.000	0.000	-118.530
2	12	44.340	0.000	0.000	0.000	0.000	130.903*
2	12	44.340	-199.546	0.000	0.000	0.000	-118.530#

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



Sections:
1 500 x 1000

PS 11 Model 2

Job: PS11BWAL, Designer: FYP, Units: m,kN,MPa, Scale: 1:45, Axes: XY
Load: 2.6 Disp: None Moment: None Shear: None Axial: None
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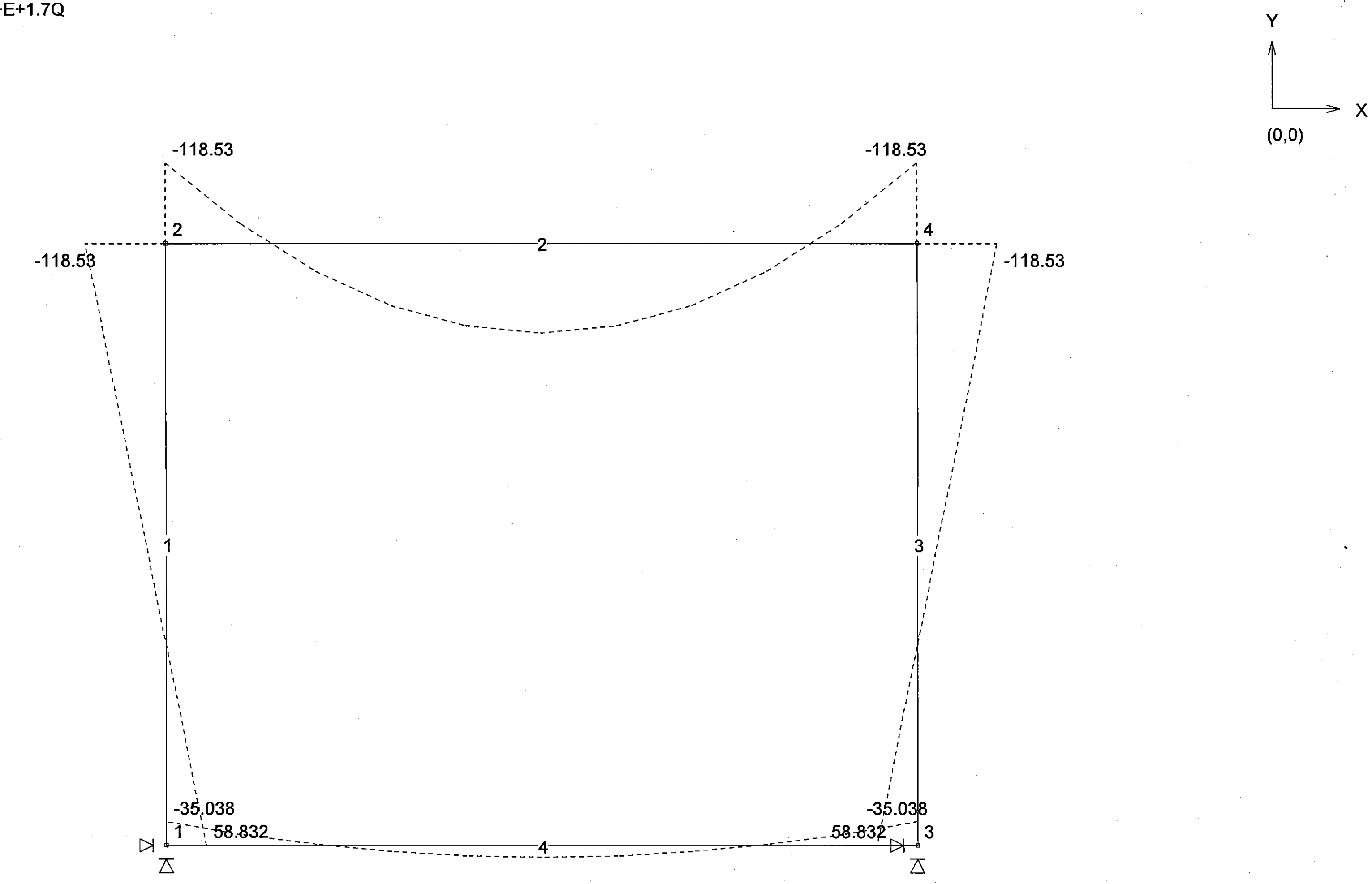
12 1.4D+1.7L+E+1.7Q



Bending Moment Diagram

Sections:
1 500 x 1000

Job: PS11BWAL, Designer: FYP, Units: m,kN,MPa, Scale: 1:45, Axes: XY
Load: 2.6 Disp: None Moment: 10 Shear: None Axial: None
POMSSUP
WALL FOR PS-11 6 Nov 2011, 11:09 am

A-4: RECEIVING WELL 1 & 2, WATER PRESSURE REDUCING WELL

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

Load Calculation

Data

1	**Density of Water (ρ) =**	9.81	**kN/m^3**
	Height of Water, h =	3.7	**m**
	Wall Thickness, tw =	0.5	**m**
	Height of Wall, hw =	6.5	**m**
	Base Slab Thickness, tb =	0.7	**m**
	Length of Base slab, Lb =	5000	**m**

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

Atmospheric Pressure, Patm =	0	**kPa**
Hydrostatic Pressure along 1 m width wall, Pw	36.30	**kN/m (=kPa/m)**

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

Water Pressure on base slab, Pw = ρgh 36.30 **kPa**

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

Flp = 36.30 **kN/m**

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.30	m
Height of Wall (H) =	3.15	m
Embedded Height of Wall (He) =	3.15	m
(Heel Depth) Thickness of Base (tb) =	0.30	m
Width of Base (wb) =	2.60	m
(Heel)/ Length of base (Lb) =	2.60	m
Toe Depth (td) =	0.30	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) =	45	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) =	16.72	LE =	1.050
PS(G) =	6.24	LS =	1.58
PS(Q) =	0.00	LS =	1.58
Total, Pa =	22.96	La =	1.19

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	16.72	1.05	17.55
PS =	6.24	1.58	9.83
Total	22.96	**1.19**	27.39

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	23.625	2.60	61.4
Wall wt =	21.38	2.60	55.6
Base wt =	50.70	1.30	65.9
Total =	95.70	1.27	182.9

CHECK GROUND BEARING FAILURE

∑MO = 27.39 kNm/m

∑MR = 182.9 kNm/m

∑F = 95.70 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

e = B/2 - [∑MR - ∑MO]/∑F 0.33 m **OK, e < B/6**

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

qtoe = ∑F/B [1+6e/B] = 0.85 kPa

qheel = ∑F/B [1-6e/B] = 0.12 kPa

Maximum Bearing Pressure = 0.85 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

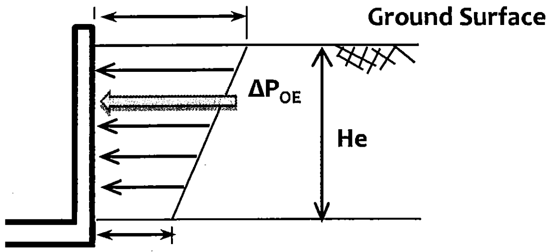
Seismic Design Data

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 = Non-volatile toxic chemicals
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 (γe) = 17.00 kN/m^3

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

fΔe = 1.5 Ch(0) γe He



fΔe = 0.5 Ch(0) γe He

Increment in at-rest pressure due to earthquake

He = embedded depth = 3.15 m
Ah(0) = 1
$C_h(0) = \alpha\ \beta\ A_h(0)\ A_p$, horizontal force coefficient = 0.131
fΔe = 1.5 Ch(0) γe He 11 kPa
fΔe = 0.5 Ch(0) γe He 4 kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	4 kN/m	UDL
Eu2 =	7 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

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = 0.3 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	2.50	5.63E-03	31.9754	0.18	0.4	0.18903	9.57	0.7159	6.8495	No	Yes	Use Winkler	75612
2	2.23	5.01E-03	31.9754	0.18	0.4	0.18360	9.57	0.7318	7.0008	No	Yes	Use Winkler	82518
3	0.30	6.75E-04	31.9754	0.18	0.4	0.11128	9.57	1.0655	10.1938	No	Yes	Use Winkler	370932

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 =	18.75	kN/m
Total G =	52.35	kN/m	Flp =	19.62	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	85.79	kN/m	COMB. 2	G + Flp =	71.97	kN/m
COMB. 3	G + Flp + Fgw =	71.97	kN/m		Long term Load Factors =	1	

2 **Beam Section Properties**

Beam Size

bw =	2500	mm	D =	300	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	2600	mm	Overall depth of sect. D =	300	mm
Effective Length,Lef =	2300	mm	Area, A =	7.50E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	5.63E+09	mm^4
Neutral Axis, xc =	1480.00	mm	Moment of Inertia, Iy-y =	3.91E+11	mm^4
Effective width, beff =	2960.00	mm	Gross Area, Ag =	7.50E+05	mm^2

3 **Design Parameters**

F'c =	40	MPa	Fsy =	410	MPa
m =	2400	kg/m^3	D =	300	mm
Ec =	31975.4	MPa	Es =	200000	MPa
Gamma =	0.766				

4 **Initial Choice of Section**

Effective width = 2960 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.038995	(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	=	85.854		
defln/Lef	=	0.004	k_1/k_2 = 17.2798	

d = 54 mm

take d = 189 mm Distance from d to extreme fibre of beam (do) = 111 mm

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

Mmax -ve	=	56.00	kNm	(left support)
Mmax +ve	=	37.00	kNm	(mid-span)
M -ve	=	53.00	kNm	(right support)
Vmax	=	92.00	kN	
V (other end)	=	92.00	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 = 111 mm d = 189

Estimate Area of main steel Ast = 944.672 mm^2

8 Y 16 Ast = 1608.5 mm^2 **SPACING = 294** mm

p = 0.0034

9 Y 16 Asc = 1809.56 mm^2

pc = 0.00383 < pd = 0.0254088

p-pc = -0.0004 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 95 mm

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

= 63 kNm > M* = 56.00 kNm **OK**

Determine - ve steel at right support :

M -ve = 53.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 111 mm dst : 189 mm

Estimate Area of main steel Ast = 894.064 mm^2

8 Y 16 Ast = 1608.5 mm^2

p = 0.0034

9 Y 16 Asc = 1809.56 mm^2

pc = 0.00383 < pd = 0.0254088

p-pc = -0.0004

Check M* <= phiMu

ku = 0 dsc = 95 mm

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

= 63 kNm > M* = 53.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 37 kNm

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

" " two " " code = 2 bar/s and soffit = 111 mm

what is the code ? 2 d = 189 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 624.1581246 mm^2

Bottom 8 Y 16 Ast = 1608.495439 mm^2

Top 9 Y 16 Asc = 1809.557368 mm^2

Assume N - A in flange

Effective width : beff = 2960 mm

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

= 0.0452629

hence dn = 9 < 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)]

= 110 kNm > M*+ve = 37.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 16
	Asc	=	9	Y 16
Right support :	Ast	=	8	Y 16
	Asc	=	9	Y 16
midspan	Ast	=	8	Y 16
	Asc	=	9	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 = 690 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 = 690 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 = 230 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 16 mm # of lines of bars = 2

a_b = 1163 mm distance between bars

If a_b > 2c C = 2c + d_b = 166 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 1179 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.062 mm^2

k2 = 2.2 a_b = 1163 mm (dist between bars)

factor = 35844

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

= 215.928 mm

take fs/Fsy = M*/Phi Mu = 0.892198

therefore

Lsy =	193 mm	≥ 25k1db	500
say =	650 mm		

For Bottom Steel :

Cover, c = 75 mm d_b = 16 mm

# of lines of bars = 2

If ab > 2c C = 2c + d_b = 166 mm

if ab < 2c C = a_b + d_b = 1353 mm

k1 = 1 A_b = 201.062 mm^2

k2 = 2.2 a_b = 1337 mm (dist between bars)

factor = 28675.2

Lsyt = 172.742 mm take fs/Fsy = M*/Phi Mu = 0.33565146

therefore

Lsy =	58 mm	≥ 25k1db	400
say =	500 mm		

9 PUNCHING SHEAR CHECK

Wall Dimension

V* = 92.00 kN b = 500 mm

Dist to centreline of bottom steel from soffit d = 500 mm

Critical shear perimeter, μ

dq = 83 mm μ = 2(b+dom) + 2(d+dom)

dom = 217 mm 2868 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 = 5,925 kN > V* = 92 kN **OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	56.00 kNm	Es	=	200000 MPa
Ml	=	53.00 kNm	Ec	=	31975.3505 MPa
Mo	=	91.5 kNm			
Ms	=	37.00 kNm			
Ig	=	5.63E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	2960 mm	A_{sc} =	1809.55737 mm^2
webwidth	b_w =	2500 mm	A_{st} =	1608.49544 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	111 mm
			d_{st} =	189 mm
			d =	189 mm
			D =	300 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)

1250 dn^2 + 19569.741 dn - 2956987) = 0

dn = 41.435333 mm

d = 189 mm

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

= 2.78E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 142.3024947 kNm

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

= 3.04E+11 mm^4

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

b_w =	2500 mm	A_{st} =	1608.5 mm^2
d_{sc} =	95 mm	A_{sc} =	1809.56 mm^2
d =	189 mm		

Ig = 5.63E+09 mm^4

At right end (arbitrary)

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

1250 dn^2 + 19569.741 dn - 2804845) = 0

dn = 40.18 mm

d = 189 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.77E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 142.3024947 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 8.80E+10 mm^4

At left end (arbitrary)

b_w = **2500** mm

d_{sc} = **95** mm

A_{st} = **1608.5** mm^2

A_{sc} = **1809.56** mm^2

d = **189** mm

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

1250 dn^2 + 19569.741 dn - 2804845) = 0

dn = 40.184101 mm

d = 189 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.77E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 142.3024947 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 1.04E+11 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 2.00E+11 mm^4

Midspan Deflection :

Lef = 2300 mm

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

= 0.0022469 mm

Total Deflection :

Long term factor	=	1			
Short term factor	=	1			
w - long term	=	85.79 kN/m			
w - short term	=	85.79 kN/m			
Defln,s,sus	=	0.002247 mm			
At midspan :			Ast	=	1608.495439 mm^2
			Asc	=	1809.557368 mm^2

Asc/Ast = 1.125

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

= 0.65

therefore

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

= 0.00371 mm

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

ALLOWABLE DEFLN = Lef / 500 : 4.6 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size 0.04926 5.5

bw	1000	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	3150	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	2750	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	775.00	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	1550.00	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	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 =	1550 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2) =			1	
k_1 =	0.045	(RECT sections)		1
=	0.03210456	(T and L sections)		2
what type of span?	5	interior span =		5
		end span =		6
k_2 =	0.0026042			
kcs =	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef =	29.96			
defln/Lef =	0.004	k_1/k_2 =	17.27977882	
d =	57 mm			
take d =	197 mm	Distance from d to extreme fibre of beam (do) =	103 mm	

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

Mmax -ve	=	61.00	kNm	(left support)
Mmax +ve	=	32.00	kNm	(mid-span)
M -ve	=	19.00	kNm	(right support)
Vmax	=	83.00	kN	
V (other end)	=	83.00	kN	
N*	=	73.00	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support) Bar Spacing 221 mm

Number of reinforcement layers 1 or 2 : 1

take do = 103 mm d = 197

Estimate Area of main steel Ast = 987.229938 mm^2

8 Y 16 Ast = 1608.495439 mm^2 OUTSIDE

p = 0.008164951

9 Y 16 Asc = 1809.557368 mm^2 INSIDE

pc = 0.00918557 < pd = 0.025409

p-pc = -0.00102062 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 95 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* =	61.00 kNm	OK

Determine - ve steel at right support :

M -ve = 19.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 103 mm dst = 197 mm

Estimate Area of main steel Ast = 307.4978495 mm^2

8 Y 16 Ast = 1608.495439 mm^2 OUTSIDE

p = 0.008164951

9 Y 16 Asc = 1809.557368 mm^2 INSIDE

pc = 0.00918557 < pd = 0.025409

p-pc = -0.00102062

Check M* <= phiMu

ku = 0 dsc = 95 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* =	19.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 32 kNm

Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 111 mm

what is the code ? 2 d = 189 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 539.812432 mm^2

8 Y 16 Ast = 1608.495439 mm^2 OUTSIDE

9 Y 16 Asc = 1809.557368 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1550 mm

ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.08643756

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)]

=	108 kNm	>	M*+ve =	32.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 16
	Asc	=	9	Y 16
Right support :	Ast	=	8	Y 16
	Asc	=	9	Y 16
midspan	Ast	=	8	Y 16
	Asc	=	9	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 = 825 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 = 825 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 = 275 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 16 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 166 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 842 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.0619298 mm^2

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

factor = 35843.99446

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

= 215.9277 mm

take fs/Fsy = M*/Phi Mu = 0.895634761

therefore

Lsy =	193 mm
say =	750 mm

≥ 25k1db 500

For Bottom Steel :

Cover, c = 75 mm d_b = 16 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 166 mm

if ab < 2c C = a_b + d_b = 1190 mm

k1 = 1 A_b = 201.0619298 mm^2

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

factor = 28675.19557

Lsyt = 172.7421 mm take fs/Fsy = M*/Phi Mu = 0.295027788

therefore

Lsy =	51 mm
say =	600 mm

≥ 25k1db 400

9 SHEAR CHECK

V* = 83.00 kN

Dist to centreline of bottom steel from soffit

critical location :	d	=	197 mm	dq	=	93 mm	
				do	=	207 mm	
V* =	79 kN	beta 1 =	1.2965	(but not less than 1.1)			
	hence	=	1.2965				
		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

= 127.31 kN

phi Vumax	=	1159.2 kN	>	V*	No web crushing
phi Vumin	=	214 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	=	5211 mm^2

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

theta = 30.43566 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 374.47 kN

Phi Vu = 262 kN > V* = 79.06 kN

hence

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

Other Support :

V* = 83.00 kN

Dist to centreline of bottom steel from soffit

critical location :	d	=	197 mm	dq	=	93 mm	
				do	=	207 mm	
V* =	79.06 kN	beta 1 =	1.2965	(but not less than 1.1)			
	hence	=	1.2965				
		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

= 127.3053 kN

phi Vumax	=	1159.2 kN	>	V*	No web crushing
phi Vumin	=	214 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	=	5210.79976 mm^2

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

theta = 31.04114 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 463.9006 kN

Phi Vu = 324.7304 kN > V* = 79.06 kN

hence

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

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	61.00 kNm		Es	=	200000 MPa
Ml	=	19.00 kNm		Ec	=	31975.35051 MPa
Mo	=	72 kNm				
Ms	=	32.00 kNm				
Ig	=	2.25E+09 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1550 mm	A_{sc} =	1809.557368 mm^2
webwidth	b_w =	1000 mm	A_{st} =	1608.495439 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	111 mm
			d_{st} =	189 mm
			d =	197 mm
			D =	300 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 + 19569.7411 dn - 3037474.044) = 0

dn = 60.7914613 mm

d = 189 mm

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

= 2.40E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

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

= 1.16E+10 mm^4

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

b_w =	1000 mm	A_{st} =	1608.495439 mm^2
d_{sc} =	95 mm	A_{sc} =	1809.557368 mm^2
d =	197 mm		

Ig = 2.25E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 19569.7411 dn - 2885331.728) = 0

dn = 58.88 mm

d = 197 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.60E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 1.88E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **95** mm

A_{st} = **1608.4954** mm^2

A_{sc} = **1809.5574** mm^2

d = **197** mm

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

500 dn^2 + 19569.7411 dn - 2885331.728) = 0

dn = 58.8753877 mm

d = 197 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.60E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 5.38E+10 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 1.97E+10 mm^4

Midspan Deflection :

Lef = 2750 mm

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

= 0.03003406 mm

Total Deflection :

Long term factor	=	0.2		
Short term factor	=	0.5		
w - long term	=	12 kN/m		
w - short term	=	15 kN/m		
Defln,s,sus	=	0.024027249 mm		
At midspan :			Ast =	1608.49544 mm^2
			Asc =	1809.55737 mm^2
Asc/Ast =		1.125		
kcs	=	2 - (1.2 x Asc/Ast)		>= 0.8
	=	0.8		

therefore

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

= 0.04926 mm

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

ALLOWABLE DEFLN = Lef / 500 = 5.5 mm

> 0.9 x Total Defln

SUMMARY

A. Base Slab

300 thk RC Slab

Reinforcement:	Y16 bars @ 250 cts	Top & Bottom Each Way
T & S	Y16 bars @ 250 cts	Top & Bottom Each Way

B. Tank Wall

300 thk RC Wall

Reinforcements:	Vertical:	Y16 - 200 cts	Outside
		Y16 - 200 cts	Inside
	Horizontal	Y12 - 300 cts	

ANALYSIS STATUS REPORT

Job	C:\SGWIN\SMIT\PMPWEL1	
Units system	Kilonewtons, Metres	
Nodes	4	(2500)
Members	3	(5000)
Restrained nodes	2	(2500)
Nodes with spring restraints	0	(2500)
Section properties	1	(100)
Material properties	1	(25)
Constrained nodes	0	(2500)
Member offsets	0	(5000)
Node loads	0	(10000)
Prescribed node displacements	0	(10000)
Member concentrated loads	2	(10000)
Member distributed forces	4	(10000)
Member distributed torsions	0	(10000)
Thermal/Prestress loads	0	(10000)
Self weight load cases	1	(100)
Combination load cases	7	(100)
Load cases with titles	12	(100)
Lumped masses	0	(10000)
Spectral load cases	0	(100)
Static analysis	Y	
Dynamic analysis	N	
Response analysis	N	
Buckling analysis	N	
Ill-conditioned	N	
Non-linear convergence	Y	
Frontwidth	6	
Total degrees of freedom	12	
Primary load cases	3	(100)
Mass load cases	0	(100)

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

900 * Load case 1: D
000 * Load case 3: E

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	43.728*	39.032	0.000	0.000	0.000	-27.322
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	43.728	58.532*	0.000	0.000	0.000	-56.616
3	12	43.728	-58.532#	0.000	0.000	0.000	-52.713
3	12	0.000	0.000	0.000	0.000	0.000	0.000*
1	12	43.728	58.532	0.000	0.000	0.000	-56.616#

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

Envelope = All Load Cases
and All Nodes

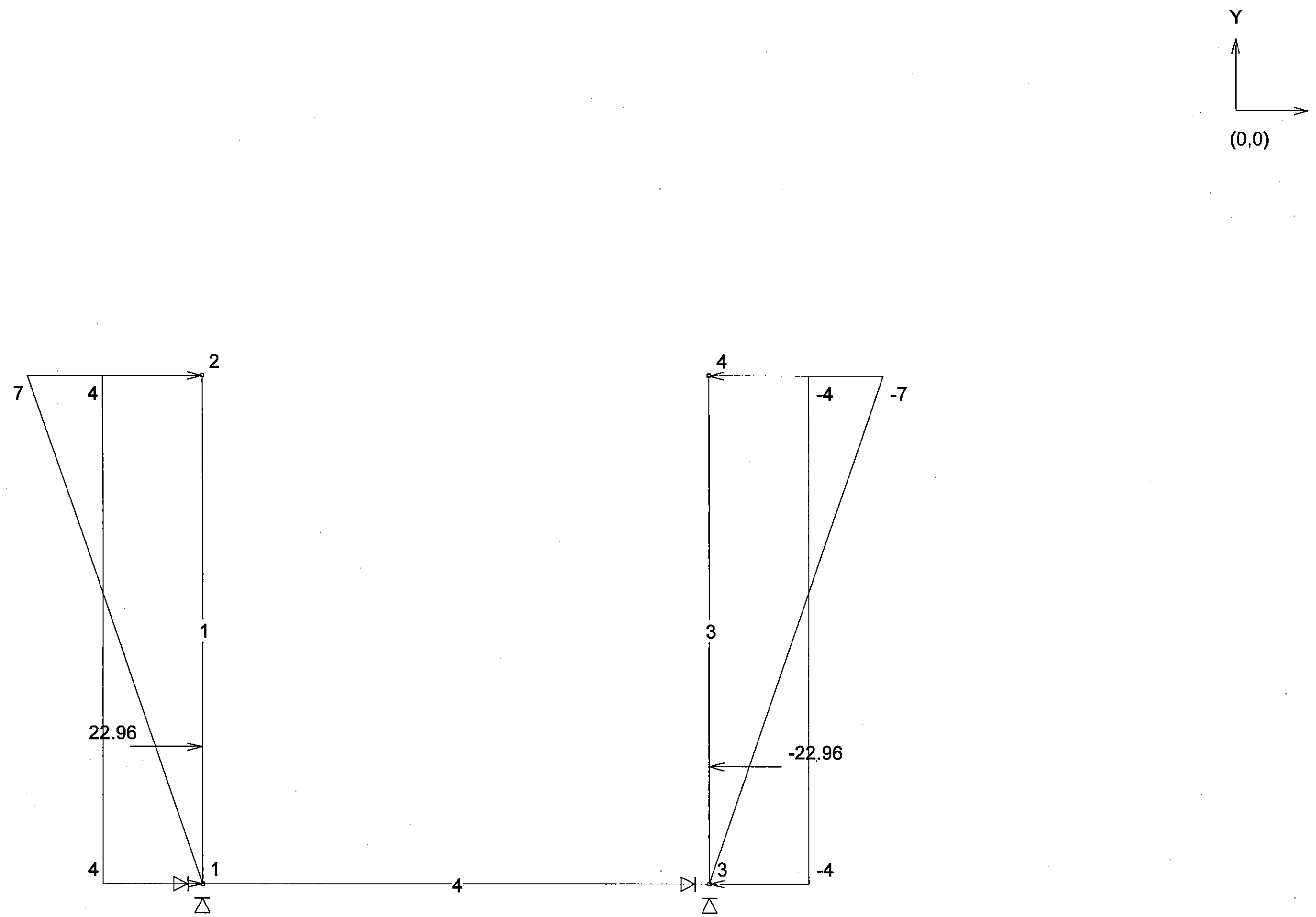
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
3	12	58.532*	65.592	0.000	0.000	0.000	-62.187
1	12	-58.532#	65.592	0.000	0.000	0.000	66.090
1	8	-39.032	65.592*	0.000	0.000	0.000	36.797
1	3	-19.500	0.000#	0.000	0.000	0.000	29.293
1	12	-58.532	65.592	0.000	0.000	0.000	66.090*
3	12	58.532	65.592	0.000	0.000	0.000	-62.187#

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	43.728*	39.032	0.000	0.000	0.000	-27.322
1	7	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	43.728	58.532*	0.000	0.000	0.000	-56.616
3	12	43.728	-58.532#	0.000	0.000	0.000	-52.713
4	8	0.000	0.000	0.000	0.000	0.000	4.737*
1	12	43.728	58.532	0.000	0.000	0.000	-56.616#

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

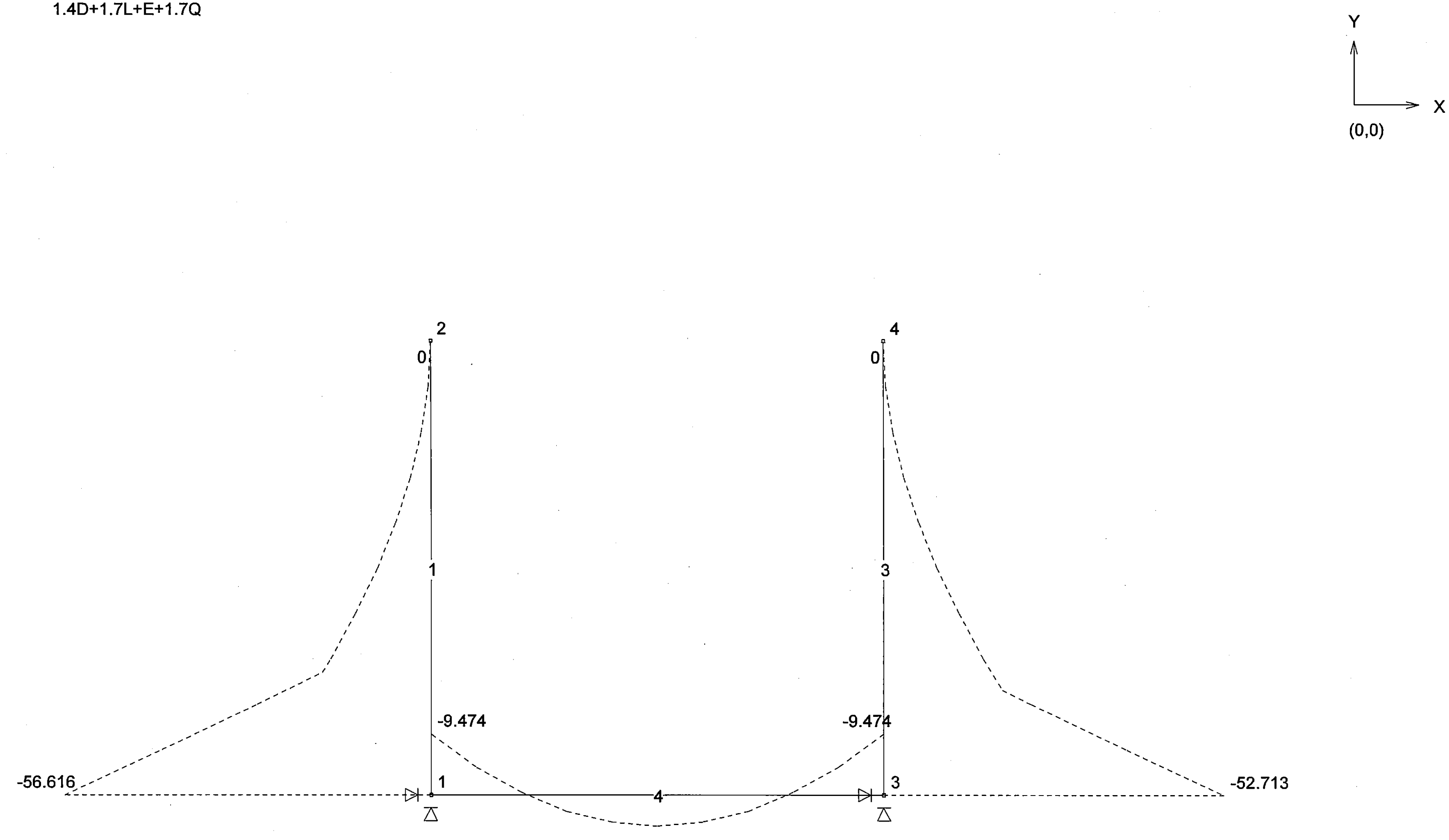


Well Model

Sections:
1 500 x 1000

Job: PMPWEL1, Designer: FYP, Units: m,kN,MPa, Scale: 1:37, Axes: XY
Load: 0.29 Disp: None Moment: None Shear: None Axial: None
POMSSUP
Well 6 Nov 2011, 1:06 pm

12 1.4D+1.7L+E+1.7Q



Bending Moment diagram

Sections:
1 500 x 1000

Job: PMPWEL1, Designer: FYP, Units: m,kN,MPa, Scale: 1:37, Axes: XY
Load: 0.29 Disp: None Moment: 1 Shear: None Axial: None
POMSSUP
Well 6 Nov 2011, 1:05 pm

A-5: Flow Meter Chamber

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

Load Calculation

Data

1	**Density of Water (ρ) =**	**9.81**	**kN/m^3**
	Height of Water, h =	**3.7**	**m**
	Wall Thickness, tw =	**0.3**	**m**
	Height of Wall, hw =	**2.45**	**m**
	Base Slab Thickness, tb =	**0.3**	**m**
	Length of Base slab, Lb =	**4.7**	**m**

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

Atmospheric Pressure, Patm =	**0**	**kPa**
Hydrostatic Pressure along 1 m width wall, Pw	**36.30**	**kN/m (=kPa/m)**

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

Water Pressure on base slab, Pw = ρgh **36.30 kPa**

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

Flp = **36.30 kN/m**

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.30	m
Height of Wall (H) =	2.45	m
Embedded Height of Wall (He) =	2.45	m
(Heel Depth) Thickness of Base (tb) =	0.30	m
Width of Base (wb) =	4.70	m
(Heel)/ Length of base (Lb) =	1.60	m
Toe Depth (td) =	0.30	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) =	42	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) = 1.5

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) =	10.11	LE =	0.817
PS(G) =	4.86	LS =	1.23
PS(Q) =	0.00	LS =	1.23
Total, Pa =	14.97	La =	0.95

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	10.11	0.82	8.26
PS =	4.86	1.23	5.95
Total	14.97	**0.95**	14.21

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	18.375	1.60	29.4
Wall wt =	16.13	1.60	25.8
Base wt =	56.40	0.80	45.1
Total =	90.90	0.78	100.3

CHECK GROUND BEARING FAILURE

$\sum$MO = 14.21 kNm/m

$\sum$MR = 100.3 kNm/m

$\sum$F = 90.90 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

$e = B/2 - [\sum MR - \sum MO]/\sum F$ 0.15 m **OK, e < B/6**

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

qtoe = $\sum F/B\,[1+6e/B]$ = 0.76 kPa

qheel = $\sum F/B\,[1-6e/B]$ = 0.22 kPa

Maximum Bearing Pressure = 0.76 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

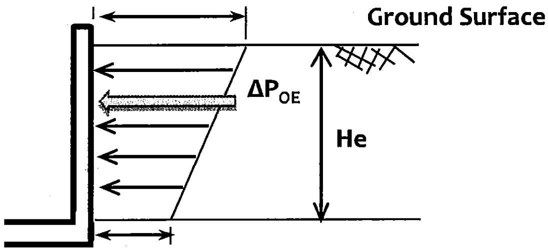
Location =	Port Moresby	
Category (pg 15) =	C	
Annual Probability of Exceedence of Design Earthquake (p): Table 1.1; pg 15		
p =	0.005	
Tank Contents =	Non-volatile toxic chemicals	
probability factor; Fig. C2.7; pg 51 (Ap) =	1.25	
geographical coefficient representing regional seismicity; Fig. C2.1; 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**

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 =	2.45	m
Ah(0) =	1	
$C_h(0) = \alpha\ \beta\ A_h(0)\ A_p$, horizontal force coefficient =	0.131	
fΔe = 1.5 Ch(0) ϒe He	8	kPa
fΔe = 0.5 Ch(0) ϒe He	3	kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	3	kN/m	UDL
Eu2 =	5	kN/m	Triangular

Base Slab Analysis

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

Design data: Compacted Material

μ = 0.4 MPa
Es = 180 MPa
π = 3.141593

D = 0.3 m
F'c = 40 MPa
ρ = 2400 kg/m^3

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

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

ks = k's B

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

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

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

Section	B m	I m^4	Ec GPa	Es GPa	μ MPa	k's	L m	λ	λL	Is $\lambda L < \pi/4$	Is $\lambda L > \pi$	Analysis method ?	ks = k'sB kNm
1	0.50	1.13E-03	31.9754	0.18	0.4	0.12643	9.57	0.9682	9.2626	No	Yes	Use Winkler	252863
2	0.50	1.13E-03	31.9754	0.18	0.4	0.12643	9.57	0.9682	9.2626	No	Yes	Use Winkler	252863
3	0.50	1.13E-03	31.9754	0.18	0.4	0.12643	9.57	0.9682	9.2626	No	Yes	Use Winkler	252863
4	0.50	1.13E-03	31.9754	0.18	0.4	0.12643	9.57	0.9682	9.2626	No	Yes	Use Winkler	252863
5	2.25	5.06E-03	31.9754	0.18	0.4	0.18412	9.57	0.7302	6.9862	No	Yes	Use Winkler	81830
6	2.25	5.06E-03	31.9754	0.18	0.4	0.18412	9.57	0.7302	6.9862	No	Yes	Use Winkler	81830
7	2.25	5.06E-03	31.9754	0.18	0.4	0.18412	9.57	0.7302	6.9862	No	Yes	Use Winkler	81830
8	2.25	5.06E-03	31.9754	0.18	0.4	0.18412	9.57	0.7302	6.9862	No	Yes	Use Winkler	81830

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

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

Design of a Continuous Beam :

1 **Loads**

1 **Serviceability Load Combination : Long term loads**

Main Load Dead Load =	33.60	kN/m	Self weight =	16.88	kN/m
Total G =	50.48	kN/m	Flp =	19.62	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	83.92	kN/m	COMB. 2	G + Flp =	70.10	kN/m
COMB. 3	G + Flp + Fgw =	70.10	kN/m		Long term Load Factors =	1	

2 **Beam Section Properties**

Beam Size

bw =	2250	mm	D =	300	mm
tff =	0	mm	Cover =	75	mm
Actual length, L =	4700	mm	Overall depth of sect. D =	300	mm
Effective Length,Lef =	4500	mm	Area, A =	6.75E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	5.06E+09	mm^4
Neutral Axis, xc =	1575.00	mm	Moment of Inertia, Iy-y =	2.85E+11	mm^4
Effective width, beff =	3150.00	mm	Gross Area, Ag =	6.75E+05	mm^2

3 **Design Parameters**

F'c =	40	MPa	Fsy =	410	MPa
m =	2400	kg/m^3	D =	300	mm
Ec =	31975.4	MPa	Es =	200000	MPa
Gamma =	0.766				

4 **Initial Choice of Section**

Effective width = 3150 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.0343921	(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	=	82.779		
defln/Lef	=	0.004	k_1/k_2 = 17.2798	

d = 103 mm

take d = 189 mm   Distance from d to extreme fibre of beam (do) = 111 mm

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

Mmax -ve	=	32.00	kNm	(left support)
Mmax +ve	=	23.00	kNm	(mid-span)
M -ve	=	35.00	kNm	(right support)
Vmax	=	95.00	kN	
V (other end)	=	95.00	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 = 111 mm d = 189

Estimate Area of main steel Ast = 539.812 mm^2

7 Y 16 Ast = 1407.43 mm^2 **SPACING = 300** mm

p = 0.00331

7 Y 16 Asc = 1407.43 mm^2

pc = 0.00331 < pd = 0.0254088

p-pc = 0 pd=0.4*0.85*μ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 95 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)]

= 49 kNm > M* = 32.00 kNm **OK**

Determine -ve steel at right support :

M -ve = 35.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 111 mm dst : 189 mm

Estimate Area of main steel Ast = 590.42 mm^2

7 Y 16 Ast = 1407.43 mm^2

p = 0.00331

7 Y 16 Asc = 1407.43 mm^2

pc = 0.00331 < pd = 0.0254088

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 95 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)]

= 49 kNm > M* = 35.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 23 kNm

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

" " two " " code = 2 bar/s and soffit = 111 mm

what is the code ? 2 d = 189 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 387.9901855 mm^2

Bottom 7 Y 16 Ast = 1407.433509 mm^2

Top 7 Y 16 Asc = 1407.433509 mm^2

Assume N - A in flange

Effective width : beff = 3150 mm

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

= 0.0372162

hence dn = 7 < 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)]

= 97 kNm > M*+ve = 23.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	7	Y 16
	Asc	=	7	Y 16
Right support :	Ast	=	7	Y 16
	Asc	=	7	Y 16
midspan	Ast	=	7	Y 16
	Asc	=	7	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 = 1350 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 = 1350 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 = 450 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 16 mm # of lines of bars = 2

a_b = 1038 mm distance between bars

If a_b > 2c C = 2c + d_b = 166 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 1054 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 201.062 mm^2

k2 = 2.2 a_b = 1038 mm (dist between bars)

factor = 35844

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

= 215.928 mm

take fs/Fsy = M*/Phi Mu = 0.655493

therefore Lsy = 142 mm ≥ 25k1db 500

say = 500 mm

For Bottom Steel :

Cover, c = 75 mm d_b = 16 mm

# of lines of bars = 2

If ab > 2c C = 2c + d_b = 166 mm

if ab < 2c C = a_b + d_b = 1228 mm

k1 = 1 A_b = 201.062 mm^2

k2 = 2.2 a_b = 1212 mm (dist between bars)

factor = 28675.2

Lsyt = 172.742 mm take fs/Fsy = M*/Phi Mu = 0.23770957

therefore Lsy = 41 mm ≥ 25k1db 400

say = 400 mm

9 PUNCHING SHEAR CHECK

Wall Dimension

V* = 95.00 kN b = 500 mm

Dist to centreline of bottom d = 500 mm

steel from soffit Critical shear perimeter, µ

dq = 83 mm µ = 2(b+dom) + 2(d+dom)

dom = 217 mm 2868 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 = 5,925 kN > V* = 95 kN **OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	32.00 kNm		Es	=	200000 MPa
Ml	=	35.00 kNm		Ec	=	31975.3505 MPa
Mo	=	56.5 kNm				
Ms	=	23.00 kNm				
Ig	=	5.06E+09 mm^4				

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	**b_{eff} =**	3150 mm	**A_{sc} =**	1407.43351 mm^2
webwidth	**b_w =**	2250 mm	**A_{st} =**	1407.43351 mm^2
Flange thickness	**t_{ff} =**	0 mm	**d_{sc} =**	111 mm
			d_{st} =	189 mm
			d =	189 mm
			D =	300 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)

1125 dn^2 + 16199.048 dn - 2484747) = 0

dn = 40.345117 mm

d = 189 mm

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

= 2.44E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 128.0722452 kNm

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

= 8.32E+11 mm^4

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

b_w =	2250 mm	**A_{st} =**	1407.43 mm^2
d_{sc} =	95 mm	**A_{sc} =**	1407.43 mm^2
d =	189 mm		

Ig = 5.06E+09 mm^4

At right end (arbitrary)

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

1125 dn^2 + 16199.048 dn - 2366414) = 0

dn = 39.23 mm

d = 189 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.43E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 128.0722452 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 3.09E+11 mm^4

At left end (arbitrary)

b_w = **2250** mm

d_{sc} = **95** mm

A_{st} = **1407.43** mm^2

A_{sc} = **1407.43** mm^2

d = **189** mm

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

1125 dn^2 + 16199.048 dn - 2366414) = 0

dn = 39.225773 mm

d = 189 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.43E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 128.0722452 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 2.36E+11 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 5.53E+11 mm^4

Midspan Deflection :

Lef = 4500 mm

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

= 0.0019462 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 83.92 kN/m

w - short term = 83.92 kN/m

Defln,s,sus = 0.001946 mm

At midspan : Ast = 1407.433509 mm^2

Asc = 1407.433509 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)

= 0.00350 mm

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

ALLOWABLE DEFLN = Lef / 500 : 9 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

Chamber Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw	1000	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	2450	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	2400	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	740.00	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	1480.00	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	mm
Fsy =	410	MPa	Ec =	31975.3505	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

4 INITIAL CHOICE OF SECTION

Effective width =		1480 mm	clause 8.8.2		
Is the section RECT or T , L (ans : 1 or 2)		=		1	
k_1	=	0.045	(RECT sections)		1
	=	0.03310143	(T and L sections)		2
what type of span?		5	interior span =		5
			end span =		6
k_2	=	0.0026042			
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)		
Fdef	=	29.96			
defln/Lef	=	0.004	k_1/k_2 =	17.2797788	
d =		50 mm			
take d =		197 mm	Distance from d to extreme fibre of beam (do) =	103 mm	

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	43.00	kNm	(left support)
Mmax +ve	=	43.00	kNm	(mid-span)
M -ve	=	43.00	kNm	(right support)
Vmax	=	39.00	kN	
V (other end)	=	39.00	kN	
N*	=	41.00	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine -ve steel required : (At arbitrary left support) Bar Spacing = 196 mm

Number of reinforcement layers 1 or 2 : 1

take	do =	103 mm	d =	197		
Estimate Area of main steel		Ast =	695.916186 mm^2			
6	Y 16	Ast =	1206.371579 mm^2	OUTSIDE		
		p =	0.00612371			
6	Y 16	Asc =	1206.371579 mm^2	INSIDE		
		pc =	0.00612371	<	pd =	0.025409
		p-pc =	0	pd=0.4*0.85*u*f'c/fsy		

Check M* <= phiMu

ku =	0	dsc =	95	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)]

=	45	kNm	>	M* =	43.00 kNm	OK

Determine - ve steel at right support :
M -ve = 43.00 kNm
Number of reinforcement layers 1 or 2 : 1
take do = 103 mm dst = 197 mm
Estimate Area of main steel Ast = 695.916186 mm^2
6 Y 16 Ast = 1206.371579 mm^2 OUTSIDE
p = 0.00612371
6 Y 16 Asc = 1206.371579 mm^2 INSIDE
pc = 0.00612371 < pd = 0.025409
p-pc = 0

Check M* <= phiMu
ku = 0 dsc = 95 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)]

=	45	kNm	>	M* =	43.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI
Tensile Steel at Midspan :
M* = 43 kNm
Steel in one layer then code = 1 take dist between centroid of
" " two " " code = 2 bar/s and soffit = 111 mm
what is the code ? 2 d = 189 mm
Ast = M*/(Phi x 0.85 x d x Fsy) = 725.372956 mm^2
6 Y 16 Ast = 1206.37158 mm^2 OUTSIDE
6 Y 16 Asc = 1206.37158 mm^2 INSIDE
Assume N - A in flange
Effective width : beff = 1480 mm
ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.06789437
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)]

=	82 kNm	>	M*+ve =	43.00 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	6	Y 16
	Asc	=	6	Y 16
Right support :	Ast	=	6	Y 16
	Asc	=	6	Y 16
midspan	Ast	=	6	Y 16
	Asc	=	6	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 = 720 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 = 720 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 = 240 mm

8 STRESS DEVELOPMENT
For Top Steel :
cover : c = 75 mm
d_b = 16 mm # of lines of bars = 1
a_b = 826 mm distance between bars
If a_b > 2c C = 2c + d_b = 166 mm
C is the outside diameter of a concrete annulus
if a_b =< 2c C = a_b + d_b = 842 mm
coaxial with and surrounding a bar
k1 = 1.25 A_b = 201.06193 mm^2
k2 = 2.2 a_b = 826 mm (dist between bars)
factor = 35843.9945
Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 215.9277 mm
take fs/Fsy = M*/Phi Mu = 0.947023641
therefore

Lsy = 204 mm	≥	25k1db	500
say = 500 mm			

For Bottom Steel:

Cover, c = 75 mm d_b = 16 mm
# of lines of bars = 1

If ab > 2c C = 2c + d_b = 166 mm
if ab < 2c C = a_b + d_b = 1190 mm
k1 = 1 A_b = 201.06193 mm^2
k2 = 2.2 a_b = 1174 mm (dist between bars)
factor = 28675.1956
Lsyt = 172.7421 mm take fs/Fsy = M*/Phi Mu = 0.524737149
therefore

Lsy = 91 mm	≥	25k1db	400
say = 400 mm			

9 **SHEAR CHECK**

V* = 39.00 kN
Dist to centreline of bottom steel from soffit
critical location : d = 197 mm dq = 93 mm
do = 207 mm
V* = 35 kN beta 1 = 1.2965 (but not less than 1.1)
hence = 1.2965
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
= 115.68 kN
phi Vumax = 1159.2 kN > V* No web crushing
phi Vumin = 203 kN > V* OK

If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 250 mm Fsyf = 410 MPa
Determine theta :
bv x s/Fsy f = 609.756098
Asv.min = 213.414634 mm^2
Asv.max = 4391 mm^2
Asv = 2 x area of stirrups = 400 mm^2 2 Ties
theta = 30.66991 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 394.23 kN
Phi Vu = 276 kN > V* = 35.06 kN
hence

PROVIDE : 1 Y12 LIGS @ 250 centres

Other Support :
V* = 39.00 kN
Dist to centreline of bottom steel from soffit
critical location : d = 197 mm dq = 93 mm
do = 207 mm
V* = 35.06 kN beta 1 = 1.2965 (but not less than 1.1)
hence = 1.2965
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
= 115.6757 kN
phi Vumax = 1159.2 kN > V* No web crushing
phi Vumin = 203 kN < V* OK
If shear reinforcement reqired then proceed as follows :
choose stirrup spacing s = 250 mm Fsyf = 410 MPa
Determine theta :
bv x s/Fsy f = 609.756098
Asv.min = 213.414634 mm^2
Asv.max = 4391.27214 mm^2
Asv = 2 x area of stirrups = 600 mm2 2 Ties
theta = 31.38798 degrees
Determine Phi Vu :
Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 499.1029 kN
Phi Vu = 349.372 kN > V* = 35.06 kN
hence

PROVIDE : 1 Y12 LIGS @ 250 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	43.00 kNm	Es	=	200000 MPa
Ml	=	43.00 kNm	Ec	=	31975.35051 MPa
Mo	=	86 kNm			
Ms	=	43.00 kNm			
Ig	=	2.25E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1480 mm	A_{sc} =	1206.371579 mm^2
webwidth	b_w =	1000 mm	A_{st} =	1206.371579 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	111 mm
			d_{st} =	189 mm
			d =	197 mm
			D =	300 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 + 13884.8979 dn - 2190148.26) = 0

dn = 53.7397049 mm

d = 189 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2
= 1.90E+08 mm4
Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Im = [Icr + (Ig - Icr) (Mcr / Ms) ^3]
= 4.97E+09 mm^4

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

b_w =	1000 mm	**A_{st}** =	1206.37158 mm^2
d_{sc} =	95 mm	**A_{sc}** =	1206.37158 mm^2
d =	197 mm		

Ig = 2.25E+09 mm^4

At right end (arbitrary)

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
500 dn^2 + 13884.8979 dn - 2088720.05) = 0
dn = 52.22 mm
d = 197 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.06E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm
Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 4.95E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm
d_{sc} = **95** mm
A_{st} = **1206.372** mm^2
A_{sc} = **1206.372** mm^2
d = **197** mm

1/2 x bdnSQ + (n - 1) Asc (dn - dsc) = nAst (d - dn)
500 dn^2 + 13884.8979 dn - 2088720.05) = 0
dn = 52.2228209 mm
d = 197 mm
Icr = bdn3/3 + n x Asc (d - dn)^2 = 2.06E+08 mm^4
Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm
Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 4.95E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 4.96E+09 mm^4

Midspan Deflection :

Lef = 2400 mm
Defln,s = Lef^2/Ec x Iav [5/48 . Mo - 1/16 ML - 1/16 Mr]

= 0.13018798 mm

Total Deflection :

Long term factor = 0.2
Short term factor = 0.5
w - long term = 12 kN/m
w - short term = 15 kN/m
Defln,s,sus = 0.104150388 mm
At midspan : Ast = 1206.37158 mm^2
Asc = 1206.37158 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)
= 0.21351 mm

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

ALLOWABLE DEFLN = Lef / 500 = 4.8 mm
> 0.9 x Total Defln

Chamber Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size 0.80121 9

bw	1000	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	4700	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	4500	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	950.00	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	1900.00	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	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 = 1900 mm clause 8.8.2
Is the section RECT or T, L (ans : 1 or 2) = 1

k_1	=	0.045	(RECT sections)	1
	=	0.02841283	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	**1.64**	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	29.96		
defln/Lef	=	**0.004**	k_1/k_2 = 17.27977882	

d = 87 mm
take d = **201** mm Distance from d to extreme fibre of beam (do) = 99 mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	**34.00**	kNm	(left support)
Mmax +ve	=	**53.50**	kNm	(mid-span)
M -ve	=	**34.00**	kNm	(right support)
Vmax	=	**56.40**	kN	
V (other end)	=	**56.40**	kN	
N*	=	**21.00**	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support) Bar Spacing 271 mm

Number of reinforcement layers 1 or 2 : 1
take do = 99 mm d = 201

Estimate Area of main steel		Ast =	539.3088757	mm^2	
8	Y 12	Ast =	904.7786842	mm^2	OUTSIDE
		p =	0.004501386		
8	Y 12	Asc =	904.7786842	mm^2	INSIDE
		pc =	0.004501386	<	pd = 0.025409
		p-pc =	0		pd=0.4*0.85*µ*f'c/fsy

Check M* <= phiMu

ku = 0 dsc = 93 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)]

=	36	kNm	>	M* =	34.00 kNm

OK

Determine -ve steel at right support :

M -ve = 34.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 99 mm dst = 201 mm

Estimate Area of main steel Ast = 539.3088757 mm^2

8 Y 12 Ast = 904.7786842 mm^2 OUTSIDE

p = 0.004501386

8 Y 12 Asc = 904.7786842 mm^2 INSIDE

pc = 0.004501386 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 93 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)]

=	36	kNm	>	M* =	34.00 kNm

OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 53.5 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 = 105 mm

d = 195 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 874.729713 mm^2

8 Y 12 Ast = 904.7786842 mm^2 OUTSIDE

8 Y 12 Asc = 904.7786842 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1900 mm

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

= 0.03844415

hence dn = 7 < 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)]

=	64 kNm	>	M*+ve =	53.50 kNm

OK

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 12
	Asc	=	8	Y 12
Right support :	Ast	=	8	Y 12
	Asc	=	8	Y 12
midspan	Ast	=	8	Y 12
	Asc	=	8	Y 12

7 STEEL CURTAILMENT REQUIREMENT CHECK

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

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

For -ve steel = 2 bars

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

= 0.3 x le = 1350 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 = 1350 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 = 450 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 12 mm   # of lines of bars = 1

a_b = 826 mm   distance between bars

If a_b > 2c   C = 2c + d_b = 162 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c   C = a_b + d_b = 838 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 113.0973355 mm^2

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

factor = 20162.24689

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

= 124.4583 mm

take fs/Fsy = M*/Phi Mu = 0.942945158

therefore

Lsy =	117 mm
say =	750 mm

≥ 25k1db 375

For Bottom Steel :

Cover, c = 75 mm d_b = 12 mm

# of lines of bars = 1

If ab > 2c   C = 2c + d_b = 162 mm

if ab < 2c   C = a_b + d_b = 1186 mm

k1 = 1 A_b = 113.0973355 mm^2

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

factor = 16129.79751

Lsyt = 99.56665 mm   take fs/Fsy = M*/Phi Mu = 0.834050965

therefore

Lsy =	83 mm
say =	600 mm

≥ 25k1db 300

9 SHEAR CHECK

V* = 56.40 kN   Dist to centreline of bottom steel from soffit

critical location : d = 201 mm   dq = 93 mm

do = 207 mm

V* = 52 kN   beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 105.11 kN

phi Vumax = 1159.2 kN > V* No web crushing

phi Vumin = 192 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 609.756098

Asv.min = 213.414634 mm^2

Asv.max = 4436 mm^2

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

theta = 30.66285 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 379.19 kN

Phi Vu = 265 kN > V* = 52.38 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**250**	**centres**

Other Support :

V* = 56.40 kN

Dist to centreline of bottom steel from soffit

critical location : d = 201 mm   dq = 93 mm
do = 207 mm

V* = 52.38 kN   beta 1 = 1.2965 (but not less than 1.1)
hence = 1.2965
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
= 105.1084 kN

phi Vumax = 1159.2 kN > V* No web crushing
phi Vumin = 192 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 609.756098
Asv.min = 213.414634 mm^2
Asv.max = 4435.74044 mm^2

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

theta = 31.37336 degrees

Determine Phi Vu :

Vu = Vuc + Vus
= Vuc + Asv / s x Fsyf x do cot theta
= 484.1985 kN

Phi Vu = 338.9389 kN > V* = 52.38 kN

hence

PROVIDE :	**1 Y12 LIGS**	**@**	**250**	**centres**

11 <u>SERVICEABILITY CHECK</u>

From an Elastic Linear Analysis :

Mr = 34.00 kNm   Es = 200000 MPa
Ml = 34.00 kNm   Ec = 31975.35051 MPa
Mo = 87.5 kNm
Ms = 53.50 kNm
Ig = 2.25E+09 mm^4

<u>Calculate Icr, Mcr, and Ief at midspan :</u>

Eff. Flange width	**b_{eff}** =	1900 mm		**A_{sc}** =	904.7786842 mm^2
webwidth	**b_w** =	1000 mm		**A_{st}** =	904.7786842 mm^2
Flange thickness	**t_{ff}** =	0 mm		**d_{sc}** =	105 mm
				d_{st} =	195 mm
				d =	201 mm
				D =	300 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 + 10413.6734 dn - 1636721.412) = 0

dn = 47.740324 mm
d = 195 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2
= 1.59E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	2.68E+09 mm^4

<u>Calculate Icr , Mcr , ILef and Iref at end supports :</u>

b_w =	1000 mm	**A_{st}** =	904.7786842 mm^2
d_{sc} =	93 mm	**A_{sc}** =	904.7786842 mm^2
d =	201 mm		

Ig = 2.25E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 10413.6734 dn - 1579668.044) = 0

dn = 46.75 mm

d = 201 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.69E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 9.93E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **93** mm

A_{st} = **904.77868** mm^2

A_{sc} = **904.77868** mm^2

d = **201** mm

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

500 dn^2 + 10413.6734 dn - 1579668.044) = 0

dn = 46.750832 mm

d = 201 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.69E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 9.93E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 6.31E+09 mm^4

Midspan Deflection :

Lef = 4500 mm

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

= 0.48854314 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 12 kN/m

w - short term = 15 kN/m

Defln,s,sus = 0.390834515 mm

At midspan : Ast = 904.778684 mm^2

Asc = 904.778684 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)

= 0.80121 mm

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

ALLOWABLE DEFLN = Lef / 500 = 9 mm

> 0.9 x Total Defln

SUMMARY

A. Base Slab

300 thk RC Slab

Reinforcement:	Y16 bars @ 250 cts	Top & Bottom Each Way
T & S	Y16 bars @ 250 cts	Top & Bottom Each Way

B. Chamber Wall

300 thk RC Wall

Reinforcements:	Vertical:	Y16 - 300 cts	Outside
		Y16 - 300 cts	Inside
	Horizontal	Y12 - 300 cts	

Job name: FMCHAMB   Designer: FYP   Date: 6 Nov 2011, 12:30 pm   Page: 1

POMSSUP

FLOW METER CHAMBER

ANALYSIS STATUS REPORT

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

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

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

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

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

MEMBER FORCES AND MOMENTS (kN,kNm)

(*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	41.205*	25.449	0.000	0.000	0.000	-24.177
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	41.205	38.924*	0.000	0.000	0.000	-43.184
3	12	41.205	-38.924#	0.000	0.000	0.000	-43.184
1	12	0.000	0.000	0.000	0.000	0.000	0.000*
3	12	41.205	-38.924	0.000	0.000	0.000	-43.184#

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	12	38.924*	54.660	0.000	0.000	0.000	-46.772
1	12	-38.924#	54.660	0.000	0.000	0.000	46.772
1	8	-25.449	54.660*	0.000	0.000	0.000	27.764
1	3	-13.475	0.000#	0.000	0.000	0.000	19.008
1	12	-38.924	54.660	0.000	0.000	0.000	46.772*
3	12	38.924	54.660	0.000	0.000	0.000	-46.772#

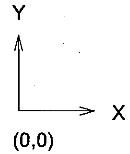
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)

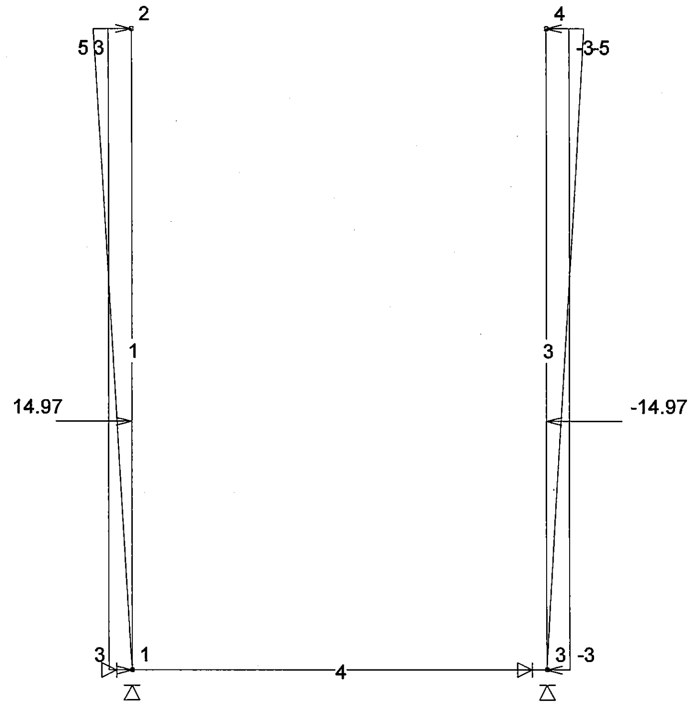
(*=Maximum, #=Minimum)

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	41.205*	25.449	0.000	0.000	0.000	-24.177
1	7	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	41.205	38.924*	0.000	0.000	0.000	-43.184
3	12	41.205	-38.924#	0.000	0.000	0.000	-43.184
4	8	0.000	0.000	0.000	0.000	0.000	1.794*
1	12	41.205	38.924	0.000	0.000	0.000	-43.184#

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



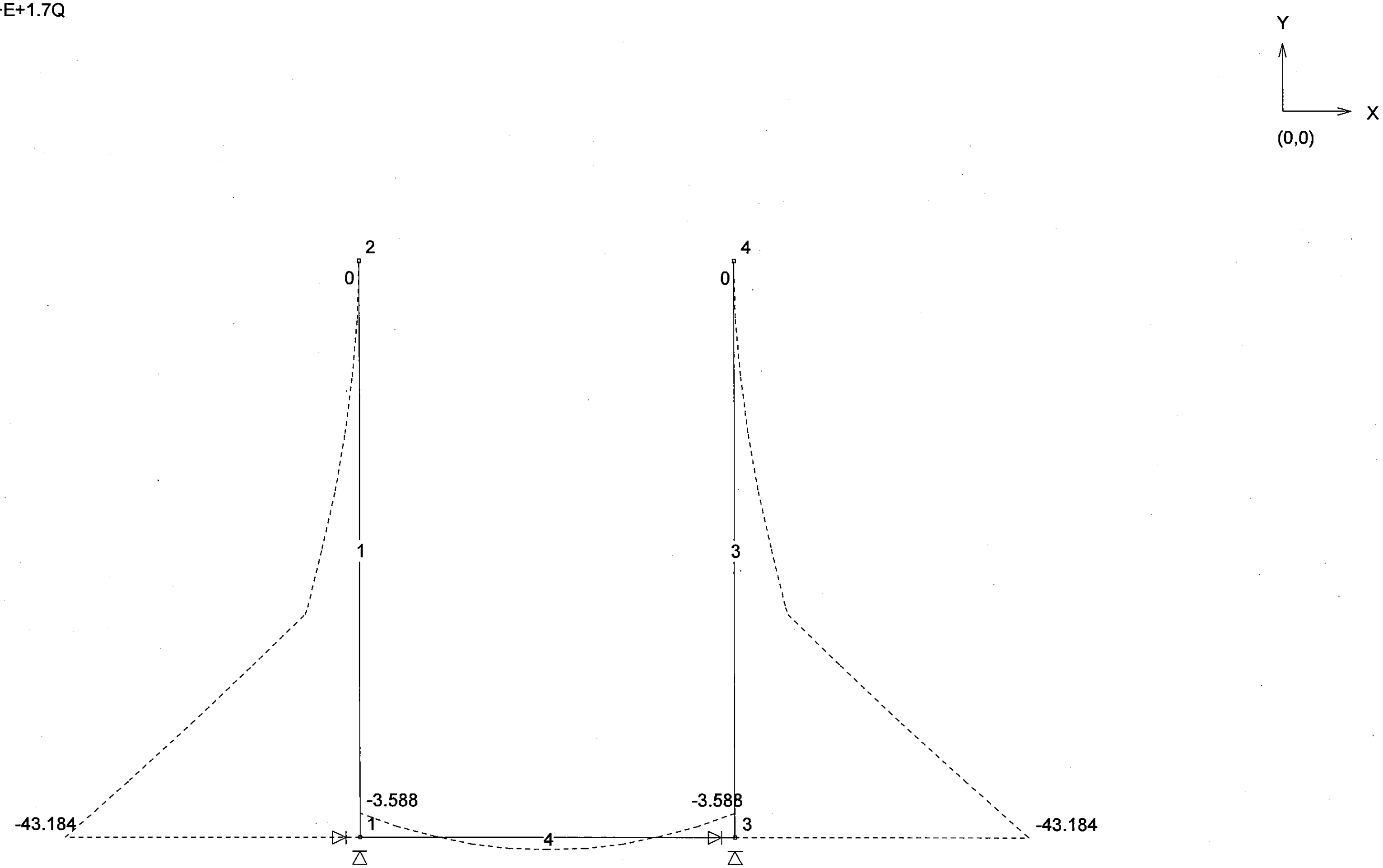


Sections:
1 500 x 1000

FLOW METER CHAMBER MODEL

Job: FMCHAMB, Designer: FYP, Units: m,kN,MPa, Scale: 1:29, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
POMSSUP
FLOW METER CHAMBER 6 Nov 2011, 12:29 pm

12 1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

BENDING MOMENT DIAGRAM

Job: FMCHAMB, Designer: FYP, Units: m,kN,MPa, Scale: 1:29, Axes: XY
Load: 1 Disp: None Moment: 1 Shear: None Axial: None
POMSSUP
FLOW METER CHAMBER 6 Nov 2011, 12:30 pm

A-6: DRAINAGE TANK

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

Load Calculation

Data

	Density of Water (ρ) =	9.81	kN/m^3
1	**Height of Water, h =**	3	m
	Wall Thickness, tw =	0.3	m
	Height of Wall, hw =	4.55	m
	Base Slab Thickness, tb =	0.35	m
	Length of Base slab, Lb =	2.6	m

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

Atmospheric Pressure, Patm =	0	kPa
Hydrostatic Pressure along 1 m width wall, Pw	29.43	kN/m (=kPa/m)

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

Water Pressure on base slab, Pw = ρgh 29.43 kPa

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

Flp = 29.43 kN/m

Earth Pressure

TANK WALL: REINFORCED CONCRETE WALL.

Ref: Foundation Analysis and Design by Joseph E. Bowles

WALL DIMENSION

Thickness of Wall (tw) =	0.30	m
Height of Wall (H) =	4.55	m
Embedded Height of Wall (He) =	4.20	m
(Heel Depth) Thickness of Base (tb) =	0.35	m
Width of Base (wb) =	1.00	m
(Heel)/ Length of base (Lb) =	2.60	m
Toe Depth (td) =	0.35	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) =	42	degree	
Design cohesion of base mat'l (Cb) =	45	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) = 1.5

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) =	29.72	LE =	1.400
PS(G) =	8.33	LS =	2.10
PS(Q) =	0.00	LS =	2.10
Total, Pa =	38.05	La =	1.55

Moment Causing Overturning

Lateral Force	Force (kN/m)	Lever arm to base (m)	Base MT. (kNm/m)
Pa =	29.72	1.40	41.61
PS =	8.33	2.10	17.48
Total	38.05	1.55	59.10

Moment Resisting Overturning

Wt of Retaining Structure	Force (kN/m)	Lever arm to Heel top (m)	Moment (kNm/m)
Dead Load on Wall =	34.125	2.60	88.7
Wall wt =	31.50	2.60	81.9
Base wt =	22.75	1.30	29.6
Total =	88.38	1.26	200.2

CHECK GROUND BEARING FAILURE

∑MO = 59.10 kNm/m

∑MR = 200.2 kNm/m

∑F = 88.38 kN/m

ECCENTRICITY FROM BASE CENTRE, e =

e = B/2 - [∑MR - ∑MO]/∑F 0.30 m **OK, e < B/6**

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

qtoe = ∑F/B [1+6e/B] = 0.82 kPa

qheel = ∑F/B [1-6e/B] = 0.15 kPa

Maximum Bearing Pressure = 0.82 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 = Non-volatile toxic chemicals

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

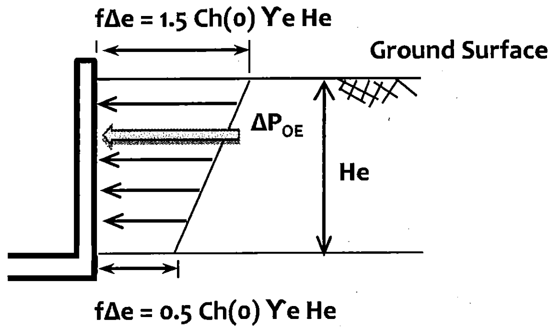
horizontal force coefficient Ch(T) = α β Ah(T) Ap = 0.13 $A_h(T)$

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 =	4.2	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	14	kPa
fΔe = 0.5 Ch(o) ϒe He	5	kPa

Earthquake Earth Pressure Load Calculation for per metre length for wall

Eu1 =	5	kN/m	UDL
Eu2 =	9	kN/m	Triangular

BEAM ON ELASTIC FOUNDATION - WINKLER ANALYSIS

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

D = [redacted] m

Design data: Compacted Material

μ = 0.4 MPa F'c = 40 MPa
Es = 180 MPa ρ = 2400 kg/m^3
π = 3.141593

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

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

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

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

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

Section	B m	I m^4	Ec GPa	Es GPa	μ MPa	k's	L m	λ	λL	Is $\lambda L < \pi/4$	Is $\lambda L > \pi$	Analysis method ?	ks = k'sB kNm
1	0.30	6.75E-04	31.9754	0.18	0.4	0.11128	2.60	1.0655	2.7703	No	No	Use Conventional	370932
2	1.00	2.25E-03	31.9754	0.18	0.4	0.15034	2.60	0.8502	2.2104	No	No	Use Conventional	150343

BASE SLAB DESIGN AS A ONTINUOUS RC BEAM

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

Design of a Continuous Beam :

1 Loads

Serviceability Load Combination : Long term loads

Main Load Dead Load =	33.60	kN/m	Self weight =	8.75	kN/m
Total G =	42.35	kN/m	Flp =	**19.62**	kN/m
Fe =	33.44	kN/m	Fgw =	0	kN/m

COMB. 1	G + Fe + Fgw =	75.79 kN/m	COMB. 2	G + Flp =	61.97	kN/m
COMB. 3	G + Flp + Fgw =	61.97 kN/m	Long term Load Factors =		1	

2 Beam Section Properties

Beam Size

bw =	**1000**	mm	D =	**350**	mm
tff =	**0**	mm	Cover =	75	mm
Actual length, L =	**2600**	mm	Overall depth of sect. D =	350	mm
Effective Length,Lef =	**2400**	mm	Area, A =	3.50E+05	mm^2
Neutral Axis, yc =	175.00	mm	Moment of Inertia, Ix-x =	3.57E+09	mm^4
Neutral Axis, xc =	740.00	mm	Moment of Inertia, Iy-y =	2.92E+10	mm^4
Effective width, beff =	1480.00	mm	Gross Area, Ag =	3.50E+05	mm^2

3 Design Parameters

F'c =	**40**	MPa	Fsy =	**410**	MPa
m =	2400	kg/m^3	D =	350	mm
Ec =	31975.4	MPa	Es =	200000	MPa
Gamma =	0.766				

4 Initial Choice of Section

Effective width	=	1480 mm	clause 8.8.2		
Is the section RECT or T, L (ans : 1 or 2)			=	**1**	
k_1	=	0.045	(RECT sections)		**1**
	=	0.0331014	(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	=	69.454			
defln/Lef	=	**0.004**	k_1/k_2 =	17.28	
d	=	67 mm			
take d =		233 mm	Distance from d to extreme fibre of beam (do) =	**117** mm	

5 Forces from an Elastic Analysis by Space Gass

Mmax -ve	=	**80.00** kNm	(left support)
Mmax +ve	=	**80.00** kNm	(mid-span)
M -ve	=	**80.00** kNm	(right support)
Vmax	=	**250.00** kN	
V (other end)	=	**250.00** 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 = 233

Estimate Area of main steel Ast = 1094.7 mm^2

8 Y 20 Ast = 2513.27 mm^2 **SPACING = 211** mm

p = 0.0108

8 Y 20 Asc = 2513.27 mm^2

pc = 0.0108 < pd = 0.0254088

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)]

= 126 kNm > M* = 80.00 kNm **OK**

Determine -ve steel at right support :

M -ve = 80.00 kNm

Number of reinforcement layers 1 or 2 : 2

take do = 117 mm dst : 233 mm

Estimate Area of main steel Ast = 1094.7 mm^2

8 Y 20 Ast = 2513.27 mm^2

p = 0.0108

8 Y 20 Asc = 2513.27 mm^2

pc = 0.0108 < pd = 0.0254088

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)]

= 126 kNm > M* = 80.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 80 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 = 233 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 1094.68401 mm^2

Bottom 8 Y 20 Ast = 2513.27412 mm^2

Top 8 Y 20 Asc = 2513.27412 mm^2

Assume N - A in flange

Effective width : beff = 1480 mm

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

= 0.1147357

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)]

= 207 kNm > M*+ve = 80.00 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 20
	Asc	=	8	Y 20
Right support :	Ast	=	8	Y 20
	Asc	=	8	Y 20
midspan	Ast	=	8	Y 20
	Asc	=	8	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 = 720 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 = 720 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 = 240 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 20 mm # of lines of bars = 2

a_b = 413 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 = 433 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 314.16 mm^2

k2 = 2.2 a_b = 413 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 = 0.634286

therefore

Lsy =	209 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 = 607 mm

k1 = 1 A_b = 314.16 mm^2

k2 = 2.2 a_b = 587 mm (dist between bars)

factor = 44805

Lsyt = 263.559 mm take fs/Fsy = M*/Phi Mu = 0.38724373

therefore

Lsy =	102 mm
say =	500 mm

≥ 25k1db 500

9 PUNCHING SHEAR CHECK

Wall Dimension

V* = **250.00** kN b = **500** mm

Dist to centreline of bottom steel from soffit d = **500** mm

Critical shear perimeter, μ

dq = 85 mm μ = 2(b+dom) + 2(d+dom)

dom = 265 mm 3060 mm

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

βh = 1 longer side/shorter side

$f_{cv} = 0.17(1+2/\beta_h)\sqrt{f'_c} \le 0.34\sqrt{f'_c}$

$f_{cv} =$ 13.60 MPa

$\phi V_{uo} =$	7,720 kN	>	$V^* =$	250 kN	**OK**

10 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	80.00 kNm	Es	=	200000 MPa
Ml	=	80.00 kNm	Ec	=	31975.3505 MPa
Mo	=	160 kNm			
Ms	=	80.00 kNm			
Ig	=	3.57E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1480 mm	A_{sc} =	2513.27412 mm^2
webwidth	b_w =	1000 mm	A_{st} =	2513.27412 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	233 mm
			d =	233 mm
			D =	350 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)

500 dn^2 + 28926.871 dn - 5E+06) = 0

dn = 77.151913 mm

d = 233 mm

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

= 5.35E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 77.4758027 kNm

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

= 3.29E+09 mm^4

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

b_w =	1000 mm	A_{st} =	2513.3 mm^2
d_{sc} =	97 mm	A_{sc} =	2513.3 mm^2
d =	233 mm		

Ig = 3.57E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 28926.871 dn - 5E+06) = 0

dn = 74.63 mm

d = 233 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 5.33E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 77.4758027 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)' = 3.29E+09 mm^4

At left end (arbitrary)

b_w = **1000** mm

d_{sc} = **97** mm

A_{st} = **2513.27** mm^2

A_{sc} = **2513.27** mm^2

d = **233** mm

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

500 dn^2 + 28926.871 dn - 5E+06) = 0

dn = 74.631984 mm

d = 233 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 5.33E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 77.4758027 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 3.29E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 3.29E+09 mm^4

Midspan Deflection :

Lef = 2400 mm

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

= 0.3645542 mm

Total Deflection :

Long term factor = 1

Short term factor = 1

w - long term = 75.79 kN/m

w - short term = 75.79 kN/m

Defln,s,sus = 0.364554 mm

At midspan : Ast = 2513.27412 mm^2

Asc = 2513.27412 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)

= 0.65620 mm

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

ALLOWABLE DEFLN = Lef / 500 : 4.8 mm

> 0.9 x Total Defln

THE SECTION IS ADEQUATE

Drainage Tank Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size

bw	2600	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	4550	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	4000	mm	Area, A =	7.80E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	5.85E+09	mm^4
Neutral Axis, xc =	1700.00	mm	Moment of Inertia, Iy-y =	4.39E+11	mm^4
Effective width, beff =	3400.00	mm	Gross Area, Ag =	7.80E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	mm
Fsy =	410	MPa	Ec =	31975.3505	MPa
m =	2400	kg/m^3	Es =	200000	MPa
			Gamma =	0.766	

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

Effective width =	3400 mm	clause 8.8.2			
Is the section RECT or T , L (ans : 1 or 2)	=			1	
k_1 =	0.045	(RECT sections)			1
=	0.03612743	(T and L sections)			2
what type of span?	5	interior span =			5
		end span =			6
k_2 =	0.0026042				
kcs =	1.64	(because there will be comp steel at midspan=say .5Ast)			
Fdef =	29.96				
defln/Lef =	0.004	k_1/k_2 =	17.2797788		
d =	64 mm				
take d =	193 mm	Distance from d to extreme fibre of beam (do) =		107 mm	

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

Mmax -ve	=	197.00	kNm	(left support)
Mmax +ve	=	27.60	kNm	(mid-span)
M -ve	=	0.00	kNm	(right support)
Vmax	=	104.60	kN	
V (other end)	=	104.60	kN	
N*	=	70.60	kN	Axial Load

6 <u>FLECTURAL STRENGTH STEEL</u>

<u>Determine - ve steel required :</u> (At arbitrary left support) bar spacing 200 mm

Number of reinforcement layers 1 or 2 :			1			
take do =	107 mm		d =	193		
Estimate Area of main steel		Ast =	3254.34525 mm^2			
18	Y 20	Ast =	5654.866776 mm^2	OUTSIDE		
		p =	0.01126916			
18	Y 20	Asc =	5654.866776 mm^2	INSIDE		
		pc =	0.01126916	<	pd =	0.025409
		p-pc =	0	pd=0.4*0.85*µ*f'c/fsy		

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

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)]

= 200 kNm > M* = 197.00 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 = 193 mm
Estimate Area of main steel Ast = 0 mm^2
18 Y 20 Ast = 5654.866776 mm^2 OUTSIDE
p = 0.01126916
18 Y 20 Asc = 5654.866776 mm^2 INSIDE
pc = 0.01126916 < 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)]

= 200 kNm > M* = 0.00 kNm **OK**

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI
Tensile Steel at Midspan :
M* = 27.6 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 = 183 mm
Ast = M*/(Phi x 0.85 x d x Fsy) = 480.85341 mm^2
18 Y 20 Ast = 5654.86678 mm^2 OUTSIDE
18 Y 20 Asc = 5654.86678 mm^2 INSIDE
Assume N - A in flange
Effective width : beff = 3400 mm
ku = 1/(0.85 x0.85) x [Ast/(bw x d)] x (Fsy/F'c)
= 0.14307658
hence dn = 26 < flange thickness
hence N -A is in flange
Phi Mu = 0.9[Fsy x Ast x d (1 - 0.5 x 0.85 x ku)]

= 361 kNm > M*+ve = 27.60 kNm **OK**

Summary of reinforcement requirement :

left support :	Ast	=	18	Y 20
	Asc	=	18	Y 20
Right support :	Ast	=	18	Y 20
	Asc	=	18	Y 20
midspan	Ast	=	18	Y 20
	Asc	=	18	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 = 2426 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 = 2446 mm
coaxial with and surrounding a bar
k1 = 1.25 A_b = 314.159265 mm^2
k2 = 2.2 a_b = 2426 mm (dist between bars)
factor = 56006.2413
Lsyt = k1 x k2 x fsy x Ab/C x sqrt(f'c) = factor/C

= 329.4485 mm

take fs/Fsy = M*/Phi Mu = 0.983436333

therefore

Lsy = 324 mm	≥	25k1db	625
say = 650 mm			

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 = 2794 mm

k1 = 1 A_b = 314.159265 mm^2

k2 = 2.2 a_b = 2774 mm (dist between bars)

factor = 44804.9931

Lsyt = 263.5588 mm   take fs/Fsy = M*/Phi Mu = 0.076468887

therefore

Lsy = 20 mm	≥	25k1db	500
say = 500 mm			

9 **SHEAR CHECK**

V* = 104.60 kN   Dist to centreline of bottom steel from soffit

critical location : d = 193 mm   dq = 93 mm

do = 207 mm

V* = 101 kN   beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 365.98 kN

phi Vumax = 3013.92 kN > V* No web crushing

phi Vumin = 592 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 1585.36585

Asv.min = 554.878049 mm^2

Asv.max = 11143 mm^2

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

theta = 29.78058 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 760.12 kN

Phi Vu = 532 kN > V* = 100.74 kN

hence

PROVIDE : 1 Y12 LIGS @ 250 centres

Other Support:

V* = 104.60 kN

Dist to centreline of bottom steel from soffit

critical location : d = 193 mm   dq = 93 mm

do = 207 mm

V* = 100.74 kN   beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 365.976 kN

phi Vumax = 3013.92 kN > V* No web crushing

phi Vumin = 592 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 1585.36585

Asv.min = 554.878049 mm^2

Asv.max = 11142.857 mm^2

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

theta = 30.06392 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 874.7136 kN

Phi Vu = 612.2995 kN > V* = 100.74 kN

hence

PROVIDE : 1 Y12 LIGS @ 250 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	197.00 kNm	Es	=	200000 MPa
Ml	=	0.00 kNm	Ec	=	31975.35051 MPa
Mo	=	126.1 kNm			
Ms	=	27.60 kNm			
Ig	=	5.85E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	3400 mm	A_{sc} =	5654.866776 mm^2
webwidth	b_w =	2600 mm	A_{st} =	5654.866776 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	117 mm
			d_{st} =	183 mm
			d =	193 mm
			D =	300 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)

1300 dn^2 + 65085.459 dn - 10303131.1) = 0

dn = 67.444858 mm

d = 183 mm

Icr = bdn3/3 +1/12(beff - bw)x t^3+(beff - bw)xt(dn-t/2)^2+nxAst (d-dn)^2
= 7.38E+08 mm4
Mcr = 0.6sqrtF'c x bD^2/6 = 147.994594 kNm

Im	=	[Icr + (Ig - Icr) (Mcr / Ms) ^3]
	=	7.89E+11 mm^4

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

b_w =	2600 mm	A_{st} =	5654.86678 mm^2
d_{sc} =	97 mm	A_{sc} =	5654.86678 mm^2
d =	193 mm		

Ig = 5.85E+09 mm^4

At right end (arbitrary)

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

1300 dn^2 + 65085.459 dn - 9708825.16) = 0

dn = 64.94 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 8.17E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 147.994594 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 2.95E+09 mm^4

At left end (arbitrary)

b_w =	2600 mm
d_{sc} =	97 mm
A_{st} =	5654.867 mm^2
A_{sc} =	5654.867 mm^2
d =	193 mm

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

1300 dn^2 + 65085.459 dn - 9708825.16) = 0

dn = 64.9391911 mm

d = 193 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 8.17E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 147.994594 kNm

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

Iav	=	[Im + (IL + Ir)/2]/2	=	3.95E+11 mm^4

Midspan Deflection :

Lef = 4000 mm

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

=	0.00104205 mm

Total Deflection :

Long term factor	=	0.2
Short term factor	=	0.5
w - long term	=	12 kN/m
w - short term	=	15 kN/m
Defln,s,sus	=	0.000833637 mm

At midspan : Ast = 5654.86678 mm^2

Asc = 5654.86678 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)
	=	0.00171 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

Chamber Wall Design

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

Design of a Continuous Beam :

1 Loads

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

2 Beam Section Properties

Beam Size 0.80121 9

bw	1000	mm	D =	300	mm
tff	0	mm	Cover =	75	mm
Actual length, L =	4700	mm	Overall depth of Sect. D =	300	mm
Effective Length,Lef =	4500	mm	Area, A =	3.00E+05	mm^2
Neutral Axis, yc =	150.00	mm	Moment of Inertia, Ix-x =	2.25E+09	mm^4
Neutral Axis, xc =	950.00	mm	Moment of Inertia, Iy-y =	2.50E+10	mm^4
Effective width, beff =	1900.00	mm	Gross Area, Ag =	3.00E+05	mm^2

3 Design Parameters

F'c =	40	MPa	D =	300	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 = 1900 mm clause 8.8.2

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

k_1	=	0.045	(RECT sections)	1
	=	0.02841283	(T and L sections)	2
what type of span?		5	interior span =	5
			end span =	6
k_2	=	0.0026042		
kcs	=	1.64	(because there will be comp steel at midspan=say .5Ast)	
Fdef	=	29.96		
defln/Lef	=	0.004	k_1/k_2 = 17.27977882	

d = 87 mm

take d = 201 mm Distance from d to extreme fibre of beam (do) = 99 mm

5 Forces from an Elastic Analysis by Space Gass :

Mmax -ve	=	34.00	kNm	(left support)
Mmax +ve	=	53.50	kNm	(mid-span)
M -ve	=	34.00	kNm	(right support)
Vmax	=	56.40	kN	
V (other end)	=	56.40	kN	
N*	=	21.00	kN	Axial Load

6 FLECTURAL STRENGTH STEEL

Determine - ve steel required : (At arbitrary left support) Bar Spacing 271 mm

Number of reinforcement layers 1 or 2 : 1

take do = 99 mm d = 201

Estimate Area of main steel		Ast =	539.3088757	mm^2	
8	Y 12	Ast =	904.7786842	mm^2	OUTSIDE
		p =	0.004501386		
8	Y 12	Asc =	904.7786842	mm^2	INSIDE
		pc =	0.004501386	<	pd = 0.025409
		p-pc =	0	pd=0.4*0.85*µ*f'c/fsy	

Check M* <= phiMu

ku = 0 dsc = 93 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)]

=	36	kNm	>	M* =	34.00 kNm	OK

Determine -ve steel at right support :

M -ve = 34.00 kNm

Number of reinforcement layers 1 or 2 : 1

take do = 99 mm dst = 201 mm

Estimate Area of main steel Ast = 539.3088757 mm^2

8 Y 12 Ast = 904.7786842 mm^2 OUTSIDE

p = 0.004501386

8 Y 12 Asc = 904.7786842 mm^2 INSIDE

pc = 0.004501386 < pd = 0.025409

p-pc = 0

Check M* <= phiMu

ku = 0 dsc = 93 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)]

=	36	kNm	>	M* =	34.00 kNm	OK

FOR DUCTILITY CHECK AT BOTH SUPPORTS USE EMPHI

Tensile Steel at Midspan :

M* = 53.5 kNm

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

" " two " " code = 2 bar/s and soffit = 105 mm

what is the code ? 2 d = 195 mm

Ast = M*/(Phi x 0.85 x d x Fsy) = 874.729713 mm^2

8 Y 12 Ast = 904.7786842 mm^2 OUTSIDE

8 Y 12 Asc = 904.7786842 mm^2 INSIDE

Assume N - A in flange

Effective width : beff = 1900 mm

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

= 0.03844415

hence dn = 7 < 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)]

=	64 kNm	>	M*+ve =	53.50 kNm	OK

Summary of reinforcement requirement :

left support :	Ast	=	8	Y 12
	Asc	=	8	Y 12
Right support :	Ast	=	8	Y 12
	Asc	=	8	Y 12
midspan	Ast	=	8	Y 12
	Asc	=	8	Y 12

7 STEEL CURTAILMENT REQUIREMENT CHECK

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

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

For -ve steel = 2 bars

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

= 0.3 x le = 1350 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 = 1350 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 = 450 mm

8 STRESS DEVELOPMENT

For Top Steel :

cover : c = 75 mm

d_b = 12 mm # of lines of bars = 1

a_b = 826 mm distance between bars

If a_b > 2c C = 2c + d_b = 162 mm

C is the outside diameter of a concrete annulus

if a_b =< 2c C = a_b + d_b = 838 mm

coaxial with and surrounding a bar

k1 = 1.25 A_b = 113.0973355 mm^2

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

factor = 20162.24689

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

= 124.4583 mm

take fs/Fsy = M*/Phi Mu = 0.942945158

therefore

Lsy =	117 mm
say =	750 mm

≥ 25k1db 375

For Bottom Steel :

Cover, c = 75 mm d_b = 12 mm

# of lines of bars = 1

If ab > 2c C = 2c + d_b = 162 mm

if ab < 2c C = a_b + d_b = 1186 mm

k1 = 1 A_b = 113.0973355 mm^2

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

factor = 16129.79751

Lsyt = 99.56665 mm take fs/Fsy = M*/Phi Mu = 0.834050965

therefore

Lsy =	83 mm
say =	600 mm

≥ 25k1db 300

9 SHEAR CHECK

V* = 56.40 kN Dist to centreline of bottom steel from soffit

critical location : d = 201 mm dq = 93 mm

do = 207 mm

V* = 52 kN beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 105.11 kN

phi Vumax = 1159.2 kN > V* No web crushing

phi Vumin = 192 kN > V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 609.756098

Asv.min = 213.414634 mm^2

Asv.max = 4436 mm^2

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

theta = 30.66285 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 379.19 kN

Phi Vu = 265 kN > V* = 52.38 kN

hence

PROVIDE :	1 Y12 LIGS	@	250	centres

Other Support :

V* = 56.40 kN

Dist to centreline of bottom steel from soffit

critical location : d = 201 mm   dq = 93 mm

do = 207 mm

V* = 52.38 kN   beta 1 = 1.2965 (but not less than 1.1)

hence = 1.2965

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

= 105.1084 kN

phi Vumax = 1159.2 kN > V* No web crushing

phi Vumin = 192 kN < V* OK

If shear reinforcement reqired then proceed as follows :

choose stirrup spacing s = 250 mm   Fsyf = 410 MPa

Determine theta :

bv x s/Fsy f = 609.756098

Asv.min = 213.414634 mm^2

Asv.max = 4435.74044 mm^2

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

theta = 31.37336 degrees

Determine Phi Vu :

Vu = Vuc + Vus

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

= 484.1985 kN

Phi Vu = 338.9389 kN > V* = 52.38 kN

hence

PROVIDE :	1 Y12 LIGS	@	250 centres

11 SERVICEABILITY CHECK

From an Elastic Linear Analysis :

Mr	=	34.00 kNm	Es	=	200000 MPa
Ml	=	34.00 kNm	Ec	=	31975.35051 MPa
Mo	=	87.5 kNm			
Ms	=	53.50 kNm			
Ig	=	2.25E+09 mm^4			

Calculate Icr, Mcr, and Ief at midspan :

Eff. Flange width	b_{eff} =	1900 mm	A_{sc} =	904.7786842 mm^2
webwidth	b_w =	1000 mm	A_{st} =	904.7786842 mm^2
Flange thickness	t_{ff} =	0 mm	d_{sc} =	105 mm
			d_{st} =	195 mm
			d =	201 mm
			D =	300 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 + 10413.6734 dn - 1636721.412) = 0

dn = 47.740324 mm

d = 195 mm

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

= 1.59E+08 mm4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

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

= 2.68E+09 mm^4

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

b_w =	1000 mm	A_{st} =	904.7786842 mm^2
d_{sc} =	93 mm	A_{sc} =	904.7786842 mm^2
d =	201 mm		

Ig = 2.25E+09 mm^4

At right end (arbitrary)

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

500 dn^2 + 10413.6734 dn - 1579668.044) = 0

dn = 46.75 mm

d = 201 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.69E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 56.9209979 kNm

Iref = [Icr + (Ig - Icr) (Mcr / Mr)^3 = 9.93E+09 mm^4

At left end (arbitrary)

b_w = 1000 mm

d_{sc} = 93 mm

A_{st} = 904.77868 mm^2

A_{sc} = 904.77868 mm^2

d = 201 mm

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

500 dn^2 + 10413.6734 dn - 1579668.044) = 0

dn = 46.750832 mm

d = 201 mm

Icr = bdn3/3 + n x Asc (d - dn)^2 = 1.69E+08 mm^4

Mcr = 0.6sqrtF'c x bD^2/6 = 56.9209979 kNm

Ilef = [Icr + (Ig - Icr) (Mcr / Ml)^3 = 9.93E+09 mm^4

Iav = [Im + (IL + Ir)/2]/2 = 6.31E+09 mm^4

Midspan Deflection :

Lef = 4500 mm

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

= 0.48854314 mm

Total Deflection :

Long term factor = 0.2

Short term factor = 0.5

w - long term = 12 kN/m

w - short term = 15 kN/m

Defln,s,sus = 0.390834515 mm

At midspan : Ast = 904.778684 mm^2

Asc = 904.778684 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)

= 0.80121 mm

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

ALLOWABLE DEFLN = Lef / 500 = 9 mm

> 0.9 x Total Defln

SUMMARY

A. Base Slab

300 thk RC Slab

Reinforcement:	Y16 bars @ 250 cts	Top & Bottom Each Way
T & S	Y16 bars @ 250 cts	Top & Bottom Each Way

B. Drainage Tank Wall

300 thk RC Wall

Reinforcements:	Vertical:	Y20 - 200 cts	Outside
		Y20 - 200 cts	Inside
	Horizontal	Y12 - 300 cts	

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ANALYSIS STATUS REPORT

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

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

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

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

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

MATERIAL PROPERTIES (kPa,Kg/m^3)

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

SELF WEIGHT (kN/kg)

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

COMBINATION LOAD CASES

Load case 6: D+1.3L+E

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

Load case 7: 0.9D+E

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

Lo' case 8: 1.4D+1.7L+1.7Q

.400 * Load case 1: D
1.700 * Load case 2: L
1.700 * Load case 4: Q

Load case 9: 0.9D+1.7Q

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

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

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

Load case 11: 0.9D+1.4F

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

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

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

LOAD CASE TITLES

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

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
1	8	70.637*	64.685	0.000	0.000	0.000	-100.262
1	1	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	70.637	104.585*	0.000	0.000	0.000	-197.282
3	12	70.637	-104.585#	0.000	0.000	0.000	-197.282
3	12	0.000	0.000	0.000	0.000	0.000	0.000*
1	12	70.637	104.585	0.000	0.000	0.000	-197.282#

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

Envelope = All Load Cases
and All Nodes

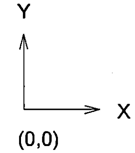
Node	Load Case	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
3	12	104.585*	84.092	0.000	0.000	0.000	-200.870
1	12	-104.585#	84.092	0.000	0.000	0.000	200.870
1	8	-64.685	84.092*	0.000	0.000	0.000	103.850
1	3	-39.900	0.000#	0.000	0.000	0.000	97.020
1	12	-104.585	84.092	0.000	0.000	0.000	200.870*
3	12	104.585	84.092	0.000	0.000	0.000	-200.870#

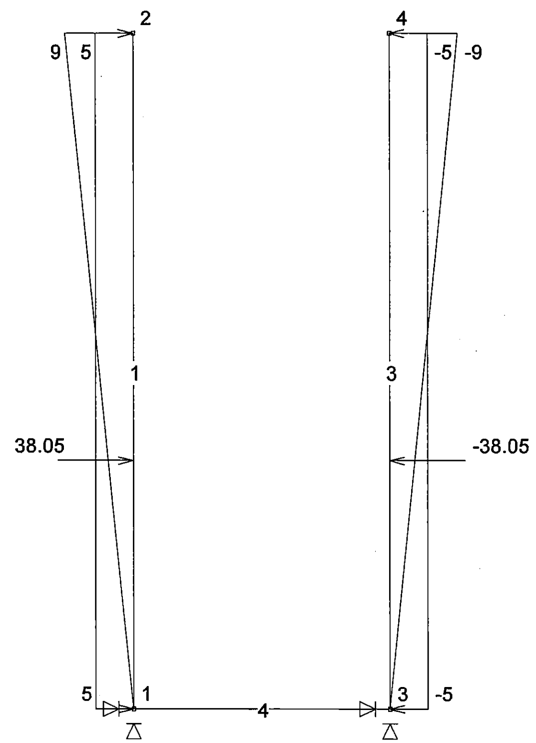
INTERMEDIATE FORCES AND MOMENTS (m,kN,kNm)
(*=Maximum, #=Minimum)

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	70.637*	64.685	0.000	0.000	0.000	-100.262
1	7	0.000#	0.000	0.000	0.000	0.000	0.000
1	12	70.637	104.585*	0.000	0.000	0.000	-197.282
3	12	70.637	-104.585#	0.000	0.000	0.000	-197.282
4	8	0.000	0.000	0.000	0.000	0.000	1.794*
1	12	70.637	104.585	0.000	0.000	0.000	-197.282#

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





Sections:
1 500 x 1000

Drainage Tank Model with All Load Cases

Job: DRTANK, Designer: FYP, Units: m,kN,MPa, Scale: 1:47, Axes: XY
Load: 1 Disp: None Moment: None Shear: None Axial: None
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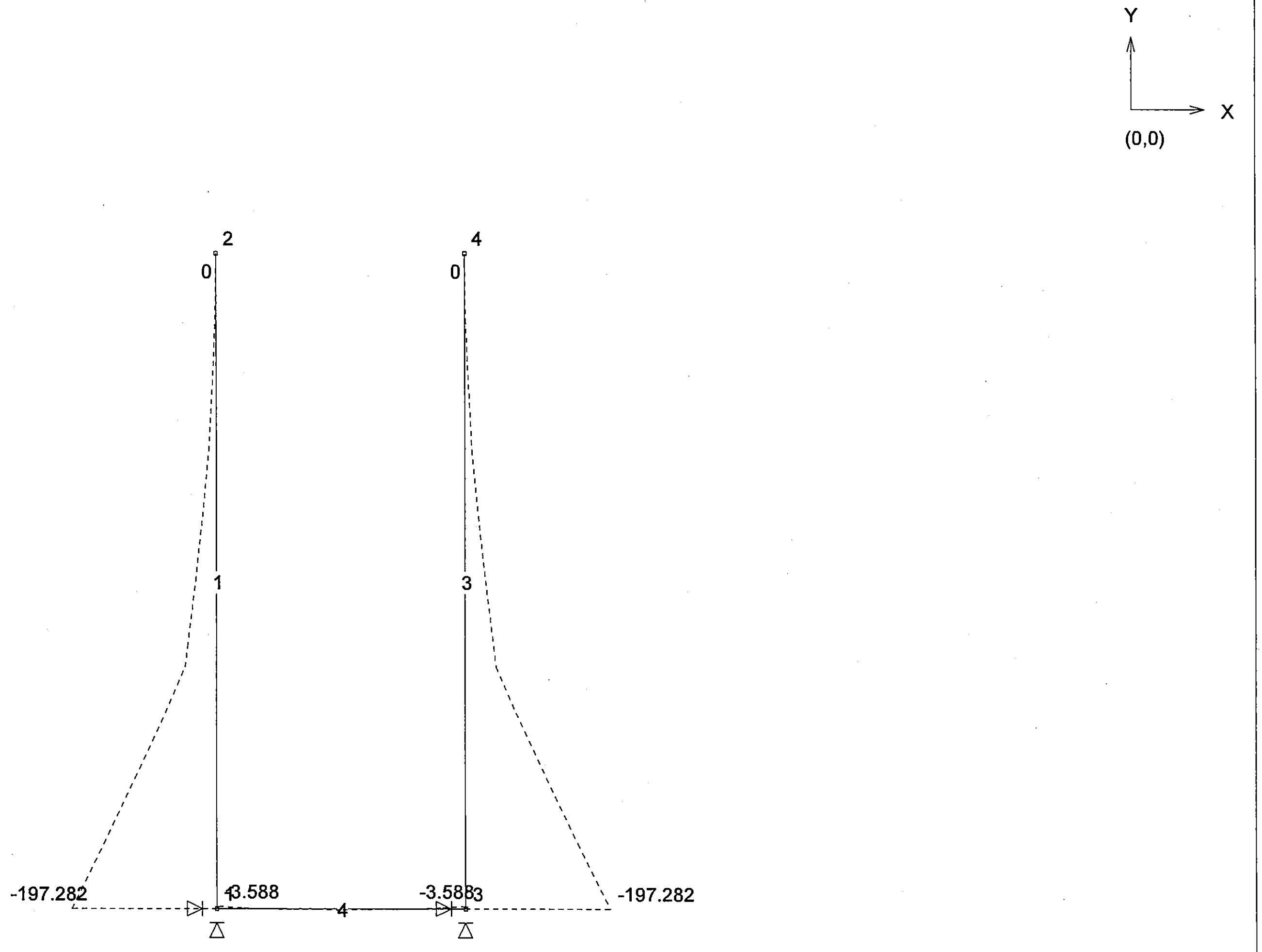
12 1.4D+1.7L+E+1.7Q



Sections:
1 500 x 1000

Bending Moment Diagram

Job: DRTANK, Designer: FYP, Units: m,kN,MPa, Scale: 1:47, Axes: XY
Load: 1 Disp: None Moment: 10 Shear: None Axial: None
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