
APPENDIX A: TRANSPORTATION – SIDRA OUTPUTS

The SIDRA output

Prunus Intersection

INTERSECTION SUMMARY

 Site: 1 [Prunus T junction -final]

Intersection Performance - Hourly Values			
Performance Measure	Vehicles	Pedestrians	Persons
Travel Speed (Average)	49.3 km/h	3.2 km/h	49.1 km/h
Travel Distance (Total)	2284.0 veh-km/h	1.0 ped-km/h	2741.8 pers-km/h
Travel Time (Total)	46.3 veh-h/h	0.3 ped-h/h	55.9 pers-h/h
Demand Flows (Total)	2416 veh/h	26 ped/h	2900 pers/h
Percent Heavy Vehicles (Demand)	0.0 %		
Degree of Saturation	0.582	0.015	
Practical Spare Capacity	54.6 %		
Effective Intersection Capacity	4150 veh/h		
Control Delay (Total)	8.00 veh-h/h	0.11 ped-h/h	9.70 pers-h/h
Control Delay (Average)	11.9 sec	14.5 sec	12.0 sec
Control Delay (Worst Lane)	24.2 sec		
Control Delay (Worst Movement)	24.2 sec	14.5 sec	24.2 sec
Geometric Delay (Average)	1.4 sec		
Stop-Line Delay (Average)	10.5 sec		
Idling Time (Average)	6.6 sec		
Intersection Level of Service (LOS)	LOS B	LOS B	
95% Back of Queue - Vehicles (Worst Lane)	6.0 veh		
95% Back of Queue - Distance (Worst Lane)	41.7 m		
Queue Storage Ratio (Worst Lane)	0.05		
Total Effective Stops	1663 veh/h	22 ped/h	2017 pers/h
Effective Stop Rate	0.69 per veh	0.85 per ped	0.70 per pers
Proportion Queued	0.80	0.85	0.81
Performance Index	67.5	0.5	67.9
Cost (Total)	1314.80 \$/h	8.22 \$/h	1323.01 \$/h
Fuel Consumption (Total)	187.6 L/h		
Carbon Dioxide (Total)	440.7 kg/h		
Hydrocarbons (Total)	0.038 kg/h		
Carbon Monoxide (Total)	0.523 kg/h		
NOx (Total)	0.133 kg/h		

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Intersection LOS value for Vehicles is based on average delay for all vehicle movements.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values			
Performance Measure	Vehicles	Pedestrians	Persons
Demand Flows (Total)	1,159,832 veh/y	12,632 ped/y	1,391,798 pers/y
Delay	3,838 veh-h/y	51 ped-h/y	4,656 pers-h/y
Effective Stops	798,025 veh/y	10,749 ped/y	968,378 pers/y
Travel Distance	1,096,298 veh-km/y	495 ped-km/y	1,316,053 pers-km/y
Travel Time	22,219 veh-h/y	157 ped-h/y	26,819 pers-h/y
Cost	631,101 \$/y	3,945 \$/y	635,047 \$/y
Fuel Consumption	90,025 L/y		

Carbon Dioxide	211,558 kg/y
Hydrocarbons	18 kg/y
Carbon Monoxide	251 kg/y
NOx	64 kg/y

MOVEMENT SUMMARY

Site: 1 [Purnus T junction -final]

Movement Performance - Vehicles											
Mov ID	OD Mov	Demand Total veh/h	Flows HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate per veh	Average Speed km/h
South: Diagnol Road											
11	T1	989	0.0	0.461	6.0	LOS A	6.0	41.7	0.64	0.56	54.7
12	R2	303	0.0	0.544	24.2	LOS C	3.0	21.1	0.97	0.80	37.8
Approach		1293	0.0	0.544	10.3	LOS B	6.0	41.7	0.72	0.62	50.6
East: purnus Street											
1	L2	232	0.0	0.208	13.7	LOS B	1.4	10.1	0.78	0.73	44.1
3	R2	8	0.0	0.028	22.0	LOS C	0.1	1.0	0.88	0.65	38.9
Approach		239	0.0	0.208	13.9	LOS B	1.4	10.1	0.78	0.73	43.9
North: Diagnol Road											
4	L2	58	0.0	0.582	15.9	LOS B	4.4	30.8	0.91	0.78	46.8
5	T1	826	0.0	0.582	13.6	LOS B	5.2	36.3	0.92	0.78	48.9
Approach		884	0.0	0.582	13.8	LOS B	5.2	36.3	0.92	0.78	48.8
All Vehicles		2416	0.0	0.582	11.9	LOS B	6.0	41.7	0.80	0.69	49.3

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Vehicle movement LOS values are based on average delay per movement.

Intersection and Approach LOS values are based on average delay for all vehicle movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians									
Mov ID	Description	Demand Flow ped/h	Average Delay sec	Level of Service	Average Back of Queue Pedestrian ped	Distance m	Prop. Queued	Effective Stop Rate per ped	
P1	East Full Crossing	26	14.5	LOS B	0.0	0.0	0.85	0.85	
All Pedestrians		26	14.5	LOS B			0.85	0.85	

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)

Pedestrian movement LOS values are based on average delay per pedestrian movement.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

LANE SUMMARY

Site: 1 [Purnus T junction -final]

Lane Use and Performance

	Demand Flows Total veh/h	HV % veh/h	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Average Delay sec	Level of Service	95% Back of Queue Veh	Queue Dist m	Lane Config	Lane Length m	Cap. Adj. %	Prob. Block. %
South: Diagnol Road													
Lane 1	495	0.0	1073	0.461	85 ⁵	6.0	LOS A	6.0	41.7	Full	500	0.0	0.0
Lane 2	495	0.0	1073	0.461	85 ⁵	6.0	LOS A	6.0	41.7	Full	500	0.0	0.0
Lane 3	152	0.0	279	0.544	100	24.2	LOS C	3.0	21.1	Full	500	0.0	0.0
Lane 4	152	0.0	279	0.544	100	24.2	LOS C	3.0	21.1	Short	100	0.0	NA
Approach	1293	0.0		0.544		10.3	LOS B	6.0	41.7				
East: purnus Street													
Lane 1	116	0.0	557	0.208	100	13.7	LOS B	1.4	10.1	Full	200	0.0	0.0
Lane 2	116	0.0	557	0.208	100	13.7	LOS B	1.4	10.1	Full	200	0.0	0.0
Lane 3	8	0.0	281	0.028	100	22.0	LOS C	0.1	1.0	Short	70	0.0	NA
Approach	239	0.0		0.208		13.9	LOS B	1.4	10.1				
North: Diagnol Road													
Lane 1	317	0.0	544	0.582	100	11.3	LOS B	4.4	30.8	Full	500	0.0	0.0
Lane 2	284	0.0	488	0.582	100	15.1	LOS B	5.2	36.3	Full	500	0.0	0.0
Lane 3	284	0.0	488	0.582	100	15.1	LOS B	5.2	36.3	Full	500	0.0	0.0
Approach	884	0.0		0.582		13.8	LOS B	5.2	36.3				
Intersection	2416	0.0		0.582		11.9	LOS B	6.0	41.7				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Lane LOS values are based on average delay per lane.

Intersection and Approach LOS values are based on average delay for all lanes.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

LANE FLOWS



Site: 1 [Purnus T junction -final]

Approach Lane Flows (veh/h)									
South: Diagnol Road									
Mov.	T1	R2	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From S To Exit:	N	E							
Lane 1	495	-	495	0.0	1073	0.461	85 ⁵	NA	NA
Lane 2	495	-	495	0.0	1073	0.461	85 ⁵	NA	NA
Lane 3	-	152	152	0.0	279	0.544	100	NA	NA
Lane 4	-	152	152	0.0	279	0.544	100	0.0	3
Approach	989	303	1293	0.0		0.544			
East: purnus Street									
Mov.	L2	R2	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From E To Exit:	S	N							
Lane 1	116	-	116	0.0	557	0.208	100	NA	NA
Lane 2	116	-	116	0.0	557	0.208	100	NA	NA

Lane 3	-	8	8	0.0	281	0.028	100	0.0	2
Approach	232	8	239	0.0		0.208			
North: Diagnol Road									
Mov.	L2	T1	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From N To Exit:	E	S							
Lane 1	58	259	317	0.0	544	0.582	100	NA	NA
Lane 2	-	284	284	0.0	488	0.582	100	NA	NA
Lane 3	-	284	284	0.0	488	0.582	100	NA	NA
Approach	58	826	884	0.0		0.582			
	Total	%HV	Deg. Satn (v/c)						
Intersection	2416	0.0	0.582						

Lane flow rates given in this report are based on the arrival flow rates subject to upstream capacity constraint where applicable.

Swimming centre Intersection

INTERSECTION SUMMARY



Site: 01 [Seagull Signals L - Final]

Intersection Performance - Hourly Values			
Performance Measure	Vehicles	Pedestrians	Persons
Travel Speed (Average)	42.8 km/h	2.7 km/h	42.2 km/h
Travel Distance (Total)	1769.2 veh-km/h	2.3 ped-km/h	2125.3 pers-km/h
Travel Time (Total)	41.3 veh-h/h	0.8 ped-h/h	50.4 pers-h/h
Demand Flows (Total)	2953 veh/h	53 ped/h	3543 pers/h
Percent Heavy Vehicles (Demand)	0.0 %		
Degree of Saturation	0.658	0.022	
Practical Spare Capacity	36.8 %		
Effective Intersection Capacity	4487 veh/h		
Control Delay (Total)	10.41 veh-h/h	0.36 ped-h/h	12.85 pers-h/h
Control Delay (Average)	12.7 sec	24.3 sec	13.1 sec
Control Delay (Worst Lane)	30.8 sec		
Control Delay (Worst Movement)	31.6 sec	24.3 sec	31.6 sec
Geometric Delay (Average)	2.8 sec		

Stop-Line Delay (Average)	9.9 sec		
Idling Time (Average)	6.8 sec		
Intersection Level of Service (LOS)	LOS B	LOS C	
95% Back of Queue - Vehicles (Worst Lane)	6.3 veh		
95% Back of Queue - Distance (Worst Lane)	43.8 m		
Queue Storage Ratio (Worst Lane)	0.25		
Total Effective Stops	1847 veh/h	47 ped/h	2264 pers/h
Effective Stop Rate	0.63 per veh	0.90 per ped	0.64 per pers
Proportion Queued	0.64	0.90	0.65
Performance Index	64.7	1.1	65.8
Cost (Total)	1239.72 \$/h	21.16 \$/h	1260.88 \$/h
Fuel Consumption (Total)	172.9 L/h		
Carbon Dioxide (Total)	406.3 kg/h		
Hydrocarbons (Total)	0.037 kg/h		
Carbon Monoxide (Total)	0.468 kg/h		
NOx (Total)	0.138 kg/h		

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Intersection LOS value for Vehicles is based on average delay for all vehicle movements.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values			
Performance Measure	Vehicles	Pedestrians	Persons
Demand Flows (Total)	1,417,263 veh/y	25,263 ped/y	1,700,716 pers/y
Delay	4,996 veh-h/y	171 ped-h/y	6,166 pers-h/y
Effective Stops	886,681 veh/y	22,762 ped/y	1,086,779 pers/y
Travel Distance	849,218 veh-km/y	1,088 ped-km/y	1,020,149 pers-km/y
Travel Time	19,821 veh-h/y	403 ped-h/y	24,189 pers-h/y
Cost	595,065 \$/y	10,158 \$/y	605,223 \$/y
Fuel Consumption	82,997 L/y		
Carbon Dioxide	195,043 kg/y		
Hydrocarbons	18 kg/y		
Carbon Monoxide	225 kg/y		
NOx	66 kg/y		

MOVEMENT SUMMARY

 Site: 01 [Seagull Signals L - Final]

Movement Performance - Vehicles											
Mov ID	OD Mov	Demand Flows Total	Deg. Satn	Average Delay	Level of Service	95% Back of Queue Vehicles	95% Back of Queue Distance	Prop. Queued	Effective Stop Rate	Average Speed	
		veh/h	%	sec		veh	m		per veh	km/h	
South: Diagnol Road											
1b	L3	89	0.0	0.614	19.4	LOS B	4.3	29.8	0.95	0.80	13.2
5	T1	563	0.0	0.614	22.7	LOS C	5.8	40.8	0.97	0.81	36.8
Approach		653	0.0	0.614	22.2	LOS C	5.8	40.8	0.97	0.81	33.5
North: Diagnol Road											
11	T1	542	0.0	0.138	0.0	LOS A	0.0	0.0	0.00	0.00	60.0
9a	R1	847	0.0	0.658	14.3	LOS B	6.3	43.8	0.91	0.82	42.2
Approach		1389	0.0	0.658	8.7	LOS A	6.3	43.8	0.55	0.50	47.7
SouthWest: Morpett Road											
30a	L1	858	0.0	0.335	10.7	LOS B	5.6	39.1	0.52	0.68	45.6
32b	R3	53	0.0	0.335	31.6	LOS C	2.5	17.2	0.92	0.76	17.0

Approach	911	0.0	0.335	11.9	LOS B	5.6	39.1	0.54	0.69	44.1
All Vehicles	2953	0.0	0.658	12.7	LOS B	6.3	43.8	0.64	0.63	42.8

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Vehicle movement LOS values are based on average delay per movement.

Intersection and Approach LOS values are based on average delay for all vehicle movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

Movement Performance - Pedestrians									
Mov ID	Description	Demand Flow ped/h	Average Delay sec	Level of Service	Average Back of Queue Pedestrian ped	Back of Queue Distance m	Prop. Queued	Effective Stop Rate per ped	
P2	South Full Crossing	26	24.3	LOS C	0.0	0.0	0.90	0.90	
P8	SouthWest Full Crossing	26	24.3	LOS C	0.0	0.0	0.90	0.90	
All Pedestrians		53	24.3	LOS C			0.90	0.90	

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)

Pedestrian movement LOS values are based on average delay per pedestrian movement.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

LANE SUMMARY

 Site: 01 [Seagull Signals L - Final]

Lane Use and Performance													
	Demand	Flows	Cap.	Deg.	Lane	Average	Level of	95% Back of Queue	Lane	Lane	Cap.	Prob.	
	Total	HV		Satn	Util.	Delay	Service	Veh	Dist	Config	Length	Adj. Block.	
	veh/h	% veh/h	v/c	%	%	sec			m		m	%	%
South: Diagnol Road													
Lane 1	250	0.0	407	0.614	100	15.1	LOS B	4.3	29.8	Full	100	0.0	0.0
Lane 2	201	0.0	328	0.614	100	26.7	LOS C	5.8	40.8	Full	100	0.0	0.0
Lane 3	201	0.0	328	0.614	100	26.7	LOS C	5.8	40.8	Full	100	0.0	0.0
Approach	653	0.0		0.614		22.2	LOS C	5.8	40.8				
North: Diagnol Road													
Lane 1	271	0.0	1970	0.138	21 ⁵	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 2	271	0.0	1970	0.138	21 ⁵	0.0	LOS A	0.0	0.0	Full	500	0.0	0.0
Lane 3	424	0.0	644	0.658	100	14.3	LOS B	6.3	43.8	Full	500	0.0	0.0
Lane 4	424	0.0	644	0.658	100	14.3	LOS B	6.3	43.8	Short	80	0.0	NA
Approach	1389	0.0		0.658		8.7	LOS A	6.3	43.8				
SouthWest: Morpett Road													
Lane 1	410	0.0	1223	0.335	100	9.9	LOS A	5.6	39.1	Full	100	0.0	0.0
Lane 2	410	0.0	1223	0.335	100	9.9	LOS A	5.6	39.1	Full	100	0.0	0.0
Lane 3	90	0.0	268	0.335	100	30.8	LOS C	2.5	17.2	Short	48	0.0	NA
Approach	911	0.0		0.335		11.9	LOS B	5.6	39.1				
Intersection	2953	0.0		0.658		12.7	LOS B	6.3	43.8				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Lane LOS values are based on average delay per lane.

Intersection and Approach LOS values are based on average delay for all lanes.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

LANE FLOWS

 Site: 01 [Seagull Signals L - Final]

Approach Lane Flows (veh/h)									
South: Diagnol Road									
Mov.	L3	T1	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From S To Exit:	SW	N							
Lane 1	89	160	250	0.0	407	0.614	100	NA	NA
Lane 2	-	201	201	0.0	328	0.614	100	NA	NA
Lane 3	-	201	201	0.0	328	0.614	100	NA	NA
Approach	89	563	653	0.0		0.614			
North: Diagnol Road									
Mov.	T1	R1	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From N To Exit:	S	SW							
Lane 1	271	-	271	0.0	1970	0.138	21 5	NA	NA
Lane 2	271	-	271	0.0	1970	0.138	21 5	NA	NA
Lane 3	-	424	424	0.0	644	0.658	100	NA	NA
Lane 4	-	424	424	0.0	644	0.658	100	0.0	3
Approach	542	847	1389	0.0		0.658			
SouthWest: Morpett Road									
Mov.	L1	R3	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From SW To Exit:	N	S							
Lane 1	410	-	410	0.0	1223	0.335	100	NA	NA
Lane 2	410	-	410	0.0	1223	0.335	100	NA	NA
Lane 3	37	53	90	0.0	268	0.335	100	0.0	2
Approach	858	53	911	0.0		0.335			
Total %HV Deg.Satn (v/c)									
Intersection	2953	0.0		0.658					

Lane flow rates given in this report are based on the arrival flow rates subject to upstream capacity constraint where applicable.

Cole's intersection INTERSECTION SUMMARY

 Site: 02 [Coles intersection - Final]

Intersection Performance - Hourly Values			
Performance Measure	Vehicles	Pedestrians	Persons
Travel Speed (Average)	52.1 km/h	3.3 km/h	51.6 km/h
Travel Distance (Total)	2853.7 veh-km/h	2.4 ped-km/h	3426.8 pers-km/h
Travel Time (Total)	54.8 veh-h/h	0.7 ped-h/h	66.5 pers-h/h
Demand Flows (Total)	2884 veh/h	53 ped/h	3461 pers/h
Percent Heavy Vehicles (Demand)	0.0 %		
Degree of Saturation	0.541	0.015	
Practical Spare Capacity	66.3 %		
Effective Intersection Capacity	5328 veh/h		
Control Delay (Total)	7.04 veh-h/h	0.21 ped-h/h	8.66 pers-h/h
Control Delay (Average)	8.8 sec	14.5 sec	9.0 sec
Control Delay (Worst Lane)	23.2 sec		
Control Delay (Worst Movement)	23.2 sec	14.5 sec	23.2 sec
Geometric Delay (Average)	0.9 sec		
Stop-Line Delay (Average)	7.9 sec		
Idling Time (Average)	4.6 sec		
Intersection Level of Service (LOS)	LOS A	LOS B	
95% Back of Queue - Vehicles (Worst Lane)	6.1 veh		
95% Back of Queue - Distance (Worst Lane)	42.6 m		
Queue Storage Ratio (Worst Lane)	0.05		
Total Effective Stops	1695 veh/h	45 ped/h	2078 pers/h
Effective Stop Rate	0.59 per veh	0.85 per ped	0.60 per pers
Proportion Queued	0.66	0.85	0.68
Performance Index	75.5	1.0	76.5
Cost (Total)	1456.59 \$/h	18.01 \$/h	1474.60 \$/h
Fuel Consumption (Total)	218.8 L/h		
Carbon Dioxide (Total)	514.1 kg/h		
Hydrocarbons (Total)	0.043 kg/h		
Carbon Monoxide (Total)	0.622 kg/h		
NOx (Total)	0.147 kg/h		

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Intersection LOS value for Vehicles is based on average delay for all vehicle movements.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Intersection Performance - Annual Values			
Performance Measure	Vehicles	Pedestrians	Persons
Demand Flows (Total)	1,384,421 veh/y	25,263 ped/y	1,661,306 pers/y
Delay	3,379 veh-h/y	102 ped-h/y	4,157 pers-h/y
Effective Stops	813,387 veh/y	21,497 ped/y	997,561 pers/y
Travel Distance	1,369,795 veh-km/y	1,131 ped-km/y	1,644,885 pers-km/y
Travel Time	26,301 veh-h/y	343 ped-h/y	31,904 pers-h/y
Cost	699,164 \$/y	8,646 \$/y	707,810 \$/y
Fuel Consumption	105,014 L/y		

Carbon Dioxide	246,783 kg/y
Hydrocarbons	21 kg/y
Carbon Monoxide	298 kg/y
NOx	71 kg/y

MOVEMENT SUMMARY

 Site: 02 [Coles intersection - Final]

Movement Performance - Vehicles											
Mov ID	OD Mov	Demand Flows Total veh/h	Deg. Satn HV % v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	Distance m	Prop. Queued	Effective Stop Rate per veh	Average Speed km/h	
South: Diagonal Rd											
11	T1	1324	0.0 0.345	3.4	LOS A	3.9	27.6	0.48	0.42	56.8	
12	R2	76	0.0 0.276	23.2	LOS C ¹¹	1.4	10.0	0.92	0.74	40.3	
Approach		1400	0.0 0.345	4.5	LOS A	3.9	27.6	0.51	0.44	55.8	
East: Morphett Rd											
1	L2	347	0.0 0.176	15.6	LOS B	1.6	11.3	0.73	0.73	44.4	
Approach		347	0.0 0.176	15.6	LOS B	1.6	11.3	0.73	0.73	44.4	
North: Diagonal Rd											
4	L2	21	0.0 0.541	17.8	LOS B	5.8	40.7	0.83	0.75	47.2	
5	T1	1116	0.0 0.541	11.9	LOS B	6.1	42.6	0.84	0.73	50.2	
Approach		1137	0.0 0.541	12.0	LOS B	6.1	42.6	0.84	0.73	50.1	
All Vehicles		2884	0.0 0.541	8.8	LOS A	6.1	42.6	0.66	0.59	52.1	

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Vehicle movement LOS values are based on average delay per movement.

Intersection and Approach LOS values are based on average delay for all vehicle movements.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

¹¹ Level of Service is worse than the Level of Service Target specified in the Parameter Settings dialog.

Movement Performance - Pedestrians								
Mov ID	Description	Demand Flow ped/h	Average Delay sec	Level of Service	Average Back of Queue Pedestrian ped	Distance m	Prop. Queued	Effective Stop Rate per ped
P4	South Full Crossing	26	14.5	LOS B	0.0	0.0	0.85	0.85
P1	East Full Crossing	26	14.5	LOS B	0.0	0.0	0.85	0.85
All Pedestrians		53	14.5	LOS B			0.85	0.85

Level of Service (LOS) Method: SIDRA Pedestrian LOS Method (Based on Average Delay)

Pedestrian movement LOS values are based on average delay per pedestrian movement.

Intersection LOS value for Pedestrians is based on average delay for all pedestrian movements.

LANE SUMMARY

 Site: 02 [Coles intersection - Final]

Lane Use and Performance										
	Demand Flows Total	Cap. HV	Deg. Satn v/c	Lane Util.	Average Delay sec	Level of Service	95% Back of Queue Veh	Distance Dist	Lane Config Length	Lane Cap. Prob. Adj. Block.

	veh/h	% veh/h	v/c	%	sec			m	m	%	%		
South: Diagonal Rd													
Lane 1	441	0.0	1281	0.345	100	3.4	LOS A	3.9	27.6	Full	500	0.0	0.0
Lane 2	441	0.0	1281	0.345	100	3.4	LOS A	3.9	27.6	Full	500	0.0	0.0
Lane 3	441	0.0	1281	0.345	100	3.4	LOS A	3.9	27.6	Full	500	0.0	0.0
Lane 4	76	0.0	274	0.276	100	23.2	LOS C ¹¹	1.4	10.0	Short	60	0.0	NA
Approach	1400	0.0	0.345			4.5	LOS A	3.9	27.6				
East: Morphett Rd													
Lane 1	116	0.0	657	0.176	100	15.6	LOS B	1.6	11.3	Full	300	0.0	0.0
Lane 2	116	0.0	657	0.176	100	15.6	LOS B	1.6	11.3	Full	300	0.0	0.0
Lane 3	116	0.0	657	0.176	100	15.6	LOS B	1.6	11.3	Full	300	0.0	0.0
Approach	347	0.0	0.176			15.6	LOS B	1.6	11.3				
North: Diagonal Rd													
Lane 1	390	0.0	721	0.541	100	12.5	LOS B	5.8	40.7	Full	500	0.0	0.0
Lane 2	373	0.0	690	0.541	100	11.7	LOS B	6.1	42.6	Full	500	0.0	0.0
Lane 3	373	0.0	690	0.541	100	11.7	LOS B	6.1	42.6	Full	500	0.0	0.0
Approach	1137	0.0	0.541			12.0	LOS B	6.1	42.6				
Intersection	2884	0.0	0.541			8.8	LOS A	6.1	42.6				

Site Level of Service (LOS) Method: Delay (SIDRA). Site LOS Method is specified in the Parameter Settings dialog (Site tab).

Lane LOS values are based on average delay per lane.

Intersection and Approach LOS values are based on average delay for all lanes.

SIDRA Standard Delay Model is used. Control Delay includes Geometric Delay.

Gap-Acceptance Capacity: SIDRA Standard (Akçelik M3D).

HV (%) values are calculated for All Movement Classes of All Heavy Vehicle Model Designation.

LANE FLOWS



Site: 02 [Coles intersection - Final]

Approach Lane Flows (veh/h)									
South: Diagonal Rd									
Mov.	T1	R2	Total	%HV	Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From S									
To Exit:	N	E							
Lane 1	441	-	441	0.0	1281	0.345	100	NA	NA
Lane 2	441	-	441	0.0	1281	0.345	100	NA	NA
Lane 3	441	-	441	0.0	1281	0.345	100	NA	NA
Lane 4	-	76	76	0.0	274	0.276	100	0.0	3
Approach	1324	76	1400	0.0		0.345			
East: Morphett Rd									
Mov.	L2	Total	%HV		Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
From E									
To Exit:	S								
Lane 1	116	116	0.0		657	0.176	100	NA	NA
Lane 2	116	116	0.0		657	0.176	100	NA	NA
Lane 3	116	116	0.0		657	0.176	100	NA	NA
Approach	347	347	0.0			0.176			
North: Diagonal Rd									
Mov.	L2	T1	Total	%HV					

From N To Exit:	E	S		Cap. veh/h	Deg. Satn v/c	Lane Util. %	Prob. SL Ov. %	Ov. Lane No.
Lane 1	21	369	390	0.0	721	0.541	100	NA
Lane 2	-	373	373	0.0	690	0.541	100	NA
Lane 3	-	373	373	0.0	690	0.541	100	NA
Approach	21	1116	1137	0.0		0.541		
	Total	%HV	Deg.Satn (v/c)					
Intersection	2884	0.0	0.541					

APPENDIX B: STRUCTURAL CALCULATIONS – RAIL BRIDGE

Superstructure Calculations

SPACE GASS 12.54 (555) - FINAL SPACEGASS MODEL - CEDP

File Structure Loads Analysis Design Output View Query Settings Window Help

Moving Loads

Scenarios

- Train Loads
 - 150LA RAIL 5-3 (User) (Oaklands Park Rail) - Unnamed Tra...
 - 150LA RAIL 5-3 (User) (Oaklands Park Rail) - Unnamed Tra...

Scenario load cases: 101 to 150, Time interval: 0.1 sec

Vehicle: 150LA RAIL 5-3 (User) (Oaklands Park Rail) Edit

Travel Path: Unnamed Travel Path 1

Start position: 0.00 m

Speed: 14.00 m/s

Delay: 0.00 sec

Load factor: 2.34

Lane factor: 1.00

Dynamic factor: 1.00

Width: 1.60 m

Length: 43.70 m

Wheels: 22

Moving/Stationary: Moving

Travel time: 4.91 sec

Generate

Close

Cancel

Help

Reset

Clear All

Select Elements to Load

Generate Extra Travel Paths

View

Vehicles

Pressures

Travel Paths

Add Scenario Delete Scenario Scenario Properties

Add Load Delete Load Load Properties

☐ Apply member loads to closest member only

☐ Ignore loads that transfer load to just one member

☐ Ignore loads that are outside the loading area

☒ Check vertical proximity Proximity (m): 1

Select Loading Area

Plan

Front

(7,52)

Headings: N Structure: YNNYYYNN Loads: NNNNNNNNNYYY Analysis: YNNNN Steel: NNNNN Concrete: NNNNNNN

X: -3.588m, Y: -3.433m, Z: 0.000m (Plane: XY)

SPACE GASS

Type here to search

14:41 13-Jun-17

SPACE GASS 12.54 (555) - FINAL SPACEGASS MODEL - CEDP

File Structure Loads Analysis Design Output View Query Settings Window Help

Vehicle Properties

Name: 150LA RAIL 5-3 (User) Source: Oaklands Park Rail

OK Cancel Help

Reset Camera

150LA RAIL 5-3 (User) (Oaklands Park Rail)

150LA RAIL 5-3 (User) (Oaklands Park Rail) - Y AXIS IS VERTICAL

X (m)	Z (m)	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
0	0.8	0	-90	0	0	0	0
0	-0.8	0	-90	0	0	0	0
-2	0.8	0	-75	0	0	0	0
-2	-0.8	0	-75	0	0	0	0
-3.7	0.8	0	-75	0	0	0	0
-3.7	-0.8	0	-75	0	0	0	0
-4.8	0.8	0	-75	0	0	0	0
-4.8	-0.8	0	-75	0	0	0	0
-6.5	0.8	0	-75	0	0	0	0
-6.5	-0.8	0	-75	0	0	0	0
-23.7	0.8	0	-75	0	0	0	0

Record 1 of 22

Headings: N Structure: YYYYYYNN Loads: NNYNNNNNNNNYY Analysis: YNNN Steel: NNNN Concrete: NNNNNN

X: -3.588m, Y: -3.433m, Z: 0.000m (Plane: XY)

14:42 13-Jun-17

Case	Member	Sub Load	Axes	Units	Start Position (m or %)	Finish Position (m or %)	Start X Force (kN/m)	Finish X Force (kN/m)	Start Y Force (kN/m)
2	6	1	G-Incl	Percent	0	100	0	0	-10
2	8	1	G-Incl	Percent	0	100	0	0	-10
2	9	1	G-Incl	Percent	0	100	0	0	-10
2	10	1	G-Incl	Percent	0	100	0	0	-10
2	11	1	G-Incl	Percent	0	100	0	0	-10
2	12	1	G-Incl	Percent	0	100	0	0	-10
2	13	1	G-Incl	Percent	0	100	0	0	-10
2	14	1	G-Incl	Percent	0	100	0	0	-10
2	15	1	G-Incl	Percent	0	100	0	0	-10
2	16	1	G-Incl	Percent	0	100	0	0	-10
2	17	1	G-Incl	Percent	0	100	0	0	-10
2	18	1	G-Incl	Percent	0	100	0	0	-10
2	19	1	G-Incl	Percent	0	100	0	0	-10
2	20	1	G-Incl	Percent	0	100	0	0	-10
2	21	1	G-Incl	Percent	0	100	0	0	-10
2	22	1	G-Incl	Percent	0	100	0	0	-10
2	23	1	G-Incl	Percent	0	100	0	0	-10
2	24	1	G-Incl	Percent	0	100	0	0	-10
2	25	1	G-Incl	Percent	0	100	0	0	-10
2	26	1	G-Incl	Percent	0	100	0	0	-10
2	27	1	G-Incl	Percent	0	100	0	0	-10
2	28	1	G-Incl	Percent	0	100	0	0	-10
2	29	1	G-Incl	Percent	0	100	0	0	-10
2	30	1	G-Incl	Percent	0	100	0	0	-10
2	31	1	G-Incl	Percent	0	100	0	0	-10
2	32	1	G-Incl	Percent	0	100	0	0	-10
2	33	1	G-Incl	Percent	0	100	0	0	-10
2	34	1	G-Incl	Percent	0	100	0	0	-10
2	35	1	G-Incl	Percent	0	100	0	0	-10
2	36	1	G-Incl	Percent	0	100	0	0	-10
2	38	1	G-Incl	Percent	0	100	0	0	-10
2	39	1	G-Incl	Percent	0	100	0	0	-10
2	40	1	G-Incl	Percent	0	100	0	0	-10
2	41	1	G-Incl	Percent	0	100	0	0	-10
2	43	1	G-Incl	Percent	0	100	0	0	-10
2	44	1	G-Incl	Percent	0	100	0	0	-10
2	45	1	G-Incl	Percent	0	100	0	0	-10
2	46	1	G-Incl	Percent	0	100	0	0	-10
2	48	1	G-Incl	Percent	0	100	0	0	-10

[illegible]

Case	Member	Sub Load	Axes	Units	Start Position (m or %)	Finish Position (m or %)	Start X Force (kN/m)	Finish X Force (kN/m)	Start Y Force (kN/m)
2	49	1	G-Incl	Percent	0	100	0	0	-10
2	50	1	G-Incl	Percent	0	100	0	0	-10
2	51	1	G-Incl	Percent	0	100	0	0	-10
2	53	1	G-Incl	Percent	0	100	0	0	-10
2	54	1	G-Incl	Percent	0	100	0	0	-10
2	55	1	G-Incl	Percent	0	100	0	0	-10
2	56	1	G-Incl	Percent	0	100	0	0	-10
2	58	1	G-Incl	Percent	0	100	0	0	-10
2	59	1	G-Incl	Percent	0	100	0	0	-10
2	60	1	G-Incl	Percent	0	100	0	0	-10
2	61	1	G-Incl	Percent	0	100	0	0	-10
2	63	1	G-Incl	Percent	0	100	0	0	-10
2	64	1	G-Incl	Percent	0	100	0	0	-10
2	65	1	G-Incl	Percent	0	100	0	0	-10
2	66	1	G-Incl	Percent	0	100	0	0	-10
2	68	1	G-Incl	Percent	0	100	0	0	-10
2	69	1	G-Incl	Percent	0	100	0	0	-10
2	70	1	G-Incl	Percent	0	100	0	0	-10
2	71	1	G-Incl	Percent	0	100	0	0	-10
2	73	1	G-Incl	Percent	0	100	0	0	-10
2	74	1	G-Incl	Percent	0	100	0	0	-10
2	75	1	G-Incl	Percent	0	100	0	0	-10
2	76	1	G-Incl	Percent	0	100	0	0	-10
2	78	1	G-Incl	Percent	0	100	0	0	-10
2	79	1	G-Incl	Percent	0	100	0	0	-10
2	80	1	G-Incl	Percent	0	100	0	0	-10
2	81	1	G-Incl	Percent	0	100	0	0	-10
2	83	1	G-Incl	Percent	0	100	0	0	-10
2	84	1	G-Incl	Percent	0	100	0	0	-10
2	85	1	G-Incl	Percent	0	100	0	0	-10
2	86	1	G-Incl	Percent	0	100	0	0	-10
2	88	1	G-Incl	Percent	0	100	0	0	-10
2	89	1	G-Incl	Percent	0	100	0	0	-10
2	90	1	G-Incl	Percent	0	100	0	0	-10
2	91	1	G-Incl	Percent	0	100	0	0	-10
2	93	1	G-Incl	Percent	0	100	0	0	-10
2	94	1	G-Incl	Percent	0	100	0	0	-10
2	95	1	G-Incl	Percent	0	100	0	0	-10
2	96	1	G-Incl	Percent	0	100	0	0	-10

[illegible]

Case	Member	Sub Load	Axes	Units	Start Position (m or %)	Finish Position (m or %)	Start X Force (kN/m)	Finish X Force (kN/m)	Start Y Force (kN/m)
2	98	1	G-Incl	Percent	0	100	0	0	-10
2	99	1	G-Incl	Percent	0	100	0	0	-10
2	100	1	G-Incl	Percent	0	100	0	0	-10
2	101	1	G-Incl	Percent	0	100	0	0	-10
2	103	1	G-Incl	Percent	0	100	0	0	-10
2	104	1	G-Incl	Percent	0	100	0	0	-10
2	105	1	G-Incl	Percent	0	100	0	0	-10
2	106	1	G-Incl	Percent	0	100	0	0	-10
2	108	1	G-Incl	Percent	0	100	0	0	-10
2	109	1	G-Incl	Percent	0	100	0	0	-10
2	110	1	G-Incl	Percent	0	100	0	0	-10
2	111	1	G-Incl	Percent	0	100	0	0	-10
2	113	1	G-Incl	Percent	0	100	0	0	-10
2	114	1	G-Incl	Percent	0	100	0	0	-10
2	115	1	G-Incl	Percent	0	100	0	0	-10
2	116	1	G-Incl	Percent	0	100	0	0	-10
2	118	1	G-Incl	Percent	0	100	0	0	-10
2	119	1	G-Incl	Percent	0	100	0	0	-10
2	120	1	G-Incl	Percent	0	100	0	0	-10
2	121	1	G-Incl	Percent	0	100	0	0	-10
2	123	1	G-Incl	Percent	0	100	0	0	-10
2	124	1	G-Incl	Percent	0	100	0	0	-10
2	125	1	G-Incl	Percent	0	100	0	0	-10
2	126	1	G-Incl	Percent	0	100	0	0	-10
2	128	1	G-Incl	Percent	0	100	0	0	-10
2	129	1	G-Incl	Percent	0	100	0	0	-10
2	130	1	G-Incl	Percent	0	100	0	0	-10
2	131	1	G-Incl	Percent	0	100	0	0	-10
2	133	1	G-Incl	Percent	0	100	0	0	-10
2	134	1	G-Incl	Percent	0	100	0	0	-10
2	135	1	G-Incl	Percent	0	100	0	0	-10
2	136	1	G-Incl	Percent	0	100	0	0	-10
2	138	1	G-Incl	Percent	0	100	0	0	-10
2	139	1	G-Incl	Percent	0	100	0	0	-10
2	140	1	G-Incl	Percent	0	100	0	0	-10
2	141	1	G-Incl	Percent	0	100	0	0	-10
2	143	1	G-Incl	Percent	0	100	0	0	-10
2	144	1	G-Incl	Percent	0	100	0	0	-10
2	145	1	G-Incl	Percent	0	100	0	0	-10

[illegible]

SPACE GASS 12.54 - STUDENT VERSION - NOT FOR COMMERCIAL USE

Path: D:\SPACEGASS\RAIL BRIDGE DEISNG\FINAL SPACEGASS MODEL - CEDP

Designer: Date: Tuesday, June 13, 2017 2:39 PM Page: 7

Case	Member	Sub Load	Axes	Units	Start Position (m or %)	Finish Position (m or %)	Start X Force (kN/m)	Finish X Force (kN/m)	Start Y Force (kN/m)
2	146	1	G-Incl	Percent	0	100	0	0	-10
2	148	1	G-Incl	Percent	0	100	0	0	-10
2	149	1	G-Incl	Percent	0	100	0	0	-10
2	150	1	G-Incl	Percent	0	100	0	0	-10
2	151	1	G-Incl	Percent	0	100	0	0	-10
2	153	1	G-Incl	Percent	0	100	0	0	-10
2	154	1	G-Incl	Percent	0	100	0	0	-10
2	155	1	G-Incl	Percent	0	100	0	0	-10
2	156	1	G-Incl	Percent	0	100	0	0	-10
2	158	1	G-Incl	Percent	0	100	0	0	-10
2	159	1	G-Incl	Percent	0	100	0	0	-10
2	160	1	G-Incl	Percent	0	100	0	0	-10
2	161	1	G-Incl	Percent	0	100	0	0	-10
3	37	1	G-Incl	Percent	0	100	0	0	-5
3	42	1	G-Incl	Percent	0	100	0	0	-5
3	47	1	G-Incl	Percent	0	100	0	0	-5
3	52	1	G-Incl	Percent	0	100	0	0	-5
3	57	1	G-Incl	Percent	0	100	0	0	-5
3	62	1	G-Incl	Percent	0	100	0	0	-5
3	67	1	G-Incl	Percent	0	100	0	0	-5
3	72	1	G-Incl	Percent	0	100	0	0	-5
3	77	1	G-Incl	Percent	0	100	0	0	-5
3	82	1	G-Incl	Percent	0	100	0	0	-5
3	87	1	G-Incl	Percent	0	100	0	0	-5
3	92	1	G-Incl	Percent	0	100	0	0	-5
3	97	1	G-Incl	Percent	0	100	0	0	-5
3	102	1	G-Incl	Percent	0	100	0	0	-5
3	107	1	G-Incl	Percent	0	100	0	0	-5
3	112	1	G-Incl	Percent	0	100	0	0	-5
3	117	1	G-Incl	Percent	0	100	0	0	-5
3	122	1	G-Incl	Percent	0	100	0	0	-5
3	127	1	G-Incl	Percent	0	100	0	0	-5
3	132	1	G-Incl	Percent	0	100	0	0	-5
3	137	1	G-Incl	Percent	0	100	0	0	-5
3	142	1	G-Incl	Percent	0	100	0	0	-5
3	147	1	G-Incl	Percent	0	100	0	0	-5
3	152	1	G-Incl	Percent	0	100	0	0	-5
3	157	1	G-Incl	Percent	0	100	0	0	-5
3	162	1	G-Incl	Percent	0	100	0	0	-5

[illegible]

SPACE GASS 12.54 - STUDENT VERSION - NOT FOR COMMERCIAL USE

Path: D:\SPACEGASS\RAIL BRIDGE DEISNG\FINAL SPACEGASS MODEL - CEDP

Designer: Date: Tuesday, June 13, 2017 2:39 PM Page: 9

Case	Member	Sub Load	Axes	Units	Start Position (m or %)	Finish Position (m or %)	Start X Force (kN/m)	Finish X Force (kN/m)	Start Y Force (kN/m)
3	357	1	G-Incl	Percent	0	100	0	0	-5
3	358	1	G-Incl	Percent	0	100	0	0	-5
3	359	1	G-Incl	Percent	0	100	0	0	-5
3	360	1	G-Incl	Percent	0	100	0	0	-5
3	361	1	G-Incl	Percent	0	100	0	0	-5
3	362	1	G-Incl	Percent	0	100	0	0	-5
3	363	1	G-Incl	Percent	0	100	0	0	-5
3	364	1	G-Incl	Percent	0	100	0	0	-5
3	365	1	G-Incl	Percent	0	100	0	0	-5
3	366	1	G-Incl	Percent	0	100	0	0	-5
3	367	1	G-Incl	Percent	0	100	0	0	-5
3	368	1	G-Incl	Percent	0	100	0	0	-5
3	369	1	G-Incl	Percent	0	100	0	0	-5
3	370	1	G-Incl	Percent	0	100	0	0	-5
3	371	1	G-Incl	Percent	0	100	0	0	-5
3	372	1	G-Incl	Percent	0	100	0	0	-5
3	373	1	G-Incl	Percent	0	100	0	0	-5
3	374	1	G-Incl	Percent	0	100	0	0	-5
3	375	1	G-Incl	Percent	0	100	0	0	-5
3	376	1	G-Incl	Percent	0	100	0	0	-5
3	377	1	G-Incl	Percent	0	100	0	0	-5
3	378	1	G-Incl	Percent	0	100	0	0	-5
3	379	1	G-Incl	Percent	0	100	0	0	-5
3	380	1	G-Incl	Percent	0	100	0	0	-5
3	381	1	G-Incl	Percent	0	100	0	0	-5
3	382	1	G-Incl	Percent	0	100	0	0	-5

[illegible]

DPC Hand Calculations

By: Liam Matthews

Date: 05/06/17

Checked by: _____

Date: / /

Wind Loads

AS Per AS 1170.2

Adelaide \Rightarrow Region A1

$$U_{lt} = V_{2000} = 48 \text{ m/s}$$

Med \Rightarrow allow 10

TC \Rightarrow Consider 500 m $z \Rightarrow$ unknown as of yet.
 \hookrightarrow TC3 (Suburban housing)

$$M_{zcat} = 0.83$$

$$M_s, M_t = 1.0$$

Site Des wind speed

$$= V_r \times 0.83$$

$$= 39.84 \Rightarrow 40 \text{ m/s}$$

Pressure

$$= V^2 \times 0.6$$

$$= 960 \text{ Pa}$$

$$= 0.96 \text{ kPa}$$

DPC Hand Calculations

By: _____

Date: / /

Checked by: _____

Date: / /

Rail Bridge Wind loads

Consider load length of 25m

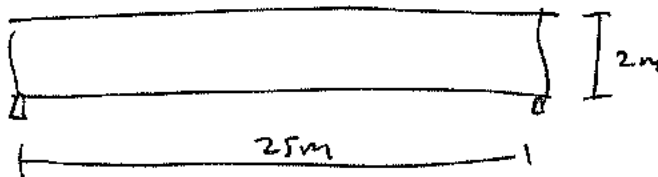
⇒ take faade to be hemispherical

↳ Reduction factor of
0.4

$$\text{load} = 0.96 \times 0.4 \times 25 \times \text{height}$$

↳ take height to be 2m
= 9.2 kN of lateral load.

load from other directions Assumed to
be Non Critical.



	Load (kN)	Super t type	Gamma	Alpha	Fc	Ast	Fpb	dt	bar D	Slab D	b	Predicted Ku	Ku	Sigma Pb	K1	K2
M*	11000	t5	0.67	0.805	65	4576	1830	1935	15.2	250	2000	0.129198966	0.0617223	0.986088	0.28	0.03329
V*	1730															
Phi	0.8															

A-Okay

Determine No. Strands
Required Strands
31.97

Nominated No. to Use
32.00

A-Okay

Actual Capacity
15810.89 kN
Factored Cap
12648.71 kN

Check Strain in PT	at Strain?	NA
0.0456	A-Okay	119.43

Check Super-T at stress transfer

Check Shear
Shear Check

Vuc
828.89

Use Area rather than bv Dv

Kv	Simplified Method
0.056	
Root F'c	
8.0	

Asv	Fsy	do	S	av	Theta
440	500	2000	250	45	90

Vus
2489.01587

Vu
3317.90

Phi
0.7

PhiVu
2322.53

A-Okay

Type	Depth
t1	750
t2	1000
t3	1200
t4	1500
t5	1800

Bar D	Area	Fpt
12.7		
15.2	143	1830

Super-t Design

Compression Block

$$\alpha_2 = 1 - 0.007 f'_c$$

$$= 0.67$$

Depth: 250 mm

$$\gamma = 1.05 - 0.007 f'_c$$

Effective width 2000 mm

$$= 0.67$$

Depth to Centre of tension

$$= 1800 - 50 - 65 + 250$$

Capacity formula

$$A_{pt} \times f_{py} \times d_{eff} - \alpha_2 f'_c b \left(\frac{\gamma k_{end}}{2} \right)^2 = \frac{11000,000,000}{\phi}$$

$$A_{pt} \times 1830 \times 1935 - 0.67 \times 65 \times 2000 \left(\frac{0.67 \times 0.12 \times 250}{2} \right)^2 =$$

$$3657$$

$$A_{pt} = 3103 \text{ mm}^2$$

$$\rightarrow 25.6 \times 15.2$$

$\rightarrow 22 \times 15.2 \text{ mm Strands}$ ☺

24 Strand

\rightarrow use $26 \div 2$

$$K_u = 0.12 < 0.36 \rightarrow \text{Ductile}$$

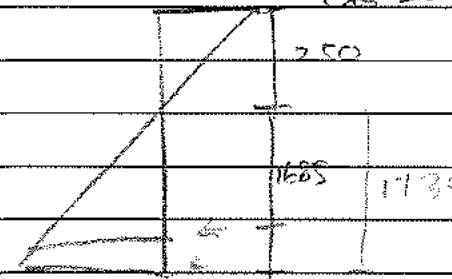
check all steel @ yield

$$\frac{0.003}{250} = \frac{\epsilon_s}{1685}$$

$$\frac{0.003}{250} = \frac{\epsilon_s}{1735}$$

$$\epsilon_s = 0.0270.0076$$

$$\epsilon_s = 0.02170.0076$$



@ yield.

check load @ transfer

I_x

$$= 3.16 \times 10^{11} \rightarrow \text{space goes.}$$

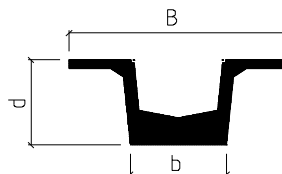
$$2x = 8.56 \times 10^8$$

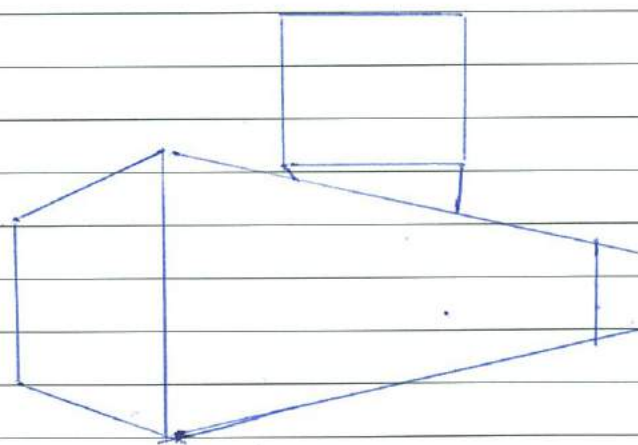
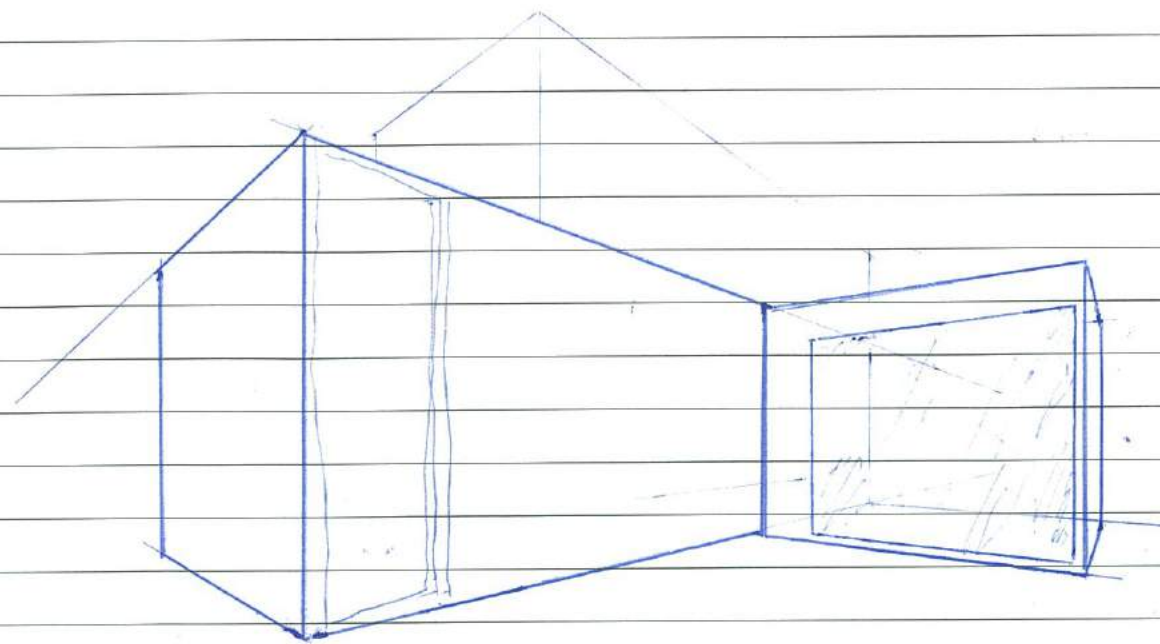
SUPER-T DATA SHEET OPEN TOP

MARK-II SECTIONS agreed with RTA !!

PRECAST PRESTRESSED CONCRETE MANUFACTURERS

Super-T (Open Top) - Precast Section Properties							
						F'c(pc)=50MPa	
T1	Flange (B)	(mm)	1800	2000	2200	2400	2500
	Precast (d)	(mm)	750	750	750	750	750
	Area	(mm2)	412,808	427,808	442,808	457,808	465,308
	Ix	(mm4)	2.46766E+10	2.70015E+10	2.91698E+10	3.11952E+10	3.21597E+10
	yb	(mm)	312	326	339	352	357
	Zt(pc)	(mm3)	5.63830E+07	6.37386E+07	7.10507E+07	7.83158E+07	8.19335E+07
	Zb(pc)	(mm3)	7.90056E+07	8.27325E+07	8.59323E+07	8.87049E+07	8.99597E+07
	Soffit (b)	(mm)	900	900	900	900	900
	V/s	(mm)	72.35	70.07	68.07	66.30	65.48
	Mass	(T/m)	1.073	1.112	1.151	1.190	1.210
T2	Flange (B)	(mm)	1800	2000	2200	2400	2500
	Precast (d)	(mm)	1000	1000	1000	1000	1000
	Area	(mm2)	449,461	464,461	479,461	494,461	501,961
	Ix	(mm4)	5.19993E+10	5.63475E+10	6.04250E+10	6.42553E+10	6.60848E+10
	yb	(mm)	416	433	450	465	473
	Zt(pc)	(mm3)	8.89788E+07	9.94241E+07	1.09827E+08	1.20187E+08	1.25350E+08
	Zb(pc)	(mm3)	1.25119E+08	1.30054E+08	1.34332E+08	1.38073E+08	1.39774E+08
	Soffit (b)	(mm)	852	852	852	852	852
	V/s	(mm)	67.88	66.15	64.60	63.22	62.58
	Mass	(T/m)	1.169	1.208	1.247	1.286	1.305
T3	Flange (B)	(mm)	1800	2000	2200	2400	2500
	Precast (d)	(mm)	1200	1200	1200	1200	1200
	Area	(mm2)	490,834	505,834	520,834	535,834	543,334
	Ix	(mm4)	8.29947E+10	8.93926E+10	9.54229E+10	1.01115E+11	1.03844E+11
	yb	(mm)	500	520	538	556	564
	Zt(pc)	(mm3)	1.18542E+08	1.31367E+08	1.44152E+08	1.56894E+08	1.63250E+08
	Zb(pc)	(mm3)	1.66032E+08	1.72068E+08	1.77353E+08	1.82019E+08	1.84154E+08
	Soffit (b)	(mm)	814	814	814	814	814
	V/s	(mm)	67.05	65.52	64.14	62.89	62.31
	Mass	(T/m)	1.276	1.315	1.354	1.393	1.413
T4	Flange (B)	(mm)	1800	2000	2200	2400	2500
	Precast (d)	(mm)	1500	1500	1500	1500	1500
	Area	(mm2)	532,229	547,229	562,229	577,229	584,729
	Ix	(mm4)	1.44275E+11	1.54071E+11	1.63347E+11	1.72140E+11	1.76368E+11
	yb	(mm)	643	666	687	707	717
	Zt(pc)	(mm3)	1.68411E+08	1.84686E+08	2.00925E+08	2.17123E+08	2.25208E+08
	Zb(pc)	(mm3)	2.24268E+08	2.31418E+08	2.37760E+08	2.43419E+08	2.46027E+08
	Soffit (b)	(mm)	756	756	756	756	756
	V/s	(mm)	63.22	62.05	60.99	60.01	59.55
	Mass	(T/m)	1.384	1.423	1.462	1.501	1.520
T5	Flange (B)	(mm)	1800	2000	2200	2400	2500
	Precast (d)	(mm)	1800	1800	1800	1800	1800
	Area	(mm2)	668,625	683,625	698,625	713,625	721,125
	Ix	(mm4)	2.47287E+11	2.61022E+11	2.74168E+11	2.86760E+11	2.92860E+11
	yb	(mm)	795	816	837	856	866
	Zt(pc)	(mm3)	2.46103E+08	2.65377E+08	2.84621E+08	3.03831E+08	3.13424E+08
	Zb(pc)	(mm3)	3.10979E+08	3.19718E+08	3.27668E+08	3.34928E+08	3.38328E+08
	Soffit (b)	(mm)	700	700	700	700	700
	V/s	(mm)	71.08	69.71	68.45	67.29	66.73
	Mass	(T/m)	1.738	1.777	1.816	1.855	1.875





Slab $M_{neg}^* = 73.76$ $V^* = 231.9$
 $M_{pos}^* = 83.0$

one way Slab Design

Try N_{10}^{20} 's @ 100 cts for M_{neg}^*

$A_{st} = 3140$

find k_u

$d = 250 - 40 - 10$

$\gamma = 0.77$

$\alpha_2 = 0.85$

$\gamma k_u d b \alpha_2 f_{ck} = A_{st} f_{sy}$

$0.77 k_u \times 200 \times 1000 \times 0.85 \times 40 = 3140 \times 500$

$5236000 k_u = 157$

$k_u = 0.29 < 0.36$

Capacity M_u

$\phi M_u = A_{st} \times f_{sy} \times d - \frac{\alpha_2 f_{ck} b (\gamma k_u d)^2}{2}$
 $= 313 \text{ kNm} \times 0.8$

$\phi M_u = 250 \text{ kNm}$ → use for top + bottom

Rural Bridge

Ethics

→ Don't Manipulate

Risks

→ W1508

Budget

→ No external Research projects
→ Save is fine

Timeline

→ Report - 31st Oct week 13

→ Poster - week 14-15 13-14

→ Viva - week 4-5

→ Effective Journal

→ project week 13

→ presentation 13 week

Super-T

$$M^* = 11042 \text{ KN}$$

$$V^* = 1687 \text{ KN } 1730$$

Slab

$$M_{\text{avg}}^* = 73.76 \text{ KN}$$

$$M_{\text{pred}}^* = 83 \text{ KN}$$

$$\text{Shear } V^* = 231.78$$

Slab Design

Depth of slab 250mm

Effective width 1000mm

Span 2800m (Max SuperT flange width.)

Design at one way slab

Space goes → Neg 9 N16's T 6 N16 B } shear
Pos 9 N16 B 6 N16 T } could
Be an
Assume.

Head Stock Design



SPACE GASS 12.54 - STUDENT VERSION - NOT FOR COMMERCIAL USE

Path: D:\SPACEGASS\RAIL BRIDGE DEISNG\FINAL SPACEGASS MODEL - CEDP

Designer: Date: Tuesday, June 13, 2017 2:45 PM Page: 1

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

Envelope = All Load Cases
and All Members
and All Sections

The following maximums and minimums are taken from either end of the member

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
3	218	0.000	1684.666*	0.000	17.710	0.000	-20.979
186	218	0.000	-1730.648#	0.000	-20.655	0.000	-18.492
213	116	0.000	447.401	0.000	132.362*	0.000	1343.240
231	109	0.000	-431.167	0.000	-127.697#	0.000	1730.360
174	212	0.000	-43.721	0.000	-4.202	0.000	11042.422*
160	231	0.000	117.220	0.000	20.577	0.000	-73.759#

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	220	0.000	1920.248*	0.000	0.000	0.000	0.000
10	101	0.000	-12.076#	0.000	0.000	0.000	0.000
5	248	0.000	1330.787	0.000	0.000*	0.000	0.000
5	209	0.000	1354.062	0.000	0.000#	0.000	0.000
4	210	0.000	1395.603	0.000	0.000	0.000	0.000*
5	218	0.000	1790.745	0.000	0.000	0.000	0.000#

INTERMEDIATE DISPLACEMENTS (m,mm)
----- (*=Maximum, #=Minimum)

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Global X Transl'n	Global Y Transl'n	Global Z Transl'n	Local X Transl'n	Local Y Transl'n	Local Z Transl'n
162	111	0.000	1.265*	0.000	0.000	1.265	0.000
174	212	0.000	-69.180#	0.000	0.000	-69.180	0.000
162	111	0.000	1.265	0.000	0.000	1.265*	0.000
174	212	0.000	-69.180	0.000	0.000	-69.180#	0.000

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

Envelope = All Load Cases
and All Members
and All Sections

Memb	Load Case	Axial Force	Y-Axis Shear	Z-Axis Shear	X-Axis Torsion	Y-Axis Moment	Z-Axis Moment
3	218	0.000	1684.666*	0.000	17.710	0.000	-20.979
186	218	0.000	-1730.648#	0.000	-20.655	0.000	-18.492
213	116	0.000	447.401	0.000	132.362*	0.000	1343.240
231	109	0.000	-431.167	0.000	-127.697#	0.000	1730.360
174	212	0.000	-43.721	0.000	-4.202	0.000	11042.422*
160	231	0.000	117.220	0.000	20.577	0.000	-73.759#

SPACE GASS 12.54 - STUDENT VERSION - NOT FOR COMMERCIAL USE

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Designer: Date: Friday, June 9, 2017 4:04 PM Page: 1

NODE LOADS (kN, kNm)

Load Case	Node	X-Axis Force	Y-Axis Force	Z-Axis Force	X-Axis Moment	Y-Axis Moment	Z-Axis Moment
1	2	0.000	-1160.000	0.000	0.000	0.000	0.000
	4	0.000	-1160.000	0.000	0.000	0.000	0.000
	5	0.000	-1160.000	0.000	0.000	0.000	0.000
	6	0.000	-1160.000	0.000	0.000	0.000	0.000
	8	0.000	-1160.000	0.000	0.000	0.000	0.000
3	2	0.000	-540.000	0.000	0.000	0.000	0.000
	4	0.000	-540.000	0.000	0.000	0.000	0.000
	5	0.000	-540.000	0.000	0.000	0.000	0.000
	6	0.000	-540.000	0.000	0.000	0.000	0.000
	8	0.000	-312.000	0.000	0.000	0.000	0.000
4	9	-24.000	0.000	0.000	0.000	0.000	0.000

SELF WEIGHT (g's)

Load Case	X-Axis Accel'n	Y-Axis Accel'n	Z-Axis Accel'n
2	0.000	-1.000	0.000

COMBINATION LOAD CASES

Load case 10: Combination Loads Based on 1.2G+15Q

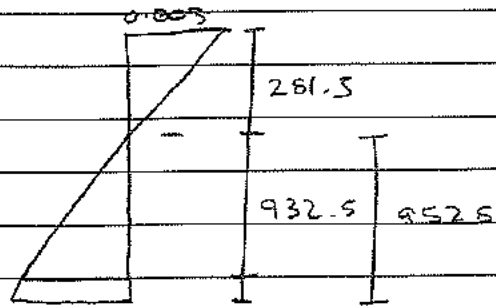
- 1.200 * Load case 1: Dead Load Loads Super imposed
- 1.200 * Load case 2: Self Weight
- 1.500 * Load case 3: Live Loads
- 1.000 * Load case 4: Wind Loads

LOAD CASE TITLES

Load Case	Title
1	Dead Load Loads Super imposed dead loads from ballast, super-t, etc
2	Self Weight Self Weight of Beam
3	Live Loads Live Loads from Train + Ped paths for Node 8
4	Wind Loads Wind Loads, need to find modification factor
10	Combination Loads Based on 1.2G+15Q

Strain Check.

1.19 - 13.10.22/17



$$\frac{0.003}{281.5} = \frac{\epsilon_{xt}}{932.5}$$

$$\epsilon_{xt} = 0.01$$

$$\frac{0.003}{281.5} = \frac{\epsilon_{sb}}{952.5}$$

$$\epsilon_{sb} = 0.011$$

Both > 0.0025 thus @ yield.

Check Moment Capacity.

$$\phi M_u \geq M^*$$

$$M_u = A_{st} f_{yk} z - \alpha_2 f'_{ck} b \left(\frac{x_{ud}}{2} \right)^2$$

$$= (22 \times 804) \times 500 \times 1232 - 0.85 \times 40 \times 1000 \left(\frac{0.77 \times 0.22 \times 1232}{2} \right)^2$$

$$= 10525.6 \text{ kNm} \quad 10451 \text{ kNm}$$

$$\phi = 0.8$$

$$\phi M_u = 8420 \text{ kNm} \quad 8361 \text{ kNm}$$

$$\begin{aligned} \text{Check } M_{u, \text{nom}} &= 1.2 \left[Z (f_{yk} + P_e / A_g + P_e / e) \right] \\ &= 1.2 \left[Z (f_{yk}) \right] \end{aligned}$$

$$Z = \frac{B d^2}{6} = \frac{1200 \times 1300^2}{6} = 3.38 \times 10^8$$

$$\begin{aligned} f_{yk} &= 0.6 \sqrt{f'_{ck}} \\ &= 3.8 \end{aligned}$$

$$M_{u, \text{nom}} = 1282 < \phi M_u \quad \text{passes}$$

Shear Check.

$$V^* > \phi V_u \quad \text{where } V_u = V_{uc} + V_{us}$$

$$V_{uc} \text{ Cl 8.2.7}$$

$$V_{us} \text{ Cl 8.2.9 \& 8.2.10}$$

Eq. 8. V_{uc} → Concrete Shear Capacity.

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cr} \left[\frac{A_{st}}{b_v d_o} \right]^{1/3}$$

$$\begin{aligned} \beta_1 &= 1.1 (1.6 - d_o / 1000) \geq 1.1 \text{ or } 0.8 \\ &= 0.8 \end{aligned}$$

$$\begin{aligned} A_{st} &= 0.6 \sqrt{f'_{ck}} b_v d_o / f_{yk} \\ &= 0.6 \sqrt{40} \times 1200 \times 1232 / 500 \\ &= 455 \end{aligned}$$

$$\beta_2 = 1$$

$$\beta_3 = 1$$

$$\begin{aligned} f_{cr} &= f'_{cu}^{1/3} \leq f_{cr, \text{max}} \\ &= 3.42 \end{aligned}$$

$$\begin{aligned} A_{st} &= 9 \times 616 \\ &= 5544 \end{aligned}$$

$$b_v = 1200 \quad d_o = 1232$$

$$V_{uc} = 864 \text{ kN}$$

U12 leg + 2 Shear ties.

$$V_{us} = 2.9 \text{ C.2.10}$$

$$= (A_{sv} f_{sy} d_o / s) \cos \theta$$

$$= (4 \times 13 \times 500 \times 1232 / 50) \cos 45$$

$$= 4176 \text{ kN}$$

$$V_u = 4176 + 864$$

$$= 5040$$

$$\phi = 0.7$$

$$\phi V_u = 3528 > V^* = 3492$$

Capacity okay.

Negative Bending

$$M^* = 2790 \text{ kN}$$

$$\leftarrow T = A_{st} f_{sy}$$

$$C = t$$

$$\rightarrow C = \gamma k_u d b K_r / c$$

N28

$$\gamma = 0.77$$

$$A_{st} = 9 \times 616$$

$$\alpha_s = 0.85$$

$$f_{sy} = 500$$

$$f_r = 40$$

$$d = 1300 - 40 - 14$$

$$b = 1206$$

$$0.77 \times k_u 1246 \times 1200 \times 0.85 \times 40 = 9 \times 616 \times 500$$

$$K_r = 0.07 < 0.36 \text{ Ductile.}$$

check strain

$$\frac{0.003}{88.2} = \frac{\epsilon_{st}}{7157}$$

$$\epsilon_{st} = 0.6397 \times 0.025 @ \text{ yield.}$$

check capacity.

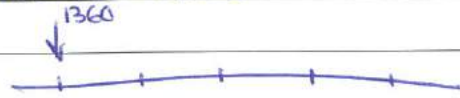
$$M_u = (9 \times 616 \times 500 \times 1246) - 0.85 \times 40 \times 1200 \left(\frac{0.77 \times 0.07 \times 1246}{2} \right)$$

$$= 3407 \text{ kNm}$$

$$\phi M_u = 2726 \text{ kNm} > M^* \text{ passes.}$$

Space Glass Design

Headstock

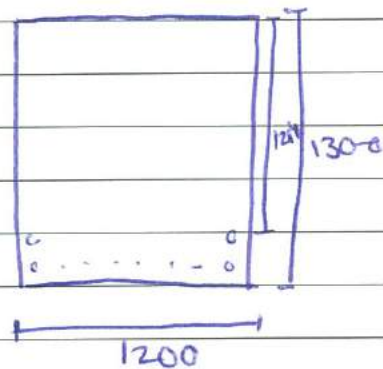


N10 Ligs @ 50 cts

↓
10

9 N28 top

10 N32 Bot + 2 N32 Bot
1214



Cover = 40mm

Column loads

$$1.2G + W_u + \psi_c$$

check Capacity of head stock

$$M_{pos}^* = 3584.15 \text{ kNm}$$

$$M_{neg}^* = 2290.12 \text{ kNm}$$

check ductility. Pos

$$T = C$$

$$T = A_{st} \times f_{sy}$$

$$C = \delta k u d b \alpha_2 f_c$$

$$f_c = 40$$

$$\alpha_2 = 0.85$$

$$\delta = 0.77$$

Steel details

20 N32's Bot Bot + 2 N32's @ 1214

find Centre of tension

$$\frac{20 \times 804 \times (1300 - 40 - 10 - 16)}{20 \times 804 + 2 \times 804} + \frac{2 \times 804 \times 1214}{20 \times 804 + 2 \times 804}$$

$$= 1232$$

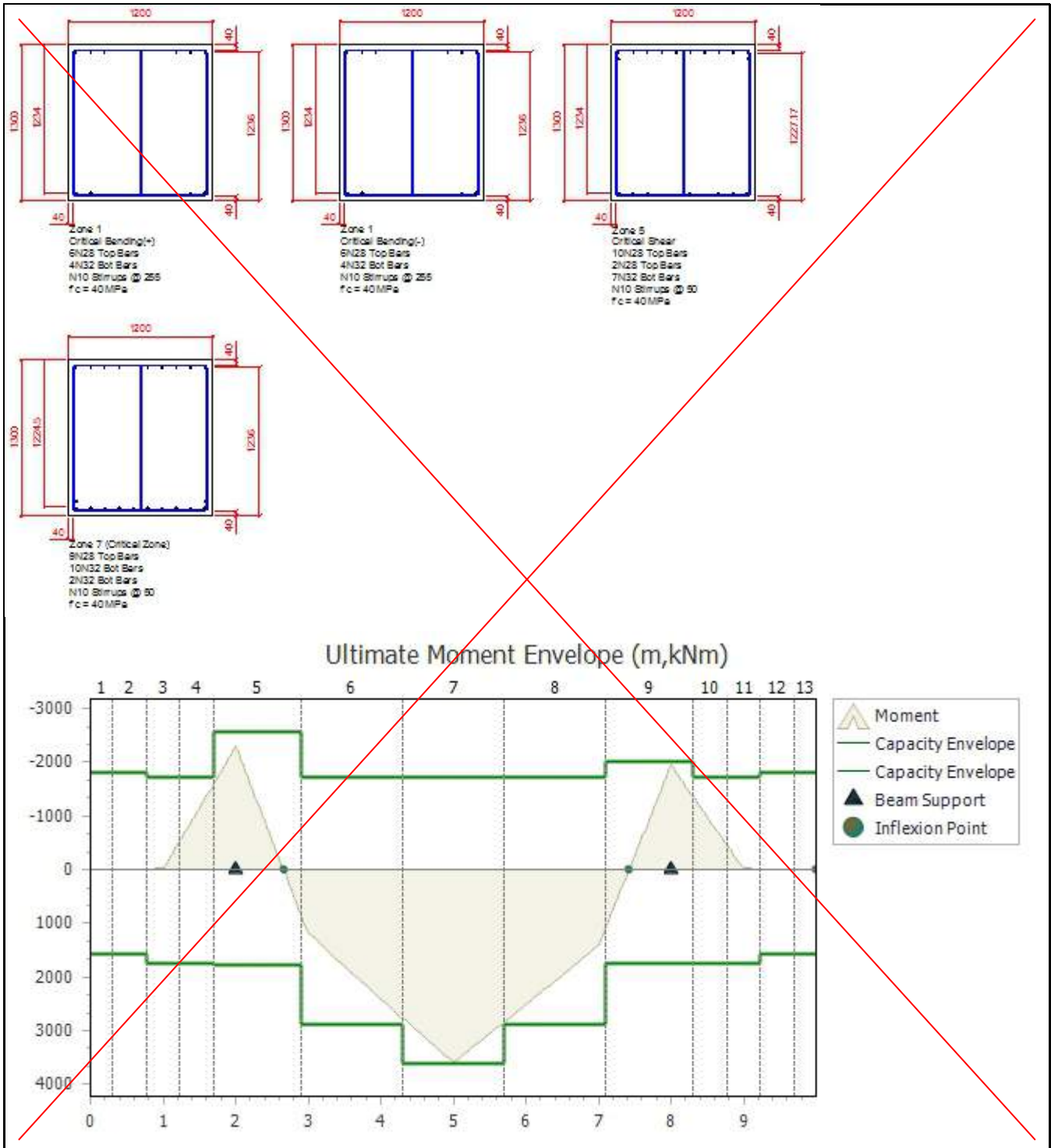
$$\rightarrow \delta k u d b \alpha_2 f_c$$

$$\leftarrow 22 \times 804 \times 500$$

$$0.77 k u 1232 \times 1200 \times 0.85 \times 40 = 22 \times 804 \times 500$$

$$38704512 k u = 8844000$$

$$k u = 0.22 < 0.36 \text{ thus Ductile}$$



Beam: 1
 Title: Headstock Design Calcs
 Member list: 1,2,3,4,5,6,7,8

Min design actions: Yes (+ve moments), Yes (-ve moments)
 Torsion: Included
 Design priority: Minimum layers
 Zones per span: 5
 Bar size continuity: Yes
 Top bar placement: Inside stirrups
 Stirrups: N10
 Bar size ranges: 12.0 to 36.0 mm (top), 16.0 to 36.0 mm (bottom)
 Layer spacing: 25.0 mm
 Cover: 40.0 top, 40.0 bottom, 40.0 sides (mm)
 Top bar anchorage: 90 Cog (start), 90 Cog (end)
 Bot bar anchorage: 90 Cog (start), 90 Cog (end)
 Torsion type: Indirect (AS3600 8.3.2)
 Crack control: Interior (AS3600 8.6.1)

REINFORCEMENT (mm,m)

Beam Title	Type	Reinforcement Layer	Start Position	Beam Length
1 Headstock Desi...	Bars	6N28 Top	1 0.000	10.000
		9N28 Top	1 0.767	
		10N28 Top	1 1.700	
		2N28 Top	2	
		9N28 Top	1 2.900	
		10N28 Top	1 7.100	
	Bars	9N28 Top	1 8.300	
		6N28 Top	1 9.233	
		4N32 Bot	1 0.000	
		7N32 Bot	1 0.767	
		10N32 Bot	1 2.900	
		10N32 Bot	1 4.300	
		2N32 Bot	2	
		10N32 Bot	1 5.700	
		7N32 Bot	1 7.100	
		4N32 Bot	1 9.233	

Total bar weight = 946.11 kg

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Beam Title	Type	Reinforcement	Layer	Start Position	Beam Length
	Stirrups	3 N10 @ 255		0.000	
		3 N10 @ 50		0.767	
		3 N10 @ 255		9.233	

AS3600 CALCULATIONS FOR REINFORCED CONCRETE BEAM 1

Design/Check: Design
Ultimate strength load cases: 10
Serviceability load cases: None
Sustained load cases: None
Critical load case: 10

Failure messages - Zone specific:

Tension at bot:
Insufficient transverse reinforcement in zones 5,6,8,9
Tension at top:
Insufficient transverse reinforcement in zones 3,4,5,9,10,11

Warning messages - Zone specific:

Check for compression reinforcement lateral restraint requirement (8.1.10.7) in zones 1,2,3,4,5,6,7,8,9,10,11,12,13
Crack control from side bars is not checked (8.6.3) in zones 1,2,3,4,5,6,7,8,9,10,11,12,13
Max centre-to-centre spacing of bars must not exceed 300 mm (8.6.1(b)) in zones 1,2,5,7,9,12,13

Notes - Zone specific:

Tension at bot:
Minimum shear reinforcement is required in zones 5,6,7,8,9
No shear reinforcement is required in zones 1,2,3,4,10,11,12,13
Tension at top:
No shear reinforcement is required in zones 6,7,8
Shear force is less than ϕV_{usmin} in zones 1,2,12,13
Minimum shear reinforcement is required in zones 1,2,3,4,5,9,10,11,12,13

Section Properties	Material Properties
D = 1300.0 mm	f'c = 40.00 MPa
Tw = 1200.0 mm	f _{sy} = 500.00 MPa
B = 1200.0 mm	f _{sy,f} = 500.00 MPa
Tf = 0.0 mm	f _{ct,f} = 3.79 MPa
Ag = 1560000.0 mm ²	Ec = 32800.00 MPa
	Es = 200000.00 MPa
	αc = 0.003

Development length multiplier: 1	(13.1.2.2)
Fire resistance period: 30 min	(5.2.5)
Max negative moment location: Not considered	(6.2.3)
Max transverse shear location: Not considered	(8.2.4)
Int moment-resisting frames: Not considered	(4.2)
β3: 1	(8.2.7.1)
θv: 45.0°	(8.2.10)
I gross: Bars not transformed	(8.5.3.1)
Design Shrinkage Strain: Calculated	(3.1.7)
Load time: 30.0 years (short), 30.0 years (long)	
Local Aggregate: Other	
Exposed environment: Interior	
th: Include top surface	

UTILIZATION RATIOS (*=Failure, #=Critical)

Zone	Positive Bending	Negative Bending	Shear Torsion	Positive Bending & Torsion	Negative Bending & Torsion	Overall
1	0.98#	0.86#	0.99	0.98	0.86	0.99
2	0.98	0.86	0.99	0.98	0.86	0.99
3	0.57	0.58	1.21	0.88	0.90	1.21*
4	0.57	0.60	1.22	0.88	0.94	1.22*
5	0.56	0.65	1.82#	0.87	0.89	1.82*
6	0.72	0.57	1.80	0.96	0.89	1.80*
7#	0.79	0.57	0.68	0.99#	0.88	0.99
8	0.74	0.57	1.74	0.99	0.89	1.74*
9	0.57	0.66	1.76	0.88	0.97#	1.76*
10	0.57	0.58	1.05	0.88	0.90	1.05*
11	0.57	0.58	1.04	0.88	0.90	1.04*
12	0.98	0.86	0.99	0.98	0.86	0.99
13	0.98	0.86	0.99	0.98	0.86	0.99

ZONE CAPACITY TABLE (kNm, kN) (#=Minimum design action governs)

(\$=Less than minimum design action)
(c=V_{uc} has been set to 0 (8.2.7.4))
(e=Earthquake action has been applied (C4.2.1))

Zone	Mx* (+)	φMuo (+)	φVu (+)	T*	φTu
1	1536.98#	1566.40	13.22	903.91	0.00
2	1541.31#	1794.85		928.97	
3	1536.98#	1566.40	33.78	903.91	0.00
4	1541.31#	1794.85		928.97	
5	1539.44#	2700.86	2256.34	2642.86	0.00
6	1538.84#	2663.55		2643.62	
7	1539.44#	2700.86	2276.90	2642.86	0.00
8	1605.07	2663.55		2643.62	
9	1531.93#	2716.55	3492.18	2642.86	0.00
10	2290.12	3502.57		2704.62	
11	2762.73	3828.70	3452.52	2719.69	0.00
12	1528.48#	2663.11		2643.62	
13	1538.15	4537.92	1188.84	2762.64	0.00
14	1522.40#	2688.48		2643.62	
15	2842.53	3828.70	3338.52	2719.69	0.00
16	1528.48#	2663.11		2643.62	
17	1536.76#	2700.74	3378.18	2642.86	0.00
18	1948.12	2952.45		2665.28	
19	1539.44#	2700.86	1934.90	2642.86	0.00
20	1538.84#	2663.55		2643.62	
21	1539.44#	2700.86	1914.34	2642.86	0.00
22	1538.84#	2663.55		2643.62	
23	1536.98#	1566.40	33.78	903.91	0.00
24	1541.31#	1794.85		903.91	
25	1536.98#	1566.40	13.22	903.91	0.00
26	1541.31#	1794.85		928.97	

Maximum N* = 0.00 kN (Not considered)
Maximum My* = 0.00 kNm (Not considered)
Maximum Vx* = 0.00 kN (Not considered)

ZONE AND DEVELOPMENT LENGTHS (mm)

Zone	Start Position	Zone Development Length	Development Length Top	Development Length Bot
1	0.0	300.0	340.0	200.0
2	300.0	466.7	340.0	200.0
3	766.7	466.7	340.0	200.0
4	1233.3	466.7	810.0	200.0
5	1700.0	1200.0	880.0	390.0
6	2900.0	1400.0	200.0	915.0
7	4300.0	1400.0	200.0	1000.0
8	5700.0	1400.0	200.0	940.0
9	7100.0	1200.0	885.0	505.0
10	8300.0	466.7	690.0	200.0
11	8766.7	466.7	340.0	200.0
12	9233.3	466.7	340.0	200.0
13	9700.0	300.0	340.0	200.0

ZONE 1 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)
Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending

Positive Moment	Negative Moment
Mx* (+) = 1536.98 kNm #	Mx* (-) = 1541.31 ...
Ast.min (+) = 2494.6 mm ²	Ast.min (-) = 2490.5 mm ²
Ast (+) = 3217.0 mm ²	Ast (-) = 3694.5 mm ²
Ast (+)/Ag = 0.2%	Ast (-)/Ag = 0.2%
d = 1234.0 mm	d = 1236.0 mm
ku = 0.05	ku = 0.05 (8.1.3)
kuo = 0.05	kuo = 0.05 (8.1.5)
φ = 0.80	φ = 0.80 (Table 2.2.2)
φMuo (+) = 1566.40 kNm	φMuo (-) = 1794.85 kNm (8.1.5)
Mx* (+) ≤ φMuo (+) ⇒ Pass	Mx* (-) ≤ φMuo (-) ⇒ Pass

Transverse bar spacing = 255.0 mm

Shear

Tension @bot	Tension @top
β1 = 1.10	β1 = 1.10
β2 = 1.00	β2 = 1.00
β3 = 1.00	β3 = 1.00
φ = 0.70	φ = 0.70
φVu.min = 1160.62 kN	φVu.min = 1186.10 kN (8.2.9)
φVu.max = 8292.48 kN	φVu.max = 8292.48 kN (8.2.6)
φVuc = 505.04 kN	φVuc = 529.46 kN (8.2.7.1)
θv = 45.0°	θv = 45.0° (8.2.10(1))
Is shear req. (+) ?	No (8.2.5a)
Is shear req. (-) ?	Yes (8.2.5a)
Min shear req. (+) ?	No (8.2.5)
Min shear req. (-) ?	Yes (8.2.5)
Asv/sv needed by V*	= 0.000 mm ² /mm (8.2.10)
Asv/sv needed final	= 0.911 mm ² /mm (8.2.8)

Torsion

T* = 0.00 kNm	φ = 0.70
Jt = 624.00x10 ⁶ mm ⁴	φTu.max = 3494.40 kNm (8.3.3)
T*/φTu.max + Vy*/φVu.max = 0.00	(8.3.3)
φTuc = 828.77 kNm	(8.3.5(1))
T* > 0.25φTuc	No (8.3.4(1))
T*/φTuc + Vy*/φVuc > 0.5	No (8.3.4(2))
T*/φTuc + Vy*/φVuc > 1 and D < Max(250.0 mm & 0.5 bw)	No (8.3.4(2))
Tension @bot	Tension @top
φTus.min = 0.00 kNm	φTus.min = 0.00 kNm (8.3.7b(ii))
ΔAst = 0.0 mm ²	ΔAst = 0.0 mm ² (8.3.6)
ΔAsc = 0.0 mm ²	ΔAsc = 0.0 mm ² (8.3.6)
Torsional reinf. req. ?	No (8.3.4)
Asw/sw needed by T*	= 0.000 mm ² /mm (8.3.5)
Asw/sw min by shear	= 0.911 mm ² /mm (8.3.7)
Asw/sw min by 0.25φTuc	= 0.237 mm ² /mm
Asw/sw needed final	= 0.000 mm ² /mm (8.3.5)

Transverse shear and torsion reinforcement:

Asv/sv + 2 Asw/sw needed	= 0.911 mm ² /mm
Asv/sv + 2 Asw/sw provided	= 0.924 mm ² /mm
Asv/sv min by shear	= 0.911 mm ² /mm
Asw/sw min by 0.25φTuc	= 0.237 mm ² /mm
Asv/sv + 2 Asw/sw min	= 0.911 mm ² /mm
Stirrup spacing	= 255.0 mm
Max Spacing by shear	
is V* < φVumini ?	Yes
s2 (max) = Min (0.5D, 300.0)	= 300.0 mm

Bending (excluding longitudinal steel required by torsion)

Positive Moment	Negative Moment
Mx* (+) = 1536.98 kNm #	Mx* (-) = 1541.31 kNm #
Ast.min (+) = 2494.6 mm ²	Ast.min (-) = 2490.5 mm ²
Ast (+) = 3217.0 mm ²	Ast (-) = 3694.5 mm ²
Ast (+)/Ag = 0.2%	Ast (-)/Ag = 0.2%
d = 1234.0 mm	d = 1236.0 mm
ku = 0.05	ku = 0.05 (8.1.3)
kuo = 0.05	kuo = 0.05 (8.1.5)
φ = 0.80	φ = 0.80 (Table 2.2.2)
φMuo (+) = 1566.40 kNm	φMuo (-) = 1794.85 kNm (8.1.5)
Mx* (+) ≤ φMuo (+) ⇒ Pass	Mx* (-) ≤ φMuo (-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10 ⁹ mm ⁴	Icr (+) = 24.37x10 ⁹ mm ⁴	Icr (-) = 27.64x10 ⁹ mm ⁴
Short-term deflection (8.5.3.1)	Ief (+) = 131.82x10 ⁹ mm ⁴	Ief (-) = 131.82x10 ⁹ mm ⁴
Ms* (+) = 0.00 kNm	Ms* (-) = 0.00 kNm	
σcs (+) = 0.26 MPa	σcs (-) = 0.34 MPa	
Mcr (+) = 1193.10 kNm	Mcr (-) = 1167.21 kNm	
Long-term deflection (8.5.3.2)	Ief (+) = 131.82x10 ⁹ mm ⁴	Ief (-) = 131.82x10 ⁹ mm ⁴
Ms (+) = 0.00 kNm	Ms (-) = 0.00 kNm	
σcs (+) = 0.26 MPa	σcs (-) = 0.34 MPa	
Mcr (+) = 1193.10 kNm	Mcr (-) = 1167.21 kNm	

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ZONE 1 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 Ten @bot = 830.7 mm Ten @top = 595.6 mm
 Min tensile bar c-c spacing
 Ten @bot = 118.7 mm Ten @top = 119.1 mm
 Spacing limit check
 Warning (8.6.1b)
 Max steel stress db limit
 Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 Ten @bot = -264.53 MPa Ten @top = -76.44 MPa (Table 8.6.1.B)
 Max steel tensile stresses
 Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
 Stress limit check
 Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored
 Development Lengths (13.1.2.2)
 @Bot @Top
 Max db = 32.0 mm Max db = 28.0 mm
 k1 = 1.00 k1 = 1.30
 k2 = 1.00 k2 = 1.04
 k3 = 0.96 k3 = 0.94
 Lsy.cb = 696.0 mm Lsy.tb = 1294.6 mm (13.1.2.2)
 Lsc = 200.0 mm Lst = 336.0 mm (13.1.2.4)
 $\sigma_{st_flex.} = 0.0$ MPa $\sigma_{st_flex.} = 0.5$ MPa
 $\sigma_{sc_flex.} = 0.1$ MPa $\sigma_{sc_flex.} = 0.0$ MPa
 $\sigma_{s_torsion} = 0.6$ MPa $\sigma_{s_torsion} = 0.6$ MPa
 Stress reduction in compressive zone due to torsion is ignored (conservative)
 Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

ZONE 2 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)
 Bending
 Positive Moment
 Mx*(+) = 1536.98 kNm #
 Ast.min(+) = 2494.6 mm²
 Ast(+) = 3217.0 mm²
 Ast(+)/Ag = 0.2%
 d = 1234.0 mm
 ku = 0.05
 kuo = 0.05
 $\phi = 0.80$
 $\phi_{Mu0}(+) = 1566.40$ kNm
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass
 Negative Moment
 Mx*(-) = 1541.31 ...
 Ast.min(-) = 2490.5 mm²
 Ast(-) = 3694.5 mm²
 Ast(-)/Ag = 0.2%
 d = 1236.0 mm
 ku = 0.05 (8.1.3)
 kuo = 0.05 (8.1.5)
 $\phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(-) = 1794.85$ kNm (8.1.5)
 Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass
 Transverse bar spacing = 255.0 mm

Shear
 Tension @bot
 $\beta_1 = 1.10$
 $\beta_2 = 1.00$
 $\beta_3 = 1.00$
 $\phi = 0.70$
 $\phi_{Vu.min} = 1160.62$ kN
 $\phi_{Vu.max} = 8292.48$ kN
 $\phi_{Vuc} = 505.04$ kN
 $\theta_v = 45.0^\circ$
 Is shear req. (+) ? No
 Is shear req. (-) ? Yes
 Min shear req. (+) ? No
 Min shear req. (-) ? Yes
 Asv/sv needed by V* = 0.000 mm²/mm (8.2.10)
 Asv/sv needed final = 0.911 mm²/mm (8.2.8)
 Tension @top
 Vy* = 33.78 kN
 Tension @top
 $\beta_1 = 1.10$
 $\beta_2 = 1.00$
 $\beta_3 = 1.00$
 $\phi = 0.70$
 $\phi_{Vu.min} = 1186.10$ kN (8.2.9)
 $\phi_{Vu.max} = 8292.48$ kN (8.2.6)
 $\phi_{Vuc} = 529.46$ kN (8.2.7.1)
 $\theta_v = 45.0^\circ$ (8.2.10(1))
 No
 Yes (8.2.5a)
 No
 Yes (8.2.5)
 Yes (8.2.5)
 = 0.000 mm²/mm (8.2.10)
 = 0.911 mm²/mm (8.2.8)

Torsion
 T* = 0.00 kNm $\phi = 0.70$
 Jt = 624.00x10⁶ mm³ $\phi_{Tu.max} = 3494.40$ kNm (8.3.3)
 T*/ $\phi_{Tu.max} + Vy*/\phi_{Vu.max} = 0.00$ (8.3.3)
 $\phi_{Tuc} = 828.77$ kNm (8.3.5(1))
 T* > 0.25 ϕ_{Tuc} No (8.3.4(1))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 0.5$ No (8.3.4(2))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 1$ and D < Max(250.0 mm & 0.5 bw) No (8.3.4(2))
 Tension @bot
 $\phi_{Tus.min} = 0.00$ kNm $\phi_{Tus.min} = 0.00$ kNm (8.3.7b(ii))
 $\Delta A_{st} = 0.0$ mm² $\Delta A_{st} = 0.0$ mm² (8.3.6)
 $\Delta A_{sc} = 0.0$ mm² $\Delta A_{sc} = 0.0$ mm² (8.3.6)
 Torsional reinf. req. ? No (8.3.4)
 Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
 Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asw/sw needed final = 0.000 mm²/mm (8.3.5)

Transverse shear and torsion reinforcement:
 Asv/sv + 2 Asw/sw needed = 0.911 mm²/mm
 Asv/sv + 2 Asw/sw provided = 0.924 mm²/mm Pass
 Asv/sv min by shear = 0.911 mm²/mm
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asv/sv + 2 Asw/sw min = 0.911 mm²/mm
 Stirrup spacing = 255.0 mm Pass
 Max Spacing by shear
 is V* < ϕ_{Vmin} ? Yes
 s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)
 Positive Moment
 Mx*(+) = 1536.98 kNm #
 Ast.min(+) = 2494.6 mm²
 Ast(+) = 3217.0 mm²
 Ast(+)/Ag = 0.2%
 d = 1234.0 mm
 ku = 0.05 (8.1.3)
 kuo = 0.05 (8.1.5)
 $\phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(+) = 1566.40$ kNm (8.1.5)
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass
 Negative Moment
 Mx*(-) = 1541.31 kNm #
 Ast.min(-) = 2490.5 mm²
 Ast(-) = 3694.5 mm²
 Ast(-)/Ag = 0.2%
 d = 1236.0 mm
 ku = 0.05 (8.1.3)
 kuo = 0.05 (8.1.5)
 $\phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(-) = 1794.85$ kNm (8.1.5)
 Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass

ZONE 2 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Deflections (no serviceability load cases selected)
 Ig = 219.70x10⁹ mm⁴
 Icr(+) = 24.37x10⁹ mm⁴ Icr(-) = 27.64x10⁹ mm⁴
 Short-term deflection (8.5.3.1)
 Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
 Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
 $\sigma_{cs}(+) = 0.26$ MPa $\sigma_{cs}(-) = 0.34$ MPa
 Mcr(+) = 1193.10 kNm Mcr(-) = 1167.21 kNm
 Long-term deflection (8.5.3.2)
 Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
 Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
 $\sigma_{cs}(+) = 0.26$ MPa $\sigma_{cs}(-) = 0.34$ MPa
 Mcr(+) = 1193.10 kNm Mcr(-) = 1167.21 kNm
 Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 Ten @bot = 830.7 mm Ten @top = 595.6 mm
 Min tensile bar c-c spacing
 Ten @bot = 118.7 mm Ten @top = 119.1 mm
 Spacing limit check
 Warning (8.6.1b)
 Max steel stress db limit
 Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 Ten @bot = -264.53 MPa Ten @top = -76.44 MPa (Table 8.6.1.B)
 Max steel tensile stresses
 Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
 Stress limit check
 Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored
 Development Lengths (13.1.2.2)
 @Bot @Top
 Max db = 32.0 mm Max db = 28.0 mm
 k1 = 1.00 k1 = 1.30
 k2 = 1.00 k2 = 1.04
 k3 = 0.96 k3 = 0.94
 Lsy.cb = 696.0 mm Lsy.tb = 1294.6 mm (13.1.2.2)
 Lsc = 200.0 mm Lst = 336.0 mm (13.1.2.4)
 $\sigma_{st_flex.} = 0.0$ MPa $\sigma_{st_flex.} = 0.5$ MPa
 $\sigma_{sc_flex.} = 0.4$ MPa $\sigma_{sc_flex.} = 0.0$ MPa
 $\sigma_{s_torsion} = 0.6$ MPa $\sigma_{s_torsion} = 0.6$ MPa
 Stress reduction in compressive zone due to torsion is ignored (conservative)
 Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

ZONE 3 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)
 Bending
 Positive Moment
 Mx*(+) = 1539.44 kNm #
 Ast.min(+) = 2494.6 mm²
 Ast(+) = 3629.7 mm²
 Ast(+)/Ag = 0.4%
 d = 1234.0 mm
 ku = 0.06 (8.1.3)
 kuo = 0.06 (8.1.5)
 $\phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(+) = 2700.86$ kNm (8.1.5)
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass
 Negative Moment
 Mx*(-) = 1538.84 ...
 Ast.min(-) = 2490.5 mm²
 Ast(-) = 5541.8 mm²
 Ast(-)/Ag = 0.4%
 d = 1236.0 mm
 ku = 0.06 (8.1.3)
 kuo = 0.06 (8.1.5)
 $\phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(-) = 2663.55$ kNm (8.1.5)
 Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass
 Transverse bar spacing = 50.0 mm
 Shear
 Tension @bot
 $\beta_1 = 1.10$
 $\beta_2 = 1.00$
 $\beta_3 = 1.00$
 $\phi = 0.70$
 $\phi_{Vu.min} = 1264.19$ kN
 $\phi_{Vu.max} = 8292.48$ kN
 $\phi_{Vuc} = 608.61$ kN
 $\theta_v = 45.0^\circ$
 Is shear req. (+) ? No
 Is shear req. (-) ? Yes
 Min shear req. (+) ? No
 Min shear req. (-) ? Yes
 Asv/sv needed by V* = 5.216 mm²/mm (8.2.10)
 Asv/sv needed final = 5.216 mm²/mm (8.2.8)
 Tension @top
 Vy* = 2256.34 kN
 Tension @top
 $\beta_1 = 1.10$
 $\beta_2 = 1.00$
 $\beta_3 = 1.00$
 $\phi = 0.70$
 $\phi_{Vu.min} = 656.64$ kN (8.2.9)
 $\phi_{Vu.max} = 8292.48$ kN (8.2.6)
 $\phi_{Vuc} = 606.08$ kN (8.2.7.1)
 $\theta_v = 45.0^\circ$ (8.2.10(1))
 No
 Yes (8.2.5a)
 No
 Yes (8.2.5)
 = 5.216 mm²/mm (8.2.10)
 = 5.216 mm²/mm (8.2.8)

Torsion
 T* = 0.00 kNm $\phi = 0.70$
 Jt = 624.00x10⁶ mm³ $\phi_{Tu.max} = 3494.40$ kNm (8.3.3)
 T*/ $\phi_{Tu.max} + Vy*/\phi_{Vu.max} = 0.27$ (8.3.3)
 $\phi_{Tuc} = 828.77$ kNm (8.3.5(1))
 T* > 0.25 ϕ_{Tuc} No (8.3.4(1))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 0.5$ No (8.3.4(2))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 1$ and D < Max(250.0 mm & 0.5 bw) No (8.3.4(2))
 Tension @bot
 $\phi_{Tus.min} = 0.00$ kNm $\phi_{Tus.min} = 207.19$ kNm (8.3.7b(ii))
 $\Delta A_{st} = 0.0$ mm² $\Delta A_{st} = 2038.2$ mm² (8.3.6)
 $\Delta A_{sc} = 0.0$ mm² $\Delta A_{sc} = 0.0$ mm² (8.3.6)
 Torsional reinf. req. ? No (8.3.4)
 Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
 Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asw/sw needed final = 0.911 mm²/mm (8.3.5)

Transverse shear and torsion reinforcement:
 Asv/sv + 2 Asw/sw needed = 7.037 mm²/mm
 Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
 Asv/sv min by shear = 0.911 mm²/mm
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asv/sv + 2 Asw/sw min = 0.911 mm²/mm
 Stirrup spacing = 50.0 mm Pass
 Max Spacing by shear
 is V* < ϕ_{Vmin} ? No
 s2 (max) = Min (0.5D, 300.0) = 300.0 mm

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ZONE 3 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Bending (excluding longitudinal steel required by torsion)
Positive Moment
Mx*(+) = 1539.49 kNm #
Ast.min(+) = 2488.1 mm²
Ast(+) = 3591.5 mm²
Ast(+)/Ag = 0.2%
d = 1237.2 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(+) = 1746.64 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1538.80 kNm #
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(-) = 1707.82 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 40.24x10⁹ mm⁴ Icr(-) = 39.82x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm

Crack Control (no serviceability load cases selected)

Max tensile bar c-c spacing
Ten @bot = 356.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm
Spacing limit check
Pass (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa
Max steel stress c-c spacing limit
Ten @bot = 115.20 MPa
Max steel tensile stresses
Ten @bot = 0.00 MPa
Stress limit check
Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths

@Bot
Max db = 32.0 mm
k1 = 1.00
k2 = 1.00
k3 = 0.96
Lsy.cb = 696.0 mm
Lsc = 200.0 mm
σst.flex. = 0.0 MPa
σsc.flex. = 13.2 MPa
σs.torsion = 0.4 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

@Top
Max db = 28.0 mm
k1 = 1.30
k2 = 1.04
k3 = 0.94
Lsy.tb = 1294.6 mm
Lst = 336.0 mm
σst.flex. = 84.9 MPa
σsc.flex. = 0.0 MPa
σs.torsion = 0.4 MPa

Fire Resistance
Min req am (continuous) = 12.0 mm (5.4.1(b))
Calculated am = 50.0 mm (5.2.1)

ZONE 4 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment
Mx*(+) = 1539.44 kNm #
Ast.min(+) = 2494.6 mm²
Ast(+) = 5629.7 mm²
Ast(+)/Ag = 0.4%
d = 1234.0 mm
ku = 0.06
kuo = 0.06
φ = 0.80
φMu(+) = 2700.86 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1605.07 kNm
Ast.min(-) = 2490.5 mm²
Ast(-) = 5541.8 mm²
Ast(-)/Ag = 0.4%
d = 1236.0 mm
ku = 0.06
kuo = 0.06
φ = 0.80
φMu(-) = 2663.55 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 1264.19 kN
φVu.max = 8292.48 kN
φVuc = 608.61 kN
θv = 45.0°
Is shear req. (+) ?
Is shear req. (-) ?
Min shear req. (+) ?
Min shear req. (-) ?
Asv/sv needed by V*
Asv/sv needed final

Vy* = 2276.90 kN
Tension @top
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 656.64 kN
φVu.max = 8292.48 kN
φVuc = 606.08 kN
θv = 45.0°
No
Yes
No
Yes
= 5.263 mm²/mm
= 5.263 mm²/mm

Torsion
T* = 0.00 kNm
Jt = 624.00x10⁶ mm⁴
T*/φTu.max + Vy*/φVu.max = 0.27
φTuc = 828.77 kNm
T* > 0.25φTuc
T*/φTuc + Vy*/φVuc > 0.5
T*/φTuc + Vy*/φVuc > 1 and D < Max(250.0 mm & 0.5 bw)
Tension @bot
φTus.min = 0.00 kNm
ΔAst = 0.0 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ?
Asw/sw needed by T*
Asw/sw min by shear
Asw/sw min by 0.25φTuc
Asw/sw needed final

φ = 0.70
φTu.max = 3494.40 kNm
(8.3.3)
(8.3.3)
(8.3.5(1))
(8.3.4(1))
(8.3.4(2))
(8.3.4(2))
Tension @top
φTus.min = 207.19 kNm
ΔAst = 2038.2 mm²
ΔAsc = 0.0 mm²
No
= 0.000 mm²/mm
= 0.911 mm²/mm
= 0.237 mm²/mm
= 0.911 mm²/mm
(8.3.7b(ii))
(8.3.6)
(8.3.6)
(8.3.4)
(8.3.5)
(8.3.7)
(8.3.5)
(8.3.5)

ZONE 4 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Transverse shear and torsion reinforcement:
Asv/sv + 2 Asw/sw needed = 7.085 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 0.911 mm²/mm
Stirrup spacing = 50.0 mm Pass
Max Spacing by shear
is V* < φVum in ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)

Positive Moment
Mx*(+) = 1539.49 kNm #
Ast.min(+) = 2488.1 mm²
Ast(+) = 3591.5 mm²
Ast(+)/Ag = 0.2%
d = 1237.2 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(+) = 1746.64 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1605.07 kNm
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(-) = 1707.82 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 40.24x10⁹ mm⁴ Icr(-) = 39.82x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm

Crack Control (no serviceability load cases selected)

Max tensile bar c-c spacing
Ten @bot = 356.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm
Spacing limit check
Pass (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa
Max steel stress c-c spacing limit
Ten @bot = 115.20 MPa
Max steel tensile stresses
Ten @bot = 0.00 MPa
Stress limit check
Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths

@Bot
Max db = 32.0 mm
k1 = 1.00
k2 = 1.00
k3 = 0.96
Lsy.cb = 696.0 mm
Lsc = 200.0 mm
σst.flex. = 0.0 MPa
σsc.flex. = 38.7 MPa
σs.torsion = 0.4 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

@Top
Max db = 28.0 mm
k1 = 1.30
k2 = 1.04
k3 = 0.94
Lsy.tb = 1294.6 mm
Lst = 806.6 mm
σst.flex. = 248.9 MPa
σsc.flex. = 0.0 MPa
σs.torsion = 0.4 MPa

Fire Resistance
Min req am (continuous) = 12.0 mm (5.4.1(b))
Calculated am = 50.0 mm (5.2.1)

ZONE 5 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment
Mx*(+) = 1531.93 kNm #
Ast.min(+) = 2494.6 mm²
Ast(+) = 5629.7 mm²
Ast(+)/Ag = 0.4%
d = 1234.0 mm
ku = 0.06
kuo = 0.06
φ = 0.80
φMu(+) = 2716.55 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 2290.12 kNm
Ast.min(-) = 2508.5 mm²
Ast(-) = 7389.0 mm²
Ast(-)/Ag = 0.5%
d = 1227.2 mm
ku = 0.08
kuo = 0.08
φ = 0.80
φMu(-) = 3502.57 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 655.58 kN
φVu.max = 8292.48 kN
φVuc = 608.61 kN
Because there is torsion in the section:
φVuc = 0.00 kN
θv = 45.0°
Is shear req. (+) ?
Is shear req. (-) ?
Min shear req. (+) ?
Min shear req. (-) ?
Asv/sv needed by V*
Asv/sv needed final

Vy* = 3492.18 kN
Tension @top
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 656.64 kN
φVu.max = 8292.48 kN
φVuc = 667.07 kN
φVuc = 0.00 kN
θv = 45.0°
Yes
Yes
Yes
Yes
= 8.086 mm²/mm
= 0.911 mm²/mm
= 8.086 mm²/mm
(8.2.9)
(8.2.6)
(8.2.7.1)
(8.3.4)
(8.2.10(1))
(8.2.5a)
(8.2.5a)
(8.2.5)
(8.2.5)
(8.2.10)
(8.2.8)
(8.2.8)

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ZONE 5 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Torsion
T* = 0.00 kNm ϕ = 0.70
Jt = 624.00x10⁶ mm³ ϕ Tu.max = 3494.40 kNm (8.3.3)
T*/ ϕ Tu.max + Vy*/ ϕ Vu.max = 0.42 (8.3.3)
 ϕ Tuc = 828.77 kNm (8.3.5(1))
T* > 0.25 ϕ Tuc No (8.3.4(1))
T*/ ϕ Tuc + Vy*/ ϕ Vuc > 0.5 Yes (8.3.4(2))
T*/ ϕ Tuc + Vy*/ ϕ Vuc > 1 and D < Max(250.0 mm & 0.5 bw) No (8.3.4(2))

Tension @bot
 ϕ Tus.min = 207.19 kNm ϕ Tus.min = 207.19 kNm (8.3.7b(ii))
 Δ Ast = 2038.2 mm² Δ Ast = 2038.2 mm² (8.3.6)
 Δ Asc = 0.0 mm² Δ Asc = 0.0 mm² (8.3.6)
Torsional reinf. req. ? Yes (8.3.4)
Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
Asw/sw min by 0.25 ϕ Tuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm (8.3.5)

Transverse shear and torsion reinforcement:
Asv/sv + 2 Asw/sw needed = 9.907 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25 ϕ Tuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
Stirrup spacing = 50.0 mm Pass
Max Spacing by torsion
s1 (max) = Min (0.12Ut, 300.0) = 300.0 mm
Max Spacing by shear
is V* < ϕ Vumin ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)
Positive Moment Negative Moment
Mx*(+) = 1531.71 kNm # Mx*(-) = 2290.12 kNm
Ast.min(+) = 2488.1 mm² Ast.min(-) = 2502.8 mm²
Ast(+) = 3591.5 mm² Ast(-) = 5350.8 mm²
Ast(+)/Ag = 0.2% Ast(-)/Ag = 0.3%
d = 1237.2 mm d = 1229.9 mm
ku = 0.05 ku = 0.06 (8.1.3)
kuo = 0.05 kuo = 0.06 (8.1.5)
 ϕ = 0.8 ϕ = 0.8 (Table 2.2.2)
 ϕ Muo(+) = 1763.41 kNm ϕ Muo(-) = 2561.95 kNm (8.1.5)
Mx*(+) ≤ ϕ Muo(+) ⇒ Pass Mx*(-) ≤ ϕ Muo(-) ⇒ Pass

Deflections (no serviceability load cases selected)
Ig = 219.70x10⁹ mm⁴
Icr(+) = 40.37x10⁹ mm⁴ Icr(-) = 50.24x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 219.70x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
 σ cs(+) = 0.40 MPa σ cs(-) = 0.65 MPa
Mcr(+) = 1148.79 kNm Mcr(-) = 1063.53 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 219.70x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
 σ cs(+) = 0.40 MPa σ cs(-) = 0.65 MPa
Mcr(+) = 1148.79 kNm Mcr(-) = 1063.53 kNm

Crack Control (no serviceability load cases selected)
Max tensile bar c-c spacing
Ten @bot = 356.0 mm Ten @top = 1072.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm Ten @top = 119.1 mm
Spacing limit check
Warning (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
Max steel stress c-c spacing limit
Ten @bot = 115.20 MPa Ten @top = 304.71 MPa (Table 8.6.1.B)
Max steel tensile stresses
Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
Stress limit check Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths (13.1.2.2)
@Bot @Top
Max db = 32.0 mm Max db = 28.0 mm
k1 = 1.00 k1 = 1.30
k2 = 1.00 k2 = 1.04
k3 = 0.96 k3 = 0.94
Lsy.tb = 1217.5 mm Lsy.tb = 1294.6 mm (13.1.2.2)
Lst = 390.0 mm Lst = 876.4 mm (13.1.2.4)
 σ st.flex. = 127.8 MPa σ st.flex. = 270.5 MPa
 σ sc.flex. = 52.2 MPa σ sc.flex. = 18.7 MPa
 σ s_torsion = 0.4 MPa σ s_torsion = 0.3 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

Fire Resistance (5)
Min req am (continuous) = 12.0 mm (5.4.1(b))
Calculated am = 50.0 mm (5.2.1)

ZONE 6 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment Negative Moment
Mx*(+) = 2762.73 kNm Mx*(-) = 1528.48 ...
Ast.min(+) = 2494.6 mm² Ast.min(-) = 2490.5 mm²
Ast(+) = 8042.5 mm² Ast(-) = 5541.8 mm²
Ast(+)/Ag = 0.5% Ast(-)/Ag = 0.4%
d = 1224.0 mm d = 1236.0 mm
ku = 0.08 ku = 0.06 (8.1.3)
kuo = 0.08 kuo = 0.06 (8.1.5)
 ϕ = 0.80 ϕ = 0.80 (Table 2.2.2)
 ϕ Muo(+) = 3828.70 kNm ϕ Muo(-) = 2663.11 kNm (8.1.5)
Mx*(+) ≤ ϕ Muo(+) ⇒ Pass Mx*(-) ≤ ϕ Muo(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

ZONE 6 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Shear Vy* = 3452.52 kN
Tension @bot Vy* = 3452.52 kN
 β 1 = 1.10 β 1 = 1.10
 β 2 = 1.00 β 2 = 1.00
 β 3 = 1.00 β 3 = 1.00
 ϕ = 0.70 ϕ = 0.70
 ϕ Vu.min = 655.58 kN ϕ Vu.min = 1262.72 kN (8.2.9)
 ϕ Vu.max = 8292.48 kN ϕ Vu.max = 8292.48 kN (8.2.6)
 ϕ Vuc = 685.44 kN ϕ Vuc = 606.08 kN (8.2.7.1)
Because there is torsion in the section:
 ϕ Vuc = 0.00 kN ϕ Vuc = 0.00 kN (8.3.4)
 θ v = 45.0° θ v = 45.0° (8.2.10(1))
Is shear reo req. (+) ? Yes (8.2.5a)
Is shear reo req. (-) ? No (8.2.5a)
Min shear reo req. (+) ? Yes (8.2.5)
Min shear reo req. (-) ? No (8.2.5)
Asv/sv needed by V* = 7.994 mm²/mm (8.2.10)
Asv/sv min = 0.911 mm²/mm (8.2.8)
Asv/sv needed final = 7.994 mm²/mm (8.2.8)

Torsion
T* = 0.00 kNm ϕ = 0.70
Jt = 624.00x10⁶ mm³ ϕ Tu.max = 3494.40 kNm (8.3.3)
T*/ ϕ Tu.max + Vy*/ ϕ Vu.max = 0.42 (8.3.3)
 ϕ Tuc = 828.77 kNm (8.3.5(1))
T* > 0.25 ϕ Tuc No (8.3.4(1))
T*/ ϕ Tuc + Vy*/ ϕ Vuc > 0.5 Yes (8.3.4(2))
T*/ ϕ Tuc + Vy*/ ϕ Vuc > 1 and D < Max(250.0 mm & 0.5 bw) No (8.3.4(2))

Tension @bot Tension @top
 ϕ Tus.min = 207.19 kNm ϕ Tus.min = 0.00 kNm (8.3.7b(ii))
 Δ Ast = 2038.2 mm² Δ Ast = 0.0 mm² (8.3.6)
 Δ Asc = 0.0 mm² Δ Asc = 0.0 mm² (8.3.6)
Torsional reinf. req. ? Yes (8.3.4)
Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
Asw/sw min by 0.25 ϕ Tuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm (8.3.5)

Transverse shear and torsion reinforcement:
Asv/sv + 2 Asw/sw needed = 9.815 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25 ϕ Tuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
Stirrup spacing = 50.0 mm Pass
Max Spacing by torsion
s1 (max) = Min (0.12Ut, 300.0) = 300.0 mm
Max Spacing by shear
is V* < ϕ Vumin ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)
Positive Moment Negative Moment
Mx*(+) = 2762.73 kNm Mx*(-) = 1528.29 kNm #
Ast.min(+) = 2490.2 mm² Ast.min(-) = 2484.8 mm²
Ast(+) = 6004.3 mm² Ast(-) = 3503.5 mm²
Ast(+)/Ag = 0.4% Ast(-)/Ag = 0.2%
d = 1236.2 mm d = 1238.9 mm
ku = 0.07 ku = 0.05 (8.1.3)
kuo = 0.07 kuo = 0.05 (8.1.5)
 ϕ = 0.8 ϕ = 0.8 (Table 2.2.2)
 ϕ Muo(+) = 2883.02 kNm ϕ Muo(-) = 1708.58 kNm (8.1.5)
Mx*(+) ≤ ϕ Muo(+) ⇒ Pass Mx*(-) ≤ ϕ Muo(-) ⇒ Pass

Deflections (no serviceability load cases selected)
Ig = 219.70x10⁹ mm⁴
Icr(+) = 54.63x10⁹ mm⁴ Icr(-) = 40.11x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
 σ cs(+) = 0.71 MPa σ cs(-) = 0.36 MPa
Mcr(+) = 1042.14 kNm Mcr(-) = 1160.83 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
 σ cs(+) = 0.71 MPa σ cs(-) = 0.36 MPa
Mcr(+) = 1042.14 kNm Mcr(-) = 1160.83 kNm

Crack Control (no serviceability load cases selected)
Max tensile bar c-c spacing
Ten @bot = 118.7 mm Ten @top = 238.2 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm Ten @top = 119.1 mm
Spacing limit check Pass (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
Max steel stress c-c spacing limit
Ten @bot = 305.07 MPa Ten @top = 209.42 MPa (Table 8.6.1.B)
Max steel tensile stresses
Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
Stress limit check Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths (13.1.2.2)
@Bot @Top
Max db = 32.0 mm Max db = 28.0 mm
k1 = 1.00 k1 = 1.30
k2 = 1.00 k2 = 1.04
k3 = 0.96 k3 = 0.94
Lsy.tb = 1217.5 mm Lsy.cb = 609.0 mm (13.1.2.2)
Lst = 911.1 mm Lst = 200.0 mm (13.1.2.4)
 σ st.flex. = 299.1 MPa σ st.flex. = 0.0 MPa
 σ sc.flex. = 0.0 MPa σ sc.flex. = 61.7 MPa
 σ s_torsion = 0.3 MPa σ s_torsion = 0.4 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

Fire Resistance (5)
Min req am (continuous) = 12.0 mm (5.4.1(b))
Calculated am = 50.0 mm (5.2.1)

ZONE 7 (Critical Zone) (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

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ZONE 7 (Critical Zone) (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Bending
Positive Moment
Mx*(+) = 3584.15 kNm
Ast.min(+) = 2513.9 mm²
Ast(+) = 9651.0 mm²
Ast(+)/Ag = 0.6%
d = 1224.5 mm
ku = 0.09
kuo = 0.09
φ = 0.80
φMu(+) = 4537.92 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1522.40 ...
Ast.min(-) = 2490.5 mm²
Ast(-) = 5541.8 mm²
Ast(-)/Ag = 0.4%
d = 1236.0 mm
ku = 0.06 (8.1.3)
kuo = 0.06 (8.1.5)
φ = 0.80 (Table 2.2.2)
φMu(-) = 2688.48 kNm (8.1.5)
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 655.58 kN
φVu.max = 8292.48 kN
φVuc = 728.39 kN
Because there is torsion in the section:
φVuc = 0.00 kN
θv = 45.0°
Is shear req. (+) ? Yes
Is shear req. (-) ? No
Min shear req. (+) ? Yes
Min shear req. (-) ? No
Asv/sv needed by V* = 2.753 mm²/mm
Asv/sv min = 0.911 mm²/mm
Asv/sv needed final = 2.753 mm²/mm

Torsion
T* = 0.00 kNm
Jt = 624.00x10⁶ mm³
T*/φTu.max + Vy*/φVu.max = 0.14
φTuc = 828.77 kNm
T* > 0.25φTuc
T*/φTuc + Vy*/φVuc > 0.5
T*/φTuc + Vy*/φVuc > 1 and D < Max(250.0 mm & 0.5 bw)

Tension @bot
φTus.min = 207.19 kNm
ΔAst = 2038.2 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ? Yes
Asw/sw needed by T* = 0.000 mm²/mm
Asw/sw min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm

Tension @top
φTus.min = 0.00 kNm
ΔAst = 0.0 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ? No
Asw/sw needed by T* = 0.000 mm²/mm
Asw/sw min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm

Tranverse shear and torsion reinforcement:

Asv/sv + 2 Asw/sw needed = 4.574 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
Stirrup spacing = 50.0 mm
Max Spacing by torsion
s1 (max) = Min (0.12Ut, 300.0) = 300.0 mm
Max Spacing by shear
is V* < φVumin ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)

Positive Moment
Mx*(+) = 3584.15 kNm
Ast.min(+) = 2509.0 mm²
Ast(+) = 7612.7 mm²
Ast(+)/Ag = 0.5%
d = 1226.9 mm
ku = 0.08
kuo = 0.08
φ = 0.8
φMu(+) = 3607.01 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1521.95 kNm #
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05 (8.1.3)
kuo = 0.05 (8.1.5)
φ = 0.8 (Table 2.2.2)
φMu(-) = 1735.14 kNm (8.1.5)
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 62.58x10⁹ mm⁴ Icr(-) = 40.16x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.86 MPa σcs(-) = 0.30 MPa
Mcr(+) = 991.12 kNm Mcr(-) = 1181.95 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
σcs(+) = 0.86 MPa σcs(-) = 0.30 MPa
Mcr(+) = 991.12 kNm Mcr(-) = 1181.95 kNm

Crack Control (no serviceability load cases selected)

Max tensile bar c-c spacing
Ten @bot = 1068.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm
Spacing limit check
Warning
Max steel stress db limit
Ten @bot = 160.00 MPa
Max steel stress c-c spacing limit
Ten @bot = 305.07 MPa
Max steel tensile stresses
Ten @bot = 0.00 MPa
Stress limit check
Pass
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Ten @top = 238.2 mm
Ten @top = 119.1 mm
(8.6.1b)
Ten @top = 184.00 MPa (Table 8.6.1.A)
Ten @top = 209.42 MPa (Table 8.6.1.B)
Ten @top = 0.00 MPa
(Table 8.6.1A/B)

ZONE 7 (Critical Zone) (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Development Lengths
@Bot
Max db = 32.0 mm
k1 = 1.00
k2 = 1.00
k3 = 0.96
Lsy.tb = 1217.5 mm
Lst = 998.6 mm
σst.flex. = 327.9 MPa
σsc.flex. = 0.0 MPa
σs.torsion = 0.2 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

@Top
Max db = 28.0 mm
k1 = 1.30
k2 = 1.04
k3 = 0.94
Lsy.cb = 609.0 mm (13.1.2.2)
Lsc = 200.0 mm (13.1.2.4)
σst.flex. = 0.0 MPa
σsc.flex. = 77.4 MPa
σs.torsion = 0.4 MPa

Fire Resistance
Min req am (continuous) = 12.0 mm
Calculated am = 52.7 mm (5)
(5.4.1(b))
(5.2.1)

ZONE 8 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)
Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment
Mx*(+) = 2842.53 kNm
Ast.min(+) = 2494.6 mm²
Ast(+) = 8042.5 mm²
Ast(+)/Ag = 0.5%
d = 1234.0 mm
ku = 0.08
kuo = 0.08
φ = 0.80
φMu(+) = 3828.70 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1528.48 ...
Ast.min(-) = 2490.5 mm²
Ast(-) = 5541.8 mm²
Ast(-)/Ag = 0.4%
d = 1236.0 mm
ku = 0.06 (8.1.3)
kuo = 0.06 (8.1.5)
φ = 0.80 (Table 2.2.2)
φMu(-) = 2663.11 kNm (8.1.5)
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 655.58 kN
φVu.max = 8292.48 kN
φVuc = 685.44 kN
Because there is torsion in the section:
φVuc = 0.00 kN
θv = 45.0°
Is shear req. (+) ? Yes
Is shear req. (-) ? No
Min shear req. (+) ? Yes
Min shear req. (-) ? No
Asv/sv needed by V* = 7.730 mm²/mm
Asv/sv min = 0.911 mm²/mm
Asv/sv needed final = 7.730 mm²/mm

Tension @top
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 1262.72 kN (8.2.9)
φVu.max = 8292.48 kN (8.2.6)
φVuc = 606.08 kN (8.2.7.1)
φVuc = 0.00 kN (8.3.4)
θv = 45.0° (8.2.10(1))
Is shear req. (+) ? Yes (8.2.5a)
Is shear req. (-) ? No (8.2.5a)
Min shear req. (+) ? Yes (8.2.5)
Min shear req. (-) ? No (8.2.5)
Asv/sv needed by V* = 7.730 mm²/mm (8.2.10)
Asv/sv min = 0.911 mm²/mm (8.2.8)
Asv/sv needed final = 7.730 mm²/mm (8.2.8)

Torsion
T* = 0.00 kNm
Jt = 624.00x10⁶ mm³
T*/φTu.max + Vy*/φVu.max = 0.40
φTuc = 828.77 kNm
T* > 0.25φTuc
T*/φTuc + Vy*/φVuc > 0.5
T*/φTuc + Vy*/φVuc > 1 and D < Max(250.0 mm & 0.5 bw)

Tension @bot
φTus.min = 207.19 kNm
ΔAst = 2038.2 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ? Yes
Asw/sw needed by T* = 0.000 mm²/mm
Asw/sw min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm

Tension @top
φTus.min = 0.00 kNm
ΔAst = 0.0 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ? No
Asw/sw needed by T* = 0.000 mm²/mm
Asw/sw min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asw/sw needed final = 0.911 mm²/mm

Tranverse shear and torsion reinforcement:

Asv/sv + 2 Asw/sw needed = 9.551 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
Stirrup spacing = 50.0 mm
Max Spacing by torsion
s1 (max) = Min (0.12Ut, 300.0) = 300.0 mm
Max Spacing by shear
is V* < φVumin ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)

Positive Moment
Mx*(+) = 2842.53 kNm
Ast.min(+) = 2490.2 mm²
Ast(+) = 6004.3 mm²
Ast(+)/Ag = 0.4%
d = 1236.2 mm
ku = 0.07
kuo = 0.07
φ = 0.8
φMu(+) = 2883.02 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1528.29 kNm #
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05 (8.1.3)
kuo = 0.05 (8.1.5)
φ = 0.8 (Table 2.2.2)
φMu(-) = 1708.58 kNm (8.1.5)
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 54.63x10⁹ mm⁴ Icr(-) = 40.11x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.71 MPa σcs(-) = 0.36 MPa
Mcr(+) = 1042.14 kNm Mcr(-) = 1160.83 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 219.70x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms(+) = 0.00 kNm Ms(-) = 0.00 kNm
σcs(+) = 0.71 MPa σcs(-) = 0.36 MPa
Mcr(+) = 1042.14 kNm Mcr(-) = 1160.83 kNm

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ZONE 8 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 Ten @bot = 118.7 mm Ten @top = 238.2 mm
 Min tensile bar c-c spacing
 Ten @bot = 118.7 mm Ten @top = 119.1 mm
 Spacing limit check Pass (8.6.1b)
 Max steel stress db limit
 Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 Ten @bot = 305.07 MPa Ten @top = 209.42 MPa (Table 8.6.1.B)
 Max steel tensile stresses
 Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
 Stress limit check Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored
 Development Lengths (13.1.2.2)
 @Bot @Top
 Max db = 32.0 mm Max db = 28.0 mm
 k1 = 1.00 k1 = 1.30
 k2 = 1.00 k2 = 1.04
 k3 = 0.96 k3 = 0.94
 Lsy.tb = 1217.5 mm Lsy.cb = 609.0 mm (13.1.2.2)
 Lst = 937.4 mm Lsc = 200.0 mm (13.1.2.4)
 $\sigma_{st_flex.} = 307.7$ MPa $\sigma_{st_flex.} = 0.0$ MPa
 $\sigma_{sc_flex.} = 0.0$ MPa $\sigma_{sc_flex.} = 63.5$ MPa
 $\sigma_{s_torsion} = 0.3$ MPa $\sigma_{s_torsion} = 0.4$ MPa
 Stress reduction in compressive zone due to torsion is ignored (conservative)
 Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

ZONE 9 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)
 Bending
 Positive Moment Negative Moment
 Mx*(+) = 1536.76 kNm # Mx*(-) = 1948.12 kNm
 Ast.min(+) = 2494.6 mm² Ast.min(-) = 2490.5 mm²
 Ast(+) = 5629.7 mm² Ast(-) = 6157.5 mm²
 Ast(+)/Ag = 0.4% Ast(-)/Ag = 0.4%
 d = 1234.0 mm d = 1236.0 mm
 ku = 0.06 ku = 0.06 (8.1.3)
 kuo = 0.06 kuo = 0.06 (8.1.5)
 $\phi = 0.80 \phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(+) = 2700.74$ kNm $\phi_{Mu0}(-) = 2952.45$ kNm (8.1.5)
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass
 Transverse bar spacing = 50.0 mm

Shear
 Tension @bot Vy* = 3378.18 kN
 $\beta_1 = 1.10 \beta_1 = 1.10$
 $\beta_2 = 1.00 \beta_2 = 1.00$
 $\beta_3 = 1.00 \beta_3 = 1.00$
 $\phi = 0.70 \phi = 0.70$
 $\phi_{Vu.min} = 655.58$ kN $\phi_{Vu.min} = 656.64$ kN (8.2.9)
 $\phi_{Vu.max} = 8292.48$ kN $\phi_{Vu.max} = 8292.48$ kN (8.2.6)
 $\phi_{Vuc} = 608.61$ kN $\phi_{Vuc} = 627.74$ kN (8.2.7.1)
 Because there is torsion in the section:
 $\phi_{Vuc} = 0.00$ kN $\phi_{Vuc} = 0.00$ kN (8.3.4)
 $\theta_v = 45.0^\circ \theta_v = 45.0^\circ$ (8.2.10(ii))
 Is shear req. (+) ? Yes (8.2.5a)
 Is shear req. (-) ? Yes (8.2.5a)
 Min shear req. (+) ? Yes (8.2.5)
 Min shear req. (-) ? Yes (8.2.5)
 Asv/sv needed by V* = 7.822 mm²/mm (8.2.10)
 Asv/sv min = 0.911 mm²/mm (8.2.8)
 Asv/sv needed final = 7.822 mm²/mm (8.2.8)

Torsion
 T* = 0.00 kNm $\phi = 0.70$
 Jt = 624.00x10⁶ mm⁴ $\phi_{Tu.max} = 3494.40$ kNm (8.3.3)
 T*/ $\phi_{Tu.max} + Vy*/\phi_{Vu.max} = 0.41$ (8.3.3)
 $\phi_{Tuc} = 828.77$ kNm (8.3.5(ii))
 T* > 0.25 ϕ_{Tuc} No (8.3.4(ii))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 0.5$ Yes (8.3.4(ii))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 1$ and D < Max(250.0 mm & 0.5 bw) No (8.3.4(ii))
 Tension @bot Tension @top
 $\phi_{Tus.min} = 207.19$ kNm $\phi_{Tus.min} = 207.19$ kNm (8.3.7b(ii))
 $\Delta Ast = 2038.2$ mm² $\Delta Ast = 2038.2$ mm² (8.3.6)
 $\Delta Asc = 0.0$ mm² $\Delta Asc = 0.0$ mm² (8.3.6)
 Torsional reinf. req. ? Yes (8.3.4)
 Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
 Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm (8.3.5)
 Asw/sw min by 2 Asw/sw min = 0.911 mm²/mm (8.3.5)
 Asw/sw needed final = 0.911 mm²/mm (8.3.5)

Tranverse shear and torsion reinforcement:
 Asv/sv + 2 Asw/sw needed = 9.643 mm²/mm
 Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
 Asv/sv min by shear = 0.911 mm²/mm
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
 Stirrup spacing = 50.0 mm Pass
 Max Spacing by torsion
 s1 (max) = Min (0.12Ut, 300.0) = 300.0 mm
 Max Spacing by shear
 is V* < ϕ_{Vumin} ? No
 s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)
 Positive Moment Negative Moment
 Mx*(+) = 1536.77 kNm # Mx*(-) = 1948.12 kNm
 Ast.min(+) = 2488.1 mm² Ast.min(-) = 2485.4 mm²
 Ast(+) = 3591.5 mm² Ast(-) = 4119.3 mm²
 Ast(+)/Ag = 0.2% Ast(-)/Ag = 0.3%
 d = 1237.2 mm d = 1238.5 mm
 ku = 0.05 ku = 0.05 (8.1.3)
 kuo = 0.05 kuo = 0.05 (8.1.5)
 $\phi = 0.8 \phi = 0.8$ (Table 2.2.2)
 $\phi_{Mu0}(+) = 1746.79$ kNm $\phi_{Mu0}(-) = 1998.73$ kNm (8.1.5)
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass

ZONE 9 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Deflections (no serviceability load cases selected)
 Ig = 219.70x10⁹ mm⁴
 Icr(+) = 40.32x10⁹ mm⁴ Icr(-) = 43.63x10⁹ mm⁴
 Short-term deflection (8.5.3.1)
 Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
 Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
 $\sigma_{cs}(+) = 0.44$ MPa $\sigma_{cs}(-) = 0.52$ MPa
 Mcr(+) = 1132.64 kNm Mcr(-) = 1106.95 kNm
 Long-term deflection (8.5.3.2)
 Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
 Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
 $\sigma_{cs}(+) = 0.44$ MPa $\sigma_{cs}(-) = 0.52$ MPa
 Mcr(+) = 1132.64 kNm Mcr(-) = 1106.95 kNm

Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 Ten @bot = 356.0 mm Ten @top = 119.1 mm
 Min tensile bar c-c spacing
 Ten @bot = 118.7 mm Ten @top = 119.1 mm
 Spacing limit check Warning (8.6.1b)
 Max steel stress db limit
 Ten @bot = 160.00 MPa Ten @top = 184.00 MPa (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 Ten @bot = 115.20 MPa Ten @top = 304.71 MPa (Table 8.6.1.B)
 Max steel tensile stresses
 Ten @bot = 0.00 MPa Ten @top = 0.00 MPa
 Stress limit check Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored

Development Lengths (13.1.2.2)
 @Bot @Top
 Max db = 32.0 mm Max db = 28.0 mm
 k1 = 1.00 k1 = 1.30
 k2 = 1.00 k2 = 1.04
 k3 = 0.96 k3 = 0.94
 Lsy.tb = 1217.5 mm Lsy.tb = 1294.6 mm (13.1.2.2)
 Lst = 500.9 mm Lst = 884.3 mm (13.1.2.4)
 $\sigma_{st_flex.} = 164.2$ MPa $\sigma_{st_flex.} = 272.9$ MPa
 $\sigma_{sc_flex.} = 45.6$ MPa $\sigma_{sc_flex.} = 25.9$ MPa
 $\sigma_{s_torsion} = 0.4$ MPa $\sigma_{s_torsion} = 0.3$ MPa
 Stress reduction in compressive zone due to torsion is ignored (conservative)
 Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

ZONE 10 (#=Minimum design action governs)
 ----- (\$=Minimum design action non-compliance)
 Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
 Positive Moment Negative Moment
 Mx*(+) = 1539.44 kNm # Mx*(-) = 1538.84 ...
 Ast.min(+) = 2494.6 mm² Ast.min(-) = 2490.5 mm²
 Ast(+) = 5629.7 mm² Ast(-) = 5541.8 mm²
 Ast(+)/Ag = 0.4% Ast(-)/Ag = 0.4%
 d = 1234.0 mm d = 1236.0 mm
 ku = 0.06 ku = 0.06 (8.1.3)
 kuo = 0.06 kuo = 0.06 (8.1.5)
 $\phi = 0.80 \phi = 0.80$ (Table 2.2.2)
 $\phi_{Mu0}(+) = 2700.86$ kNm $\phi_{Mu0}(-) = 2663.55$ kNm (8.1.5)
 Mx*(+) $\leq \phi_{Mu0}(+) \Rightarrow$ Pass Mx*(-) $\leq \phi_{Mu0}(-) \Rightarrow$ Pass

Transverse bar spacing = 50.0 mm
 Shear Vy* = 1934.90 kN
 Tension @bot Tension @top
 $\beta_1 = 1.10 \beta_1 = 1.10$
 $\beta_2 = 1.00 \beta_2 = 1.00$
 $\beta_3 = 1.00 \beta_3 = 1.00$
 $\phi = 0.70 \phi = 0.70$
 $\phi_{Vu.min} = 1264.19$ kN $\phi_{Vu.min} = 656.64$ kN (8.2.9)
 $\phi_{Vu.max} = 8292.48$ kN $\phi_{Vu.max} = 8292.48$ kN (8.2.6)
 $\phi_{Vuc} = 608.61$ kN $\phi_{Vuc} = 606.08$ kN (8.2.7.1)
 $\theta_v = 45.0^\circ \theta_v = 45.0^\circ$ (8.2.10(ii))
 Is shear req. (+) ? No (8.2.5a)
 Is shear req. (-) ? Yes (8.2.5a)
 Min shear req. (+) ? No (8.2.5)
 Min shear req. (-) ? Yes (8.2.5)
 Asv/sv needed by V* = 4.473 mm²/mm (8.2.10)
 Asv/sv needed final = 4.473 mm²/mm (8.2.8)

Torsion
 T* = 0.00 kNm $\phi = 0.70$
 Jt = 624.00x10⁶ mm⁴ $\phi_{Tu.max} = 3494.40$ kNm (8.3.3)
 T*/ $\phi_{Tu.max} + Vy*/\phi_{Vu.max} = 0.23$ (8.3.3)
 $\phi_{Tuc} = 828.77$ kNm (8.3.5(ii))
 T* > 0.25 ϕ_{Tuc} No (8.3.4(ii))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 0.5$ No (8.3.4(ii))
 T*/ $\phi_{Tuc} + Vy*/\phi_{Vuc} > 1$ and D < Max(250.0 mm & 0.5 bw) No (8.3.4(ii))
 Tension @bot Tension @top
 $\phi_{Tus.min} = 0.00$ kNm $\phi_{Tus.min} = 207.19$ kNm (8.3.7b(ii))
 $\Delta Ast = 0.0$ mm² $\Delta Ast = 2038.2$ mm² (8.3.6)
 $\Delta Asc = 0.0$ mm² $\Delta Asc = 0.0$ mm² (8.3.6)
 Torsional reinf. req. ? No (8.3.4)
 Asw/sw needed by T* = 0.000 mm²/mm (8.3.5)
 Asw/sw min by shear = 0.911 mm²/mm (8.3.7)
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asw/sw min by 2 Asw/sw min = 0.911 mm²/mm (8.3.5)
 Asw/sw needed final = 0.911 mm²/mm (8.3.5)

Tranverse shear and torsion reinforcement:
 Asv/sv + 2 Asw/sw needed = 6.294 mm²/mm
 Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
 Asv/sv min by shear = 0.911 mm²/mm
 Asw/sw min by 0.25 ϕ_{Tuc} = 0.237 mm²/mm
 Asv/sv + 2 Asw/sw min = 2.732 mm²/mm
 Stirrup spacing = 50.0 mm Pass
 Max Spacing by shear
 is V* < ϕ_{Vumin} ? No
 s2 (max) = Min (0.5D, 300.0) = 300.0 mm

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ZONE 10 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Bending (excluding longitudinal steel required by torsion)
Positive Moment
Mx*(+) = 1539.49 kNm #
Ast.min(+) = 2488.1 mm²
Ast(+) = 3591.5 mm²
Ast(+)/Ag = 0.2%
d = 1237.2 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(+) = 1746.64 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1538.80 kNm #
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(-) = 1707.82 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 40.24x10⁹ mm⁴ Icr(-) = 39.82x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm

Crack Control (no serviceability load cases selected)

Max tensile bar c-c spacing
Ten @bot = 356.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm
Spacing limit check
Pass (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa
Max steel stress c-c spacing limit
Ten @bot = 115.20 MPa
Max steel tensile stresses
Ten @bot = 0.00 MPa
Stress limit check
Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths

@Bot
Max db = 32.0 mm
k1 = 1.00
k2 = 1.00
k3 = 0.96
Lsy.cb = 696.0 mm
Lsc = 200.0 mm
σst.flex. = 0.0 MPa
σsc.flex. = 32.9 MPa
σs.torsion = 0.4 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

@Top
Max db = 28.0 mm
k1 = 1.30
k2 = 1.04
k3 = 0.94
Lsy.tb = 1294.6 mm
Lst = 686.5 mm
σst.flex. = 211.8 MPa
σsc.flex. = 0.0 MPa
σs.torsion = 0.4 MPa

Fire Resistance

Min req am (continuous) = 12.0 mm (5)
Calculated am = 50.0 mm (5.4.1(b)) (5.2.1)

ZONE 11 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment
Mx*(+) = 1539.44 kNm #
Ast.min(+) = 2494.6 mm²
Ast(+) = 3529.7 mm²
Ast(+)/Ag = 0.4%
d = 1234.0 mm
ku = 0.06
kuo = 0.06
φ = 0.80
φMu(+) = 2700.86 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1538.84 ...
Ast.min(-) = 2490.5 mm²
Ast(-) = 3541.8 mm²
Ast(-)/Ag = 0.4%
d = 1236.0 mm
ku = 0.06
kuo = 0.06
φ = 0.80
φMu(-) = 2663.55 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 50.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 1264.19 kN
φVu.max = 8292.48 kN
φVuc = 608.61 kN
θv = 45.0°
Is shear req. (+) ?
Is shear req. (-) ?
Min shear req. (+) ?
Min shear req. (-) ?
Asv/sv needed by V*
Asv/sv needed final

Vy* = 1914.34 kN
Tension @top
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 656.64 kN
φVu.max = 8292.48 kN
φVuc = 606.08 kN
θv = 45.0°
No
Yes
No
Yes
= 4.425 mm²/mm
= 4.425 mm²/mm

Torsion

T* = 0.00 kNm
Jt = 624.00x10⁶ mm⁴
T*/φTu.max + Vy*/φVu.max = 0.23
φTuc = 828.77 kNm
T* > 0.25φTuc
T*/φTuc + Vy*/φVuc > 0.5
T*/φTuc + Vy*/φVuc > 1 and D < Max(250.0 mm & 0.5 bw)
Tension @bot
φTus.min = 0.00 kNm
ΔAst = 0.0 mm²
ΔAsc = 0.0 mm²
Torsional reinf. req. ?
Asw/sw needed by T*
Asw/sw min by shear
Asw/sw min by 0.25φTuc
Asw/sw needed final

φ = 0.70
φTu.max = 3494.40 kNm
(8.3.3)
(8.3.5(1))
(8.3.4(1))
(8.3.4(2))
No
Tension @top
φTus.min = 207.19 kNm
ΔAst = 2038.2 mm²
ΔAsc = 0.0 mm²
No
= 0.000 mm²/mm
= 0.911 mm²/mm
= 0.237 mm²/mm
= 0.911 mm²/mm
(8.3.7b(ii))
(8.3.6)
(8.3.6)
(8.3.4)
(8.3.5)
(8.3.7)
(8.3.5)
(8.3.5)

ZONE 11 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Transverse shear and torsion reinforcement:
Asv/sv + 2 Asw/sw needed = 6.247 mm²/mm
Asv/sv + 2 Asw/sw provided = 4.710 mm²/mm Fail
Asv/sv min by shear = 0.911 mm²/mm
Asw/sw min by 0.25φTuc = 0.237 mm²/mm
Asv/sv + 2 Asw/sw min = 0.911 mm²/mm
Stirrup spacing = 50.0 mm Pass
Max Spacing by shear
is V* < φVum ? No
s2 (max) = Min (0.5D, 300.0) = 300.0 mm

Bending (excluding longitudinal steel required by torsion)

Positive Moment
Mx*(+) = 1539.49 kNm #
Ast.min(+) = 2488.1 mm²
Ast(+) = 3591.5 mm²
Ast(+)/Ag = 0.2%
d = 1237.2 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(+) = 1746.64 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1538.80 kNm #
Ast.min(-) = 2484.8 mm²
Ast(-) = 3503.5 mm²
Ast(-)/Ag = 0.2%
d = 1238.9 mm
ku = 0.05
kuo = 0.05
φ = 0.8
φMu(-) = 1707.82 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Deflections (no serviceability load cases selected)

Ig = 219.70x10⁹ mm⁴
Icr(+) = 40.24x10⁹ mm⁴ Icr(-) = 39.82x10⁹ mm⁴
Short-term deflection (8.5.3.1)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm
Long-term deflection (8.5.3.2)
Ief(+) = 131.82x10⁹ mm⁴ Ief(-) = 131.82x10⁹ mm⁴
Ms*(+) = 0.00 kNm Ms*(-) = 0.00 kNm
σcs(+) = 0.47 MPa σcs(-) = 0.45 MPa
Mcr(+) = 1124.56 kNm Mcr(-) = 1129.15 kNm

Crack Control (no serviceability load cases selected)

Max tensile bar c-c spacing
Ten @bot = 356.0 mm
Min tensile bar c-c spacing
Ten @bot = 118.7 mm
Spacing limit check
Pass (8.6.1b)
Max steel stress db limit
Ten @bot = 160.00 MPa
Max steel stress c-c spacing limit
Ten @bot = 115.20 MPa
Max steel tensile stresses
Ten @bot = 0.00 MPa
Stress limit check
Pass (Table 8.6.1A/B)
User has specified beam location as 'Interior'
Table 8.6.1(A & B) might be ignored

Development Lengths

@Bot
Max db = 32.0 mm
k1 = 1.00
k2 = 1.00
k3 = 0.96
Lsy.cb = 696.0 mm
Lsc = 200.0 mm
σst.flex. = 0.0 MPa
σsc.flex. = 11.3 MPa
σs.torsion = 0.4 MPa
Stress reduction in compressive zone due to torsion is ignored (conservative)

@Top
Max db = 28.0 mm
k1 = 1.30
k2 = 1.04
k3 = 0.94
Lsy.tb = 1294.6 mm
Lst = 686.5 mm
σst.flex. = 72.5 MPa
σsc.flex. = 0.0 MPa
σs.torsion = 0.4 MPa

Fire Resistance

Min req am (continuous) = 12.0 mm (5)
Calculated am = 50.0 mm (5.4.1(b)) (5.2.1)

ZONE 12 (#=Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
Positive Moment
Mx*(+) = 1536.98 kNm #
Ast.min(+) = 2494.6 mm²
Ast(+) = 3217.0 mm²
Ast(+)/Ag = 0.2%
d = 1234.0 mm
ku = 0.05
kuo = 0.05
φ = 0.80
φMu(+) = 1566.40 kNm
Mx*(+) ≤ φMu(+) ⇒ Pass

Negative Moment
Mx*(-) = 1541.31 ...
Ast.min(-) = 2490.5 mm²
Ast(-) = 3694.5 mm²
Ast(-)/Ag = 0.2%
d = 1236.0 mm
ku = 0.05
kuo = 0.05
φ = 0.80
φMu(-) = 1794.85 kNm
Mx*(-) ≤ φMu(-) ⇒ Pass

Transverse bar spacing = 255.0 mm

Shear
Tension @bot
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 1160.62 kN
φVu.max = 8292.48 kN
φVuc = 505.04 kN
θv = 45.0°
Is shear req. (+) ?
Is shear req. (-) ?
Min shear req. (+) ?
Min shear req. (-) ?
Asv/sv needed by V*
Asv/sv needed final

Vy* = 33.78 kN
Tension @top
β1 = 1.10
β2 = 1.00
β3 = 1.00
φ = 0.70
φVu.min = 1186.10 kN
φVu.max = 8292.48 kN
φVuc = 529.46 kN
θv = 45.0°
No
Yes
No
Yes
= 0.000 mm²/mm
= 0.911 mm²/mm
(8.2.5a)
(8.2.5a)
(8.2.5)
(8.2.5)
(8.2.10)
(8.2.8)

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ZONE 12 (Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Torsion
 $T^* = 0.00 \text{ kNm}$ $\phi = 0.70$
 $J_t = 624.00 \times 10^6 \text{ mm}^4$ $\phi T_u \text{ max} = 3494.40 \text{ kNm}$ (8.3.3)
 $T^* / \phi T_u \text{ max} + V_y^* / \phi V_u \text{ max} = 0.00$ (8.3.3)
 $\phi T_{uc} = 828.77 \text{ kNm}$ (8.3.5(1))
 $T^* > 0.25 \phi T_{uc}$ No (8.3.4(1))
 $T^* / \phi T_{uc} + V_y^* / \phi V_{uc} > 0.5$ No (8.3.4(2))
 $T^* / \phi T_{uc} + V_y^* / \phi V_{uc} > 1$ and $D < \text{Max}(250.0 \text{ mm} \ \& \ 0.5 \text{ bw})$ (8.3.4(2))
 Tension @bot
 $\phi T_{us \text{ min}} = 0.00 \text{ kNm}$ $\phi T_{us \text{ min}} = 0.00 \text{ kNm}$ (8.3.7b(ii))
 $\Delta A_{st} = 0.00 \text{ mm}^2$ $\Delta A_{st} = 0.00 \text{ mm}^2$ (8.3.6)
 $\Delta A_{sc} = 0.00 \text{ mm}^2$ $\Delta A_{sc} = 0.00 \text{ mm}^2$ (8.3.6)
 Torsional reinf. req. ? No (8.3.4)
 $A_{sw/sw} \text{ needed by } T^* = 0.000 \text{ mm}^2/\text{mm}$ (8.3.5)
 $A_{sw/sw} \text{ min by shear} = 0.911 \text{ mm}^2/\text{mm}$ (8.3.7)
 $A_{sw/sw} \text{ min by } 0.25 \phi T_{uc} = 0.237 \text{ mm}^2/\text{mm}$
 $A_{sw/sw} \text{ needed final} = 0.000 \text{ mm}^2/\text{mm}$ (8.3.5)

Transverse shear and torsion reinforcement:
 $A_{sv/sv} + 2 A_{sw/sw} \text{ needed} = 0.911 \text{ mm}^2/\text{mm}$
 $A_{sv/sv} + 2 A_{sw/sw} \text{ provided} = 0.924 \text{ mm}^2/\text{mm}$ Pass
 $A_{sv/sv} \text{ min by shear} = 0.911 \text{ mm}^2/\text{mm}$
 $A_{sw/sw} \text{ min by } 0.25 \phi T_{uc} = 0.237 \text{ mm}^2/\text{mm}$
 $A_{sv/sv} + 2 A_{sw/sw} \text{ min} = 0.911 \text{ mm}^2/\text{mm}$
 Stirrup spacing = 255.0 mm Pass
 Max Spacing by shear
 is $V^* < \phi V_{u \text{ min}}$? Yes
 $s_2 \text{ (max)} = \text{Min} (0.5D, 300.0) = 300.0 \text{ mm}$

Bending (excluding longitudinal steel required by torsion)
 Positive Moment Negative Moment
 $M_x^* (+) = 1536.98 \text{ kNm}$ $M_x^* (-) = 1541.31 \text{ kNm}$ #
 $A_{st \text{ min}} (+) = 2494.6 \text{ mm}^2$ $A_{st \text{ min}} (-) = 2490.5 \text{ mm}^2$
 $A_{st} (+) = 3217.0 \text{ mm}^2$ $A_{st} (-) = 3694.5 \text{ mm}^2$
 $A_{st} (+) / A_g = 0.2\%$ $A_{st} (-) / A_g = 0.2\%$
 $d = 1234.0 \text{ mm}$ $d = 1236.0 \text{ mm}$
 $k_u = 0.05$ $k_u = 0.05$ (8.1.3)
 $k_{uo} = 0.05$ $k_{uo} = 0.05$ (8.1.5)
 $\phi = 0.8$ $\phi = 0.8$ (Table 2.2.2)
 $\phi M_{uo} (+) = 1566.40 \text{ kNm}$ $\phi M_{uo} (-) = 1794.85 \text{ kNm}$ (8.1.5)
 $M_x^* (+) \leq \phi M_{uo} (+) \Rightarrow \text{Pass}$ $M_x^* (-) \leq \phi M_{uo} (-) \Rightarrow \text{Pass}$

Deflections (no serviceability load cases selected)
 $I_g = 219.70 \times 10^9 \text{ mm}^4$
 $I_{cr} (+) = 24.37 \times 10^9 \text{ mm}^4$ $I_{cr} (-) = 27.64 \times 10^9 \text{ mm}^4$
 Short-term deflection (8.5.3.1)
 $I_{ef} (+) = 131.82 \times 10^9 \text{ mm}^4$ $I_{ef} (-) = 131.82 \times 10^9 \text{ mm}^4$
 $M_s^* (+) = 0.00 \text{ kNm}$ $M_s^* (-) = 0.00 \text{ kNm}$
 $\sigma_{cs} (+) = 0.26 \text{ MPa}$ $\sigma_{cs} (-) = 0.34 \text{ MPa}$
 $M_{cr} (+) = 1193.10 \text{ kNm}$ $M_{cr} (-) = 1167.21 \text{ kNm}$
 Long-term deflection (8.5.3.2)
 $I_{ef} (+) = 131.82 \times 10^9 \text{ mm}^4$ $I_{ef} (-) = 131.82 \times 10^9 \text{ mm}^4$
 $M_s (+) = 0.00 \text{ kNm}$ $M_s (-) = 0.00 \text{ kNm}$
 $\sigma_{cs} (+) = 0.26 \text{ MPa}$ $\sigma_{cs} (-) = 0.34 \text{ MPa}$
 $M_{cr} (+) = 1193.10 \text{ kNm}$ $M_{cr} (-) = 1167.21 \text{ kNm}$

Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 $T_{en} \text{ @bot} = 830.7 \text{ mm}$ $T_{en} \text{ @top} = 595.6 \text{ mm}$
 Min tensile bar c-c spacing
 $T_{en} \text{ @bot} = 118.7 \text{ mm}$ $T_{en} \text{ @top} = 119.1 \text{ mm}$
 Spacing limit check
 Warning (8.6.1b)
 Max steel stress db limit
 $T_{en} \text{ @bot} = 160.00 \text{ MPa}$ $T_{en} \text{ @top} = 184.00 \text{ MPa}$ (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 $T_{en} \text{ @bot} = -264.53 \text{ MPa}$ $T_{en} \text{ @top} = -76.44 \text{ MPa}$ (Table 8.6.1.B)
 Max steel tensile stresses
 $T_{en} \text{ @bot} = 0.00 \text{ MPa}$ $T_{en} \text{ @top} = 0.00 \text{ MPa}$
 Stress limit check Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored

Development Lengths (13.1.2.2)
 @Bot @Top
 $\text{Max db} = 32.0 \text{ mm}$ $\text{Max db} = 28.0 \text{ mm}$
 $k_1 = 1.00$ $k_1 = 1.30$
 $k_2 = 1.00$ $k_2 = 1.04$
 $k_3 = 0.96$ $k_3 = 0.94$
 $L_{sy \text{ cb}} = 696.0 \text{ mm}$ $L_{sy \text{ tb}} = 1294.6 \text{ mm}$ (13.1.2.2)
 $L_{sc} = 200.0 \text{ mm}$ $L_{st} = 336.0 \text{ mm}$ (13.1.2.4)
 $\sigma_{st \text{ flex.}} = 0.0 \text{ MPa}$ $\sigma_{st \text{ flex.}} = 3.0 \text{ MPa}$
 $\sigma_{sc \text{ flex.}} = 0.4 \text{ MPa}$ $\sigma_{sc \text{ flex.}} = 0.0 \text{ MPa}$
 $\sigma_{s \text{ torsion}} = 0.6 \text{ MPa}$ $\sigma_{s \text{ torsion}} = 0.6 \text{ MPa}$
 Stress reduction in compressive zone due to torsion is ignored (conservative)

Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

ZONE 13 (Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Calc bef = 1200.00 mm Design bef = 1200.00 mm (8.8.2)

Bending
 Positive Moment Negative Moment
 $M_x^* (+) = 1536.98 \text{ kNm}$ $M_x^* (-) = 1541.31 \text{ kNm}$...
 $A_{st \text{ min}} (+) = 2494.6 \text{ mm}^2$ $A_{st \text{ min}} (-) = 2490.5 \text{ mm}^2$
 $A_{st} (+) = 3217.0 \text{ mm}^2$ $A_{st} (-) = 3694.5 \text{ mm}^2$
 $A_{st} (+) / A_g = 0.2\%$ $A_{st} (-) / A_g = 0.2\%$
 $d = 1234.0 \text{ mm}$ $d = 1236.0 \text{ mm}$
 $k_u = 0.05$ $k_u = 0.05$ (8.1.3)
 $k_{uo} = 0.05$ $k_{uo} = 0.05$ (8.1.5)
 $\phi = 0.8$ $\phi = 0.8$ (Table 2.2.2)
 $\phi M_{uo} (+) = 1566.40 \text{ kNm}$ $\phi M_{uo} (-) = 1794.85 \text{ kNm}$ (8.1.5)
 $M_x^* (+) \leq \phi M_{uo} (+) \Rightarrow \text{Pass}$ $M_x^* (-) \leq \phi M_{uo} (-) \Rightarrow \text{Pass}$
 Transverse bar spacing = 255.0 mm

ZONE 13 (Minimum design action governs)

----- (\$=Minimum design action non-compliance)

Shear
 $V_y^* = 13.22 \text{ kN}$
 Tension @bot
 $\beta_1 = 1.10$ $\beta_1 = 1.10$
 $\beta_2 = 1.00$ $\beta_2 = 1.00$
 $\beta_3 = 1.00$ $\beta_3 = 1.00$
 $\phi = 0.70$ $\phi = 0.70$
 $\phi V_{u \text{ min}} = 1160.62 \text{ kN}$ $\phi V_{u \text{ min}} = 1186.10 \text{ kN}$ (8.2.9)
 $\phi V_{u \text{ max}} = 8292.48 \text{ kN}$ $\phi V_{u \text{ max}} = 8292.48 \text{ kN}$ (8.2.6)
 $\phi V_{uc} = 505.04 \text{ kN}$ $\phi V_{uc} = 529.46 \text{ kN}$ (8.2.7.1)
 $\theta_v = 45.0^\circ$ $\theta_v = 45.0^\circ$ (8.2.10(1))
 Is shear req. (+) ? No (8.2.5a)
 Is shear req. (-) ? Yes (8.2.5a)
 Min shear req. (+) ? No (8.2.5)
 Min shear req. (-) ? Yes (8.2.5)
 $A_{sv/sv} \text{ needed by } V^* = 0.000 \text{ mm}^2/\text{mm}$ (8.2.10)
 $A_{sv/sv} \text{ needed final} = 0.911 \text{ mm}^2/\text{mm}$ (8.2.8)

Torsion
 $T^* = 0.00 \text{ kNm}$ $\phi = 0.70$
 $J_t = 624.00 \times 10^6 \text{ mm}^4$ $\phi T_u \text{ max} = 3494.40 \text{ kNm}$ (8.3.3)
 $T^* / \phi T_u \text{ max} + V_y^* / \phi V_u \text{ max} = 0.00$ (8.3.3)
 $\phi T_{uc} = 828.77 \text{ kNm}$ (8.3.5(1))
 $T^* > 0.25 \phi T_{uc}$ No (8.3.4(1))
 $T^* / \phi T_{uc} + V_y^* / \phi V_{uc} > 0.5$ No (8.3.4(2))
 $T^* / \phi T_{uc} + V_y^* / \phi V_{uc} > 1$ and $D < \text{Max}(250.0 \text{ mm} \ \& \ 0.5 \text{ bw})$ (8.3.4(2))
 Tension @bot
 $\phi T_{us \text{ min}} = 0.00 \text{ kNm}$ $\phi T_{us \text{ min}} = 0.00 \text{ kNm}$ (8.3.7b(ii))
 $\Delta A_{st} = 0.00 \text{ mm}^2$ $\Delta A_{st} = 0.00 \text{ mm}^2$ (8.3.6)
 $\Delta A_{sc} = 0.00 \text{ mm}^2$ $\Delta A_{sc} = 0.00 \text{ mm}^2$ (8.3.6)
 Torsional reinf. req. ? No (8.3.4)
 $A_{sw/sw} \text{ needed by } T^* = 0.000 \text{ mm}^2/\text{mm}$ (8.3.5)
 $A_{sw/sw} \text{ min by shear} = 0.911 \text{ mm}^2/\text{mm}$ (8.3.7)
 $A_{sw/sw} \text{ min by } 0.25 \phi T_{uc} = 0.237 \text{ mm}^2/\text{mm}$
 $A_{sw/sw} \text{ needed final} = 0.000 \text{ mm}^2/\text{mm}$ (8.3.5)

Transverse shear and torsion reinforcement:
 $A_{sv/sv} + 2 A_{sw/sw} \text{ needed} = 0.911 \text{ mm}^2/\text{mm}$
 $A_{sv/sv} + 2 A_{sw/sw} \text{ provided} = 0.924 \text{ mm}^2/\text{mm}$ Pass
 $A_{sv/sv} \text{ min by shear} = 0.911 \text{ mm}^2/\text{mm}$
 $A_{sw/sw} \text{ min by } 0.25 \phi T_{uc} = 0.237 \text{ mm}^2/\text{mm}$
 $A_{sv/sv} + 2 A_{sw/sw} \text{ min} = 0.911 \text{ mm}^2/\text{mm}$
 Stirrup spacing = 255.0 mm Pass
 Max Spacing by shear
 is $V^* < \phi V_{u \text{ min}}$? Yes
 $s_2 \text{ (max)} = \text{Min} (0.5D, 300.0) = 300.0 \text{ mm}$

Bending (excluding longitudinal steel required by torsion)
 Positive Moment Negative Moment
 $M_x^* (+) = 1536.98 \text{ kNm}$ $M_x^* (-) = 1541.31 \text{ kNm}$ #
 $A_{st \text{ min}} (+) = 2494.6 \text{ mm}^2$ $A_{st \text{ min}} (-) = 2490.5 \text{ mm}^2$
 $A_{st} (+) = 3217.0 \text{ mm}^2$ $A_{st} (-) = 3694.5 \text{ mm}^2$
 $A_{st} (+) / A_g = 0.2\%$ $A_{st} (-) / A_g = 0.2\%$
 $d = 1234.0 \text{ mm}$ $d = 1236.0 \text{ mm}$
 $k_u = 0.05$ $k_u = 0.05$ (8.1.3)
 $k_{uo} = 0.05$ $k_{uo} = 0.05$ (8.1.5)
 $\phi = 0.8$ $\phi = 0.8$ (Table 2.2.2)
 $\phi M_{uo} (+) = 1566.40 \text{ kNm}$ $\phi M_{uo} (-) = 1794.85 \text{ kNm}$ (8.1.5)
 $M_x^* (+) \leq \phi M_{uo} (+) \Rightarrow \text{Pass}$ $M_x^* (-) \leq \phi M_{uo} (-) \Rightarrow \text{Pass}$

Deflections (no serviceability load cases selected)
 $I_g = 219.70 \times 10^9 \text{ mm}^4$
 $I_{cr} (+) = 24.37 \times 10^9 \text{ mm}^4$ $I_{cr} (-) = 27.64 \times 10^9 \text{ mm}^4$
 Short-term deflection (8.5.3.1)
 $I_{ef} (+) = 131.82 \times 10^9 \text{ mm}^4$ $I_{ef} (-) = 131.82 \times 10^9 \text{ mm}^4$
 $M_s^* (+) = 0.00 \text{ kNm}$ $M_s^* (-) = 0.00 \text{ kNm}$
 $\sigma_{cs} (+) = 0.26 \text{ MPa}$ $\sigma_{cs} (-) = 0.34 \text{ MPa}$
 $M_{cr} (+) = 1193.10 \text{ kNm}$ $M_{cr} (-) = 1167.21 \text{ kNm}$
 Long-term deflection (8.5.3.2)
 $I_{ef} (+) = 131.82 \times 10^9 \text{ mm}^4$ $I_{ef} (-) = 131.82 \times 10^9 \text{ mm}^4$
 $M_s (+) = 0.00 \text{ kNm}$ $M_s (-) = 0.00 \text{ kNm}$
 $\sigma_{cs} (+) = 0.26 \text{ MPa}$ $\sigma_{cs} (-) = 0.34 \text{ MPa}$
 $M_{cr} (+) = 1193.10 \text{ kNm}$ $M_{cr} (-) = 1167.21 \text{ kNm}$

Crack Control (no serviceability load cases selected)
 Max tensile bar c-c spacing
 $T_{en} \text{ @bot} = 830.7 \text{ mm}$ $T_{en} \text{ @top} = 595.6 \text{ mm}$
 Min tensile bar c-c spacing
 $T_{en} \text{ @bot} = 118.7 \text{ mm}$ $T_{en} \text{ @top} = 119.1 \text{ mm}$
 Spacing limit check
 Warning (8.6.1b)
 Max steel stress db limit
 $T_{en} \text{ @bot} = 160.00 \text{ MPa}$ $T_{en} \text{ @top} = 184.00 \text{ MPa}$ (Table 8.6.1.A)
 Max steel stress c-c spacing limit
 $T_{en} \text{ @bot} = -264.53 \text{ MPa}$ $T_{en} \text{ @top} = -76.44 \text{ MPa}$ (Table 8.6.1.B)
 Max steel tensile stresses
 $T_{en} \text{ @bot} = 0.00 \text{ MPa}$ $T_{en} \text{ @top} = 0.00 \text{ MPa}$
 Stress limit check Pass (Table 8.6.1A/B)
 User has specified beam location as 'Interior'
 Table 8.6.1(A & B) might be ignored

Development Lengths (13.1.2.2)
 @Bot @Top
 $\text{Max db} = 32.0 \text{ mm}$ $\text{Max db} = 28.0 \text{ mm}$
 $k_1 = 1.00$ $k_1 = 1.30$
 $k_2 = 1.00$ $k_2 = 1.04$
 $k_3 = 0.96$ $k_3 = 0.94$
 $L_{sy \text{ cb}} = 696.0 \text{ mm}$ $L_{sy \text{ tb}} = 1294.6 \text{ mm}$ (13.1.2.2)
 $L_{sc} = 200.0 \text{ mm}$ $L_{st} = 336.0 \text{ mm}$ (13.1.2.4)
 $\sigma_{st \text{ flex.}} = 0.0 \text{ MPa}$ $\sigma_{st \text{ flex.}} = 0.5 \text{ MPa}$
 $\sigma_{sc \text{ flex.}} = 0.1 \text{ MPa}$ $\sigma_{sc \text{ flex.}} = 0.0 \text{ MPa}$
 $\sigma_{s \text{ torsion}} = 0.6 \text{ MPa}$ $\sigma_{s \text{ torsion}} = 0.6 \text{ MPa}$
 Stress reduction in compressive zone due to torsion is ignored (conservative)

Fire Resistance (5)
 Min req am (continuous) = 12.0 mm (5.4.1(b))
 Calculated am = 50.0 mm (5.2.1)

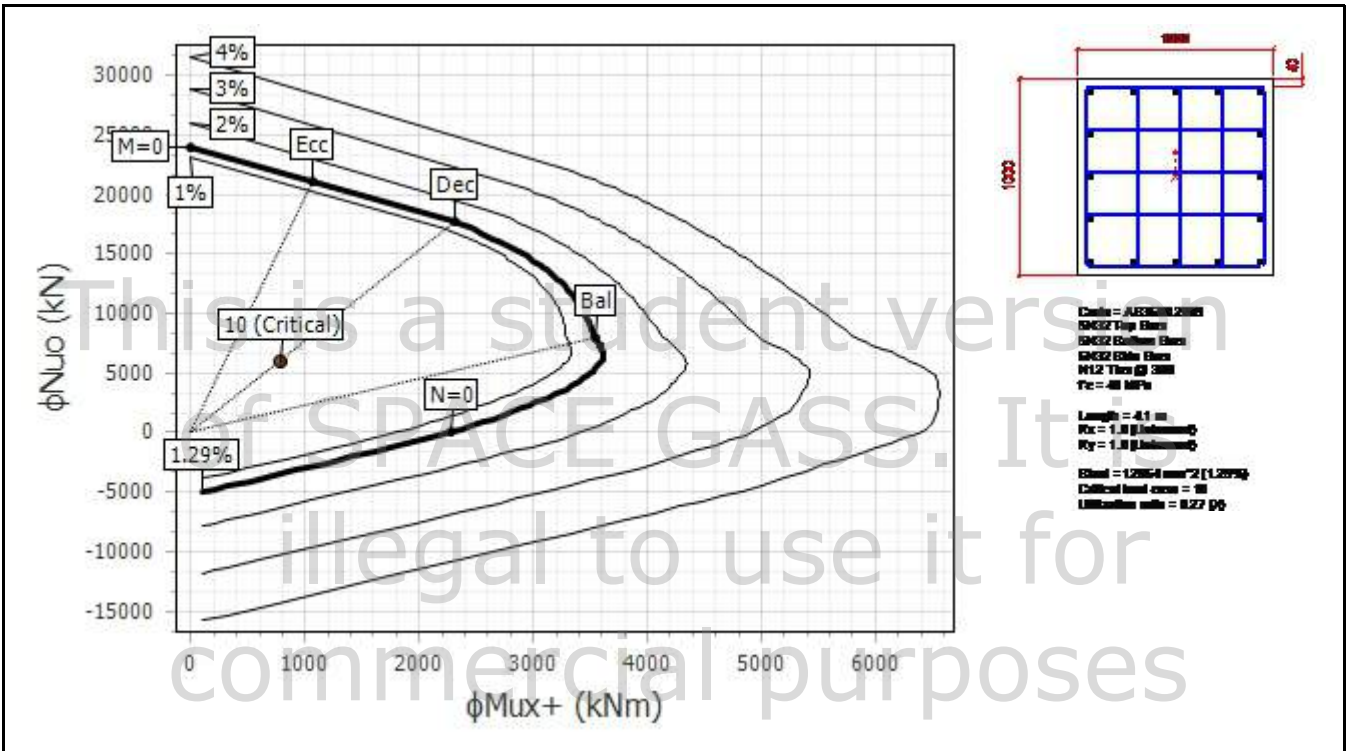
Column Design



SPACE GASS 12.54 - STUDENT VERSION - NOT FOR COMMERCIAL USE

Path: C:\USERS\SAMUEL\SHAREPO...\COLUMNS\HEADSTOCK DESIGN REV 2 WITH COLUMNS

Designer: Date: Monday, May 29, 2017 7:02 PM Page: 1



Column: 9
Member list: 9
Biaxial bending check: Included
Shear check: Included
Min design actions: Yes (Mmin=0.05DxN*) (AS3600 10.1.2)
δx: Calculated (AS3600 10.4.2)
δy: Calculated (AS3600 10.4.2)
βd: 1 (AS3600 10.4.3)
λuc: 1000 (AS3600 10.4.3)
kx: 1 (Not Braced)
ky: 1 (Not Braced)
Av/Ag: 0.83
Steel range: 1% to 4%
Bar size range: 12.0 to 36.0 mm
Cover: 40.0 mm
Design Priority: Minimum bars

DESIGN ACTIONS (#=Minimum design action governs)
----- (\$=Less than minimum design action)

Axial and Bending (Critical load case = 10):
N* = 5898.83 kN φNu = 21857.22 kN
Mx* = 783.13 kNm φMux = 3592.27 kNm
My* = 294.94 kNm φMuy = 3592.27 kNm
N*/φNu = 0.27
Mx*/φMux = 0.22
My*/φMuy = 0.08
(Mx*/φMux)^an + (My*/φMuy)^an = 0.24 (an = 1.118)

Shear (Critical load case = 10):
Vx* = 0.00 kN φVux = 250.17 kN
Vy* = 295.64 kN φVuy = 1026.88 kN
Vu.max = 6666.67 kN Av = 833333.31 mm^2
Asv = 226.19 mm^2 Asv.min = 0 mm^2

REINFORCEMENT (mm,m)

Column Title: 9
Type: Bars
Reinforcement: 5N32 Top Bars, 5N32 Bottom Bars, 6N32 Side Bars
Column Length: 4.100
Total bar weight = 414.03 kg
Ties: N12 @ 300

Vx*/φVux = 0.00
Vy*/φVuy = 0.29
LOADS (kN,kNm)

Case δbx kmx Ncx M1x/M2x
δby kmy Ncy M1y/M2y
N* Mx+ Mag.Mx+ Des.Mx+ Sht/ Util
Nc My+ Mag.My+ Mag.My- Des.My+ Sht Ratio
10 1.00 0.40 176069.47 0.55
1.03 1.00 176069.47 -1.00
5898.83 783.13 -428.98 783.13 783.13 Sht
176069.47 0.00 0.00 294.94 -294.94 294.94 Sht
0.27 (X)

BAR TABLE (mm,MPa,mm^2)

Column Bar	Name	Diameter	fsy	Area	X	Y
9 1	N32	32.00	500.00	804.00	-432.00	432.00
2	N32	32.00	500.00	804.00	-216.00	432.00
3	N32	32.00	500.00	804.00	0.00	432.00
4	N32	32.00	500.00	804.00	216.00	432.00
5	N32	32.00	500.00	804.00	432.00	432.00
6	N32	32.00	500.00	804.00	-432.00	-432.00
7	N32	32.00	500.00	804.00	-216.00	-432.00
8	N32	32.00	500.00	804.00	0.00	-432.00
9	N32	32.00	500.00	804.00	216.00	-432.00
10	N32	32.00	500.00	804.00	432.00	-432.00
11	N32	32.00	500.00	804.00	-432.00	-216.00
12	N32	32.00	500.00	804.00	-432.00	0.00
13	N32	32.00	500.00	804.00	-432.00	216.00
14	N32	32.00	500.00	804.00	432.00	-216.00
15	N32	32.00	500.00	804.00	432.00	0.00
16	N32	32.00	500.00	804.00	432.00	216.00

KEY POINTS (kNm,kN,mm) (Ecc=Min Eccentricity, Dec=Decompression, Bal=Balanced)

Pnt	φMux	φNux	φ	e	φMuy	φNuy	φ	e
Ecc	1055.62	21112.37	0.60	50	1055.62	21112.37	0.60	50
Dec	2303.64	17700.93	0.60	130	2303.64	17700.93	0.60	130
Bal	3535.86	7899.11	0.60	448	3535.86	7899.11	0.60	448
M=0	0.00	23997.88	0.60	0	0.00	23997.88	0.60	0
N=0	2272.07	0.00	0.80	>10000	2272.07	0.00	0.80	>10000

INTERACTION TABLE (kNm,kN,mm)

Pnt	φMux	φNux	φ	e	φMuy	φNuy	φ	e
1	0.00	23997.88	0.60	0	0.00	23997.88	0.60	0
2	2303.64	17700.93	0.60	130	2303.64	17700.93	0.60	130
3	2351.88	17521.44	0.60	134	2351.88	17521.44	0.60	134
4	2399.04	17341.51	0.60	138	2399.04	17341.51	0.60	138
5	2445.13	17161.09	0.60	142	2445.13	17161.09	0.60	142
6	2490.15	16980.20	0.60	147	2490.15	16980.20	0.60	147
7	2534.10	16798.80	0.60	151	2534.10	16798.80	0.60	151
8	2576.99	16616.89	0.60	155	2576.99	16616.89	0.60	155
9	2618.83	16434.44	0.60	159	2618.83	16434.44	0.60	159
10	2659.52	16284.25	0.60	163	2659.52	16284.25	0.60	163
11	2692.26	16100.68	0.60	167	2692.26	16100.68	0.60	167
12	2730.96	15916.52	0.60	172	2730.96	15916.52	0.60	172
13	2768.62	15731.75	0.60	176	2768.62	15731.75	0.60	176
14	2805.26	15546.35	0.60	180	2805.26	15546.35	0.60	180
15	2840.87	15360.30	0.60	185	2840.87	15360.30	0.60	185
16	2875.46	15173.58	0.60	190	2875.46	15173.58	0.60	190
17	2909.05	14986.17	0.60	194	2909.05	14986.17	0.60	194
18	2941.63	14798.02	0.60	199	2941.63	14798.02	0.60	199

AS3600 CALCULATIONS FOR REINFORCED CONCRETE COLUMN 9

Design/Check: Design
Load cases considered: 10
Critical load case: 10

Section Properties
D = 1000.0 mm
Ag = 1000000.0 mm^2
Material Properties
f'c = 40.00 MPa
fsy = 500.00 MPa
fsy.f = 500.00 MPa
Ec = 32800.00 MPa
Es = 200000.00 MPa
gc = 0.003

Columns

$$L_e/r < 25 \text{ (braced)}$$

$$L_e/r < 22 \text{ (unbraced)}$$

$$0.3D = 0.3 \times 1000$$

$$r = 300$$

$$L_e = \frac{4100}{300}$$

$$= 14 \text{ (unbraced)}$$

$$N_{uo} = \alpha_1 f'_c A_g + f_y A_{st}$$

Where N_{uo} = ultimate compressive strength.
 $\alpha_1 = 1.0 - 0.003 f'_c$ $0.72 \leq \alpha_1 \leq 0.85$
 A_g = concrete area of the column.
 $A_s = A_{st} + A_{sc}$

$$\alpha_1 = 1.0 - 0.003 \times 50$$

$$= 0.85$$

$$A_g = 1000^2$$

$$A_{st} = 8 \text{ N32} = 804 \text{ } 6432$$

$$A_{sc} = 8 \text{ N32} = 6432$$

$$A_s = 12864$$

$$N_{uo} = 48932 \text{ kN}$$
~~$$48932 \text{ kN}$$~~

$$1. N_{uo} (N_u = 0)$$

$$C_c = \alpha_2 f'_c b k u d$$

$$\gamma = 1.05 - 0.007 f'_c$$

$$= 1.05 - 0.007 \times 50$$

$$= 0.7$$

$$C_s = A_{sc} f_{sc}$$

$$= 6432 \times 500$$

$$= 3216 \text{ kN}$$

$$f_{sy} = 500 \text{ MPa}$$

$$T = A_{st} f_{sy}$$

$$= 804 \times 500$$

$$= 3216 \text{ kN}$$

$$C_c = 0.85 \gamma f'_c k u d b$$

$$= 0.85 \times 0.77 \times 50 \times 40 \times$$

$$k u = 1$$

$$k u \times d$$

$$= 1 \times 40$$

$$= 40 \text{ mm}$$

GRANOR XJS® EXPANSION JOINT SYSTEM



THE GRANOR XJS® POLYMER NOSED EXPANSION JOINT SYSTEM

The SSI / GRANOR XJS® Expansion Joint System has revolutionised expansion joint construction for both new works and rehabilitation outcomes. XJS combines a tough, impact resistant wear resistant polymer joint nosing and a rapid two part cure joint sealing system with high movement capacity.

The total expansion joint system, which is cold applied with minimal specialised equipment, is specifically designed to provide a watertight, chemically resistant seal to accommodate high traffic volumes and to remain pliable in cold and warm temperatures. An important feature is that the two component silicone sealant in the system will readily bond to itself. This is ideal for maintenance applications where only one traffic lane can be closed at a time, but particularly where a continuous seal is required between adjacent lanes when they are eventually rehabilitated.

The rapid curing ability of the total XJS® system makes it an excellent choice for highways, bridges, airfields, parking decks and other high volume traffic areas that require short closure times. Installation to completion during non peak hour traffic time is possible, thus helping to avoid frustrating traffic backups and costly overtime.

The XJS® system is cost competitive, easily repairable if needed. It is also highly effective for skew joint applications due to the non-directional strain capacity attributes of the rapid cure seal.

Replacement of failed expansion joint systems at a fraction of the cost of conventional joint repair alternatives is achieved by use of the XJS® Expansion Joint System.

SYSTEM COMPONENTS

SILSPEC® 900 POLYMER NOSING SYSTEM (PNS)

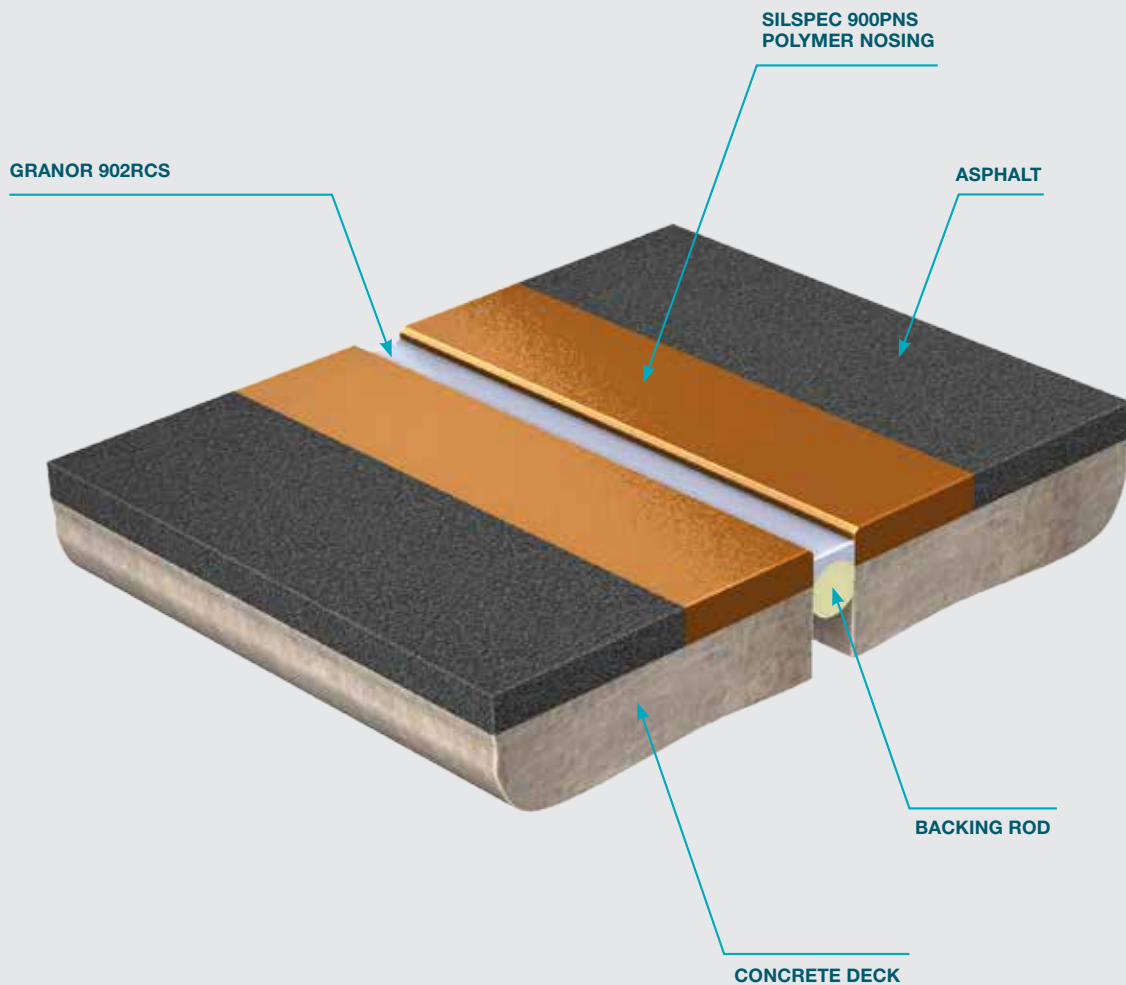
Is a two-component, rapid curing liquid polymer. Due to its relatively low viscosity, Silspec® 900 PNS is easy to mix and place. It cures to a dense, semi-flexible polymer that is resistant to chemicals, weather, abrasion and impact. The polymer is mixed with Silspec® Blended Aggregate. This combination forms a polymer-based mortar for joint repair or joint nosing repairs.

DOW CORNING® 902 RCS JOINT SEALANT

Is a two component, easy to install, 100% percent silicone rubber sealant designed to seal the expansion joint gap. The rapid curing ability of Dow Corning® 902 RCS joint sealant allows it to accommodate typical daily thermal movements and/or differential joint movement caused by traffic. Since it is self-levelling, Dow Corning® 902 RCS Joint Sealant can conform to irregularly shaped joints without tooling.

RAPID INSTALL
WITH PROVEN
LONGEVITY

PROVIDING OUTSTANDING
ASSET MANAGEMENT



DESIGN FEATURES

FAST, USER FRIENDLY INSTALLATION

- XJS joint repairs can be completed between peak hours. The Silspec nosing typically cures to trafficable hardness within 2 – 4 hours (depending on temperature); the 902RCS sealant skins over and is trafficable 15 minutes after application.
- Can be installed within lane closures, lane by lane.
- New joints are completed quickly cutting labour costs. As expansion joints are often one of the last components in a bridge build, this can help to achieve earlier completion dates
- Contractor or road authority maintenance crews can be easily trained to install XJS.
- Required tools and equipment are simple and readily available.
- A comprehensive installation guide and video guide are both available.

DURABLE

- The original XJS joint was installed in Oklahoma in 1991. The original XJS installation in Australia was in Adelaide in 1997.
- Durable in all climates, XJS has been used to repair joints from Alaska to South America and throughout Australia, New Zealand and Papua New Guinea. XJS has been used in every capital city and every major highway in Australia.
- The multiple XJS joints installed between 1998 and 2002 on the Westgate Bridge in Melbourne are still in excellent condition despite over 160,000 vehicles per day.

VERSATILE

- XJS is an excellent choice for new construction in a wide range of climates and engineering configurations.
- XJS can be used to repair or replace a vast range of systems in use today.
- XJS can fully replace Asphaltic Plug Joints suffering from potholes, cracking, rutting or shoving—significantly extending joint life cycle and reducing probability of hazardous defects.
- XJS is the most widely used joint rehabilitation system in Australia. Nothing else comes close in terms of usage or outcomes.

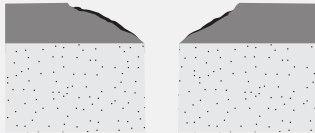
XSJ® INSTALLATION – LANE BY LANE



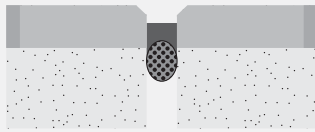
WATCH OUR INSTALLATION VIDEO
GRANOR.COM.AU



ASPHALT OVERLAY REPAIR

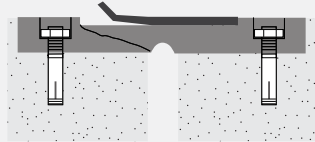


Before

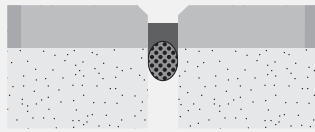


After

MODULAR JOINT REPAIR

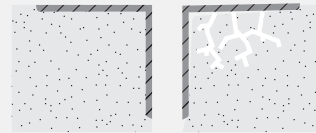


Before

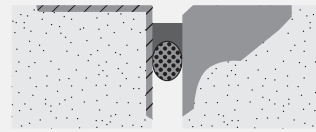


After

ARMOR JOINT REPAIR

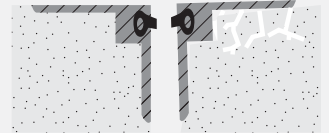


Before

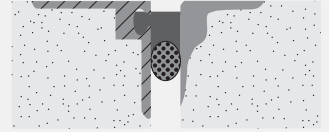


After

STRIP SEAL JOINT REPAIR

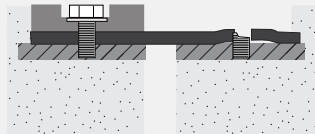


Before

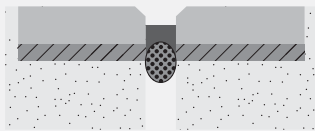


After

NEOPRENE STRIP JOINT REPAIR

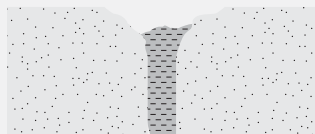


Before

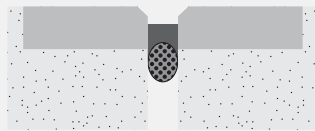


After

SPALLED JOINT FACE REPAIR

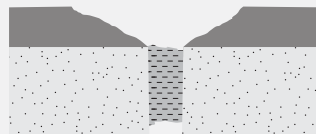


Before

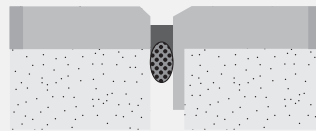


After

DOWN SIZE JOINTS

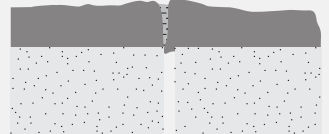


Before

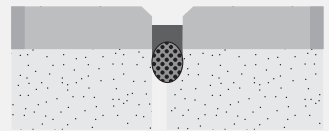


After

CLOSED JOINT RESIZE



Before



After

THE MOST WIDELY
USED EXPANSION JOINT
REHABILITATION SYSTEM
IN AUSTRALIA

ULTRA HIGH MOVEMENT CAPACITY

NEUTRAL POSITION



view from side



view from top

HORIZONTAL MOVEMENT



view from side



view from top

SKEWED MOVEMENT



view from side



view from top

ULTRA HIGH MOVEMENT CAPACITY

- > Dow Corning® 902RCS has the highest movement capacity of any formed-in-place sealant currently available to market.
- > +100% / -50% of installed gap width for joint 25mm – 75mm wide.
- > 902RCS is also ideal for skew joints because of its non-directional strain capacity attributes. The joint gap width should be nominated such that the resultant movement vector will be less than 100% of the installed gap width.

HORIZONTAL MOVEMENT CAPACITY			
Joint Install Width (mm)	"Joint Min (-50%)" (mm)	"Joint Max (+100%)" (mm)	Linear Yield per 40oz Kit (m)
25	12.5	50	2.74
30	15	60	2.29
35	17.5	70	1.83
40	20	80	1.75
45	22.5	90	1.52
50	25	100	1.37
55	27.5	110	1.22
60	30	120	1.15
65	32.5	130	1.00
70	35	140	0.92
75	37.5	150	0.77

*Maximum seal bead thickness should be 13mm at centre. N.B. AS5100.4 limits gaps to 85mm ULS for stripseal expansion joints at full opening.

902RCS DURING CYCLIC MOVEMENT TEST

A sample of 902RCS being tested 4,250 cycles with the ultimate limit state +100% / -50% design movement range (equivalent to over 11 years with maximum strain acting every single day).



REPAIRABLE

- > If an XJS joint is damaged, only the damaged area needs to be replaced, unlike many other systems which require total removal and replacement.
- > Both Silspec 900 Polymer Nosing and 902RCS materials will tenaciously bond to themselves (new to old) ensuring high quality patch repairs.
- > XJS joint patching can be accomplished quickly and economically with maintenance personnel.

ECONOMICAL

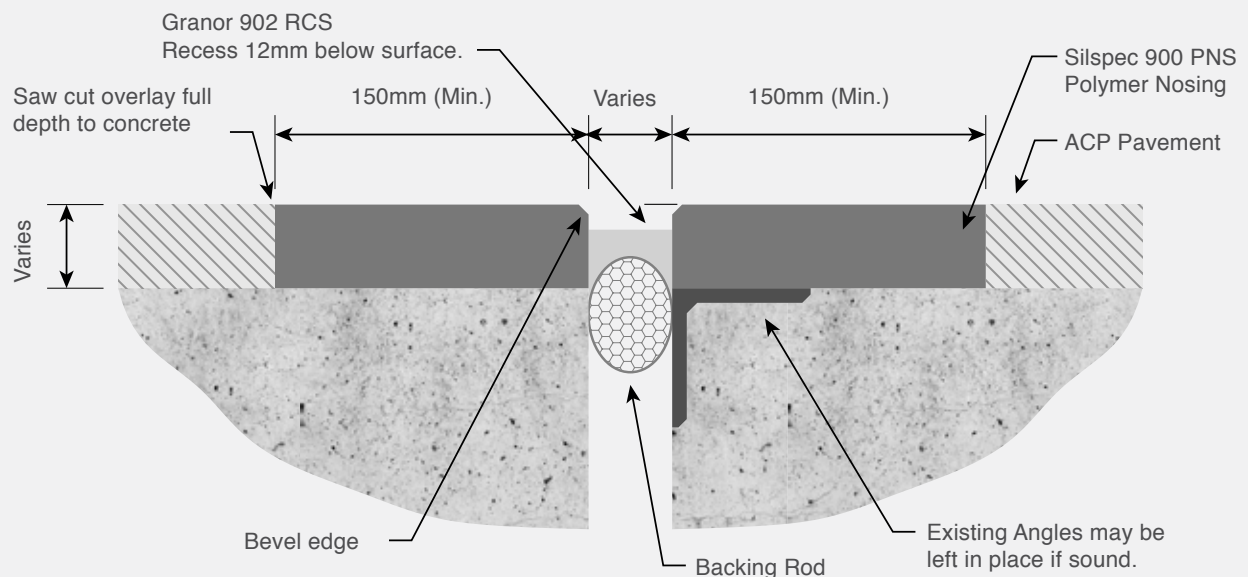
- > XJS has very competitive initial installation costs.
- > The proven longevity across a vast range of climates and the ease of maintenance combine to give XJS one of the lowest life cycle costs of any joint system available.

AVAILABILITY

- > The XJS expansion joint system is available from Granor Rubber and Engineering Pty Ltd. The material is stocked in the Melbourne warehouse and delivered to projects anywhere in Australia, New Zealand, PNG and South East Asia.
- > Silspec 900PNS is available in 14 Litre "Bucket" kits, and 28 Litre "Large" bagged kits.
- > Granor has available mortar mixers tailored for use with Silspec.
- > 902RCS is available in 40oz yield kits.
- > Granor has available pneumatic applicator guns for the 902RCS.

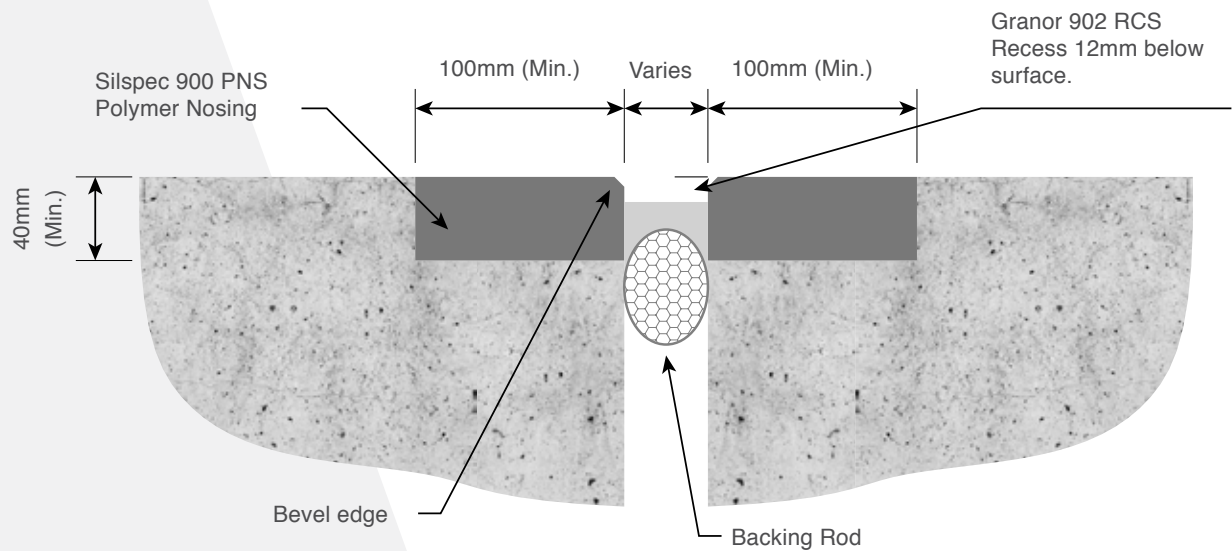
SPECIFICATIONS

XJS STANDARD DRAWING FOR ACP OVERLAYS.



- > Minimum 150mm nosing width when asphalt adjacent
- > Minimum 40mm depth Silspec 900 Polymer Nosing
- > For asphalt depths deeper than 150mm the nosing width should be increased to maintain a minimum 1:1 Width to Depth ratio. (eg. 200mm asphalt depth required minimum 200mm wide nosing)


XJS STANDARD DRAWING FOR NEW WORK INTO CONCRETE DECK.



- > Minimum 100mm nosing width when concrete adjacent
- > Minimum 40mm depth Silspec 900 Polymer Nosing


FOR A COMPLETE INSTALLATION
GUIDE VISIT GRANOR.COM.AU





**GRANOR XJS®
IS A TOUGH
VERSATILE RAPID
CURING EXPANSION
JOINT SYSTEM**

GRANOR.COM.AU



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BAYSWATER, VICTORIA 3153
AUSTRALIA

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TELEPHONE. +61 3 9762 9699

GRANOR.COM.AU



Product Data Sheet

InstaProof HA80

Two part, Polyurea Hybrid Waterproofing and Protective Coating System

Description

Duram InstaProof HA80 is a two component, 100% solids (solvent free), spray applied, elastomeric, hybrid polyurea waterproofing membrane and protective coating.

Duram InstaProof HA80 cures, within seconds to provide a tough, high tensile strength, abrasion and corrosion resistant, seamless membrane and protective coating.

Uses

Duram InstaProof HA80 is suitable as a:

- Waterproofing membrane over most construction substrates.
 - Abrasion resistant protective coating.
 - Corrosion resistant protective coating.
 - Chemical resistant coating.
 - Protective coating in marine environments.
 - Protective liner for landfill and waste materials
 - Wear resistant coating / liner for mining operation.
 - Protective truck, horse float, ute liner.
 - Trafficable membrane for vehicle parking decks and bridges.
 - Protective liner for cargo holds and boat liner.
 - Sewerage tank liner.
-

Suitable Surfaces

Duram InstaProof HA80 is suitable for use over suitably primed and prepared:

- Concrete and cementitious surfaces
 - Timber
 - Steel, aluminium, galvanised metal.
 - Fibreglass resin coating
-

Additional Uses

Duram InstaProof HA80 is an excellent waterproofing, abrasion resistant, chemical resistant, trafficable protective coating and therefore has numerous applications on to numerous substrates.

In all cases, an adhesion test is recommended prior to application.

Specification

The information contained in this product data sheet is typical but does not constitute a full specification as conditions and specific requirements may vary from project to project. The instructions should be considered as a minimum requirement but the applicator or contractor must use their skill, knowledge and experience to carry out additional works as may be necessary to meet the requirements of the project. Specification for specific projects should be sought from the Company in writing.

Limitations

- As InstaProof HA80 is a dual component, fast setting, spray applied polyurea membrane coating, requiring appropriate skills in machine set-up, machine functionality and spraying techniques and it should only be used and applied by an experienced applicator.
 - Do not apply if material and substrate temperatures are below 25°C or 2°C respectively or if the substrate temperature is greater than 50°C.
 - **UV Resistance:** Due to its aromatic composition, the product will discolour and yellow when subject to exposure to UV light. It is recommended to top coat the product with an aliphatic polyurethane top coat or other suitable coating which should be done with 6 hours of applying InstaProof HA80.
 - Structural movement and crack development must be below the tolerances of the membrane coating or rupture may occur.
-

Benefits and Advantages

Duram InstaProof HA80 is:

- Fast curing.
- 100% solids (solvent free)
- Flexible - Elongation \pm 350%
- High Tensile strength
- Tough
- Seamless
- Abrasion, impact and tear resistant
- Good chemical resistance.
- Single coat application
- Rain resistant within 2 minutes.
- No VOC's

Precautions in Use

Health

Do not spray in confined areas.

Wear suitable breathing respirators.

Gloves, boots and safety goggles or glasses should be worn.

Application

Mask off areas that are not to be coated.

Be wary of wind conditions to avoid over-spray.

Do not dilute the material. Viscosity can be controlled by the temperature of the material.

Regularly check the machine and equipment particularly the filters for build up of material.

Do not open until ready for use.

Priming and Surface Preparation

Good preparation is essential. Surfaces must be sound, stable, dry, clean and free of dust, loose, flaking, friable material and substances that may diminish adhesion.

All surfaces must be properly and suitably prepared. Problems are mainly attributable to poor surface preparation. Polyurea coatings rely on the soundness and structural integrity of the substrate to which they are applied.

Substrates vary from project to project and may have differing porosity, strength, tensile strength, movement, moisture content and water vapour transmissions. Therefore, preparation requirements may likewise vary and information should be sought from the Company.

All surfaces must be clean, dry, sound and free of dirt, dust, loose material, oil, grease, wax, contaminants, laitance, efflorescence, rust, salts and adhesion detracting substances.

Detailing Preparation

Concrete:

- Concrete must be cured for at least 28 days.
- Surface defects such as spalling, concrete cancer, cracks must be suitably corrected.
- Concrete surfaces must have a reasonable degree of porosity and surface profile to allow adhesion.
- Shiny, mirror finish or helicopter finished concrete must be suitably abraded by grinding or shot blasting.
- Prime with one or two coats of Duram Primeseal SP as per the product data sheet. Where water vapour transmission is expected apply two coats of Duram Primeseal SP as per the product data sheet. Apply InstaProof HA80 as soon as the primer is cured and best within 2 hours but must be within 6 hours.
- The product has good tolerance to application in high humidity conditions. However, if applied directly to moist surfaces some surface blistering may occur.

Where Appropriate or necessary:

- Comply with Concrete Surface Preparation ICRI 03732
- Standard for Cleaning Concrete: ASTM D4258.
- Standard for Abrading Concrete: ASTM D4259.
- Comply with Standard for Etching Concrete: D4260.
- Comply with Standard for Measuring Moisture Vapour Emissions: ASTM F1869

For metals, timber, aluminium, galvanised surfaces contact the Company.

Product Preparation Before Use

Thoroughly stir parts A and B separately using a clean and dry mechanical stirrer taking care Not to cross-contaminate the products, then ideally every four hours.

Application

Introduction

Duram InstaProof HA80 is formulated for application through a heated, plural component, high pressure airless spray machine.

Pre-Conditioning

The A & B Components should be at 25°C to 30°C.

Recommended Machine Parameters

Primary and hose heaters: 55°C to 60°C.

Spray Pressure at nozzle: Minimum of 2000 psi.

Suitable spray machines include Gusmer H-3500 or Graco Topcoater using a Gusmer GX7-400 spray gun with a 452 or 453 module.

A Probler gun with 00 or 01 round pattern mix chamber of flat spray tip greater than 0.043" is suitable. For further information regarding the equipment contact the Company.

Transfer pumps of at least 2:1 ratio are required to prevent cavitation of the proportioning pumps.

Mix Ratio

Parts A:B 1:1 (v/v)

Spraying

Spray in a single coat using a 50% overlap to ensure that the coating is evenly applied over the surface at 0.5mm to 0.7mm per pass. The product can be sprayed in multiple passes to any desired thickness. Spraying should be continuous and avoid triggering the gun. The applicator should be aware of even small changes in pressure as this may effect the product and if so, spraying should be suspended and the machine and other factors checked before continuing. Check filters regularly for material build up.

Dry Film Thickness (Minimum)

Waterproofing membrane: 1.5mm

Abrasion, chemical or protective resistant coating: 2mm to 4mm

The guidelines are subject to membrane or coating performance and function requirements, project requirements and conditions of substrates and usages must be increased accordingly. Usage should be project specific and confirmation should be sought from the Company.

Application Temperatures

Material (minimum) 25°C

Substrate (minimum) 2°C Maximum 50°C

Recoating of Older Coatings

Clean with high pressure water spray, Allow to dry.

Remove contaminates.

Abrade surface by lightly grinding to remove top film and oxidised material.

Re-apply the product.

.

Coverage

The stated average coverage rate may vary depending upon type, condition, porosity, texture of the surface and application technique.

The coating can be build up to any thickness, provided it is done in a single, continuous application in a sweeping overlapping technique.

Theoretical Coverage

1mm of thickness requires 1 liter applied to 1m².

Minimum Usage and Dry Film Thicknesses

As a waterproofing membrane: 1.5 litres per 1m² (1.5mm thick)

As a chemical or corrosion resistant coating: 2 litres to 3 litres per 1m² (2mm to 3mm thick).

As an abrasive resistant coating: 3 litres to 4 litres per 1m² (3mm to 4mm thick).

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Colours

Natural: Cream colour

Tinting: The product can be tinted to a variety of colours. For UV and colour resistance deeper colours are recommended.

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Drying and Curing

Drying and curing of the product is affected by type, dryness and porosity of the surface, temperature, humidity, ventilation, climate conditions and application technique and therefore drying and curing can only be given as a guide.

Tack free: 5 to 10 seconds of spraying.

Full Cure: 24 hours

Rain resistant: 2 minutes

Foot Trafficable: 2 to 4 hours

Vehicle Trafficable: 24 hours to 48 hours

Note: The product attains its full properties, strength and resiliance over a 7 day period. It is recommended to allow the product this period before being subjected to heavy traffic.

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Storage

Store in dry, cool, ventilated area away from direct sunlight.

Storage Life: 6 months at recommended temperatures of 20°C to 25°C and minmum of 10°C. Temperatures below 10°C may result in cystallisation of the product requiring the product to be pre-heated before use. Protect from frost.

Rotate drums periodically if stored for long periods.

If crystallisation occurs then contact the Company for melting instructions.

Clean Up

The product cures very fast and is chemical and abrasion resistant making the clean up of spills very difficult requiring surface grinding.

Protect, cover or mask off areas that are not to be coated. Protect unwanted areas from application and over-spray.

Drums should be placed on plastic sheets so as to avoid product from contacting the substrate.

Wet spills of unmixed product, should also be avoided, but should be able to be wiped with cloth and Duram solvent.

.

Tiling, Topping or Top Coating

Exposed Areas:

The cured membrane / coating should be protected from exposure to sunlight and uv by overcoating it with an aliphatic top coat (Multithane ATC A80) which must be applied as soon after the product has been applied but must be within 6 hours.

Not usually directly tiled but can be landscaped.

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Safety & Precautions

When spraying or using InstaProof HA80, observe usual good industrial safety standards and hygiene.

The following safety equipment should be worn: Appropriate gloves, safety glasses, respirator, D-grade protection suit and boots. Avoid breathing in mist.

Although the product does not contain volatile or flammable solvents, keep all sources of ignition away from uncured product.

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For full safety data refer to the products Material Safety Data Sheet. Observe precautions as per label.

Tests and Technical Data

Hardness Shore A: $\pm 85 (\pm 45 D)$

Elongation: $\pm 350\%$

Tensile Strength 12 MPa ASTM D 412-92

Tear Strength: 44 N/mm ATM D 624-86

Abrasion resistance: 175mg H18 wheel, 1000g, 1000 revs

Issued: 1st January 2013 | Valid to: 30th June 2015

Conditions of Use and Disclaimer

The information contained in this Material Data Sheet is given in good faith based upon our current knowledge and does not imply warranty, express or implied. The information is provided and the product is sold on the basis that the product is used for its intended purpose and is used in a proper workmanlike manner in accordance with the instructions of the Product Data Sheet in suitable and safe working conditions. Under no circumstances will the Company be liable for loss, consequential or otherwise, arising from the use of the product.

DURAM PTY LTD ABN 50 612 836 718

The Ultimate in Waterproofing & Protective Coating Technology

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QUEENSLAND: Unit 4, 29 Collinsvale St Rocklea, QLD 4106

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APPENDIX C: STRUCTURAL CALCULATIONS – RAIL PLATFORM

Rail Platform Calculations

1.1 Top reinforcement for critical negative bending moments

Minimum steel for negative M^* at the middle of the support is -3.91 kNm,

$$d = D - \text{cover} - \frac{1}{2} \text{bar dia} - \text{ligature}$$

$$= 800 - 25 - \left(\frac{12}{2}\right) - 12 = 757 \text{ mm}$$

$$\frac{A_{st \min}}{bd} = \frac{0.19 \left(\frac{D}{d}\right)^2 f'_{ct.f}}{f_{sy}}$$

$$f'_{ct.f} = 0.6 \sqrt{f'_c}$$

$$\text{So that, } A_{st,min} = \frac{0.19 \times \left(\frac{800}{757}\right)^2 \times (0.6)(\sqrt{40})}{500} \times 1000 \times 757$$

$$= 1219.13 \text{ mm}^2$$

$$M^* = \phi M_u$$

$$M_u = \frac{M^*}{\phi} = \frac{3.91}{0.8} = 4.89 \text{ kNm}$$

$$Z_u = 0.925 d$$

$$M_u = T Z_u = T \times 0.925 d$$

$$T = \frac{M_u}{0.925 d} = \frac{4.89 \times 10^3}{0.925 \times 757} = 6.98 \text{ kN}$$

$$T = f_{sy} A_{st}$$

$$A_{st} = \frac{T}{f_{sy}} = \frac{6.98 \times 10^3}{500} = 13.96 \text{ mm}^2 < A_{st,min}$$

Choose min steel area of N12@75 cts, so $A_{st} = 1467 \text{ mm}^2$

Re-calculate actual $T = f_{sy} A_{st} = 1467 \times 500 \div 1000 = 733.5 \text{ kN}$

Check the beam is ductile:

$$\alpha_2 = 1.0 - 0.003 f'_c = 0.85 \quad (0.67 \leq \alpha_2 \leq 0.85)$$

$$\gamma = 1.05 - 0.007 f'_c = 0.77 \quad (0.67 \leq \gamma \leq 0.85)$$

$$k_u = \frac{T}{\alpha_2 f'_c b \gamma d} = \frac{733.5 \times 10^3}{0.85 \times 40 \times 1000 \times 0.77 \times 757} = 0.04 < 0.36 \text{ so OK}$$

Re-calculate accurate capacity:

Accurate lever arm, $Z_u = d - \frac{1}{2} \gamma k_u d = 757 - \frac{1}{2} \times 0.77 \times 0.04 \times 757 = 746.21 \text{ mm}$

$$M_u = T Z_u = 605 \times 746.21 \times 10^{-3} = 547.35 \text{ kNm}$$

Design strength $\phi M_u = 0.8 \times 547.35 = 437.88 \text{ kNm} > M^* = 3.91$, hence OK

Adopt N12@75cts at the top, $f'_c = 40 \text{ MPa}$, 25 mm cover.

1.2 Bottom reinforcement for critical positive bending moments

Minimum steel for the maximum positive bending moment M^* is 28.23 kNm at the middle of first slab.

$$d = D - \text{cover} - \frac{1}{2} \text{ bar dia}$$

$$= 225 - 25 - \left(\frac{12}{2}\right) = 194 \text{ mm}$$

$$\frac{A_{st \min}}{bd} = \frac{0.19 \left(\frac{D}{d}\right)^2 f'_{ct.f}}{f_{sy}}$$

$$f'_{ct.f} = 0.6 \sqrt{f'_c}$$

$$\text{So that, } A_{st, \min} = \frac{0.19 \times \left(\frac{225}{194}\right)^2 \times (0.6)(\sqrt{40})}{500} \times 1000 \times 194$$

$$= 376.29 \text{ mm}^2$$

$$M^* = \phi M_u$$

$$M_u = \frac{M^*}{\phi} = \frac{28.23}{0.8} = 35.29 \text{ kNm}$$

$$Z_u = 0.925 d$$

$$M_u = T Z_u = T \times 0.925 d$$

$$T = \frac{M_u}{0.925 d} = \frac{35.29 \times 10^3}{0.925 \times 194} = 196.64 \text{ kN}$$

$$T = f_{sy} A_{st}$$

$$A_{st} = \frac{T}{f_{sy}} = \frac{196.64 \times 10^3}{500} = 393.29 \text{ mm}^2 > A_{st, \min}$$

Choose N12@275 cts, so $A_{st} = 400 \text{ mm}^2$

Re-calculate actual $T = f_{sy} A_{st} = 400 \times 500 \div 1000 = 200 \text{ kN}$

Check the beam is ductile:

$$\alpha_s = 1.0 - 0.003 f'_c = 0.85 (0.67 < \alpha_s < 0.85)$$

$$\gamma = 1.05 - 0.007f'_c = 0.77 \quad (0.67 \leq \gamma \leq 0.85)$$

$$k_u = \frac{T}{\alpha_2 f'_c b \gamma d} = \frac{200 \times 10^3}{0.85 \times 40 \times 1000 \times 0.77 \times 194} = 0.04 < 0.36 \text{ so OK}$$

Re-calculate accurate capacity:

$$\text{Accurate lever arm, } Z_u = d - \frac{1}{2} \gamma k_u d = 194 - \frac{1}{2} \times 0.77 \times 0.04 \times 194 = 191.06 \text{ mm}$$

$$M_u = T Z_u = 220 \times 191.06 \times 10^{-3} = 38.21 \text{ kNm}$$

$$\text{Design strength } \phi M_u = 0.8 \times 38.21 = 30.57 \text{ kNm} > M^* = 28.23, \text{ hence OK}$$

Adopt N12@275cts at the bottom, $f'_c = 40 \text{ MPa}$, 25 mm cover.

APPENDIX D: STRUCTURAL CALCULATIONS – BEAM

Beam Calculations

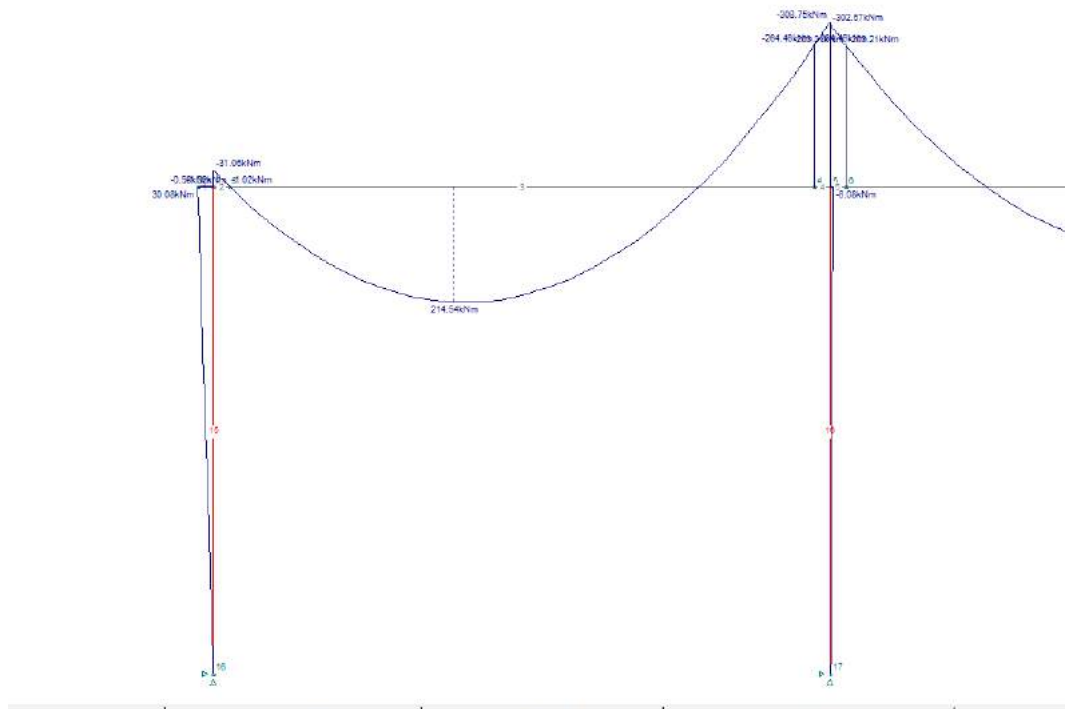


Figure 1: Bending Moments for Beam

It is clearly that the maximum negative bending moment at the middle of the support is -308.75 kNm, and the maximum positive bending moment is 214.54 kNm at the middle of first span.

1.3 Top reinforcement for critical negative bending moments

1.3.1 2.3.1.1 Minimum steel for negative M^*

The minimum steel for negative bending moment at the middle of the support is -308.75 kNm

$$d = D - \text{cover} - \frac{1}{2} \text{ bar dia} - \text{ligature}$$

$$= 800 - 25 - \left(\frac{20}{2}\right) - 12 = 753 \text{ mm}$$

$$\frac{A_{st \min}}{bd} = \frac{0.19 \left(\frac{D}{d}\right)^2 f'_{ct.f}}{f_{sy}}$$

$$f'_{ct.f} = 0.6 \sqrt{f'_c}$$

$$\text{So that, } A_{st, \min} = \frac{0.19 \times \left(\frac{800}{753}\right)^2 \times (0.6)(\sqrt{40})}{500} \times 1000 \times 753$$

$$= 1225.60 \text{ mm}^2$$

$$M^* = \phi M_u$$

$$M_u = \frac{M^*}{\phi} = \frac{308.75}{0.8} = 385.94 \text{ kNm}$$

$$Z_u = 0.925 d$$

$$M_u = T Z_u = T \times 0.925 d$$

$$T = \frac{M_u}{0.925 d} = \frac{385.94 \times 10^3}{0.925 \times 753} = 554.09 \text{ kN}$$

$$T = f_{sy} A_{st}$$

$$A_{st} = \frac{T}{f_{sy}} = \frac{554.09 \times 10^3}{500} = 1108.18 \text{ mm}^2 < A_{st, \min}$$

Choose min steel area of 4N12, so $A_{st} = 1240 \text{ mm}^2$

Re-calculate actual $T = f_{sy} A_{st} = 1240 \times 500 \div 1000 = 620 \text{ kN}$

Check the beam is ductile:

$$\alpha_2 = 1.0 - 0.003 f'_c = 0.85 \quad (0.67 \leq \alpha_2 \leq 0.85)$$

$$\gamma = 1.05 - 0.007 f'_c = 0.77 \quad (0.67 \leq \gamma \leq 0.85)$$

$$k_u = \frac{T}{\alpha_2 f'_c b \gamma d} = \frac{620 \times 10^3}{0.85 \times 40 \times 1000 \times 0.77 \times 753} = 0.03 < 0.36 \text{ so OK}$$

Re-calculate accurate capacity:

Accurate lever arm, $Z_u = d - \frac{1}{2} \gamma k_u d = 753 - \frac{1}{2} \times 0.77 \times 0.03 \times 753 = 743.88 \text{ mm}$

$$M_u = T Z_u = 620 \times 743.88 \times 10^{-3} = 461.21 \text{ kNm}$$

Design strength $\phi M_u = 0.8 \times 461.21 = 368.97 \text{ kNm} > M^* = 308.75$, hence OK

Adopt 4N20 at the top, $f'_c = 40 \text{ MPa}$, 25 mm cover.

1.4 Bottom reinforcement for critical bending moments

Minimum steel for the maximum positive bending moment M^* is 214.54 kNm at the middle of first slab.

$$d = D - \text{cover} - \frac{1}{2} \text{ bar dia} - \text{ligature}$$

$$= 800 - 25 - \left(\frac{20}{2}\right) - 12 = 753 \text{ mm}$$

$$\frac{A_{st \min}}{bd} = \frac{0.19 \left(\frac{D}{d}\right)^2 f'_{ct.f}}{f_{sy}}$$

$$f'_{ct.f} = 0.6 \sqrt{f'_c}$$

$$\text{So that, } A_{st,min} = \frac{0.19 \times \left(\frac{800}{753}\right)^2 \times (0.6)(\sqrt{40})}{500} \times 1000 \times 753$$

$$= 1225.60 \text{ mm}^2$$

$$M^* = \phi M_u$$

$$M_u = \frac{M^*}{\phi} = \frac{214.54}{0.8} = 268.18 \text{ kNm}$$

$$Z_u = 0.925 d$$

$$M_u = T Z_u = T \times 0.925 d$$

$$T = \frac{M_u}{0.925 d} = \frac{268.18 \times 10^3}{0.925 \times 753} = 385.02 \text{ kN}$$

$$T = f_{sy} A_{st}$$

$$A_{st} = \frac{T}{f_{sy}} = \frac{385.02 \times 10^3}{500} = 770.04 \text{ mm}^2 < A_{st,min}$$

Choose min steel area of 4N12, so $A_{st} = 1240 \text{ mm}^2$

Re-calculate actual $T = f_{sy} A_{st} = 1240 \times 500 \div 1000 = 620 \text{ kN}$

Check the beam is ductile:

$$\alpha_2 = 1.0 - 0.003 f'_c = 0.85 \quad (0.67 \leq \alpha_2 \leq 0.85)$$

$$\gamma = 1.05 - 0.007 f'_c = 0.77 \quad (0.67 \leq \gamma \leq 0.85)$$

$$k_u = \frac{T}{\alpha_2 f'_c b \gamma d} = \frac{620 \times 10^3}{0.85 \times 40 \times 1000 \times 0.77 \times 753} = 0.03 < 0.36 \text{ so OK}$$

Re-calculate accurate capacity:

Accurate lever arm, $Z_u = d - \frac{1}{2} \gamma k_u d = 753 - \frac{1}{2} \times 0.77 \times 0.03 \times 753 = 743.88 \text{ mm}$

$$M_u = T Z_u = 620 \times 743.88 \times 10^{-3} = 461.21 \text{ kNm}$$

Design strength $\phi M_u = 0.8 \times 461.21 = 368.97 \text{ kNm} > M^* = 308.75$, hence OK

Adopt 4N20 at the bottom, $f'_c = 40 \text{ MPa}$, 25 mm cover.

1.5 Shear Design

$$V_{uc} = \beta_1 \times \beta_2 \times \beta_3 \times b_v \times d_o \times f_{cv} \times \frac{A_{st}}{b_v \times d_o}^{\frac{1}{3}}$$

Where,

$$d_o = 765 \text{ mm}$$

$$\beta_1 = 1.1 \times \left[1.6 - \frac{765}{1000} \right]$$

$$= 0.91 > 0.8$$

$$\beta_2 = 1 \text{ (As there is no axial tension or compression)}$$

$$\beta_3 = 1 \text{ (No concentrated load near support)}$$

$$A_{st} = 1240 \text{ mm}^2 \text{ (4 N20 Bars)}$$

$$f'_c = 40 \text{ MPa}$$

$$f'_{cv} = 40^{\frac{1}{3}} = 3.41 < 4 \text{ MPa, OK}$$

$$d_o = 765$$

$$b_v = 1000$$

$$V_{uc} = 0.91 \times 1 \times 1 \times 1000 \times 765 \times 3.41 \times \frac{1240}{1000 \times 765}^{\frac{1}{3}}$$

$$= 282.3 \text{ KN}$$

Now,

$$0.5 \times \phi \times V_{uc} \geq V^*$$

$$= 0.5 \times 0.7 \times 282.3$$

$$= 98.8 < V^*, \text{ NOT OK}$$

Shear reinforcement is required

$$V_{u,min} = V_{uc} + (0.1 \times \sqrt{f'_c} \times b_v \times d_o) \geq V_{uc} + (0.6 \times b_v \times d_o) \quad (\text{AS/NZ3600-2009, CL 8.2.9})$$

$$\begin{aligned} L.H.S &= V_{uc} + (0.1 \times \sqrt{f'_c} \times b_v \times d_o) \\ &= 282.3 \times 10^3 + (0.1 \times \sqrt{40} \times 1000 \times 765) \\ &= 766.1 \text{ KN} \end{aligned}$$

$$\begin{aligned} R.H.S &= V_{uc} + (0.6 \times b_v \times d_o) \\ &= 282.3 \times 10^3 + (0.6 \times 1000 \times 765) \\ &= 741.3 \text{ KN} \end{aligned}$$

$$\begin{aligned} \phi \times V_{u,min} &= 0.7 \times 766.1 \\ &= 536.3 \text{ KN} > 181 \text{ KN} \end{aligned}$$

Since, $\phi V_{u,min} > V^*$ condition is satisfied we will find Minimum Shear Reinforcement, $A_{sv,min}$

$$A_{sv,min} = \frac{0.06 \times \sqrt{f'_c} \times b_v \times s}{f'_{sy}} \geq \frac{0.35 \times b_v \times s}{f'_{sy}}$$

$$\frac{A_{sv,min}}{s} = \frac{0.06 \times \sqrt{f'_c} \times b_v}{f'_{sy}} \geq \frac{0.35 \times b_v}{f'_{sy}}$$

$$L.H.S = \frac{0.06 \times \sqrt{40} \times 1000}{500}$$

$$L.H.S = 0.75$$

$$R.H.S = \frac{0.35 \times 1000}{500}$$

$$R.H.S = 0.7 \frac{mm^2}{mm}$$

➤ Calculating Longitudinal Spacing

$$D = 800 \text{ mm}$$

$$S_{max} = 800 \times 0.75 = 600 \text{ mm}$$

Here, 6 N12 ligs are used

$$\frac{A_{sv,min}}{s} = 0.75 \frac{mm^2}{mm}$$

$$\frac{660}{s} = 0.75 \frac{mm^2}{mm}$$

$$s = 870 \text{ mm}$$

Hence Consider longitudinal spacing equal to 800 mm

1.6 Beam Deflection

At Mid-span

$$b_{eff} = b_w + 0.1 \times 0.7 \times L = 1000 + (0.1) \times (0.7) \times (10000)$$

(AS 3600-2009, CL 8.8.2)

$$b_{eff} = 1700 \text{ mm}$$

$$\text{Neutral axis depth, } d_n = \frac{1700 \times 225 \times 112.5 + 575 \times 1000 \times (287.5 - 225)}{1700 \times 225 + 575 \times 1000} = 352.7 \text{ mm}$$

$$I_g = \sum I + Ay^2$$

$$I_g = [1700 \times \frac{250^3}{12} + 1700 \times 225 \times (112.5 - 352.7)^2] + [1000 \times \frac{(800 - 225)^3}{12} + 1000 \times 575 \times (287.5 + 225 - 352.7)^2]$$

$$I_g = 54.8 \times 10^9 \text{ mm}^4$$

Now,

$$I_{eff} = I_{cr} + (I + I_{cr}) \left(\frac{M_{cr}}{M_s} \right)^3 \leq I_{ef,max}$$

(AS 3600, CL 8.5.3.1(1))

➤ **Determining I_{cr} :**

$$I_{cr} = \frac{bx^3}{3} + (n-1)A_{sc}(x-d_c)^2 + nA_{st}(d-x)^2$$

$$d_c = \text{cover} + \text{ligature diameter} + 0.5 * \text{bar diameter} = 25 + 12 + 0.5 * 20 = 47 \text{ mm}$$

$$d = D - \text{Cover} - \text{ligature diameter} - 0.5 * \text{bar diameter} = 800 - 25 - 12 - 0.5 * 20 = 753 \text{ mm}$$

where, x = Neutral axis depth

$$n = \frac{E_{steel}}{E_{concrete}}$$

$$f'_c = 40 \text{ MPa} \quad E_{steel} = 200,000 \text{ MPa}$$

$$E_{concrete} = 32800 \text{ MPa} \quad (\text{AS3600-2009 Table 3.1.2})$$

$$n = \frac{200000}{32800} = 6.1 \approx 7$$

$$A_{sc} = A_{st} = 1240 \text{ mm}^2$$

$$(n-1)A_{sc} = (7-1) \times 1240 = 7440 \text{ mm}^2$$

$$nA_{st} = 7 \times 1240 = 8680 \text{ mm}^2$$

In order to find neutral axis depth(x) we have the following equation:

$$\frac{bx^2}{2} + (n-1)A_{sc}(x-d_c) = nA_{st}(d-x) \quad (\text{Reinforced concrete Course notes})$$

$$\frac{1700x^2}{2} + (x-47) = (753-x)$$

$$x = 84.6 \text{ mm} \rightarrow \text{neutral axis within the flange} \Rightarrow OK$$

$$I_{cr} = \frac{1700 \times 84.6^3}{3} + 7740(84.6-47)^2 + 8680(753-84.6)^2$$

$$I_{cr} = 4.2 \times 10^9 \text{ mm}^4$$

➤ **Determining M_{cr} :**

(From AS 3600 2009, CL 8.5.3.1)

$$M_{cr} = z \left(f'_{ct,f} - \sigma_{cs} + \frac{P}{Ag} \right) + P_e \geq 0$$

Here, $P = 0$ (Tendon force)

$$P_e = 0 \text{ (Prestressed Force)}$$

$$M_{cr} = z \left(f'_{ct,f} - \sigma_{cs} \right)$$

$$f'_{ct,f} = 0.6 \sqrt{f'_c} = 0.6 \sqrt{40} = 3.79 \text{ MPa}$$

$$\sigma_{cs} = \frac{2.5p_\omega - 0.8p_{c\omega}}{1+50p_\omega} \times E_s E_{cs} \quad (\text{AS 3600 - 2009, CL 8.5.3.1})$$

where, $p\omega = \frac{A_{st} + A_{pt}}{b_w d}$

$$p\omega = \frac{1240 + 0}{1000 \times 753} = 0.002$$

$$p_{c\omega} = \frac{A_{sc}}{b_w d}$$

$$p_{c\omega} = \frac{1240}{1000 \times 753} = 0.002$$

ϵ_{cs}^* : typical final design shrinkage strain

$$th = \frac{2Ag}{u_e} \quad (\text{Hypothetical thickness})$$

$$u_e = \text{closed perimeter} + \frac{1}{2} \text{closed perimeter}$$

$$th = \frac{2(1700 \times 225 + 1000 \times 575)}{1700 + 1700 + 800 + 575} = 401 \text{ mm}$$

$$E_{cs}^* = 390 \times 10^{-6}$$

$$\sigma_{cs} = \frac{(2 \times 0.002) - (0.8 \times 0.002)}{1 + 50(0.002)} \times 200000 \times 390 \times 10^{-6}$$

$$\sigma_{cs} = 0.42 \text{ MPa}$$

$$z = \frac{I}{Y} = \frac{54.8 \times 10^9}{(753 - 352.7)} = 1.36 \times 10^8 \text{ mm}^3$$

$$M_{cr} = 1.36 \times 10^8 (3.79 - 0.42)$$

$$M_{cr} = 458.3 \text{ KN.m}$$

$$M_s = \frac{\omega l^2}{8}$$

where, $\omega = G + \psi_s Q$

$$\psi_s = 0.7 \quad (\text{AS 1170.1, Table 4.1})$$

G = self-weight of beam + Gslab

$$= 24 \times 1 \times (0.8 - 0.225) + (24 \times 0.225 \times 2.5)$$

$$= 27.3 \text{ KN/m}$$

Q = 5 KPa x Load width

$$= 5 \text{ KPa} \times 2.5 \text{ m}$$

$$= 12.5 \text{ KN/m}$$

$$\omega = 27.3 + 0.7(12.5)$$

$$L = 10000 \text{ mm}$$

$$M_s = \frac{36.1(1000)^2}{8} = 451.25 \text{ KN.m}$$

$$\text{Therefore, } I_{eff} = I_{cr} + (I_g - I_{cr})\left(\frac{M_{cr}}{M_s}\right)^3$$

$$I_{eff} = 4.2 \times 10^9 + (54.8 \times 10^9 - 4.2 \times 10^9)\left(\frac{458.3}{451.25}\right)^3$$

$$I_{eff} = 57.2 \times 10^9 \text{ mm}^4$$

Checking Maximum Limit for I_{eff} :

$$I_{ef, \max} = 0.6 \times I_g$$

$$I_{ef, \max} = 0.6 \times 54.8 \times 10^9 = 32.8 \times 10^9 \text{ mm}^4$$

Since $I_{eff} > I_{ef, \max}$

Use I_{eff} to calculate deflection

$$\Delta_{\text{short term}} = \frac{5}{384} \times \frac{(G + \psi_s Q)L^4}{E_c \times I_{eff}}$$

$$\Delta_{\text{short term}} = 2.5 \text{ mm}$$

➤ Long Term Deflection

$$k_{cs} = \left(2 - 1.2 \times \frac{A_{sc}}{A_{st}}\right) \geq 0.8$$

$$k_{cs} = \left(2 - 1.2 \times \frac{15 \times 310}{15 \times 310}\right) = 0.8$$

$$\omega_L = G + \psi_L Q$$

$$\psi_L = 0.4$$

(AS 1170.1, Table 4.1)

$$\omega = 27.3 + 0.4(12.5) = 32.3$$

$$\Delta'_{\text{short term}} = \Delta_{\text{short term}} \times \frac{(G + \psi_L Q)}{G + \psi_s Q}$$

$$\Delta'_{\text{short term}} = 2.5 \times \frac{32.3}{36.1}$$

$$\Delta'_{\text{short term}} = 2.2 \text{ mm}$$

$$\Delta_{\text{Long Term}} = \Delta'_{\text{short term}} \times k_{cs}$$

$$\Delta_{\text{Long Term}} = 2.2 \times 0.8$$

$$\Delta_{\text{Long Term}} = 1.8 \text{ mm}$$

Hence, $\Delta_{\text{Total}} = \Delta_{\text{Long Term}} + \Delta_{\text{short term}}$

$$\Delta_{\text{Total}} = 2.5 + 1.8 = 4.3 \text{ mm}$$

➤ **Span of Depth Ratio:**

$$\frac{Lef}{250} > \Delta_{\text{Total}}$$

where, $lef = L = 8500 \text{ mm}$

$$\frac{10000}{250} = 40 \text{ mm} > 4.3 \text{ mm}$$

Beam is satisfactory for deflection

Hence longitudinal spacing of 300 mm is used.

APPENDIX E: STRUCTURAL CALCULATIONS – COLUMN

Column Calculations

Noted that we are dealing with braced columns, thus k will be determined using AS3600 Figure 10.5.3(B).

$$\gamma_1 = \frac{\left(\frac{I}{L}\right)_c}{\left(\frac{EI}{L}\right)_b}$$

$$\beta = 1.0 \quad (\text{Rigidly connected to column}) \quad \text{CI 10.5.6}$$

$$I_c = \frac{BD^3}{12} = \frac{(500)(500)^3}{12} = 5208 * 10^6 \text{ mm}^4$$

$$L_c = 7700 \text{ mm}$$

Length of beam, $L_b = 40000 \text{ mm}$

$$\begin{aligned} b_{ef} &= b_w + 0.1a \\ &= 1000 + 0.1 * (0.7 * 40000) \\ &= 3800 \text{ mm} \end{aligned}$$

$$\begin{aligned} N.A &= \frac{3800 * 225 * \left(800 - \frac{225}{2}\right) + 1000 * (800 - 225) * \left(\frac{800 - 225}{2}\right)}{3800 * 225 + 1000 * (800 - 225)} \\ &= 527 \text{ mm} \end{aligned}$$

$$I = \sum I + A\bar{y}^2$$

$$\begin{aligned} I_b &= \frac{(3800)(225)^3}{12} + 3800 * 225 * \left(\frac{225}{2} + (800 - 225) - 447\right)^2 + \frac{1000 * (800 - 225)^3}{12} + 1000 * (800 - 225) * \\ &\quad ((800 - 225)/2 - 447)^2 \\ &= 7.45 * 10^{10} \text{ mm}^4 \end{aligned}$$

$$\begin{aligned} \gamma_1 &= \frac{\left(\frac{I}{L}\right)_c}{\left(\frac{EI}{L}\right)_b} = \frac{\left(\frac{5208 * 10^6}{7700}\right)}{\left(\frac{(1.0)(7.45 * 10^{10})}{40000}\right)} \\ &= 0.363 \end{aligned}$$

$$\gamma_2 = 10 \quad \text{CI 10.5.6}$$

After γ_1 and γ_2 have been calculated, k can be found using Figure 10.5.3(B) which gives 0.78.

$$\begin{aligned} L_u &= 7700 - 800 \\ &= 6900 \text{ mm} \end{aligned}$$

For rectangular cross-section column, r is taken as 0.3D,

$$\begin{aligned} r &= 0.3 * 500 \\ &= 150 \end{aligned}$$

$$\begin{aligned} L_e &= kL_u \quad \text{CI 10.5.3} \\ &= 0.78 * 6900 \end{aligned}$$

$$\begin{aligned}
 &= 5382 \text{ mm} \\
 \frac{Le}{r} &= \frac{5382}{150} \\
 &= 33.88 > 25 \quad \textbf{(Slender Column)}
 \end{aligned}$$

1. 3.2 Determine the total design loads on the columns

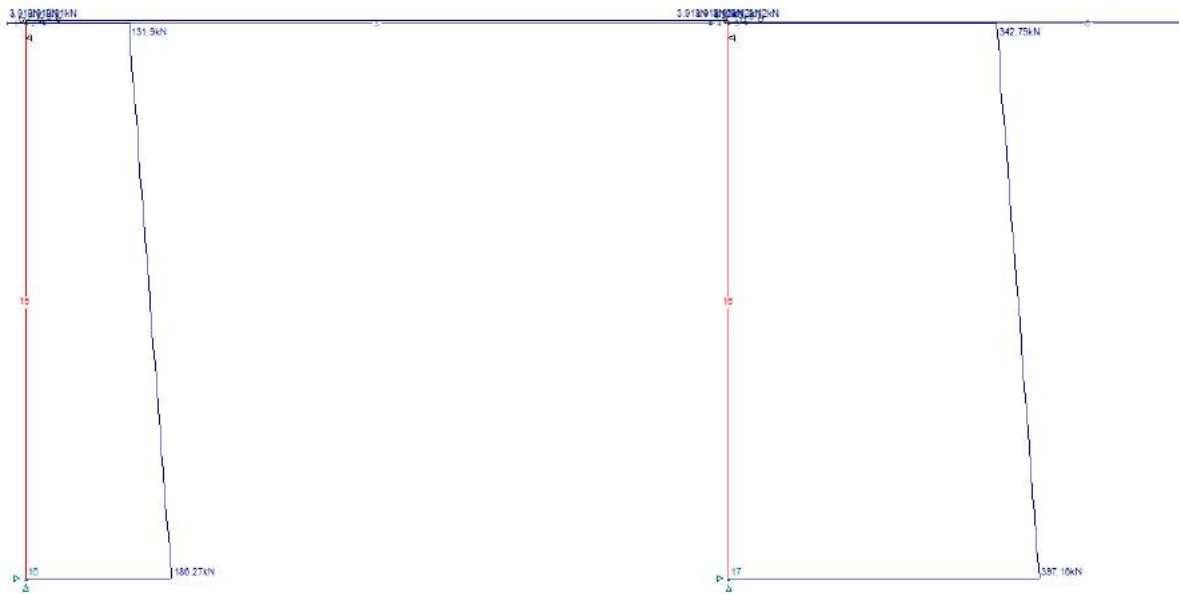


Figure 2: Axial Force Diagram

At first support, $N^* = 186.27 \text{ kN}$

$$\begin{aligned}
 \text{Minimum bending moment, } M_{\min} &= 0.05N^*D \quad \text{CI 10.1.2} \\
 &= 0.05 \times 186.27 \times 0.500 \\
 &= 4.66 \text{ kNm}
 \end{aligned}$$

At second column, $N^* = 397.16 \text{ kN}$

$$\begin{aligned}
 \text{Minimum bending moment, } M_{\min} &= 0.05 \times 397.16 \times 0.500 \\
 &= 9.93 \text{ kNm}
 \end{aligned}$$

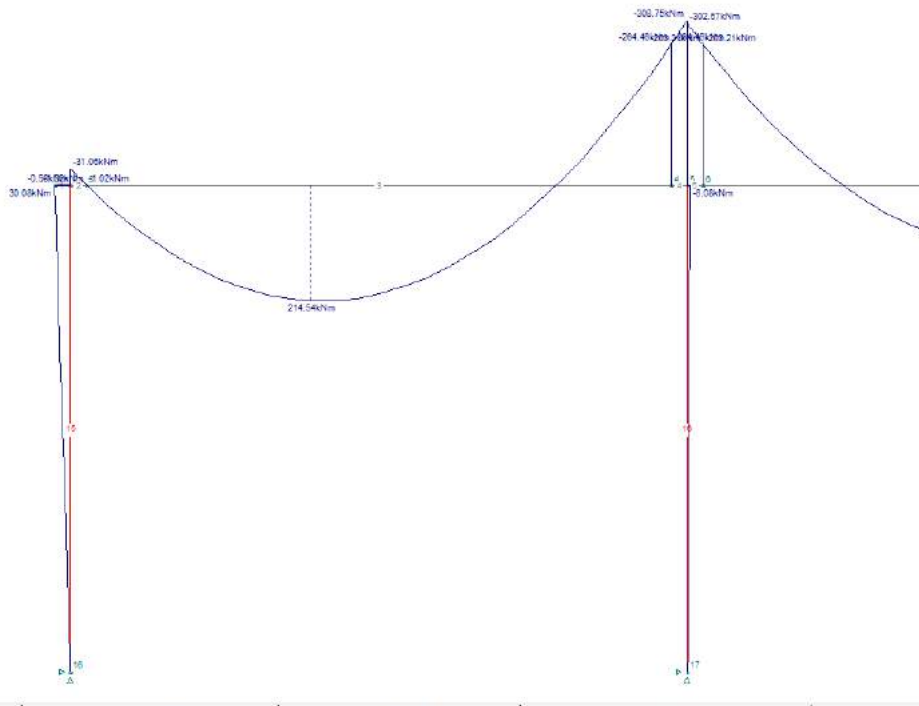


Figure 3: Bending Moment Diagram

As shown in figure 2, column 1 and 2 have 30.08 kNm and 6.08 kNm bending moment respectively. We will adopt the critical force 186.27 kN, critical bending moment 30.08 kNm for column 1 design, and adopt the critical force 397.16 kN, critical bending moment 9.93 kNm for column 2 design.

Minimum cross-sectional area of longitudinal reinforcement shall not be less than $0.01 A_g$.

CI 10.7.1

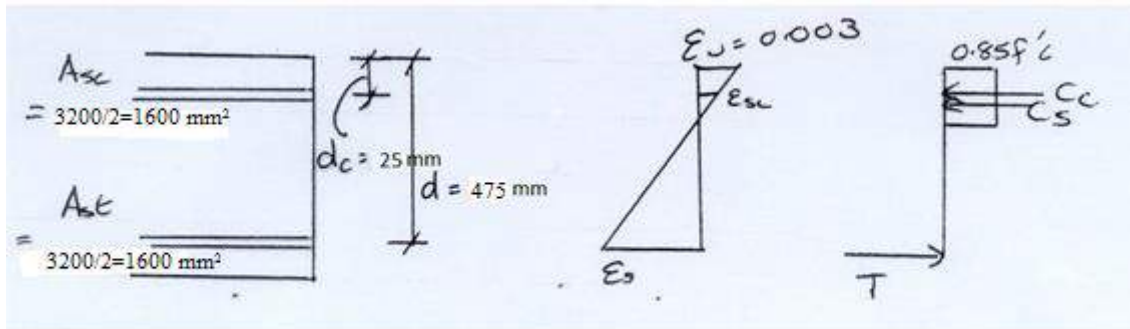
$$\begin{aligned} A_{min} &= 0.01 A_g \\ &= 0.01 * 500^2 \\ &= 2500 \text{ mm}^2 \end{aligned}$$

To provide the fitments in four point to form a square shape, 4 bars are required and satisfy the requirements at the same time.

Adopt 4N32 (3200 mm²)

2. 3.3 Determine column design curve assuming minimum reinforcement

Assume same minimum reinforcement for both columns.



3. 3.3.1 Squash load point (0, N_{uo})

$$\begin{aligned}
 N_{uo} &= \alpha(f'_c)(A_g) + (f_{sy})(A_s) \quad \alpha = 1.0 - 0.003 \cdot 40 = 0.88, \text{ take } 0.85 \\
 &= 0.85 \cdot 40 \cdot 500^2 + 500 \cdot 3200 \\
 &= 10100 \text{ kN}
 \end{aligned}$$

4. 3.3.2 Pure bending point (M_{uo}, 0)

$$\begin{aligned}
 T &= f_{sy} A_{st} = 500 \cdot \left(\frac{3200}{2} \right) / 1000 \\
 &= 800 \text{ kN}
 \end{aligned}$$

Assume $k_{ud} = 25 \text{ mm}$, $d_c = 25 \text{ mm}$, $C_s = 0$

$$\begin{aligned}
 C_c &= 0.85 \gamma k_{ud} b f'_c, \quad \gamma = 1.05 - 0.007(40) = 0.77 \\
 &= 0.85 \cdot 40 \cdot 500 \cdot 0.77 \cdot 25 / 1000 \\
 &= 327.25 \text{ kN}
 \end{aligned}$$

$C_c < T$, need higher k_{ud}

Try $k_{ud} = 30 \text{ mm}$,

$$\begin{aligned}
 C_c &= 0.85 \cdot 40 \cdot 500 \cdot 0.77 \cdot 30 / 1000 \\
 &= 392.7 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 \epsilon_{sc} &= \frac{5}{30} \times 0.003 \\
 &= 5 \times 10^{-4}
 \end{aligned}$$

$$\begin{aligned}
 C_s &= A_{sc} E \epsilon_{sc} \\
 &= 1600 \cdot 200 \cdot 10^3 \cdot 5 \times 10^{-4} \\
 &= 160 \text{ kN}
 \end{aligned}$$

$$\begin{aligned}
 C_s + C_c &= 160 + 392.7 \\
 &= 552.70 \text{ kN} < 800 \text{ kN}
 \end{aligned}$$

Use Excel goal seek function find

$k_{ud} = 37 \text{ mm}$,

$$\begin{aligned}
 C_c &= 0.85 \cdot 40 \cdot 500 \cdot 0.77 \cdot 37 \\
 &= 484.33 \text{ kN} \\
 \epsilon_{sc} &= \frac{12}{37} \cdot 0.003 \\
 &= 9.73 \cdot 10^{-4} \\
 C_s &= A_{sc} E \epsilon_{sc} \\
 &= 1600 \cdot 200 \cdot 10^3 \cdot 9.73 \cdot 10^{-4} \\
 &= 311.35 \text{ kN} \\
 T &= C_s + C_c \\
 &= 484.33 + 311.35 \\
 &= 795.68 \approx T = 800 \text{ kN}
 \end{aligned}$$

Therefore N.A. is at 37 mm.

$$\begin{aligned}
 Z_s &= d - d_c \\
 &= 475 - 25 \\
 &= 450 \text{ mm} \\
 Z_c &= d - 0.5 \gamma_k u d \\
 &= 475 - 0.5 \cdot 0.77 \cdot 37 \\
 &= 460.76 \text{ mm}
 \end{aligned}$$

$$\begin{aligned}
 M_{uo} &= C_s Z_s + C_c Z_c \\
 &= 311.35 \cdot 450 + 484.33 \cdot 460.76 \\
 &= 363.27 \text{ kNm}
 \end{aligned}$$

5. 3.3.3 Balanced point ($k_u = 0.545$)

$$\begin{aligned}
 k_u &= 0.545 \\
 d_n &= 0.545 \cdot 475 \\
 &= 258.88 \text{ mm} \\
 \epsilon_{sc} &= \frac{233.88}{258.88} \cdot 0.003 \\
 &= 2.71 \cdot 10^{-3} \\
 C_s &= 1600 \times 200 \times 10^3 \times 2.71 \times 10^{-3} \\
 &= 867.29 \text{ kN}
 \end{aligned}$$

Summation $F = 0$,

$$\begin{aligned}
 N_{ub} &= C_c + C_s - T \\
 &= 0.85 f'_c \gamma_k u d b + C_s - (f_{sy} \cdot A_{st})
 \end{aligned}$$

$$\begin{aligned}
 &= 0.85 \cdot 40 \cdot 0.77 \cdot 0.545 \cdot 475 \cdot 10^{-3} \cdot 500 + 867.29 - (1600 \cdot 500 \cdot 10^{-3}) \\
 &= 3388.67 + 867.29 - 800 \\
 &= 3455.97 \text{ kN}
 \end{aligned}$$

M_{ub} : Take moments about tensile steel

$$\begin{aligned}
 C_s &= 867.29 \text{ kN} & \text{Lever arm} &= 475 - 25 = 450 \text{ mm} \\
 C_c &= 3388.67 \text{ kN} & \text{Lever arm} &= 475 - 0.5 \cdot 0.77 \cdot 0.545 = 474.8 \text{ mm} \\
 N_{ub} &= 3455.97 \text{ kN} & \text{Lever arm} &= h_{ub} \\
 N_{ub} \times h_{ub} &= C_c Z_c + C_s Z_s \\
 H_{ub} &= (3388.67 \cdot 0.45 + 867.29 \cdot 0.4748) / 3455.97 \\
 &= 0.560 \text{ m} \\
 &= 560 \text{ mm} \\
 v &= \frac{500}{2} - 25 \\
 &= 225 \text{ mm} \\
 e &= h - v \\
 &= 560 - 225 \\
 &= 335 \text{ mm} \\
 M_{ub} &= N_u \times e \\
 &= 3455.97 \cdot 335 \cdot 10^{-3} \\
 &= 1159.09 \text{ kNm}
 \end{aligned}$$

6. 3.3.4 Decompression point ($k_u=1$)

$T = 0$,

Summation F:

$$\begin{aligned}
 N_u &= 0.85 f'_c \gamma_k u d b + A_{sc} f_{sy} \\
 &= 0.85 \cdot 40 \cdot 0.77 \cdot 1 \cdot 475 \cdot 10^{-3} \cdot 500 + 1600 \cdot 500 \cdot 10^{-3} \\
 &= 6217.75 + 800 \\
 &= 7017.75 \text{ kN}
 \end{aligned}$$

Take moments about tensile steel

$$\begin{aligned}
 N_u \cdot h &= C_c Z_c + C_s Z_s \\
 7017.75 h &= 6217.75 \cdot (475 - 0.5 \cdot 0.77 \cdot 1 \cdot 475) + 800 \cdot (475 - 25) \\
 h &= 310.12 \text{ mm} \\
 v &= 225 \text{ mm as before} \\
 e &= 310.12 - 225
 \end{aligned}$$

$$\begin{aligned}
 &= 85.12 \text{ mm} \\
 M_u &= N_u \cdot e \\
 &= 7017.75 \cdot 85.12 \cdot 10^{-3} \\
 &= 597.37 \text{ kNm}
 \end{aligned}$$

Form all above, we can produce the table as shown below:

Moment	Axial Load
363.27	0
1159.09	3455.97
597.37	7017.75
0	10100

Table 1. Unfactored interaction curve.

For $N_u = 0$, $\Phi = 0.8$

For $N_u > N_{ub}$, $\Phi = 0.6$

For $N_u < N_{ub}$, $\Phi = 0.6 + [\Phi - 0.6] \left(1 - \frac{N_u}{N_{ub}} \right)$, where $\Phi = 0.8$ for $k_u < 0.36$

Factored interaction curve calculations,

$$N_{uo} = 0.6 \times 10100 = 2871 \text{ kN}$$

$$M_{uo} = 0.8 \times 363.27 = 78.22 \text{ kNm}$$

$$N_u = 0.6 \times 7017.75 = 1972.8 \text{ kN}$$

$$M_u = 0.6 \times 597.37 = 117 \text{ kNm}$$

$$N_{ub} = 0.6 \times 3455.97 = 981 \text{ kN}$$

$$M_{ub} = 0.6 \times 1159.09 = 160.9 \text{ kNm}$$

Table 2: Factored Interaction Curve

Moment	Axial Load
290.616	0
695.454	2073.582
358.422	4210.65
0	6060

The interaction curve is shown below in Figure 4

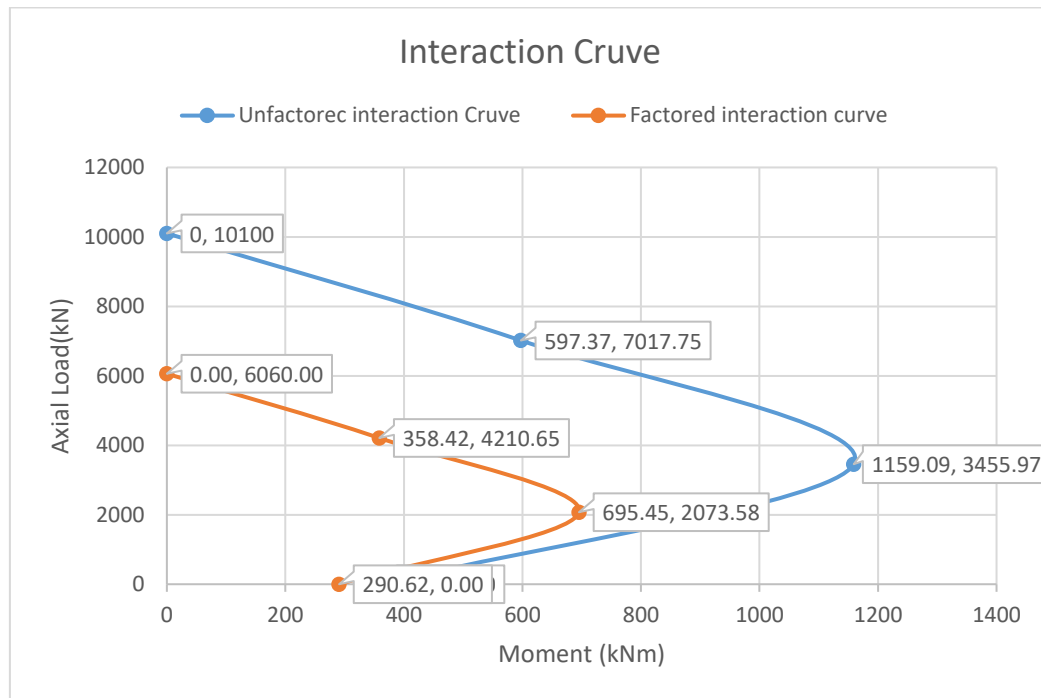


Figure 4: Interaction curve of columns

7. 3.4 Demonstrate that minimum steel is adequate (or not)

By plotting M vs. N for column 1 and 2 in the interaction curve, the dots are observed being within the factored interaction curve (fig 4); hence, the minimum steel is adequate.

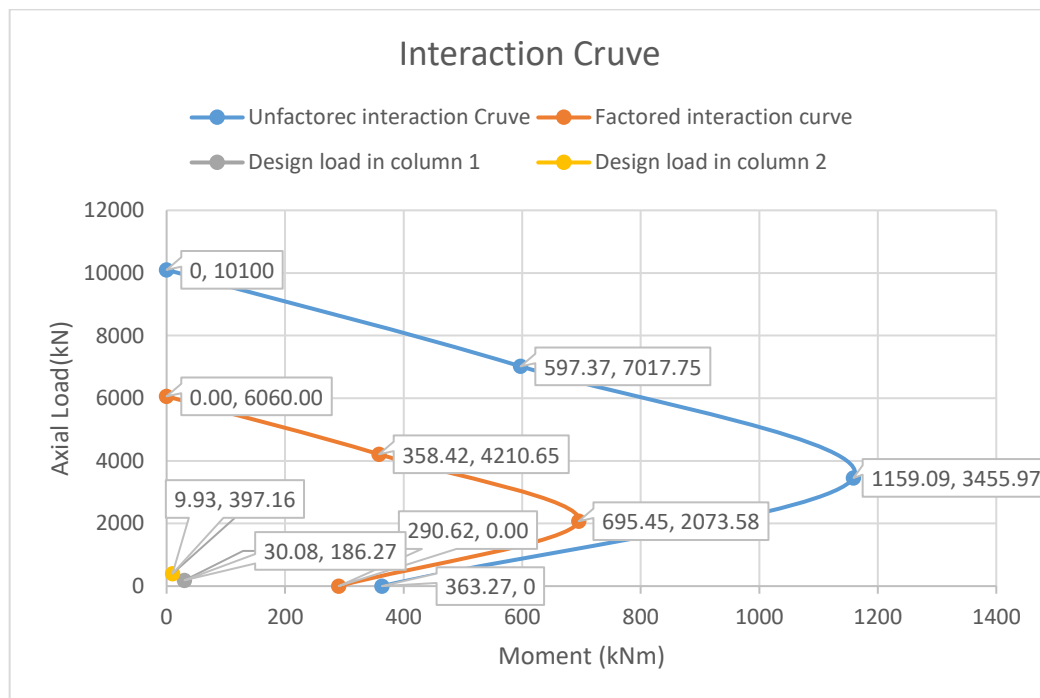


Figure 5: Interaction curve of columns

From the figure, it is clear that the design load is safty.

8. 3.5 Reinforcement detailing

9. 3.5.1 Stirrup Size and Spacing

Stirrup size = 12 mm for N32 bar according to AS3600 Table 10.7.4.3

$$\text{Stirrup spacing, } s = \text{smaller} \left\{ \begin{array}{l} D_c \\ 15d_b \end{array} \right.$$

Where $D_c = 500 \text{ mm}$, $15d_b = 15 \times 32 = 480 \text{ mm}$

Hence, stirrup spacing, $S = 300 \text{ mm}$

10. 3.5.2 Laps with bars from footing below

$$\begin{aligned} \text{Lap length} &= 40 \times \text{bar dia.} \\ &= 40 \times 32 \\ &= 1280 \text{ mm} \end{aligned}$$

Spacing of fitment = lesser of 500 mm or $15 \times 32 = 480 \text{ mm}$, we will take 480 mm.

Adopt 4N32 bars, R6 ligs @ 480 CTS

11. 3.6 Check design using the column charts

$$g = \frac{gD}{D} = \frac{(500 - 2 \times 25 - 2 \times 12 - 32)}{500} = 0.788$$

Use column chart with $g = 0.7$.

$$\begin{cases} N_1^* = 186.27 \text{ kN} \\ M_1^* = 30.08 \text{ kNm} \end{cases}$$

$$\begin{cases} N_2^* = 397.16 \text{ kN} \\ M_2^* = 9.93 \text{ kNm} \end{cases}$$

$$\begin{cases} \frac{N_1^*}{bD} = \frac{186.27 \times 10^3 \text{ N}}{500^2 \text{ mm}^2} = 0.75 \text{ MPa} \\ \frac{M_1^*}{bD^2} = \frac{30.08 \text{ Nmm}}{500^3 \text{ mm}^3} = 2.4 \times 10^{-7} \text{ MPa} \end{cases}$$

$$\begin{cases} \frac{N_2^*}{bD} = \frac{397.16 \times 10^3 \text{ N}}{500^2 \text{ mm}^2} = 1.59 \text{ MPa} \\ \frac{M_2^*}{bD^2} = \frac{9.93 \text{ Nmm}}{500^3 \text{ mm}^3} = 7.94 \times 10^{-8} \text{ MPa} \end{cases}$$

From figure 5 (show as below), two points are all located within curve $p=0$, it means these forces are tension. Hence, the design is satisfactory.

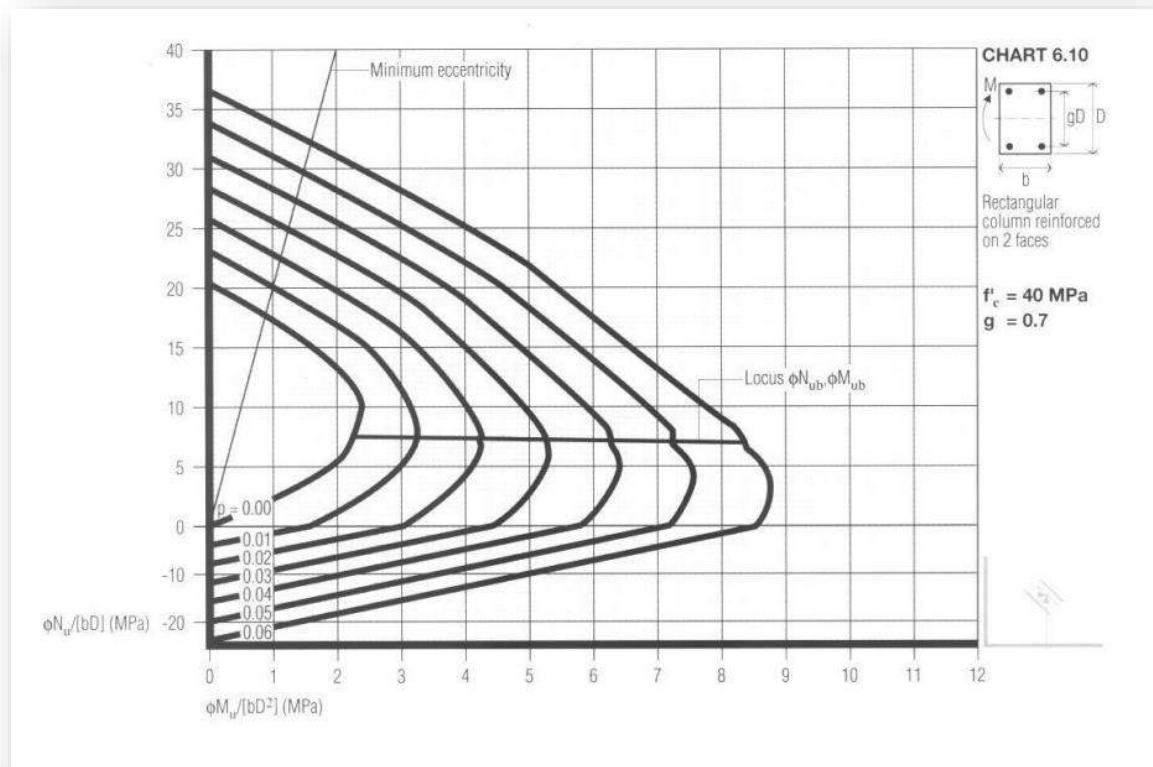


Figure 6: Column charts

APPENDIX F: STRUCTURAL CALCULATIONS – FOOTING

Footing Calculations

1.7 Footing Basic Requirement

Allowable bearing pressure of $q_a = 250 \text{ kPa}$ (From geotechnical group)

Concrete footing strength $f'_c = 25 \text{ MPa}$

Cover

From table 4.3 the surface of maritime structure in sea water then choose permanently submerged, the exposure classification B2.

Table 4.10.3.2. $f'_c = 25 \text{ MPa}$ then cover is 30mm

Check clause 4.10.3.5 the cover must to plus 20mm

Total cover is 50mm

1.7.1 Footing Design Load

Dead Load:

Dead load from column $G = G_{\text{slab}} + G_{\text{beam}} + G_{\text{column}} + G_{\text{service}}$

$$G_{\text{slab}} = 0.225 \times 2.5 \times 5 \times 24 = 67.5 \text{ kN}$$

$$G_{\text{beam}} = 0.575 \times 1 \times 5 \times 24 = 69 \text{ kN}$$

$$G_{\text{column}} = 7.7 \times 0.5 \times 0.5 \times 24 = 46.2 \text{ kN}$$

$$G_{\text{service}} = 1 \times 2.5 \times 5 = 12.5 \text{ kN}$$

Dead load from column $G = 67.5 + 69 + 46.2 + 12.5 = 195.2 \text{ kN}$

Live load from column $Q = 5 \times 2.5 \times 5 = 62.5 \text{ kN}$

1.7.2 Determine Footing Area

Allowable bearing pressure of $q_a = 250 \text{ kPa}$

$$N_{\text{allowable}} = G + Q = 195.2 + 62.5 = 257.7 \text{ kN}$$

$$\frac{N_{\text{allowable}}}{\text{area}} \leq q_a$$

$$\frac{257.7}{\text{area}} \leq 250 \rightarrow \text{area} = \frac{257.7}{250} = 1.03 \text{ m}^2$$

$$\sqrt{1.03} = 1.015 \text{ m then choose } B = 1.5 \text{ m}$$

Try the area $1.5 \times 1.5 = 2.25 \text{ m}^2$

$$\text{thickness of footing } t = \frac{B}{4 \text{ to } 5} = \frac{1500}{4} = 375 \text{ mm}$$

Footing self-weight $= 1.5 \times 1.5 \times 0.375 \times 24 = 20.25 \text{ kN}$

Total dead load include footing self-weight $G=195.2+20.25=215.45$ kN

Check the bearing pressure

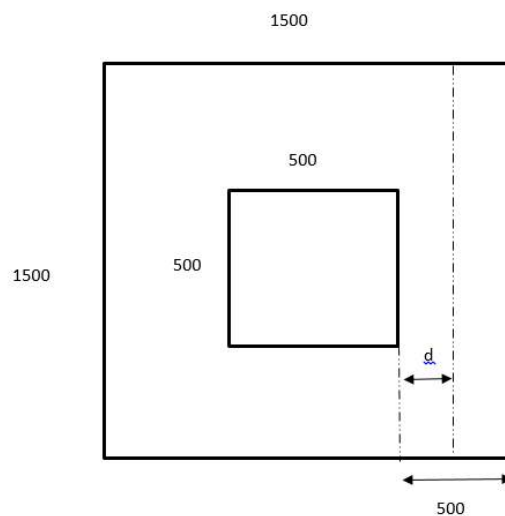
$$\frac{215.45 + 62.5}{2.25} = 123.53 \text{ kPa} \leq 250 \text{ ok}$$

1.7.3 Determine ultimate net effective upward pressure, q_u

$$N^* = 1.2G + 1.5Q = 1.2 * 215.45 + 1.5 * 62.5 = 352.29 \text{ KN}$$

$$q_u = \frac{N^*}{\text{area}} = \frac{352.29}{2.25} = 156.57 \text{ kPa}$$

1.7.4 Determine design M^* , N^* , punching shear and d



1.7.4.1 Beam shear critical section is at 'd' from column face

$$V^* = q_u * B * \left(\frac{B}{2} - \frac{\text{column}}{2} - d \right)$$

$$V^* \leq \phi V_{uc} \text{ with } \phi = 0.7$$

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d f_{cv} \left(\frac{A_{st}}{bd} \right)^{1/3}$$

From clause 9.1.1 assumed the minimum steel

$$\text{assumed } \frac{D}{d} = \frac{1}{0.9}$$

$$\frac{A_{st}}{bd} \geq 0.24 * \left(\frac{D}{d} \right)^2 \frac{f'_{ct.f}}{f_{sy}} = 0.24 * \left(\frac{1}{0.9} \right)^2 \frac{0.6 * \sqrt{25}}{500} = 0.00178$$

$\beta_1 = 1.1$ ($1.6 - d / 1000$) and adopt $\beta_2 = \beta_3 = 1$ (Usually these equal 1)

$$f_{cv} = (f'_c)^{1/3} = 25^{1/3} = 2.92 \text{ MPa}$$

Then

$$V_{uc} = 1.1 * \left(1.6 - \frac{d}{1000} \right) 1 * 1 * 1500 * d * 2.92 (0.00178)^{1/3}$$

$$V_{uc} = 934.24d - 0.584d^2$$

$$V^* = 156.57 * 10^{-3} * 1500 * \left(\frac{1500}{2} - \frac{500}{2} - d\right)$$

$$V^* = 234.86 * (500 - d)$$

Then

$$V^* \leq \phi V_{uc} \text{ with } \phi = 0.7$$

$$234.86 * (500 - d) = 0.7(934.24d - 0.584d^2)$$

$$0.409d^2 - 888.830d + 117430 = 0$$

d=141mm

Check

$$V^* = 234.86 * (500 - d)$$

$$V^* = 84.31 \text{ kN}$$

$$\phi V_{uc} = 0.7 * (934.24d - 0.584d^2)$$

$$\phi V_{uc} = 0.7 * (934.24 * 141 - 0.584 * 141^2)$$

$$\phi V_{uc} = 84.08 \text{ kN within } 1\% \text{ ok}$$

1.7.4.2 Punching shear

$$u = (500+d)*4 = (500+141)*4=2564\text{mm}$$

$$M_v^*=0$$

From clause 9.2.3

$$V^* \leq \phi V_{uo}$$

$$V^* = q_u * (B^2 - (\text{column} + d)^2)$$

$$V^* = 156.57 * (1500^2 - (500 + 141)^2)10^{-6} = 287.96\text{kN}$$

From clause 9.2.3

$$V_{uo} = ud * (f_{cv} + 0.3\sigma_{cp})$$

Because no pre-stressing then $\sigma_{cp} = 0$

$$V_{uo} = u * d * f_{cv}$$

$$f_{cv} = 0.17 * \left(1 + \frac{2}{\beta_h}\right) * \sqrt{f'_c} \leq 0.34 * \sqrt{f'_c}$$

Clause 9.2.1.5: $\beta_h=1$ square column

$$f_{cv} = 0.34 * \sqrt{f'_c} = 0.34 * \sqrt{25} = 1.7 \text{ MPa}$$

$$\phi V_{uo} = 0.7 * 2564 * 141 * 1.7 * 10^{-3} = 430.21\text{kN} > V^* = 287.96 \text{ kN punching shear is ok}$$

1.7.4.3 Bending moment

Check at $0.7 a_{sup}$ from center line to the column

$$M^* = q_u * B * \frac{\left(\frac{B}{2} - 0.7 * \frac{column}{2}\right)^2}{2}$$

$$M^* = 156.57 * 1.5 * \left(\frac{\left(\frac{1.5}{2} - 0.7 * \frac{0.5}{2}\right)^2}{2}\right) = 38.83 kNm$$

Using the minimum A_{st}/bd we calculated in step 3, determine A_{st} min accurately

$$A_{st} = 0.00178 * 1500 * 141 = 376 \text{ mm}^2 \rightarrow \text{choose } 4N12 \text{ (} A_{st} = 440 \text{ mm}^2 \text{)}$$

$$\text{Hence } T = 440 * 500 / 1000 = 220 \text{ kN}$$

$$C = \alpha_2 f'_c b \gamma k_u d = 0.85 * 25 * 1500 * 0.826 * k_u * \frac{141}{1000} = 3712.35 * k_u$$

$$T = C$$

$$K_u = 220 / 3712.35 = 0.059 < 0.36 \text{ ductile ok}$$

$$\phi M_u = 0.8 * 220 * (141 - 0.5 * 0.826 * 0.059 * 141) / 1000 = 24.21 < 38.83 \text{ kNm not ok}$$

Then choose more steel

$$\text{Choose } 8N12 \text{ (} A_{st} = 880 \text{ mm}^2 \text{)}$$

$$\text{Hence } T = 880 * 500 / 1000 = 440 \text{ kN}$$

$$C = \alpha_2 f'_c b \gamma k_u d = 0.85 * 25 * 1500 * 0.826 * k_u * \frac{141}{1000} = 3712.35 * k_u$$

$$T = C$$

$$K_u = 440 / 3712.35 = 0.119 < 0.36 \text{ ductile ok}$$

$$\phi M_u = 0.8 * 440 * (141 - 0.5 * 0.826 * 0.119 * 141) / 1000 = 47.20 > 38.83 \text{ kNm ok for bending}$$

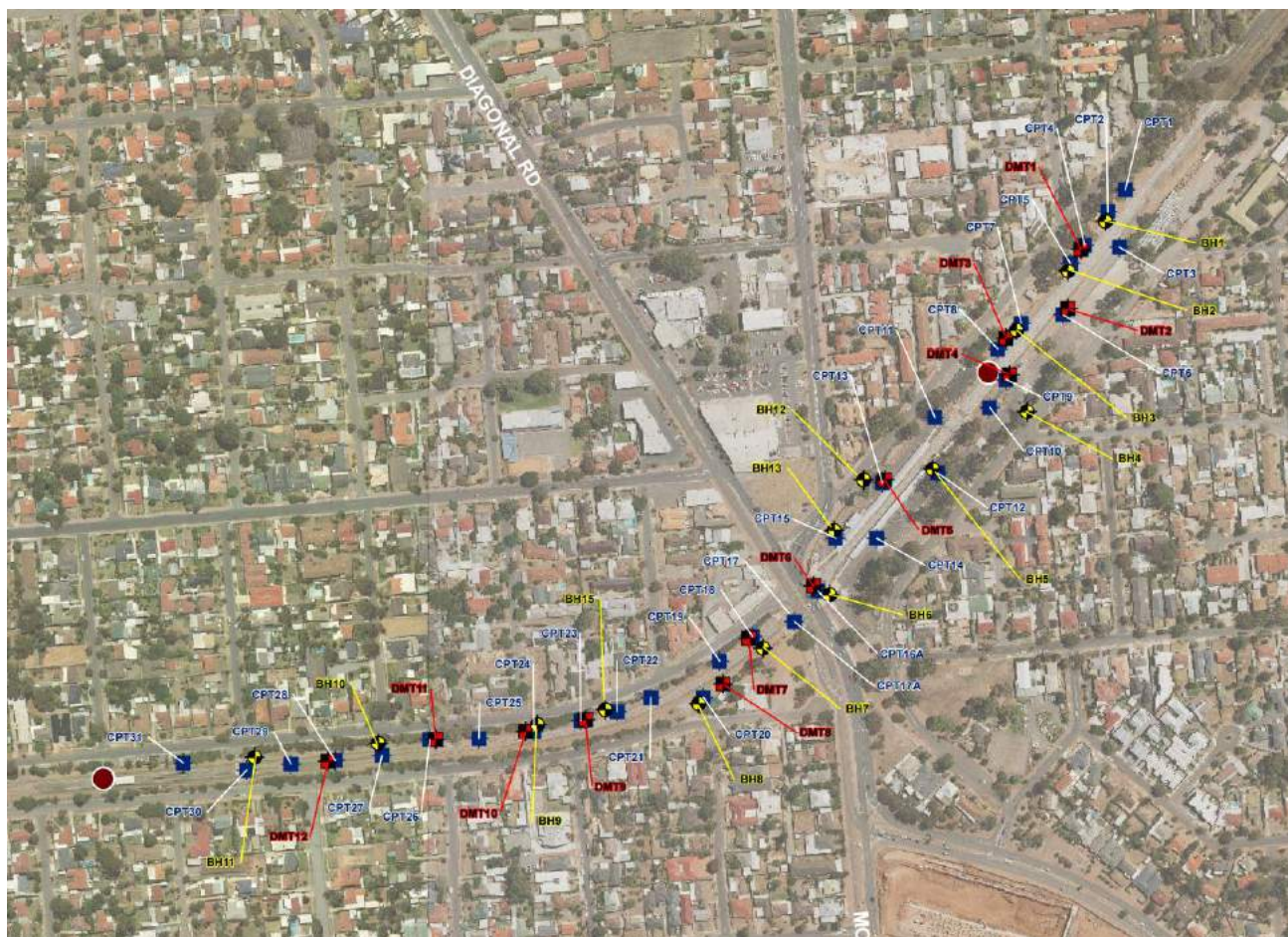
Adopt 8N12 bars each way bottom in 1.5m x 1.5m x 0.375m footing

APPENDIX G: GEOTECHNICAL

Appendix G

G1. Geotechnical investigation

G1.1 Map of borehole location



G1.2 Summarized soil layer profile for OPBH06 – OPBH08

Summary of general subsurface condition for OPBH06-OPBH08

General material description	Typical depth to top of unite (m)	Minimum encountered depth (m)	Maximum recorded depth of material (m)
FILL: Asphalt	0.0	0.0	0.2
Fill: Ballast, Gravel; grey dense	0.0	0.0	0.7
FILL: Clayey sand/ clayey gravelly sand; brown	0.2	0.2	1.5

Top soil: Sandy clay, red-brown, very stiff	0.7	0.7	1.0
Silty CLAY; medium to high plasticity, dark brown, purple-brown, hard	1.0	0.7	5.3
FILL: Sandy clay; brown mottled red-brown, dense	1.5	1.5	2.3
Clayey SAND/Sandy CLAY; low to medium plasticity, brown, orange-brown, red-brown mottled green, grey and white, dense-very dense/very stiff-hard	2.0	0.7	24.8
Clayey Sandy Gravel/Sandy Clayey Gravel; light brown, grey, pink, red, black, very dense	6.5	6.00	9.0
Gravelly Clay SAND/Clayey gravelly SAND; orange0brown, dense to very dense	7.0	6.20	9.0
Sandy CLAY/Silty CLAY; medium to high plasticity, orange-brown mottled green-grey and brown, firm to stiff	12.0	12.0	20.5
Sandy CLAY/Silty CLAY; medium to high plasticity, orange brown, red-brown mottled green-grey, very stiff-hard	13.0	9.0	30.0
Sandy Gravelly CLAY-Sandy Clayey GRAVEL; low plasticity, orange-brown mottled green- grey, very stiff to hard/dense to very dense	14.0	13.4	16.8

G1.3 OPBH 06 soil layer description

Depth (m)	Layers Depth (m)	SOIL/ROCK MATERIAL FIRLD DESCRIPTION	Relative Density/Consistency
0.00	0.2	ASPHALT	
0.20	1.3	FILL: Clayey Gravelly Sand; fine to coarse grained, brown, low plasticity clay, gravel to 40mm, highly calcareous	

1.50	0.8	FILL: Sandy Clay; low to medium plasticity, brown mottled red-brown, fine to medium grained sand, trace gravel to 40mm	VST
2.30	0.2	Clayey SAND; fine to coarse grained, brown mottled orange-brown, low plasticity clay, trace gravel to 30mm	D
2.50	1.3	Sandy CLAY; low to medium plasticity, dark brown, fine to coarse grained sand, trace fine grained gravel	VST
3.80	0.14	CLAY; medium plasticity, with fine to medium grained sand	VST-H
3.94	1.36	CLAY; low plasticity, orange-brown mottled orange, dark brown, with fine to coarse grained sand, with organic at 5.0m	H
5.30	0.45	CLAY; medium plasticity, brown, with fine to medium grained sand, trace fine gravel	H
5.75	0.25	Sandy Clay; low to medium plasticity, orange-brown, fine to coarse grained sand	H
6.00	0.8	Clayey Sandy GRAVEL; fine to coarse grained light brown, grey, pink, low plasticity clay, fine to coarse grained sand, quartzite gravel to 50mm	VD
6.80	0.5	Clayey SAND; fine to coarse grained, brown, low plasticity, clay, quartzite fragment to 30mm	VD
7.30	0.7	Clayey SAND; fine to coarse grained, orange mottled light brown and brown, low plasticity clay, fine to medium grained gravel	VD
8.00	0.5	Clayey Gravelly SAND; fine to coarse grained orange mottled light brown, red and grey, fine to coarse grained gravel to 45mm, low to	VD

		medium plasticity clay, quartzite	
8.50	0.9	Clayey SAND; fine to medium grained, brown, low plasticity clay	VD
9.40	0.4	CLAY; high plasticity, brown, trace fine to coarse grained sand	H
9.80	0.5	CLAY; medium plasticity, orange mottled grey and white, fine to medium grained sand, highly calcareous white veins throughout	H
10.30	1.0	Silty Sandy CLAY; low to medium plasticity, orange-brown mottled green-grey, fine to coarse grained sand	H
11.30	0.45	CLAY; medium plasticity, grey-brown, trace of fine to medium sand	VST-H
11.75	1.05	Sandy CLAY; low plasticity, orange-brown mottled green-grey, fine to coarse grained sand	H
12.80	0.55	CLAY; medium plasticity, grey mottled orange-brown, trace of fine to medium grained sand	ST-VST
13.35	0.95	Sandy Gravelly CLAY; low plasticity, orange-brown mottled green-grey, fine to coarse grained sand	VST-H
14.30	0.45	CLAY; low plasticity, grey-brown, with fine to coarse grained sand, trace fine to medium grained gravel	F-ST
14.75	0.45	Sandy Clayey GRAVEL; fine to medium grained green-grey, black, red-brown, sub-angular to angular, fine to coarse grained sand	H
15.20	0.6	Sandy CLAY; low plasticity, brown mottled, green-grey, red-brown, orange-brown, fine to coarse grained sand, trace fine to medium	VST-H

		grained sub-angular to angular gravel	
15.80	0.45	Clayey Sandy GRAVEL; fine to coarse grained grey, black, green-grey, sub-rounded to sub-angular, low plasticity clay, fine to coarse grained sand	VD
16.25	1.05	Silty CLAY; medium to high plasticity, orange-brown mottled green-grey and brown with black inclusions, with fine to coarse grained sand, trace fine to medium grained gravel	H
17.30	0.45	CLAY; low plasticity, orange-brown mottled grey-green with black inclusions, trace fine to medium grained sand	H
17.75	0.85	Silty CLAY; medium to high plasticity, orange-brown mottled green-grey with black inclusions, with fine to coarse grained sand, trace fine to medium grained gravel	H
18.60	0.7	Silty CLAY; medium to high plasticity, orange-brown mottled green-grey and red-brown with black inclusions, with fine to coarse grained sand, trace fine to medium grained gravel	H
19.30	1.0	Silty CLAY; medium to high plasticity, red mottled orange with black inclusions, with fine to coarse grained sand, trace fine to medium grained gravel	H
20.30	0.45	CLAY; low plasticity, grey-brown, trace of fine to medium grained sand	H
20.75	0.75	Silty CLAY; high plasticity, green-grey mottled orange-brown with dark red inclusions, with fine to coarse grained sand, trace fine to medium	H

		grained gravel	
21.50	1.4	Silty Sand CLAY; medium to high plasticity, orange-brown mottled green-grey, fine to coarse grained sand, trace fine to medium grained sub-angular gravel	H
22.90	0.5	Clayey GRAVEL; fine to coarse grained up to 10mm, white, red, green-grey, medium plasticity clay, fine to coarse grained sand, quartz, quartzite, siltstone gravel	VD
23.40	0.6	Silty Sandy CLAY; medium to high plasticity, orange-brown mottled green-grey, fine to coarse grained sand, with fine to coarse grained sub-angular quartzite gravel	H
24.00	0.5	Silty Sandy CLAY; medium to high plasticity, green-grey mottled orange-brown, fine to coarse grained sand	H
24.50	0.3	CLAY; medium plasticity, pale brown, trace of fine to coarse grained sand	H
24.80	0.5	Silty CLAY; high plasticity, orange-brown mottled grey-green, with fine to coarse grained sand, trace fine to medium grained angular gravel	H
25.30	0.5	Sandy CLAY; medium to high plasticity, orange-brown mottled green-grey, with some dark red-brown, trace fine grained gravel	H
25.80	0.5	Clayey CLAY; fine to coarse grained, brown mottled green-grey, low plasticity clay	H
26.30	0.7	Clayey SAND; fine to coarse grained, brown mottled green-grey, low plasticity clay, trace fine to medium grained gravel	VD

27.00	1.0	CORE LOSS: no recovery	
28.00	1.0	Clayey Gravelly SAND; fine to coarse grained, green-grey, white, red-brown, fine to coarse grained gravel, sub-angular to angular, quartz, low plasticity clay	VD
29.00	0.64	CORE LOSS: no recovery	
29.64	/	Sandy clayey Gravel; fine to medium grained, green-brown, grey, white, sub-angular to angular-fine to coarse grained sand	VD

G2. Other used design information

G2.1 Soil parameters determination table

Was form Section 3 – commentary foundations, AUSTRROADS Bridge Design Code

Table C3.5(B) Properties of Materials

Notes: Characteristic values of undrained shear strength of cohesive soils may also be based on the results of in place tests. The material factor for unit weight is equal to unity so that in Tables C3.5(B) (1, 2 and 3) below, the design values equal characteristic values.

Table C3.5(B)(1) Cohesionless Soils

Soil	Unit Weight (kN/m ³)	Angle of Shearing Resistance ϕ_k (degrees)
Loose gravel with sand content	16-19	28-30
Medium dense gravel with low sand content	18-20	30-36
Dense to very dense gravel with low sand content	19-21	36-45
Loose well graded sandy gravel	18-20	28-30
Medium dense clayey sandy gravel	19-21	30-35
Dense to very dense clayey sandy gravel	21-22	35-40
Loose, coarse to fine sand	17-20	28-30
Medium dense, coarse to fine sand	20-21	30-35
Dense to very dense, coarse to fine sand	21-22	35-40
Loose, fine and silty sand	15-17	28-30
Medium dense, fine & silty sand	17-19	30-35
Dense to very dense, fine and silty sand	19-21	35-40

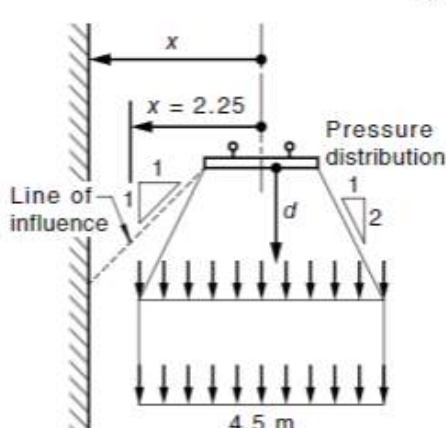
Table C3.5(B)(2) Cohesive and Organic Soils

Soil	Unit Weight (kN/m ³)	Undrained Cohesion c_{uk} (kPa)
Soft plastic clay	16-19	10-25
Firm plastic clay	17.5-20	25-50
Stiff plastic clay	18-21	50-100
Soft slightly plastic clay	17-20	10-25
Firm slightly plastic clay	18-21	25-50
Stiff slightly plastic clay	21-22	50-100
Stiff to hard, glacial clay	20-23	100+
Organic clay	14-17	-
Peat	10.5-14	-

G2.2 Railway surcharge determination table

Was captured from AS5100.2, part of Table14.3

TABLE 14.3
VALUES OF UNFACTORED VERTICAL PRESSURE vs. DEPTH BELOW 2.5 m
LONG SLEEPER (300LA TRAIN LOAD)

Distance from centreline of track (x), m									Depth below sleeper (d) m
x = 7.0	x = 6.0	x = 5.0	x = 4.0	x = 3.0	x = 2.25	x = 1.5	x = 1.25	x = 0.0 abutment approach	
							131	131	0.00
							92	97	0.30
							70	77	0.60
							63	69	0.80
No vertical surcharge							54	58	1.00
							48	53	1.20
							44	50	1.40
							41	48	1.60
							39	45	1.80
							36	43	2.00
							35	42	2.20
							35	45	2.40
							34	41	2.60
							33	40	2.80
							33	39	3.00

G2.3 Ballast profile determination table

Track Class	Max Axle Load (Tonnes)	Max Train Operating Speed			Nominal Maximum MGT/year
		H.S Passenger (km/h)	Passenger (km/h)	Freight (km/h)	
Main line					
1	25	160	115	80	10
2	21	120	100	80	6
3 / 3G	19	-	-	70	5
5	19	-	-	40	1
Siding					
1	25	25	25	25	6
2	21	25	25	25	6
3	19			25	5

Table. Train speed and axle load

(Source: CRN-CS-200, track system, John Holland 2016, Engineering Standard)

Track Class	Rail Size (kg/m)	
	Preferred	Approved Alternatives
Main Line		
1	60	53, 60H
2	50	47, 53
3	41	40, 47
3G	53	
5	30	40, 41,
Sidings		
1	60	53, 50
2	50	47, 53
3	41	40, 47

Table. Selection of rail size

(Source: CRN-CS-220 Rail and rail joints, John Holland 2016, Engineering Standard)

Rail Section Kg/m	Original dimensions		Category 1 (White Rail)		Category 2 (Blue Rail)		Category 3 (Red Rail)		Category 4 (Green Rail)	
	Width mm	Depth mm	Width mm	Depth mm	Width mm	Depth mm	Width mm	Depth mm	Width mm	Depth mm
60 Kg/m	70	44	≥ 66.5	≥ 35	≥ 63.5	≥ 35	> 46	> 26	≤ 46	≤ 26
53 Kg/m *	70	40	≥ 66.5	≥ 35	≥ 63.5	≥ 35	> 46	> 22	≤ 46	≤ 22
50 kg/m	70	40	≥ 66.5	≥ 35	≥ 63.5	≥ 35	> 47	> 22	≤ 47	≤ 22
47 Kg/m *	70	37	≥ 66.5	≥ 33	≥ 63.5	≥ 33	> 46	> 24	≤ 46	≤ 24
41 Kg/m *	63	35	≥ 60	≥ 30	≥ 5	≥ 30	> 41	> 23	≤ 41	≤ 23
80 lb/yard AS (1928) "B" (NEW)	64	40	≥ 60	≥ 30	≥ 57	≥ 30	> 41	> 23	≤ 41	≤ 23
80 lb/yard AS (1928) "A" (OLD)	70	37	≥ 66.5	≥ 27	≥ 63.5	≥ 30	> 46	> 23	≤ 46	≤ 23
80 lb/yard AS (1916) (OLD)	70	37	≥ 66.5	≥ 27	≥ 63.5	≥ 30	> 46	> 23	≤ 46	≤ 23
80 lb/yard AA (1907)	64	40	≥ 60	≥ 30	≥ 57	≥ 35	> 41	> 23	≤ 41	≤ 23

Table. Rail categories by wear limit

(Source: CRN-CS-220 Rail and rail joints, John Holland 2016, Engineering Standard)

Parameter	Heavy Duty	Medium Duty
Length	2390 - 2500mm	
Width (at base)	220 - 255 mm	
depth (centre of rail seat)	230mm maximum	180mm maximum
Rail seat area (flat surface)	28800mm ²	28800mm ²

Table?. Concrete sleeper dimensions

(Source: CRN-CS-230 sleepers and track support, John Holland 2016, Engineering Standard)

Track Class	Rail Length	Design Ballast shoulder width (mm)	
		Minimum	Maximum
Main line			
1	CWR / LWR	400	700
2	CWR / LWR	400	700
3/3G	CWR / LWR	400	700
3	Loose	250	700
5	Loose	250	700
Siding			
1	CWR / LWR	400	700
2	CWR / LWR	400	700
3	CWR / LWR	400	700
3	Loose	250	700

Table. Ballast shoulder widths

(Source: CRN-CS-230 sleepers and track support, John Holland 2016, Engineering Standard)

Track Class	Sleeper / rail type	Design Ballast Depth	
		Minimum	Maximum
Main Line			
1	Concrete – 60kg rail	300	500
	Concrete – 53kg rail	270	500
	Steel	270	500
	Timber – 60kg rail	300	500
	Timber – 53kg rail	270	500
2	Timber/Steel	270	500
3	Timber/Steel	200	500
3G	Timber/Steel	150	500
5	Timber/Steel	150	500

Table. Ballast depth

(Source: CRN-CS-240 ballast, John Holland 2016, Engineering Standard)

G3. Design process and calculations

G3.1 Calculations for pile footing design

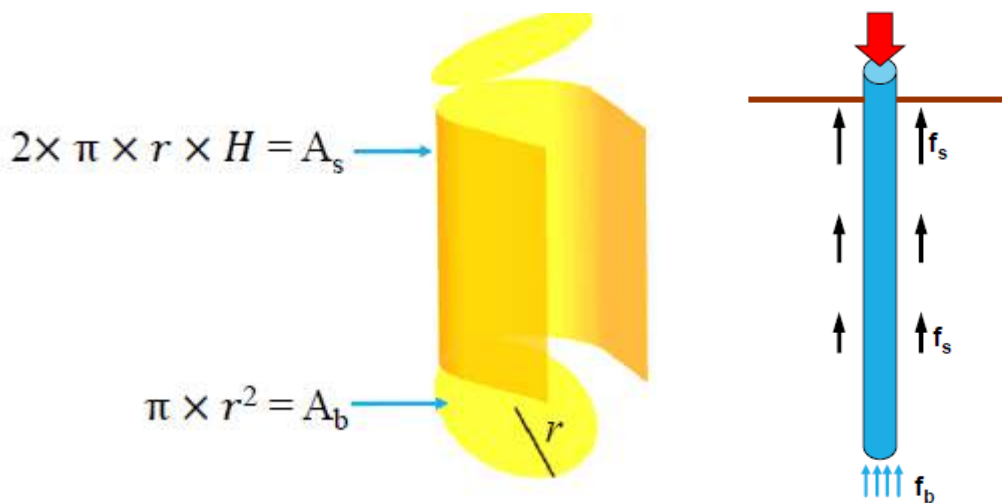
All the design calculation procedures are complying with AS2159-2009: piling design and installation.

G3.1.1 Design for ultimate geotechnical strength

The calculation of the design ultimate geotechnical strength of a single or isolated pile is the priority step to assess the performance of piled foundation. Calculation of ultimate geotechnical strength of a single pile is a necessary prerequisite for estimating group piles performance. In addition, vertical load capacity is the most important for piled foundations and other factors are also critical, such as settlement and lateral capacity.

The design of ultimate geotechnical strength of single pile is the sum of the soil friction on the pile shaft and the resistance on the base of the pile. The basic capacity equation is:

$$\text{Ultimate Geotechnical Capacity (force): } R_{ug} = \overline{F}_s A_s + f_b A_b$$



900mm diameter with enlarger basement as 1.6m diameter base, piles with depth of 20m is assumed to be used as the deep foundation.



Diameter of pile = 1.2m
Height of the pile = 20m

$$A_s = 2 * \pi * \frac{1.2}{2} * 20 = 75.4m^2$$

$$A_b = \pi * \left(\frac{1.2}{2}\right)^2 = 1.13m^2$$

Deep Foundations usually $\frac{L}{B} > 5$

Where L = length of the pile

B = pile diameter

$$\frac{20}{1.2} = 16.667, \text{ok}$$

Accordance with the Australian Standard 2519-2009 Clause 4.4.1

Design ultimate geotechnical strength in compression

$$R_u = \overline{f_s} A_s + f_b A_b$$

f_s = average, fully mobilized unit shaft resistance

f_b = ultimate base bearing pressure

A_s = surface area

A_b = base area

The calculation will be divided into sand layer and clay layer in ultimate axial capacity.

Clay layer:

$$R_u = \overline{f_s} A_s + f_b A_b$$

$$A_s = 2 * \pi * r * H$$

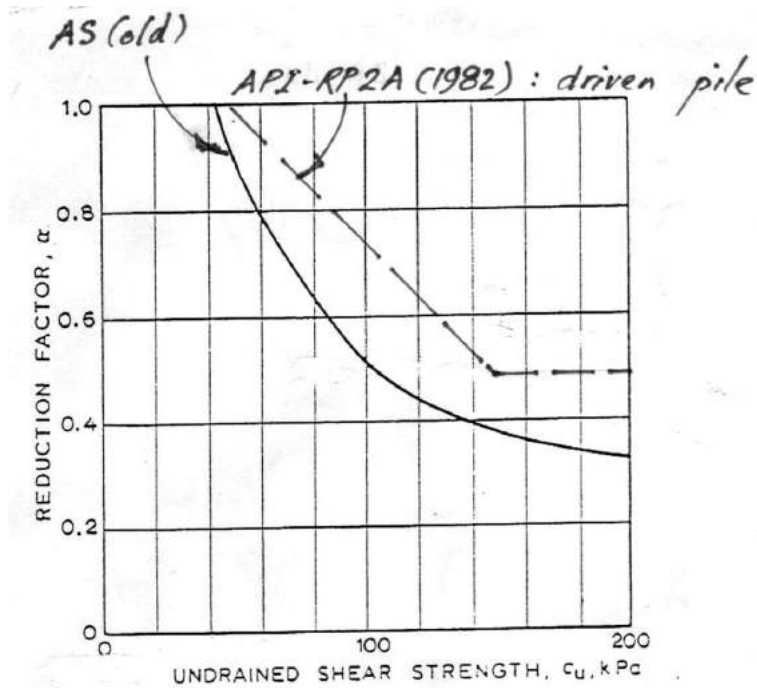
$$A_b = \pi * r^2$$

Bearing capacity/end bearing

$$f_b = N_{cb} * C_{ub}$$

$$N_{cb} = 9 \text{ to } 12,$$

take N_{cb} 10 in the case



shaft resistance

α Method:

$$f_s = \alpha * C_u$$

where,

$$\alpha = 0.3 - 1.0$$

C_u :

$$\text{Layer 2: } C_u = 30$$

$$\text{Layer 4: } C_u = 200$$

$$\text{Layer 5: } C_u = 150$$

$$\text{Layer 6: } C_u = 200$$

Reduction factor α :

$$\text{Layer 2: } \alpha = 1.0$$

$$\text{Layer 4: } \alpha = 0.3$$

$$\text{Layer 5: } \alpha = 0.4$$

$$\text{Layer 6: } \alpha = 0.3$$

Layer 2:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 1.0 * 30 * 2 * \pi * 0.6m * 3.5m = 30 * 395.8kN$$

Layer 4:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.3 * 200 * 2 * \pi * 0.6m * 1.9m = 429.8kN$$

Layer 5:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.4 * 150 * 2 * \pi * 0.6m * 3.45m = 780.4kN$$

Layer 6:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.3 * 200 * 2 * \pi * 0.6m * 5.25m = 1187.5N$$

Bearing capacity/end bearing:

$$f_b = N_{cb} * C_{ub}$$

$$N_{cb} = 9 \text{ to } 12,$$

take N_{cb} 10 in the case

$$\text{therefore: } f_b = 10 * 200 = 2000$$

$$f_b A_b = 2000 * \pi * 0.6^2 = 2262kN$$

Sand layer:

Accordance with the simplified soil profile, the first and third layer were sand. Sand layer and clay layer in ultimate axial capacity:

TABLE C1 (draft code, April 1991)

VALUES OF F , f_{s2} , N_q and f_{b2} FOR PILES IN SILICA SAND

Condition of soil	Relative density	Skin resistance				Ultimate base resistance			
		Driven piles ¹		Bored or cast-in situ piles ²		Driven piles		Bored piles ²	
		F β	f_{s2} kPa	F β	f_{s2} kPa	N_q	f_{b2} MPa	N_q	f_{b2} MPa
Loose	0.2 - 0.3	0.8	25	0.3	10	60	2.0	25	0.8
Medium-loose	0.3 - 0.4	0.9	35	0.4	16	75	3.0	40	1.6
Medium	0.4 - 0.6	1.0	50	0.5	25	100	5.0	60	3.0
Medium-dense	0.6 - 0.75	1.2	65	0.65	35	130	7.0	80	4.2
Dense	0.75 - 0.9	1.5	85	0.8	45	180	10.0	100	5.5
Very dense	> 0.9	1.75	100	0.9	50	210	12.0	120	7.0

1 Including cast-in-situ piles of hammered shaft construction.

2 Assuming close supervision of construction is exercised.

(Source: AS 2159 - 1991-draft)

$$R_u = \bar{f}_s A_s + f_b A_b$$

$$A_s = 2 * \pi * r * H$$

$$A_b = \pi * r^2$$

$f_s - \beta$ method

$$f_s = \beta * \sigma_{v0}$$

β Value:

$$\text{layer 1: } \beta = 0.8$$

$$f_s \text{ limite} = 45kPa$$

$$\text{layer 2: } \beta = 0.9$$

$$f_s \text{ limite} = 50kPa$$

In the β method water level will be considered in sand layers

Shaft resistance

for σ_v :

$$\text{level 1 sand: } \sigma_v = 2.5 * 20.5 = 51.25kPa$$

$$\text{level 2 clay: } \sigma_v = 3.5 * 20 = 70kPa$$

$$\text{level 3 sand: } \sigma_v = 3.4 * 21 = 71.4kPa$$

for v , water level:

$$\text{level 1 sand: } v = 2.5 * 9.81 = 24.5kPa$$

$$\text{level 2 clay: } v = 3.5 * 9.81 = 34.3kPa$$

$$\text{level 3 sand: } v = 3.4 * 9.81 = 33.4kPa$$

for $\sigma_{v0} = \sigma_v - v$:

$$\text{level 1 sand: } \sigma_{v0} = 51.25 - 24.5 = 26.75kPa$$

$$\text{level 2 clay: } \sigma_{v0} = 70 - 34.3 = 35.7kPa$$

$$\text{level 3 sand: } \sigma_{v0} = 71.4 - 33.4 = 37kPa$$

for f_s sand level:

$$\text{level 1: } f_s = \beta \sigma_{v0} = 0.8 * 26.75 = 21.4kPa < 45kPa, \text{ok, using the smaller value}$$

$$\text{level 3: } f_s = \beta \sigma_{v0} = 0.9 * 33.4 = 30.1kPa < 50kPa, \text{ok, using the smaller value}$$

$\overline{F_s} A_s$:

$$A_s = 2 * \pi * r * H$$

$$\overline{F_s} A_s = 21.4 * 2 * \pi * 0.6 * 2.5 + 30.1 * 2 * \pi * 0.6 * 3.4 = 587.5kN$$

Therefore:

$$R_u = \overline{f_s} A_s + f_b A_b$$

$$R_u = 5643 \text{ kN}$$

G3.1.2 Design for geotechnical strength

Accordance with the AS 2159-2009, clause 4.3, general principles of geotechnical strength design, 4.3.1 design geotechnical strength, the design geotechnical strength R_{dg} will be calculated as the design ultimate geotechnical strength R_{dug} multiplied by a geotechnical strength reduction factor ϕ_g :

$$R_{d,g} = \phi_g * R_{dug}$$

The geotechnical strength reduction factor ϕ_g be determined as:

$$\phi_g = \phi_{gb} + (\phi_{tf} - \phi_{gb}) * K \geq \phi_{gb}$$

ϕ_{gb} = basic geotechnical strength reduction factor

ϕ_{tf} = intrinsic test factor

K = test benefit factor

Since there is no pile test will be considered in this case, therefore:

$$\phi_{tf} = \phi_{gb}$$

$$\phi_g = \phi_{gb}$$

$$K = 1$$

Accordance with AS 2159-2009, Clause 4.3.2 assessment of basic geotechnical strength reduction factor (ϕ_{gb}):

Average risk rating:

$$ARR = \sum(w_i IRR_i) / \sum w_i$$

$$ARR = \frac{2 * 3 + 2 * 1 + 2 * 1 + 1 * 5 + 2 * 5 + 1 * 3 + 2 * 1 + 2 * 3 + 0.5 * 1}{14.5}$$

$$= 2.52 \text{ (AS2159.2009 TABLE 4.3.2)}$$

Therefore: from TABLE 4.3.2(C)

$$\phi_{gb} = 0.60$$

Therefore:

$$R_{d,g} = \phi_g * R_{dug}$$

$$R_{d,g} = 0.60 * 5643 = 3386 \text{ kN}$$

Accordance with the design load for each column from structure, $P^* = 5145kN$

Therefore, assume there are total CAF 4 piles will be designed in a footing system:

$$3386 * 4 = 13536kN$$

$$FOS = \frac{13536}{5145} = 2.63, \text{ok within } 2 - 3$$

G3.1.3 Group piles efficiency design

Ultimate Capacity of Pile Group Treat the pile group as a single mega PILE

piles in a row: 2

row of pile: 2

pile length: 20m

diameter of pile: 1.2m

minimum distance of pile edge to cap edge: 0.2m

$$\text{pile spacing: } S = 2.5D * 0.02L = 3.4m$$

Therefor:

$$x = 3.4 + 0.6 + 0.6 = 4.6m$$

$$y = 3.4 + 0.6 + 0.6 = 4.6m$$

$$A_b = 4.6 * 4.6 = 21.16m^2$$

Clay layer:

Layer 2:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 1.0 * 30 * 2 * \pi * (4.6 * 3.5) * 4m = 30 * 395.8kN$$

Layer 4:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.3 * 200 * 2 * \pi * (4.6m * 1.9) * 4m = 429.8kN$$

Layer 5:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.4 * 150 * 2 * \pi * (4.6m * 3.45) * 4m = 780.4kN$$

Layer 6:

$$f_s A_s = \alpha * C_u * 2 * \pi * r * H = 0.3 * 200 * 2 * \pi * (4.6m * 5.25) * 4m = 1187.5N$$

Bearing capacity/end bearing:

$$f_b = N_{cb} * C_{ub}$$

$$N_{cb} = 9 \text{ to } 12,$$

take N_{cb} 10 in the case

therefore: $f_b = 10 * 200 = 2000$

$$f_b A_b = 2000 * \pi * 21.16 = 42320kN$$

Sand layer:

$\overline{F}_s A_s$:

$$A_s = 4xy$$

$$\overline{F}_s A_s = 21.4 * (4.6 * 2.5) * 4 + 30.1 * (4.6 * 3.4) * 4 = 13190kN$$

Therefore:

$$R_u = \overline{f}_s A_s + f_b A_b$$

$$R_{u_megapile} = 69145kN$$

Group efficiency:

$$\eta = 1 - \arctan\left[\frac{(n-1)m + (m-1)n}{mn}\right]/90$$

Where:

m = number of rows

n = number of piles in a row

d = pile diameter

s = center to center spacing of piles

Therefore:

$$\eta = 1 - \arctan\left[\frac{(n-1)m + (m-1)n}{mn}\right]/90$$

$$\eta = 1 - \arctan\left[\frac{(2-1)2 + (2-1)2}{2 * 2}\right]/90 = 0.99 < 1$$

Thus:

$$R_{ug} = 0.99 * 13536kN = 13400$$

$$FOS = 13400/5146 = 2.6$$

OK

G3.1.4 Pile cap design and reinforcement

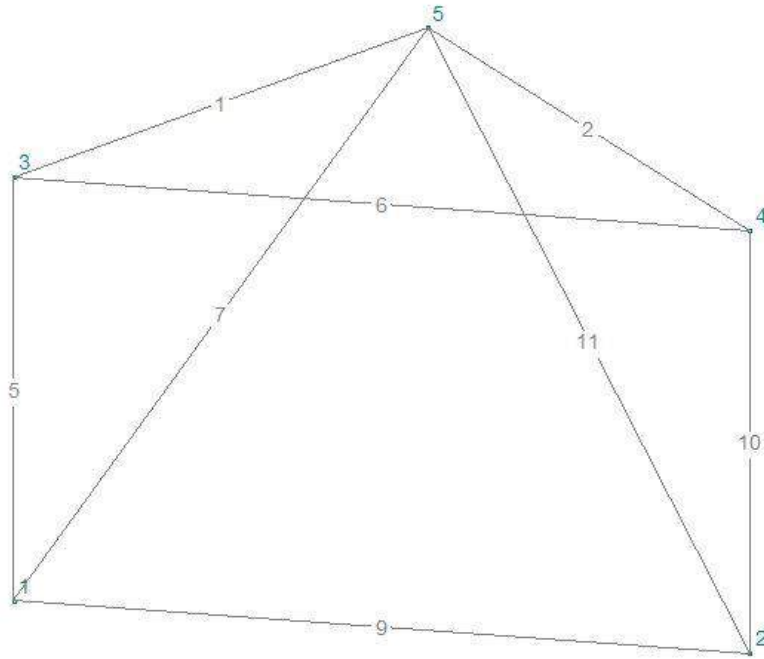
Step 1: determine the loads

The load from a column is 5145kN

The pile cap transferring the loads to four pile evenly, and load for each pile is $5145/4 = 1286.25kN$

Step 2: develop strut and tie model and structural analysis

The space-truss model for the pile cap is developed as the below:



The space-truss model for the pile cap as the figure below, it is a pyramid, with node 1,2,3,4 at bottom level and node 5 at top. The angel between inclined strut and ties' plane is 45 deg

Member 5, 6, 9, 10 are tie member and under tension, and length is two piles center to center distance, which is 3.4 meters

Member 1, 2,7,11 are strut member and under compression, and length is 3.4m

Then,

$$\text{The distance form node 5 to ties'plane is} = \frac{\sqrt{2}}{2} * 3.4 = 2.4 \text{ m}$$

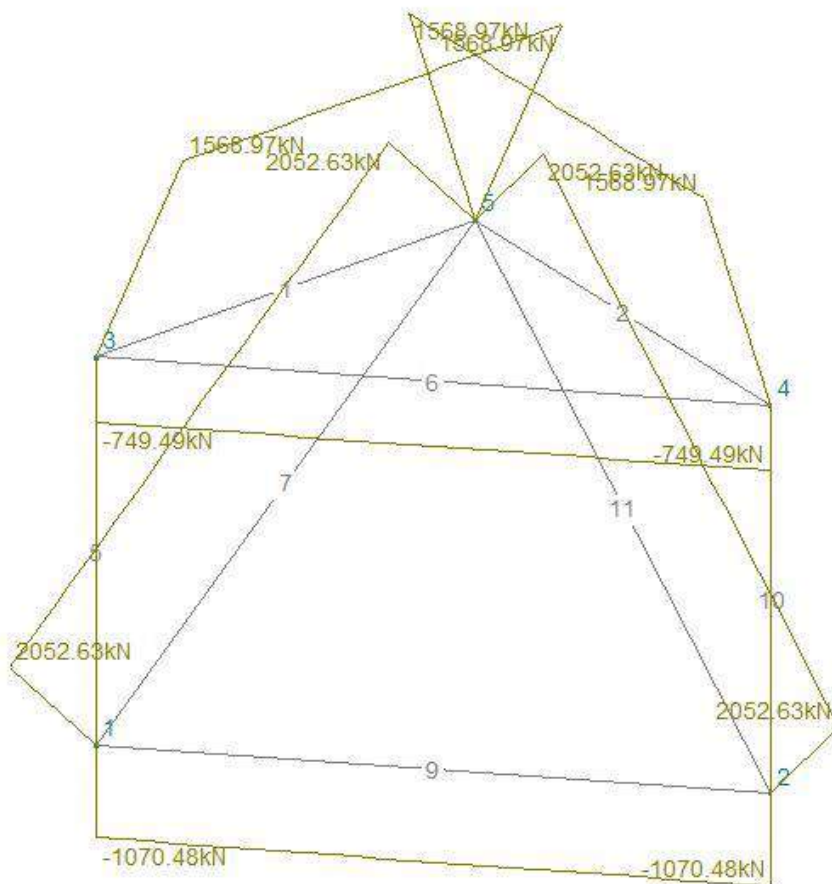
Hence, the

$$\text{pile cap depth} = 2400\text{mm} + 75\text{mm} + 50\text{mm} = 2525\text{mm}$$

say 2550mm

Step 3: bottom reinforcement determination

Using SpaceGass, member force inside each member as the figure below:



Then, maximum tension force is 1070.48kN while maximum compression force is 2052.63kN

And, bottom reinforcement should take 1070.48kN tension force

$$f_{sy} = 500\text{Mpa steel bar used}$$

Thus,

$$A_{st} = 1070.48 * \frac{1000}{500} = 2141\text{mm}^2$$

Hence, 5N24 bar used, which provides a cross section of 2250mm^2

G3.2 Calculations for retaining wall design

There are four parts in the retaining wall design process, an initial retaining wall dimension were set at first, then adjust the design dimension by checking overturning, basing sliding and ultimate bearing capacity to achieve design factor of safety.

G3.2.1 Initial dimensions

Figure 5 show the initial dimensions of retaining wall with maximum 3 meters' maximum height. Triangle is the ground level.

All the dimension has relative to the height of 3 meters stem wall. The stem width is 0.1 times the stem height and the length of slab is 1.55 times the stem height.

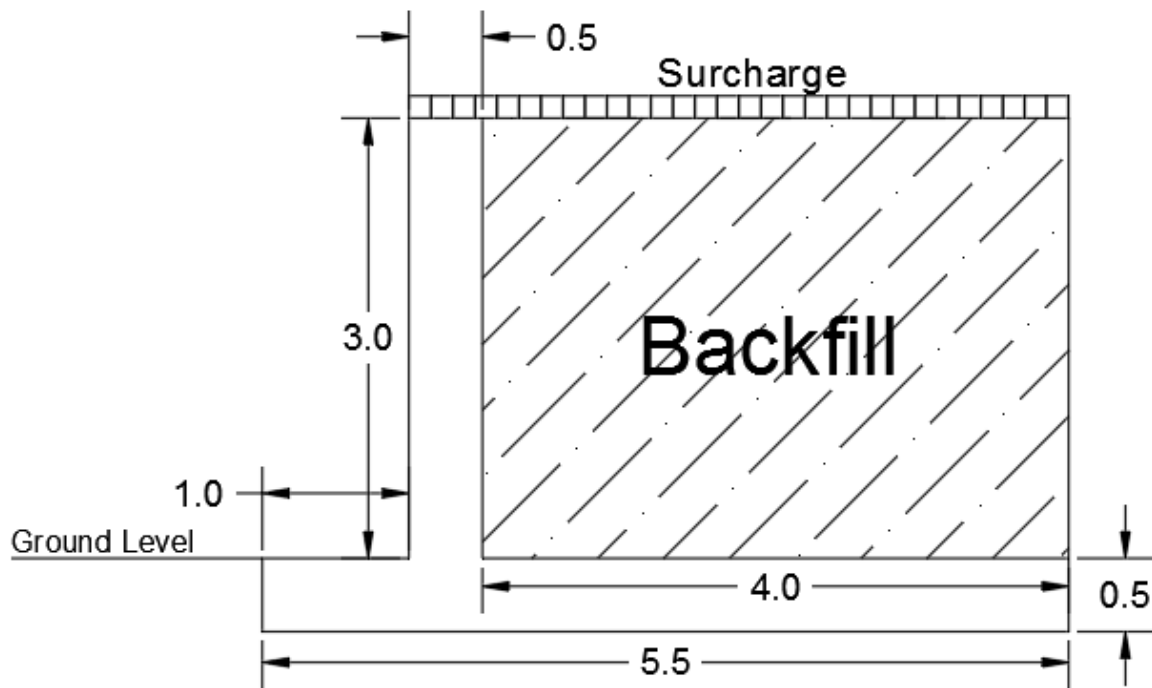


Figure 1 - T-shape cantilever retaining wall (meter)

Figure 6 shows the lateral pressure diagram. There are 90 kPa surcharge on top of the retaining wall and the backfill will be using saturated coarse sand with dense which is 21.5 kN/m^3 . The factor for the backfill will be 21.5 kN/m^3 for density, 32 for the ϕ' and 0 for c' . From figure 6, the long rectangle is the surcharge force and the triangle is the backfill force.

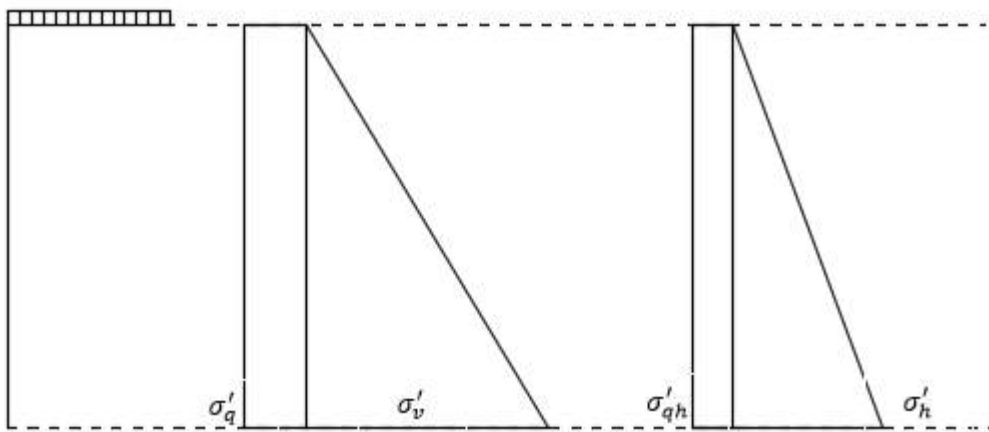


Figure 2 - Lateral pressure diagram of horizontal force and vertical force

$$\begin{aligned}\sigma'_q &= 90 \text{ kPa} \\ \sigma'_v &= 21.5 * 3.5 = 75.25 \text{ kPa} \\ k_a &= \frac{1 - \sin(32)}{1 + \sin(32)} = 0.307 \\ \sigma'_{qh} &= \sigma'_q * k_a\end{aligned}$$

$$\sigma'_{qh} = 90 * 0.307 = 27.65 \text{ kPa}$$

$$\sigma'_{vh} = \sigma'_v * k_a$$

$$\sigma'_{vh} = 70.95 * 0.307 = 23.1 \text{ Pa}$$

There are three sections divided by the retaining wall for calculations which show at figure 7. For table 1, it shows the calculations such as the force, arm from toe and the moment for each section. Pa is calculate by the area of σ'_{qh} and σ'_{vh} . Moment is Force multiply by arm.

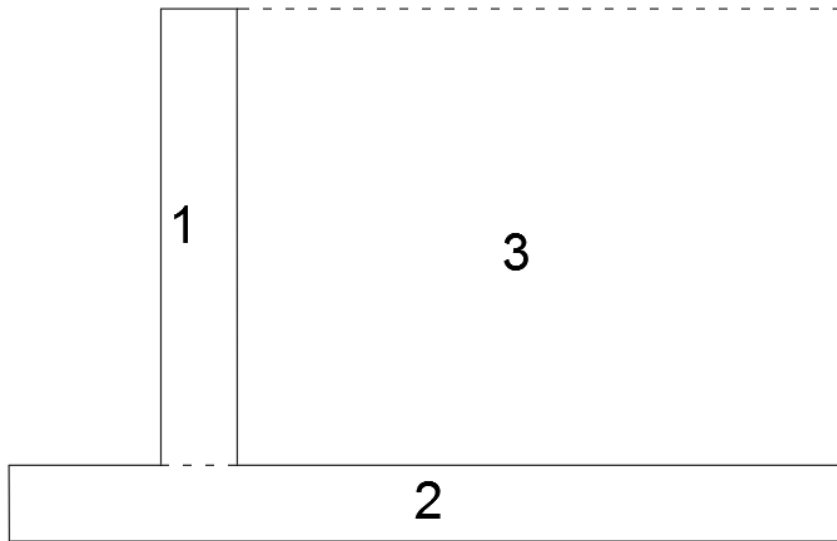


Figure 3 - Diagram for stem part of Cantilever Wall (1), slab section (2) and backfill soil (3)

Active Earth pressure (for backfill soil)

$$P_a = k_a \gamma H$$

Passive Earth Pressure:

$$P_p = k_p \gamma H$$

(Active pressure) Lateral pressure due to water at the depth h

$$P_a = k_a \gamma H + \gamma_w h$$

(Passive pressure) Lateral pressure due to water at the depth h

$$P_p = k_p \gamma H + \gamma_w h$$

Where:

P_a = Intensity of active earth pressure trying to move the wall away from the wall.

$$k_a = \text{Coefficient of active earth pressure} = \frac{1 - \sin \phi}{1 + \sin \phi}$$

(when $\beta = 0$ horizontal ground surface)

ϕ = Angle of friction for backfill

γ = Unit weight of backfill soil

H = Height of retaining wall (Rahul, 2013)

Section	Force (kN)	Arm(m)	Moment kNm
---------	------------	--------	------------

Pa	$\left(\frac{1}{2} * 23.1 + 27.65\right) * 3.5 = 137.2$	$\frac{1}{3} * 3.5 = 1.2$	160.1 (M_o)
1	$0.5 * 3 * 24 = 36$	$\frac{0.5}{2} + 1 = 1.25$	45
2	$0.5 * 5.5 * 24 = 66$	$\frac{5.5}{2} = 2.75$	181.5
3	$3 * 4 * 21.5 = 258$	$1 + 0.5 + \frac{4}{2} = 3.5$	903
Sum of section 1,2,3	360(V)		1129.5(M_R)

Table 1 - Calculation of each section

Location of vertical force from toe: $x = \frac{M_R - M_O}{v} = \frac{1129.5 - 160.1}{360} = 2.69$

Eccentricity: $e = \frac{B}{2} - x = \frac{5.5}{2} - 2.69 = 0.1 < \frac{B}{2} = 0.7m$

G3.2.2 Overturning design

Figure 8 shows the overturning point of retaining wall. Overturning failure is the rotation of wall about its toe due to overweight of moment caused due to overturning force larger than resisting forces. Increasing of wall mass and enlarging foundation can prevent from overturning phenomenon.

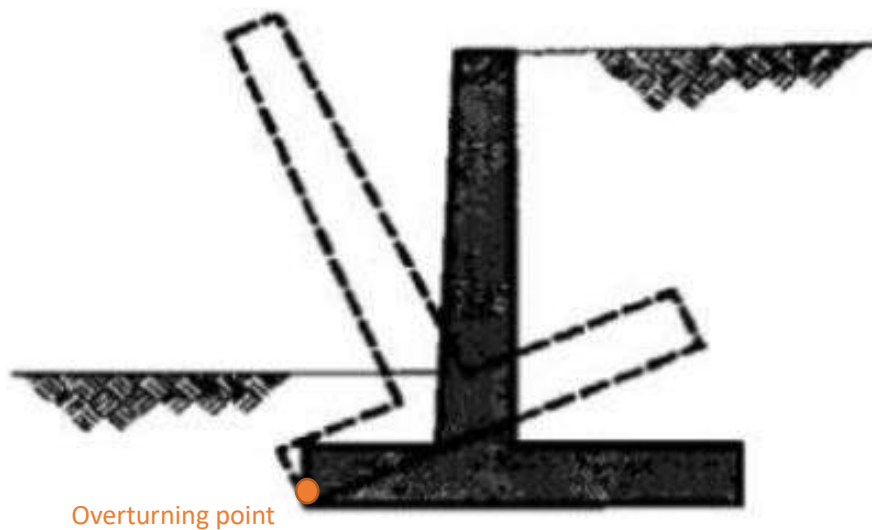


Figure 4 - Overturning failure diagram (Ritesh, 2013)

The factor of safety for avoiding from overturning is given by:

$$F_o = \frac{\Sigma M_R}{\Sigma M_O}$$

ΣM_R = sum of resisting moment about toe

ΣM_O = sum of overturning moment about toe

$$F_o = \frac{1129.5}{160} = 7.05 > 2.5 \quad \therefore \text{SAFE}$$

G3.2.3 Base sliding design

Lateral pressure due to water ($\delta w = 10\text{kPa}$) will be considered in calculation of sliding check. Since it will be the critical case of sliding. The factor for the soil under footing will be 18kN/m^3 for density, 38° for the ϕ' and 2 for c' . From figure 9, the long rectangle is the surcharge force and the triangle is the backfill force. Instead of wall overturning, the sliding horizontally is caused by thrust of soil. Increase foundation and more massive walls are the solution for reducing the failure percentage of wall sliding. Construction of drainage system is crucial to install behind the wall for reducing hydrostatic pressure. (John 2013)

$$\begin{aligned}\sigma'_v &= \gamma * h - h * \delta w \\ \sigma'_v &= 18 * 0.5 - 0.5 * 10 \\ \sigma'_v &= 4 \text{ kPa} \\ k_p &= \frac{1 + \sin(38)}{1 - \sin(38)} = 4.2 \\ \sigma'_h &= \sigma'_v * k_p + 2c' \sqrt{k_p} \\ \sigma'_h &= 4 * 4.2 + 2 * 2\sqrt{4.2} \\ \sigma'_h &= 25.01 \text{ kPa}\end{aligned}$$

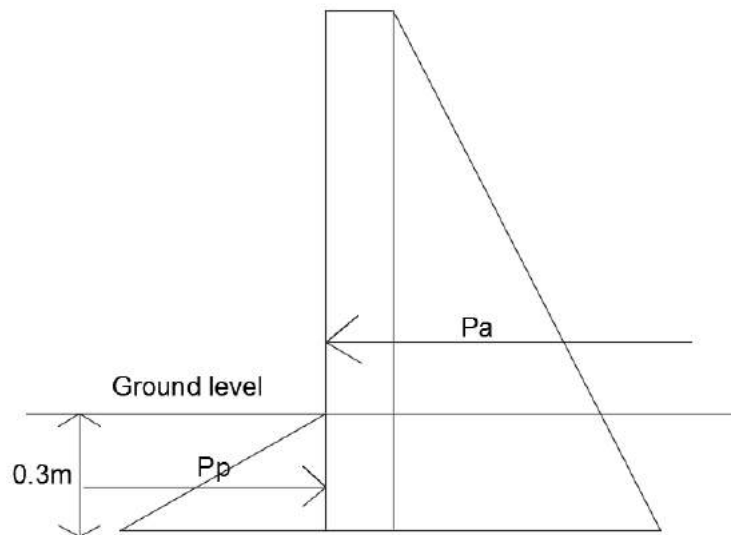


Figure 5 - Diagram of passive earth pressure and active earth pressure

$$\text{Sliding force} = \left(\frac{1}{2} * 23.1 + 27.65 \right) * 3.5 = 137.2$$

$$\text{Resistance Force} = P_p + v \tan\left(\frac{2}{3} * \phi'\right)$$

$$\text{Resistance Force} = 25.01 + 360 \tan\left(\frac{2}{3} * 38\right)$$

$$\text{Resistance Force} = 195.4 \text{ kN}$$

$$\text{Factor of Safety} = \frac{\text{Sliding Force}}{\text{Resistance Force}} > 1.5$$

$$\text{Factor of Safety} = \frac{195.4}{137.2} = 1.4 < 1.5 \text{ Fail}$$

Since the factor of safety fail at sliding, it will be economic to add a key with 0.5 meter height under the footing to provide sliding rather than increase the size of the footing. So, the total width for bottom underground is 1 meter.

$$\sigma'_v = \gamma * h - h * \delta w$$

$$\sigma'_v = 18 * 1 - 1 * 10$$

$$\sigma'_v = 8 \text{ kPa}$$

$$k_p = \frac{1 + \sin(38)}{1 - \sin(38)} = 4.2$$

$$\sigma'_h = \sigma'_v * k_p + 2c' \sqrt{k_p}$$

$$\sigma'_h = 8 * 4.2 + 2 * 2\sqrt{4.2}$$

$$\sigma'_h = 137.2 \text{ kPa}$$

$$\text{Sliding force} = \left(\frac{1}{2} * 23.1 + 27.65\right) * 3.5 = 137.2$$

$$\text{Resistance Force} = P_p + v \tan\left(\frac{2}{3} * \phi'\right)$$

$$\text{Resistance Force} = 137.2 + 360 \tan\left(\frac{2}{3} * 38\right)$$

$$\text{Resistance Force} = 212.3 \text{ kN}$$

$$\text{Factor of Safety} = \frac{\text{Sliding Force}}{\text{Resistance Force}} > 1.5$$

$$\text{Factor of Safety} = \frac{212.3}{137.2} = 1.55 > 1.5 \text{ Safe}$$

G3.2.4 Ultimate bearing capacity design

Bearing capacity is ensuring to safely carry the pressure on the soil without undergoing a shear failure, in other words the ultimate bearing capacity is on the average vertical pressure on the ground which makes to failure by shear force. Ensure the settlement of the foundation is within allowable limit for bearing pressure.

The maximum safe bearing capacity is the maximum pressure value that the soil can be subjected without any risk of shear failure, in additional this can be calculated simply on the ultimate bearing capacity divided by factor of safety.

q_u = ultimate bearing capacity

The vertical force at toe of retaining wall must not exceed acceptable bearing capacity for the soil.

$$\text{Effective width } B' = B - 2e = 5.5 - 2 * 0.1 = 5.3$$

$$q_u = cN_c + q_0N_q + \frac{\gamma B'}{2} N_\gamma$$

$$N_c = 63.62, N_q = 51.84, N_\gamma = 66.32 \text{ with } \phi' = 38$$

$$q_u = 2 * 63.32 + 0 + \frac{21.5 * 5.3}{2} 66.32$$

$$q_u = 3341.7$$

$$q_{safe} = \frac{3341.7}{3} = 1113.9 \text{ kPa}$$

$$\text{Force per area, } q_{design} = \frac{360}{5.3 * 1} = 66.8 \text{ kPa}$$

The factor of safety(FOS) against bearing failure present:

$$F_b = \frac{Q_{safe}(\text{allowable bearing pressure})}{\text{Force per area}}$$

$$\text{Factor of Safety} = \frac{1113.9}{66.8} = 16.6 > 1.5 \text{ Safe}$$

G3.2.5 Detailed reinforcement schedule calculation

Retaining wall reinforcement design was designed as four proportioning, wall proportioning, heel proportioning, toe proportioning and key proportioning, each proportioning will be provided vertical reinforcement bars and longitudinal reinforcement bars.

A) Proportioning of wall reinforcement

Basing on the previous calculation results and wall dimensions

$$\text{Active force from backfill } P_{A1} = \frac{1}{2} * k_a * \gamma * H^2 = 0.5 * 0.307 * 21.5 * 3^2 = 29.7kN$$

$$\text{Acting at } \frac{3}{3} \text{ meters from ground level}$$

$$\text{Active force from surcharge } P_{A2} = k_a * s * H = 0.307 * 90 * 3 = 82.89kN$$

$$\text{Acting at } \frac{3}{2} \text{ meters from ground level}$$

For the strength limit state, multiply all earth pressure by 1.5

Then,

$$M^* = 1.5 * \left(29.7 * \frac{3}{3} + 82.89 * \frac{3}{2} \right) = 231.05kNm$$

Wall thickness = 500mm, use cover = 50mm, and 20mm fitments

$$\text{effective depth} = 500 - 50 - \frac{20}{2} = 440mm$$

This requires a steel proportion of

$$p = 2.7 * \frac{231.05}{440^2} = 0.00322$$

$$f'_c = 40 \text{ Mpa concrete was used, } f'_{ct.f} = 0.6\sqrt{f'_c} = 3.8Mpa$$

The minimum p for one-way slab is

$$p_{min} = 0.2 * \left(\frac{D_s}{d} \right)^2 * \left(\frac{f'_{ct.f}}{f_{sy}} \right) = 0.2 * \left(\frac{500}{440} \right)^2 * \left(\frac{3.8}{500} \right) = 0.00196$$

Then adopt

$$p = 0.00322$$

Design reinforcement for each 1 meter, b=1000mm

Thus,

$$A_{st} = p * b * d_{eff} = 0.00322 * 1000 * 440 = 1417mm^2$$

Therefore,

The design team will provide vertical reinforcement of **N20 bars at 200mm centers.**

For shrinkage in the longitudinal direction of the wall, use $p = 0.0035bD$ for a moderate degree of crack control, hence,

$$A_{st} = p * b * D = 0.0035 * 1000 * 500 = 1750mm^2$$

Therefore,

The design team will provide horizontal reinforcement of **N20 bars at 175mm centers** at back of wall face. The required rebar area at front wall face considered as half of back, then, $A_{st} = 875 \text{ mm}^2$, adopt **N16 bars at 225mm centers**.

B) Proportioning of heel reinforcement

As bending moment at heel, there will tension at top of base slab, but will be slightly smaller than that in the wall, adopt same reinforcement as wall, **N20 bars at 200mm centers**.

For the longitudinal direction, same crack control method as wall, use $p = 0.0035bD$, provide two layers of reinforcement bars of **N16 bars at 200mm centers**.

C) Proportioning of toe reinforcement

For the toe part, use cover = 75mm, assuming 16mm fitment bars, effective depth is

$$d_{eff} = 500 - 75 - \frac{16}{2} = 417mm$$

The design bearing strength $q_{design} = 66.8 \text{ kPa}$, for strength limit state, use factor 1.5

Then, the distance from front wall to the end of toe part is 1.0 meter

$$M^* = 1.5 * 66.8 * 1.0 * \frac{1.0}{2} = 50.1kNm$$

And,

$$p = 2.7 * \frac{50.1}{417^2} = 0.00078$$

$$p_{min} = 0.2 * \left(\frac{D_s}{d}\right)^2 * \left(\frac{f'_{ct.f}}{f_{sy}}\right) = 0.2 * \left(\frac{500}{417}\right)^2 * \left(\frac{3.8}{500}\right) = 0.0022$$

Then, reinforcement area is

$$A_{st} = p * b * d_{eff} = 0.0022 * 1000 * 417 = 917mm^2$$

Therefore,

The design team will adopt **N16 bars at 200mm centers**

Again, for a moderate crack control, the design team provide two layer of **N16 bars at 200mm centers**

For key part, providing cover=50mm, **N20 bars at 200mm centers** and **N20 bars at 225mm centers** for longitudinal direction at top and bottom layers.

G3.3 Traffic data analysis

G3.3.1 Summarized traffic volume

Morphett Road / Diagonal Road – South of Rail Crossing

No	Road	AADT	Trucks	%HV (Heavy Vehicles)
1	Morphett Road [N]	60100	2020	3.3
2	Diagonal Road [E]	27100	920	3.3
3	Morphett Road [S]	40400	1200	3.0

Morphett Road / Railway Terrace / Diagonal Road

No	Road	AADT	Trucks	%HV (Heavy Vehicles)
1	Morphett Road [NE]	300	0	0
2	Morphett Road [SE]	60100	2020	3.3
3	Railway Terrace [SW]	900	0	0
4	Diagonal Road [NW]	60100	2020	3.3

Diagonal Road / Dunrobin Road

No	Road	AADT	Trucks	%HV (Heavy Vehicles)
1	Diagonal Road [SE]	60100	2020	3.3
2	Dunrobin Road [W]	5500	160	2.9
3	Diagonal Road [NW]	60000	1980	3.3

Prunus Street / Diagonal Road/ Egmont Avenue

No	Road	AADT	Trucks	%HV (Heavy Vehicles)
1	Prunus Street [E]	21500	790	3.6
2	Diagonal Road [SE]	60000	1980	3.3
3	Egmont Avenue [W]	600	0	0
4	Diagonal Road [NW]	39700	1210	3.0

Morphett Road / Sturm Court / Prunus Street

No	Road	AADT	Trucks	%HV (Heavy Vehicles)
1	Morphett Road [N]	23700	820	3.4
2	Sturm Court [E]	0	0	0

3	Morphett Road [S]	2800	50	1.7
4	Prunus Street [W]	21700	790	3.6

G3.3.2 DESA calculation process – Morphett Road and Diagonal Road

Design Parameters:

Design Period = 30 years

Annual Average Daily Traffic (AADT) = 60100

Direction Factor = 0.5

Percentage Heavy vehicles = 3.3%

Lane Distribution Factor = 1.0

Total Number of Heavy vehicles Axle Groups

N_{DT} = Cumulative number of heavy vehicle axel group over the design period

$$N_{DT} = 365 \times (AADT \times DF) \times \left(\% \frac{HV}{100} \right) \times (N_{HVAG}) \times (LDF \times CGF)$$

where

AADT = Average Annual Daily Traffic

DF = Direction Factor

%HV = Percentage of Heavy vehicles

LDF = lane distribution factor = 1.00 (3 lanes each direction)

CGF = Cumulative Growth Factor

N_{HVAG} = average number of Axle Groups per Heavy vehicles

Cumulative Growth Factor

$$CGF = \frac{(1 + 0.01R)^P - 1}{0.01R}$$

Road	Indicative Annual Growth (2013 to 2020) (%)	Indicative Annual Growth (2021 onwards) (%)
Highways, motorways, and other interstate routes	5	3
Other state controlled roads	3	2

Table above showing the indicative growth rates for traffic flow based on freight forecasts – (AUSTROADS)

$R = \text{annual growth rate (\%)} = 2\%$

$P = \text{design period (years)} = 30 \text{ years}$

$$CGF = \frac{(1 + 0.01 \times 2)^{30} - 1}{0.01 \times 2} = CGF = 40.56$$

$$N_{DT} = 365 \times 60100 \times 0.5 \times \left(\frac{3.3}{100}\right) \times (1 \times 40.56) \times N_{HVAG}$$

$N_{HVAG} = \text{presumptive values} = \text{urban road design} = 2.5$

$$N_{DT} = 1.47 \times 10^7 \times 2.5 = 3.7 \times 10^7$$

DESA = Design Traffic in Equivalent Standard Axles

$$\begin{aligned} DESA &= \frac{ESA}{HVAG} \times N_{DT} \\ &= 0.9 \times 3.7 \times 10^7 \\ &= 3.3 \times 10^7 \end{aligned}$$

G3.3.3 DESA calculation process – Prunus Street

Design Parameters:

Design Period = 30 years

Annual Average Daily Traffic (AADT) = 21500

Direction Factor = 0.5

Percentage Heavy vehicles = 3.6%

Lane Distribution Factor = 1.0

Total Number of Heavy vehicles Axle Groups

N_{DT} = Cumulative number of heavy vehicle axel group over the design period

$$N_{DT} = 365 \times (AADT \times DF) \times \left(\% \frac{HV}{100}\right) \times (N_{HVAG}) \times (LDF \times CGF)$$

where

$AADT$ = Average Annual Daily Traffic

DF = Direction Factor

$\%HV$ = Percentage of Heavy vehicles

LDF = lane distribution factor = 1.00 (3 lanes each direction)

CGF = Cumulative Growth Factor

N_{HVAG} = average number of Axle Groups per Heavy vehicles

Cumulative Growth Factor

$$CGF = \frac{(1 + 0.01R)^P - 1}{0.01R}$$

Road	Indicative Annual Growth (2013 to 2020) (%)	Indicative Annual Growth (2021 onwards) (%)
Highways, motorways, and other interstate routes	5	3
Other state controlled roads	3	2

Table above showing the indicative growth rates for traffic flow based on freight forecasts – (AUSTROADS)

$R = \text{annual growth rate (\%)} = 2\%$

$P = \text{design period (years)} = 30 \text{ years}$

$$CGF = \frac{(1 + 0.01 \times 2)^{30} - 1}{0.01 \times 2} = CGF = 40.56$$

$$N_{DT} = 365 \times 21500 \times 0.5 \times \left(\frac{3.3}{100}\right) \times (1 \times 40.56) \times N_{HVAG}$$

$N_{HVAG} = \text{presumptive values} = \text{urban road design} = 2.5$

$$N_{DT} = 5.7 \times 10^6 \times 2.5 = 1.4 \times 10^7$$

DESA = Design Traffic in Equivalent Standard Axles

$$\begin{aligned} \text{Target DESA} &= \frac{ESA}{HVAG} \times N_{DT} \\ &= 0.9 \times 1.4 \times 10^7 \\ &= 1.3 \times 10^7 \end{aligned}$$

G3.3.4 CIRCLY design output results – Morphett Road and Diagonal Road

Calculation option:

☒ Calculate damage factors ☐ Calculate selected results at user-defined z-values

☐ Parametric Analysis

Traffic Spectrum: 10⁷ ESAs

Summary | Reliability |

☐ Design thickness of layer highlighted below ☐ Calculate Cost

No.	ID	Title	Current Thickness	CDF
1	AC-20M	AC20M Mix Size 80km/h	150.00	2.19E-05
2	Cement3500	Cemented, E=3500 MPa	200.00	6.29E-01
3	Sub_CBR3	Subgrade, CBR=3, Aniso	175.00	3.54E-03
4	Type A Fil	Type A E<70	400.00	
5	Sub_CBR15	Subgrade, CBR15, Aniso	150.00	2.62E-08

Performance Criteria and Traffic multipliers:

No.	Material Type	Performance Criterion	Multiplier
1	Asphalt	AC Mix Designation	1.10
2	Cement Stabilised	Fatigue criterion for Cemented materials, E=3500 MP.	10.00
3	Subgrade (Austroads 2004)	Subgrade failure criterion (Austroads, 2004)	1.60
5	Subgrade (Austroads 2004)	Subgrade failure criterion (Austroads, 2004)	1.60

CIRCLY Version 5.0u (8 April 2013)

Job Title: Pavement Morphet Road

Damage Factor Calculation

Assumed number of damage pulses per movement:

One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: Eco 10 7 Title: 10⁷ ESAs

Load	Load	Movements
No.	ID	
1	ESA75-Full	1.00E+07

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Full	SA750-Full	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Full	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Full	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Full	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Full	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:

Xmin: -200 Xmax: 2200 Xdel: 10
Y: 0

Details of Layered System:

ID: Morphett R Title: Pavement Design Morphett RD

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh)	F	Eh	vh
1	rough	AC-20M	Iso.	3.80E+03	0.40			
2	rough	Cement3500	Iso.	3.50E+03	0.20			
3	rough	Sub_CBR3	Aniso.	3.00E+01	0.45	2.07E+01	1.50E+01	0.45
4	rough	Type A Fil	Aniso.	7.00E+01	0.45	3.50E-01	1.00E+02	0.35
5	rough	Sub_CBR15	Aniso.	1.50E+02	0.45	1.03E+02	7.50E+01	0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	AC	ETH	0.004000	5.000	1.100
2	bottom	Cement3500	ETH	0.000350	12.000	10.000
3	top	Sub_2004	EZZ	0.009300	7.000	1.600
5	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors: Not Used.

Details of Layers to be sublayered:

Layer no. 4: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	150.00	AC-20M	ESA75-Full	-1.83E-05	2.19E-05
2	200.00	Cement3500	ESA75-Full	-7.25E-05	6.29E-01
3	175.00	Sub_CBR3	ESA75-Full	3.88E-04	3.54E-03
4	400.00	Type A Fil		n/a	n/a
5	150.00	Sub_CBR15	ESA75-Full	7.18E-05	2.62E-08

G3.3.5 CIRCLY design output results – Prunus Street

☐ Parametric Analysis

Traffic Spectrum:

Austrads 2004 - Example 3 - Full Depth Asphalt Pavement

Summary

Reliability

☐ Design thickness of layer highlighted below
 ☐ Calculate Cost

No.	ID	Title	Current Thickness	CDF
1	AC-20M	AC-20M - 80km/h	150.00	3.15E-01
2	Gran_350	Granular, E=350MPa	250.00	
3	Cement2000	Cemented, E=2000 MPa	175.00	5.17E-03
4	Type A	Type A Fill E<70	300.00	
5	Sub_CBR12	Subgrade, CBR12, Aniso	0.00	6.75E-08

Performance Criteria and Traffic multipliers:

No.	Material Type	Performance Criterion	Multiplier
1	Asphalt	AC-20M 80km/h	1.10
3	Cement Stabilised	Fatigue criterion for Cemented materials, E=2000MPa	10.00
5	Subgrade (Austrads 2004)	Subgrade failure criterion (Austrads, 2004)	1.60

CIRCLY Version 5.0u (8 April 2013)

Job Title: Prunus Street Pavement Design

Damage Factor Calculation

Assumed number of damage pulses per movement:

One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: 2004-3 Title: Austrads 2004 - Example 3 - Full Depth Asphalt Pavement

Load No.	Load ID	Movements
----------	---------	-----------

1 ESA75-Fulll 7.00E+06

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Fulll	SA750-Fulll	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Fulll	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Fulll	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Fulll	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Fulll	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:

Xmin: -200 Xmax: 2200 Xdel: 10
Y: 0

Details of Layered System:

ID: Aust2004-1 Title: Austroads 2004 - Example 1 - Unbound Granular Pavement

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh) F	Eh	vh
1	rough	AC-20M	Iso.	3.80E+03	0.40		
2	rough	Gran_350	Aniso.	3.50E+02	0.35	2.60E+02	1.75E+02 0.35
3	rough	Cement2000	Iso.	2.00E+03	0.20		
4	rough	Type A	Aniso.	5.20E+01	0.35	3.50E-01	7.00E+01 0.35
5	rough	Sub_CBR12	Aniso.	1.20E+02	0.45	8.28E+01	6.00E+01 0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	AC20M	ETH	0.003735	5.000	1.100
3	bottom	Cement2000	ETH	0.000442	12.000	10.000
5	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors: Not Used.

Details of Layers to be sublayered:

Layer no. 2: Austroads (2004) sublayering
Layer no. 4: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	150.00	AC-20M	ESA75-Full	-1.24E-04	3.15E-01
2	250.00	Gran_350		n/a	n/a
3	175.00	Cement2000	ESA75-Full	-6.32E-05	5.17E-03
4	300.00	Type A		n/a	n/a
5	0.00	Sub_CBR12	ESA75-Full	8.65E-05	6.75E-08

G4. Quality calculation for costing estimate

Quality determination for pile design

- Total bored pile length:
Total 184 piles and 20 meters each, then $total\ length = 184 * 20 = 3860m$
- Total concrete required for bored piles:
1200mm diameters, $total\ volume = \pi * 0.6^2 * 3860 = 4364m^3$
- Mass of steel reinforcement used for piles:
The mass was estimated basing on retaining wall reinforcement details approximately.
Total volume of steel used for retaining wall was estimated as 18.5 cum
Density of steel is 7700kg per cum
Total mass is $7700 * 18.5 = 142450kg = 142.45\ t$

Quality determination for retaining wall

- Filling materials volume:
Assume same width as railway bridge, using 10.6 meter width
 $total\ volume = 150 * 3 * 10.6 * 2 = 9540m^3$

- Total concrete volume for base slab part:

Underneath wall:

$$(5.5 * 0.5 + 0.5 * 0.5) * 10.6 * 2 = 63.6m^3$$

Side walls:

7 different heights of wall are being designed, total volume calculated in table below:

height	wall length	base section area	cross key section area	total cross section area	total cross section area	total volume
3	25	2.75	0.25		3	75
2.5	25	2.25	0.25		2.5	62.5
2	25	2	0.25		2.25	56.25
1.5	25	1.75	0.25		2	50
1	25	1.75	0.25		2	50
0.5	20	1.75	0.25		2	40

0.1	5	1.75	0.25	2	10
total =					343.75 cum

For both sides the total volume is $343.75 * 2 * 2 = 1375$ cum

Total volume of base = $63.6 + 1375 = 1438.6$ cum

- Total concrete volume for wall part:
Underneath wall:

$$0.5 * 3 * 10.6 * 2 = 31.8m^3$$

Side walls:

height	wall length	wall thickness	total volume
3	25	0.5	37.5
2.5	25	0.5	31.25
2	25	0.4	20
1.5	25	0.3	11.25
1	25	0.3	7.5
0.5	20	0.3	3
0.1	5	0.3	0.15

110.65 cum

For both side, total volume is $110.65 * 2 * 2 = 663.9$ cum

Total volume of base = $31.8 + 663.9 = 695.7$ cum

- Mass of steel reinforcement used for piles:
The mass was estimated basing on retaining wall reinforcement details approximately.
Total volume of steel used for retaining wall was estimated as 13.6 cum
Density of steel is 7700kg per cum
Total mass is $7700 * 13.6 = 104720\text{kg} = 104.72$ t

- Total area for retaining wall:
Formwork was estimated based on the total area, basing on Rawlinsons Construction Handbook, Class 3 formwork was used for the project.
The total area was calculated as two underneath walls and four triangle side walls.
 $10.6 * 3 * 2 + 150 * 3 / 2 * 4 = 963.6$ sqm

Total pavement area calculation

Based on new road development management.

740 meters of road be widened from two lanes to three lanes for both sides (Diagonal Road and Morphett Road), and 150 meters of road be widen from one lane to two lanes (Prunus Street)

130 meters of road be re-alignment (Morphett Road), 70 meters are designed 5 lanes both sides and 60 meters are designed 4 lanes both side

Road width is designed as 3.5 meters.

Then the total area of new developed road is

$$\text{widened area} = (740 + 150) * 3.5 * 2 = 6230m^2$$

$$\text{re-alignment area} = 70 * 3.5 * 5 + 60 * 4 * 3.5 = 5127.5m^2, \text{ say } 5130 m^2$$

Thus,

$$\text{total pavement area} = 6230 + 5130 = 11360m^2$$

Total track length determination for ballast design

The materials volume used for formation capping level:

Total length of fill length is approach parts at each side, which is 330 meters each side, the width is 10.6 meters and filling depth 30mm, then

$$\text{Total volume} = 330 * 2 * 0.03 * 10.6 = 210 \text{ cum}$$

Based on project design concept and drawings, the total length of the designed structure was 850 meters, which including 160 meters of bridge and 330 meters of bridge approaching at each side. The unit price of ballast design was estimated by per track meter, since two gauge track was used in the project, and then the total track length is: $850 * 2 = 1700$ meters.

APPENDIX H: SERVICES



ROAD DESIGN Standards & Guidelines

A GUIDE TO CONDUIT DESIGN FOR TRAFFIC SIGNALS AND ITS – TS/ITS 002

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1 INTRODUCTION

This Design Standard specifies the requirements for undertaking the design and documentation of conduits for electrical and communications systems associated with DPTI road infrastructure, including:

- (a) consumer mains;
- (b) traffic signals; and
- (c) Intelligent Transport Systems (ITS).

It does not cover the design of conduit systems for railways (refer http://www.dpti.sa.gov.au/contractor_documents/public_transport_technical_standards).

2 DESIGN DEVELOPMENT – GENERAL INFORMATION

The design shall comply with:

- (a) *Electricity Act 1996* (SA);
- (b) *Electricity (General) Regulations 2012* (SA);
- (c) SA Power Networks: Service Rules and Regulations;
- (d) AS 3000: “Electrical Installations”;
- (e) AS 3008.1: “Electrical Installation – Cable Selection”;
- (f) AS 2053: “Non-metallic Conduits and Fittings”;
- (g) Austroads Publication: Guide to Road Design Part 6B: Roadside Environment;
- (h) Relevant Australian Standards and Australian Communications & Media Authority (ACMA) standards; and
- (i) DPTI Intelligent Transport Systems Project Policies and Procedures PPP01 and PPP02: Available upon request from the Principal.

All conduit design shall be approved by DPTI before the commencement of construction.

3 DESIGNER’S RESPONSIBILITIES

The person/organisation undertaking the design (“Designer”) shall comply with this Design Standard, and liaise with relevant DPTI staff.

4 DRAWINGS

Drawings and other documentation shall comply with the requirements specified in <http://www.dpti.sa.gov.au/standards>, including DP001, DP002, DP12, DP013.

5 DESIGN REQUIREMENTS

The design of the conduit system shall:

- (a) optimise the layout of electrical and communications conduits and pits;
- (b) be compatible with existing DPTI infrastructure;
- (c) where specified, connect to existing DPTI or other conduit systems;
- (d) maximise the ease of installation and maintenance of hardware and cabling;
- (e) take into consideration other services/utilities, road furniture, watercourses, drainage infrastructure and landscaping;

- (f) take into consideration the future provision and/or expansion of ITS;
- (g) minimise the possibility of the ingress of water, vermin and contaminants that may affect the performance of cable systems;
- (h) comply with the requirements of the applicable ACMA standards and relevant Australian Standards; and
- (i) ensure that the conduits and pits comply with Part R53 “Installation of Conduits and Pits” of the DPTI Master Specification (refer: http://www.dpti.sa.gov.au/contractor_documents/specifications).

Unless otherwise specified, the Consumer Mains/Sub-Mains service connection cable shall not share pits or conduits with other services. Where the use of shared pits is approved and/or specified, the Consumer Mains/Sub-Mains service connection cable shall be fixed to the walls of the shared pit with saddles and clearly labelled as “Consumer Mains/Sub-Mains Service”. The label shall also indicate the source and destination of the cable.

6 CONDUIT

6.1 Layout

Unless otherwise specified, the conduits shall:

- (a) be placed in straight lines avoiding unnecessary bends;
- (b) generally run either parallel or normal to the carriageway;
- (c) use large sweeping bends for entry into junction boxes and pile footings; and
- (d) not exceed a 90° total change in direction in any run between pits.

Where a telecommunications service is required for a device, a telecommunications conduit shall be installed directly from the Telecommunications Service Pit (housing the Telecommunications Service Point) to the device.

Where an ITS device requires LV, ELV, communications and/or detector feed in cables, conduits for these required cables may terminate in a common cable draw-in pit. Each conduit shall have a minimum of 60% spare cross-sectional capacity on completion of construction. Segregation requirements shall be in accordance with AS 3000 and ACMA standards.

Unless otherwise specified, the conduit design shall provide separate dedicated conduits for each cable use, e.g. road lighting cables shall be installed in separate conduits to traffic signals.

Conduits installed on bridges and other structures shall, wherever possible, be incorporated into the structure and not be visible. Conduits shall terminate in pits no smaller than P4 external to any bridge structure.

Conduits installed under rail corridors shall terminate in pits no smaller than P4 either side and external to the rail corridor.

6.2 Spare Conduits – ITS

The following spare conduits along the entire length of any ITS backbone shall be installed as a minimum:

- 1x spare communications conduit, minimum diameter 100 mm unless otherwise specified
- 1x spare electrical conduit, minimum diameter 100 mm unless otherwise specified

Where trenches are required for purposes other than ITS backbone cabling, e.g. to ITS equipment located off the backbone or for Road Lighting which does not use the backbone trench, 1x spare

communications conduit, minimum diameter 100 mm, shall be installed in the trench and connected to the ITS backbone conduit system, unless otherwise specified. The conduit shall terminate in a pit no smaller than P4 at the end of the trench.

Spare conduits installed on bridges and under rail crossings shall include 100% capacity over the number of spare conduits specified. For example, if 1 spare communications and 2 spare electrical conduits are specified in a project and the conduits pass under a rail crossing or across a bridge, the number of spare conduits will be 2 communications and 4 electrical at bridge and rail crossing locations. Spare conduits shall terminate in pits no smaller than P4 outside the rail corridor and/or external to any bridge structure.

6.3 Spare Conduits – Traffic Signals

The following spare conduits along the entire length of any Traffic Signals backbone shall be installed as a minimum:

- 1x spare communications conduit, minimum diameter 80 mm unless otherwise specified
- 1x spare electrical conduits, minimum diameter 80 mm unless otherwise specified

Spare conduits installed on bridges and under rail crossings shall include 100% capacity over the number of spare conduits specified. For example, if 1 spare communications and 2 spare electrical conduits are specified in a project and the conduits pass under a rail crossing or across a bridge, the number of spare conduits will be 2 communications and 4 electrical at bridge and rail crossing locations. Spare conduits shall terminate in pits no smaller than P4 outside the rail corridor and/or external to any bridge structure.

6.4 Separation and Size of Conduits

Conduit separation shall be as specified in AS 3000 and/or relevant ACMA standards. If trenches are to be shared with other services, whenever practicable conduits and other services shall be placed side-by-side (in the horizontal plane) as opposed to overlaying services vertically.

Conduit size shall be determined by consideration of all of the following criteria:

- (a) ability to haul in additional cable infrastructure;
- (b) for telecommunications conduit, cable accommodation and technology mix e.g. twisted pair, coaxial and optical fibre cables;
- (c) costs of conduit laying (direct material and labour costs); and
- (d) costs of trenching and/ or horizontal boring.

6.4.1 Electrical Power Conduit Size and Colour

Electrical power conduit shall be coloured orange. The diameter shall not be smaller than:

- | | |
|----------------------------|--------|
| (a) Flexible Conduit: | 25 mm |
| (b) Traffic Signal Conduit | 80 mm |
| (c) ITS Conduit | 100 mm |

6.4.2 Communications Conduit Size and Colour

Communications conduit shall be coloured white. The diameter shall not be smaller than:

- | | |
|-------------------------------------|--------|
| (a) Flexible Conduit: | 25 mm |
| (b) ITS Conduit | 100 mm |
| (c) Traffic Signal Detector Conduit | 50 mm |

6.4.3 Telecommunications Conduit Size and Colour

The telecommunications conduit shall be coloured white and have a typical internal diameter of 20 mm.

6.5 Cover

Underground consumer mains shall be installed at a depth and method as specified by AS 3000 & SAPN service rules & installation requirements.

7 PITS

7.1 Pit Types

Pits shall comply with the following:

With the exception of pits directly servicing combined road lighting, traffic signal and/or ITS devices, unless otherwise specified, road lighting, traffic signals and ITS shall not share pits. This does not apply to the use of combination road lighting and traffic signal poles at intersections, where road lighting power is fed through the intersection's backbone from a distribution board in the traffic signal controller.

7.2 Service Disconnection Pit

Refer drawing S4055 sheet 56.

7.3 Pit Sizes

DPTI utilises the generally accepted industry standard of "P" classification for pit sizes, e.g. P2, P3, P4, P5 and so forth.

Pit sizes shall be selected on the basis of the intended use, including the number and size of conduits entering and exiting the pit, and shall enable the cable(s) to be installed and operated according to the manufacturer's specifications, particularly with regard to minimum bending radiuses. As a guide, nominal pit uses are listed below.

Pit Size	Nominal Pit use
P1	Not used by DPTI
P2	Road lighting slip base pole base pits; direct connects; earth stake pit; vehicle detector; switchboard terminating pit (type A)
P3	Road crossings for lighting only; draw-in pits for road lighting only
P4	Road lighting impact absorbing frangible pole base pits; road crossings; large switchboard terminating pit; traffic signal controller terminating pit; combination lighting signal pole. If the total number of conduit penetrations into the pit exceeds 4 conduits, then a P4 pit shall be used
P5	As required for cable size, cable numbers and bending radius
P6	As required for cable size, cable numbers and bending radius
P7	As required for cable size, cable numbers and bending radius
P8	As required for cable size, cable numbers and bending radius
S	Traffic signal pull in pits; road crossing pits for traffic signals
D	Traffic signal backbone (typically at corners and in front of controller)

7.4 Pit Location

Pits shall not be located in positions where it is likely that they will be driven over unless design constraints render this unachievable.

Pits and pit lids located in any area which may be subject to being driven over, e.g. carriageways, verges, hard shoulders or service bays/maintenance areas, shall be designed to meet the appropriate load classification as described in AS 3996-2006 Table 3.1.

The pit locations shall wherever possible:

- (a) be installed at the splice point between transverse and longitudinal connections of conduit;
- (b) be located not less than 1 m from the kerb and not less than 3 m back from the intersection of property lines at street corners or from their projection at truncated corners;
- (c) be positioned longitudinally with the 'line' of the roadway/street with sides parallel to the property boundary, footpath or kerb;
- (d) not be located at vehicle crossovers or at places where congestion of services and future maintenance activities by other service agencies could affect the security of the Principal's plant/assets;
- (e) not be installed in flat painted islands or medians;
- (f) not located within restricted zones of LV electricity distribution pedestals, pads, domes, Stobie poles or service pits;
- (g) not be located in driveways (an existing pit may be left in situ provided that it is in sound condition and is safe);
- (h) not be located in pedestrian ramps;
- (i) not be located in bicycle paths unless absolutely necessary, and if so, the pit lid is to be provided with a suitable permanent non slip treatment;
- (j) not be located within rail corridors, unless specified otherwise; and
- (k) have a pit spacing no larger than 70m, unless otherwise specified.

Draw-in pits of a size suitable for allowing subsequent cable replacement and/or repair shall be located to enable cabling as follows:

- | | |
|--------------------|---------------|
| (a) 80 mm conduit: | 50 m spacing |
| (b) 100 mm conduit | 100 m spacing |

7.5 Earth Stake

Where the switchboard is mounted on an SA Power Networks pole, an earth stake pit shall be installed between 3 m and 5 m from the pole.

7.6 Drawing Examples

Refer to Appendix: Instructional Drawings and DP012 for drawing examples.

8 NETWORK IDENTIFICATION

Pits and conduits shall be clearly labelled on the drawings to show the following, as per standard drawings and the DPTI Drawing Legend:

- (a) pit use and size (e.g. L3 = Lighting pit, size P3, IT4 = ITS pit, size P4);
- (b) non-secure (no label), secure, or lockable (as per labels on Legend);

(c) lid type (e.g. concrete, metal, composite); and

Conduits shall be labelled as follows:

CONDUIT ALLOCATION	LABEL
Traffic Signals power	TS
Road Lighting power	RL
Intelligent Transport Systems power	ITS
Intelligent Transport Systems communication	COMMS
Telecommunications service	T
Electrical Service	ES

Conduit labelling shall also include conduit number and size.

9 RECORDS

The Designer shall prepare and provide the following records:

Drawings

The design drawings in accordance with DPTI Design Presentation Standards, in particular “DP001 – General requirements”, DP011 “Traffic Signals” and DP012 “Traffic Signal Conduit”. As of September 2015, drawing examples for ITS are yet to be published.

Update of any existing infrastructure drawings.

If specified, the designer shall also provide “As Constructed” drawings showing actual locations of pits and conduits at the completion of the project.

10 APPENDIX: INSTRUCTIONAL DRAWINGS

Instructional drawings are being developed.



TRAFFIC SIGNAL DESIGN - TS 100

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TS 100 Version Register

Date-Version	Details of Revision	Authorised By
5 Nov 2014 – Ver 0		

1 INTRODUCTION

This Design Standard specifies the requirements for undertaking the design and documentation of traffic signals to be installed on Department of Planning, Transport and Infrastructure (DPTI) roads. The DPTI Traffic Operations Group (Norwood Office) is responsible for managing DPTI traffic signal assets and the provision of any information required for the design of traffic signals.

2 DEFINITIONS

“**AIMSUN**” means the software product produced by TSS.

“**TMC**” means the DPTI Traffic Management Centre located at Norwood.

“**TRANSYT**” means the software produced by TRL Software.

“**TSOPR**” means Traffic Signal Operation Performance Report.

“**SIDRA**” means the SIDRA Solutions software produced by Akcelik and Associates.

“**SCATS®**” means the Sydney Coordinated Adaptive Traffic System, property of RMS NSW (refer <http://www.scats.com.au>).

3 REFERENCES

Unless specified otherwise, all design must be undertaken in accordance with the following:

Road Traffic Act 1961 (SA).

DPTI: Code of Technical Requirements for the Legal Use of Traffic Control Devices.

DPTI: Road Design Standards and Guidelines, available from: <http://www.dpti.sa.gov.au/standards>.

DPTI: Operational Instruction 14.2 Traffic Signal Faces, available from:
<http://www.dpti.sa.gov.au/standards/tass>

DPTI Master Specification Part 255: “Installation of Traffic Signals”, available from:
http://www.dpti.sa.gov.au/contractor_documents/specifications_-_division_2_roadworks

AS 2144: Traffic Signal Lanterns.

AS 2578: Traffic Signal Controllers.

AUSTROADS: Guide to Traffic Management.

AS 1742: Manual of Uniform Traffic Devices.

National Heavy Vehicle Register, National Transport Commission Performance Based Standards Scheme Network Classification Guidelines (“NTC Guidelines”) July 2007.

DPTI: AIMSUN Model Development Manual.

DPTI: Traffic Modelling Guidelines – TRANSYT 15

The following drawings apply to the design of traffic signals:

S-6841:sheet 1 and 2	Traffic Signal Design Guide, detectors, Signal Groups, Phasing and Pole numbering
S-4500 sheets 1 and 2	Detector Loop Layouts
GD 704	PAC Standard - No Median
GD 705	PAC Standard Median up to 3m (solid or painted)
GD 706	PAC Standard Dual Carriage Way Raised Median more than 3m

DPTI standard drawings are available from the following web site: <http://www.dpti.sa.gov.au/standards>.

4 TRAFFIC SIGNAL OPERATIONAL PERFORMANCE

4.1 General

Traffic signal performance analysis must be undertaken, including:

- a) capacity analysis to guide the design and operational requirements of traffic signals;
- b) modelling using current and specified future design flows, and
- c) detailed assessment of the traffic impacts of alternative traffic management arrangements to be used during construction of the project.

The following design and operational requirements must be achieved:

- a) lane, phasing and coordination requirements at traffic signals that ensures the safe and efficient operation of road network for the current and future design flows, as specified;
- b) traffic signals integration within DPTI's current systems used to monitor and control traffic signal operation; and
- c) the needs of all road users (e.g. buses, heavy vehicles, freight, cycles, and pedestrians) are taken into account.

4.1.1 Traffic Signal Operation Performance Report (TSOPR) –reporting requirements

A report describing all aspects of traffic signal operational performance must be provided at the following stages of design:

Notional 30% design stage:	the TSOPR must be completed including calibration and validation of the base case and assessment of all the options/scenarios
Notional 70% design stage:	the TSOPR will be reviewed to account for design changes since the 30% design stage
Final design stage:	the TSOPR will be reviewed to account for design changes since the 70% design stage

At each design stage, the TSOPR must be accompanied by the respective model files from which the conclusions are drawn and recommendations made.

The report must include:

- a) an executive summary including all recommendations;
- b) the intersection performance, which must list outcomes using the specified design traffic flows and the following:
 - Intersection layout plans, traffic signal phasing, phase sequences and time settings,
 - Offset parameters for linking to adjacent traffic signal sites;
- c) an outline of the traffic modeling methodologies adopted for the analysis of intersection and network performance including reasons for model selection and statement of compliance with and any DPTI specified parameters (and reasons for and list of deviations from these);
- d) a description of the existing traffic conditions including the signalised intersection performance at the intersections within close proximity that influence/may be influenced by the operation of the intersections which are included in the project;

- e) a summary of the assessment of the existing (base case) and forecast am and pm peak hour turning volumes at subject signalised intersections and any assumptions;
- f) degree of saturation, LOS and 95th percentile queue lengths for individual traffic movements;
- g) discussion of alternative schemes considered and reasons for the recommendations;
- h) report on calibration and validation of models;
- i) history of design changes with supporting reasons from 30% Nominal Design through to the Final Design;
- j) discussion of alternative schemes considered; and
- k) any variations from the requirements of this Design Standard or any specified requirement.

4.2 Operational Analysis

4.2.1 General

The traffic signals analysis must include:

- a) the base model, fully calibrated and validated to reflect the existing traffic conditions, and
- b) the projected model options, using the calibrated and validated base case with the projected scenario criteria.

The SCATS Summaries provided by DPTI must be used to calibrate and validate base case traffic models.

All traffic models must use as far as possible the labelling conventions for signal groups, detectors, and phasing as described in DPTI standard drawings S-6841 sheets 1 and 2, except for existing traffic signal controls which should use the existing labels in the personality.

The impacts on intersection and network performance of proposed traffic management strategies to be adopted during construction must be assessed. Analysis must include all major construction staging of traffic signal arrangements, including a detailed assessment of the performance of the proposed interim traffic signal operations.

All traffic signals analysis must be fully documented in the TSOPR.

4.2.2 SIDRA Analysis

In addition to any other modelling, modelling to determine traffic flow performance SIDRA for analysis of individual sites must be undertaken. The SIDRA analysis must confirm that the proposed traffic signal design and operation will satisfy the following performance criteria:

	New Signals	Temporary Signals
Degree of Saturation for each signal site	≤ 0.9	≤ 1.0
Level of Service (LOS) (use Standard left)	"E" or better	"E" or better

The SIDRA analysis must:

- a) for existing sites; compare the current intersection performance with proposed options;
- b) use the data available in the SCATS summaries available from DPTI for configuring the base case models;
- c) assume fixed time traffic signal control for models;
- d) model to the cycle length of the current maximum cycle length set in SCATS® or use the optimised cycle length up to a maximum value of 150 seconds;

- e) calibrate saturation flows in the base model to reflect those experienced in existing traffic conditions (SCATS MF data which represents measured saturation flows in passenger car equivalent values are included in the SCATS summaries);
- f) for new intersections, use default values from SIDRA modified to reflect intersection geometry, traffic composition, road environment, and grade; these adjusted saturation values should be typical of MF values experienced at adjacent SCATS sites;
- g) where analysis includes existing signal sites, use the current controller settings for red/yellow and pedestrian times;
- h) for new and proposed upgraded traffic signal sites, the settings for minimum green, red/yellow and pedestrian times must be calculated using the DPTI phase time setting template; and
- i) incorporate pedestrian and cycle movements in the phasing at all sites on all approaches except where there is expected to be no current or future demand.

4.2.3 Traffic Network Analysis

Traffic modelling techniques must be used to demonstrate that the traffic signals will be capable of being effectively coordinated with adjacent traffic signal controlled sites, which may include traffic signals located outside of the extent of the project. The recommended offsets are to be included in the TSOPR.

The intent of this modelling is also to determine any adverse impacts of the project on traffic signal operation. Where such impacts are evident, the design must (as appropriate):

- a) develop and assess alternative concepts and select a preferred concept for capacity improvements at the intersections or the interchange ramps;
- b) inform DPTI of other locations that may require improvements. and
- c) identify locations at which temporary capacity improvements may be required to cater for traffic diversions resulting from proposed full or partial road closures.

A calibrated and validated base case using existing traffic volumes must be provided. Proposed Options must be based on the Projected Traffic Design Volumes.

The modelling must be undertaken by a modelling tool used by DPTI, which currently includes:

- a) TRANSYT;
- b) AIMSUN.

TRANSYT must be used to analyse intersection performance of all intersection and interchanges. The models must extend to include all intersections whose performance may influence, or be influenced by, the operation of other intersections.

The TRANSYT model must be developed in accordance with the DPTI "Traffic Modelling Guidelines - TRANSYT 15".

DPTI must be provided with a calibrated micro simulation AIMSUN which must be used as the basis for assessing traffic impacts.

In creating an AIMSUN model for network analysis, the model/s must be developed in accordance with the DPTI AIMSUN Model Development Manual.

In consideration of the predicted queuing effects, the model analysis using any of the above software must be consistent with the "SIDRA Analysis". (refer Clause 4.2.2)

4.3 Design and Operational Requirements

4.3.1 Turn Movements

All turning paths must cater for the type and size of specified Design Vehicle. Consideration must be given to these turning paths in designing the most effective phasing for the intersections and the implications for the physical intersection arrangement to maximise the traffic throughput.

Right turn lanes and left turn lanes must be designed with turn bays to remove through lane interaction and short queue effects.

4.3.2 Heavy Vehicles and Buses

The design must clearly show how the affect of heavy vehicles and buses on design flows and signal timings are taken into account. In particular the design must allow for the yellow and all red clearances for long vehicles required by the NTC Guidelines, Section 2.5 “Signalised Intersections”.

5 DESIGN REQUIREMENTS

The traffic signal design must comply with the following:

- a) the operational performance requirements documented in the TSOPR are achieved;
- b) cater for all turn movements of the maximum size Specified Design Vehicle;
- c) the design of the turn movements is checked to ensure that adequate provision has been made to cater for all possible traffic signal phase arrangements and sequences;
- d) signal groups, detectors, phasing and poles are designed in accordance with DPTI standard drawings S-6841 sheets 1 and 2;
- e) the design of pedestrian activated crossings is in accordance with DPTI standard drawings:
 - GD 704 “PAC Standard - No Median”,
 - GD 705 “PAC Standard Median up to 3m (solid or painted)”, and
 - GD 706 “PAC Standard Dual Carriage Way Raised Median more than 3m”;
- f) pedestrian displays are incorporated in the phasing at all sites except where there is no demand identified by the design process;
- g) provision of vehicle and pedestrian detection is included at all signalised intersections;
- h) a vehicle stopline detector is required for every discrete traffic lane at an intersection including left turn lanes which are not controlled by a signal group, (These loops in left turn lanes will initially be used for counting, but can be used to control traffic in adjacent lanes); and
- i) Provide queue detectors at intersections and level crossings where blocking back is anticipated from upstream intersections or level crossings.
- j) the design is documented in accordance with the Design Presentation – Construction Drawings DP011 - Traffic Signals.

6 DESIGN DOCUMENTS

6.1 Drawings and schedules

At a minimum, the design must be documented in the following drawings and schedules:

- a) traffic signal layout drawings;
- b) signal duct layout drawings;
- c) site wiring diagrams;

- d) cable connection schedules, including the identification of spare cores;
- e) hardware schedules;
- f) traffic signal phasing plan; and
- g) equipment drawings;

6.2 Traffic Signal Design Report

A Traffic Signal Design Report must be prepared, which at a minimum includes details of the following:

- a) system description;
- b) reference to all applicable standards and specifications;
- c) details of design inputs from the Traffic Signal Operation Performance Report;
- d) details of Interfaces to existing or third party systems;
- e) operational description, including phasing philosophy;
- f) location of all equipment;
- g) explanations for non-standard equipment locations;
- h) controller input/output allocations;
- i) signal group allocations;
- j) detector input allocations;
- k) power system, volt drop and fault loop impedance calculations;
- l) details of all proposed construction materials;
- m) construction designs for all equipment;
- n) traffic signal hardware documentation;
- o) specification for supply and installation of traffic signals; and
- p) Design, procurement, installation integration and commissioning program.

7 TRAFFIC SIGNAL HARDWARE

7.1 Approved Products

Details of DPTI approved traffic signal products are available from http://www.dpti.sa.gov.au/contractor_documents/specifications.

7.2 Traffic Signal hardware Documentation Requirements

Traffic Signal hardware requirements must be documented in the Traffic Signal Design Report, including:

- a) Traffic signal site number;
- b) Telstra PSTN and port service numbers; and
- c) Confirmation of site specific parameters.

Upon written request, DPTI will supply the necessary information to enable the purchase of the communications services. DPTI does not warrant the time of delivery of this information and at least 2 weeks from the request to delivery must be allowed.

7.3 Traffic signal Operating Voltage

Unless otherwise specified, the Traffic Signal Design must incorporate extra low voltage (ELV) traffic signal controllers, Lanterns and other electrical equipment.

7.4 Traffic signal wiring

A traffic signal cable connection chart must be prepared for approval of DPTI. A minimum of 10 spare cores in road crossings and 3 spare cores to each pole top must be provided.

7.5 Traffic Signal Lanterns

At new traffic signal sites, all traffic signal lanterns must use a Light Emitting Diode (LED) optical system, comply with AS 2144 and be approved by DPTI.

Hardware compliance details must be provided in the Traffic Signal Design Report. Where additional lanterns are to be provided at an existing signal site, all the lanterns must be changed to LED. All lanterns must be of the same brand and version. 200mm or 300mm aspects must be provided in accordance with DPTI: Operational Instruction 14.2 "Traffic Signal Faces".

7.6 Location of Signal Equipment and Signal Face Layouts and Display Sequences

The arrangement of traffic signal faces must be in accordance with DPTI: Operational Instruction 14.2 "Traffic Signal Faces".

7.7 Traffic Signal Controller

All new traffic signal controllers must conform to Transport for New South Wales Road and Marine Services, NSW specification TSC/4 and be approved by DPTI.

The location of the traffic signal controller must be determined using the following criteria:

- a) minimisation of obstruction to pedestrians;
- b) minimisation of visual obstruction to drivers;
- c) minimisation of the risk of accidental damage by traffic;
- d) provision of a safe and easy access for maintenance personnel and associated vehicles;
- e) permits maintenance staff to have a clear view of the whole of the intersection from the controller as far as is practicable;
- f) orientation so that the cabinet door(s) open away from the centre of the intersection; and
- g) close location to the power supply and telecommunications service.

7.8 Video Surveillance (CCTV cameras)

Where specified, the design must include video surveillance.

Video surveillance must be included to provide coverage at approaches to level crossings where queue management strategies are required to prevent queuing over the level crossing. The video surveillance system will be incorporated in the traffic signal controller hardware.

Note: CCTV monitoring is required to monitor where changes to traffic signal operation is expected to be required on a regular basis. The requirements for CCTV coverage are specified elsewhere, but will include, the location of new permanent traffic signals, new signals under construction (or temporary signals) where intersection geometry and phasing requirements will be subject to ongoing changes over an extended period of time (e.g. 6 months), reversible lanes, variable lane use signals, variable speed limits, and locations impacted by special events.

Where CCTV equipment is required to be installed at intersections, the roadside equipment must be housed in an extension housing attached to the top of the traffic signal controller cabinet.

CCTV equipment must be approved by DPTI.

7.9 Programming of Traffic Signal Controller(s)

The Programmable Controller Personality Module (PCPM) contains the personality for the controller.

DPTI does not supply the PCPM; however it will provide a personality programming service, subject to the following conditions:

- a) all Design Documents (including all reports including the TSOPR and drawings) are complete and all applicable approvals or hold points released; and
- b) not less than 8 weeks prior to the required delivery date of the program personality, DPTI has been requested (in writing) to provide programming of the controller and has also been provided all documentary information necessary for the programming.

7.10 Uninterrupted Power supply

Uninterrupted power supply (UPS) must be provided at sites nominated by DPTI. The UPS must operate over a period of not less than 4 hours.

UPS equipment must be approved by DPTI

Subject to the approval of DPTI, an internal UPS integrated into the signal controller cabinet may be allowed.

7.11 Provision of Telecommunications for SCATS®

7.11.1 SCATS® Communication

All traffic signal controllers must be connected to SCATS® via a DPTI approved method which will normally involve an IP-Wan service or a connection directly to the DPTI fibre optic cable network. The form of the connection is subject to approval by DPTI.

7.11.2 Vehicle Detection

Where inductive loops are utilised for vehicle detection, the loops must be as detailed on DPTI Drawing No. S-4500 sheets 1 and 2. Detector loops located at the intersection stop bar must be the quadrupole type. Other loops will be designed appropriate to their function.

Vehicle stop line type detectors suitable for SCATS® operations must be provided for all lanes irrespective of the control function. Loops on non-controlled lanes e.g. left turn lanes, must be provided and connected to traffic controllers and must be used for counting.

Advanced detectors must be considered on approaches with speed limits 80 km/h and over. Locations to be calculated on the basis of the design speeds and must consider the approach gradient. Advanced loops will be located in accordance with the DPTI design template. Alternative technologies to loop detection will be considered providing they are compatible with SCATS® operation.

Where queue detectors are recommended in the TSOPR their final position on the road will be determined in consultation with DPTI.

7.11.3 Pedestrian and Bicycle Detection

Push buttons provided for this purpose must be a product approved by DPTI.

All pedestrian movements must be demand actuated by audio tactile pedestrian push buttons. Pedestrian push buttons must:

- a) be installed at 1 m height above the standing place used by pedestrians when operating the button. Pedestrian;
- b) be orientated so that the face of the push button is in line with or parallel to the crosswalk marking;

- c) incorporate arrow legends (in the audio tactile display) on all pedestrian push buttons;
- d) be correctly oriented to guide visually impaired pedestrians in the same direction indicated by cross walk markings;
- e) be provided on median traffic islands; and
- f) Incorporate microwave pedestrian sensors where pedestrian displays are controlling marked foot crossings 15 m and longer.

Cycle push buttons must be provided on side road approaches and at locations where extended clearance is required. Cycle Push button faces are to be parallel to the kerb.

In-ground loop detection in cycle lanes must be provided between marked vehicle lanes

7.11.4 Level Crossings

Interlocking between level crossings and adjacent traffic signal installations must be provided with a hard-wired connection.

Four standard inputs are to be provided via the hard wire connection and these will be documented in the Traffic Signal Design Report.

Level crossings are considered adjacent to traffic signals where the traffic signals could be interpreted to conflict with the level crossing wigwag signs or where the level crossing is expected to create queues across the intersection, or visa versa. There must be a cable link provided in duct between the level crossing controller and the traffic controller, which will enable the traffic signal controller to be forced to a "safe" state before the level crossing closes to permit the passage of a train.

7.11.5 Traffic Signal Poles

The location of traffic signal poles must take into account site constraints such as services and roadside furniture. The Pole location(s) must ensure that the signals can be sited where they can be clearly seen by approaching drivers. (Refer to DPTI: Operational Instruction 14.2 "Traffic Signal Faces"). Notwithstanding the above care must be taken not to locate poles in locations where they are likely to be struck by vehicles following a conventional turning path, or where long vehicles may mount the kerb while negotiating a turn.

Traffic signal posts must not be located within painted medians/islands; i.e. where there are no kerbed islands or there is an area physically raised from the road, which is designed only to discourage overrunning by large vehicles.

Wherever possible, combination poles comprising road lighting and or mast arms must be used to minimise the number of signal poles. Where a site is specified to be ELV operation and low voltage (LV) road lighting is used, appropriate electrical segregation must be maintained between the two systems.

Where combination poles are to be incorporated into the traffic signal design consideration will be given to source of the energy supply for the street lighting. Provision of this type of pole is not to compromise the design of either the street lighting or the traffic signals.

The provision of road lighting is not to compromise the safe location and distribution of traffic signal poles. Traffic signal poles are to be located for optimal design and if the location is suitable for a road lighting unit the combining of the units on a common pole may be considered.

7.11.6 Pavement Markings

Where the speed limit is 80 km/h or higher, 600 mm stop lines on approaches must be provided.

7.11.7 Signs

Where regulatory signs are to be provided for part time regulation, a symbolic internally illuminated sign (no left turn/no right turn) must be provided at a stop line pole and a secondary (or tertiary) pole.

Sign equipment must be approved by DPTI.

Signs must be equipped with fault monitoring, and report faults back to the signal controller.

8 RECORDS

The following records must be provided to DPTI:

Drawings

The drawings described in Clause 6 “Design Documents”

Relevant standard drawings from the DPTI: Road Design Standards and Guidelines

Reports

Traffic Signal Operational Performance Report

Traffic Signal Design Report

APPENDIX I: URBAN – BARRIER HAND CALCULATION

DPC Hand Calculations

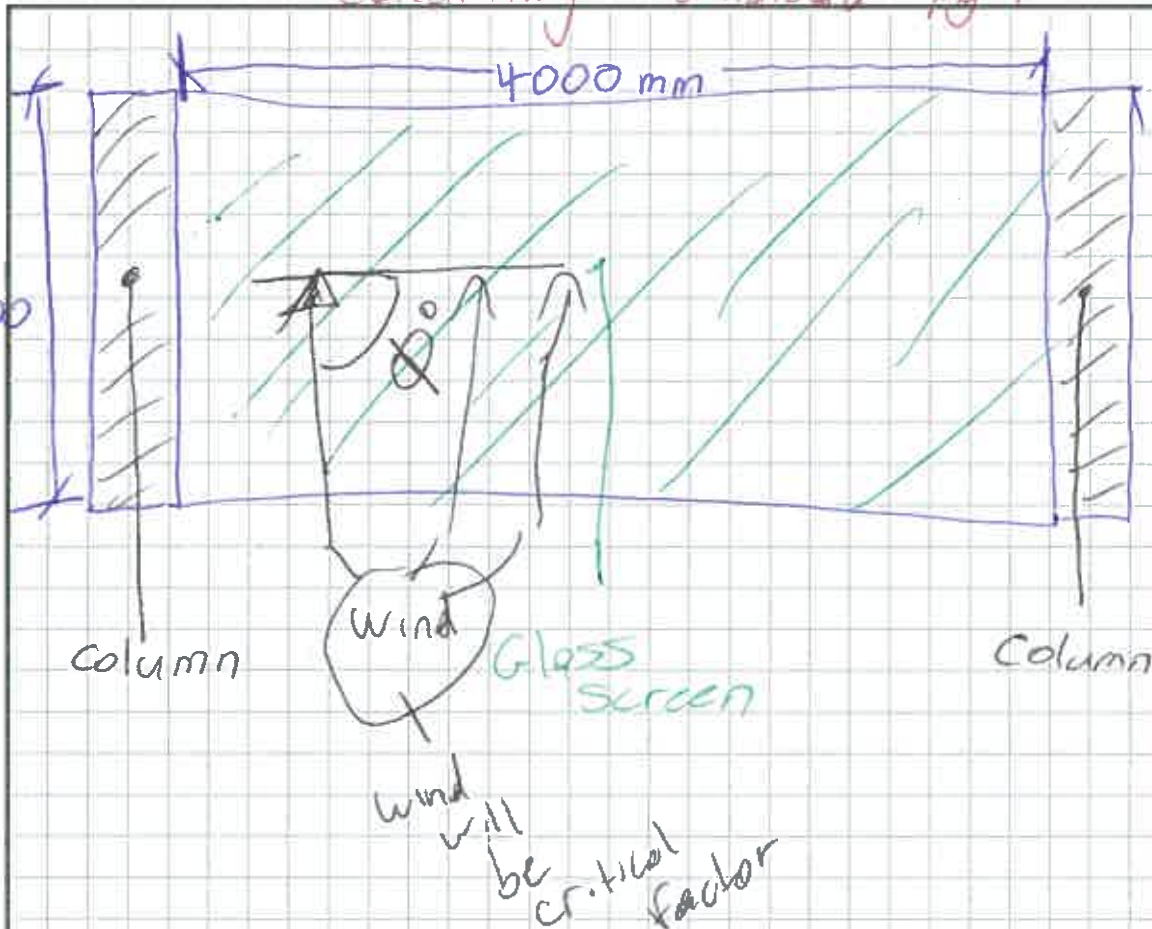
By: Dino

Date: 28/5/2017

Checked by: _____

Date: / /

Calculating windload Pg 1



Just a sketch

Glass pannels

max size : 2400mm x 4200mm x 20mm

Assume glass used is strong enough

∴ only design columns

Chevron
glass
spec
book
2017

DPC Hand Calculations

By: Dino

Date: 29/5/17

Checked by: _____

Date: 1/1

calculating windload on barrier pg 2

Find site windspeed (Only critical direction)

$$V_{site,b} = V_R M_d (M_{z,cat}, M_s, M_e)$$

AS 1170.2
Section 2.2

V_R

$$V_R = V_{500} = 45 \text{ m/s}$$

- Importance 1v 2
- Region A1
- 1/500 years

AS 1170.0
cl 3.2

M_d

$$W = 1.00 \quad (\text{West is critical})$$

AS 1170.2
cl 3

AS 1170.2
tbl 3.2

$M_{z,cat}$

- Barrier sits high not much shielding

$$M_{z,cat} = 0.93 \quad (\text{Interpolation})$$

AS 1170.2
cl 2.1
tbl 4.1

M_e

$$M_e = M_h = 1 \quad - \text{Assumption}$$

AS 1170.2
cl 4
tbl 4.3

cl 4.2.2

DPC Hand Calculations

By: Dine

Date: 29/5/17

Checked by: _____

Date: 1/1

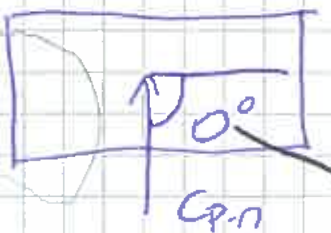
calculating windload barrier pg 3

Calculating $V_{s1,b}$

$$V_{s1,b} = V_R M_d (M_{Z1,cat} M_s M_G)$$

$$V_{s1,west} = 45 \times 1000 (0.9 \times 0.9 \times 1) = 37.6 \text{ m/s}$$

Free standing wall / barrier



critical case

$$P_{air} = 1.2 \text{ g/m}^3$$

$$C_{ydn} = 1$$

$$K_p = 1$$

$$C_{F,y} = C_{p,n} \times K_p$$

$$C_{p,n} = 1.28$$

$$P = (0.5 P_{air}) (V_{des,w})^2 (C_{F,y}) (C_{ydn})$$

$$= 0.5 \times 1.2 \times 37.6^2 \times 1.28 \times 1$$

$$= 1088 \text{ pa (acting perpendicular to barrier)}$$

$$= 1.085 \text{ kpa}$$

As 1170.2
Appendix
D1.4

DZ
D2

$$C_{p,n} = 1.3105$$

$$(0.3 + \log(6.4))$$

$$(0.8 (-1/5))$$

$$= 1.28$$

DPC Hand Calculations

By: Duo

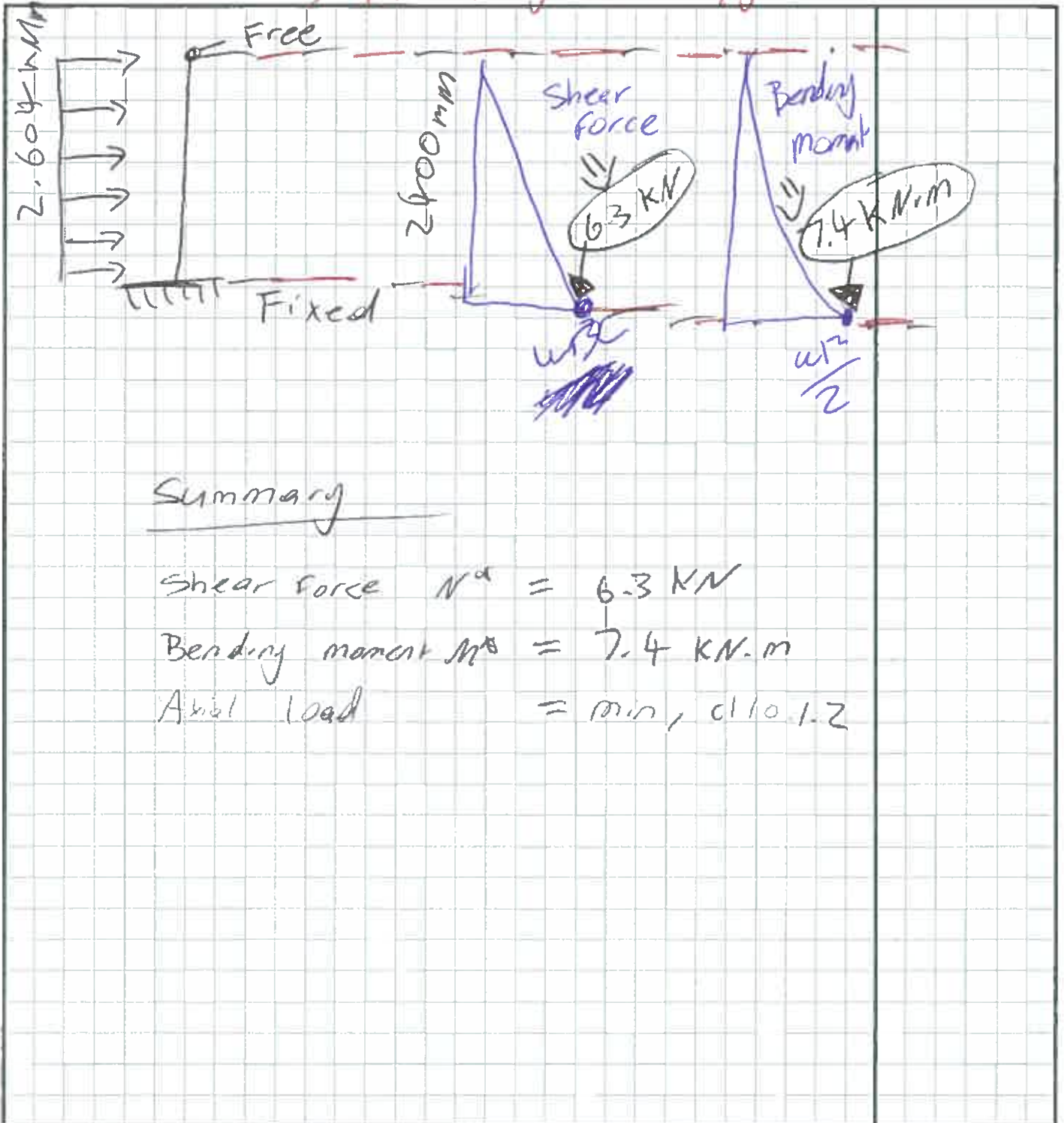
Date: 29/5/17

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Date: / /

BM & SF Diagram

pg 4



Summary

Shear force $N^a = 6.3 \text{ kN}$

Bending moment $M^a = 7.4 \text{ kN.m}$

Abut load = min, cllo 1.2

DPC Hand Calculations

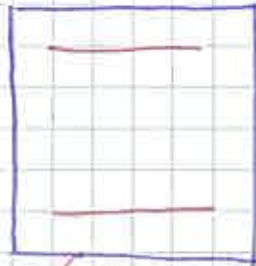
By: Dino

Date: 29/5/2017

Checked by: _____

Date: / /

Designing interaction curve for concrete column pg 5



N^u = Design axial load = 6.1 kN

M^u = Design bending = 7.4 kNm

$$M_{min}^u = 0.5D \times N^u$$

$$\frac{M_{min}^u}{0.5D} = N^u$$

$$\frac{7.4}{0.5 \times 24} = 6.1$$

AS 3600

min
Axial
load
cl 10.1.2

A_{st} = unknown

Find A_{st} for
column

$$0.01 A_g = A_{st, min}$$

thus, Adopt

4 N12 bars
 $A_{st} = 440 \text{ mm}^2$

number
size

AS 3600
 $A_{st, min}$
10.7.1

ARC
reinforce
had to do
N Bars

DPC Hand Calculations

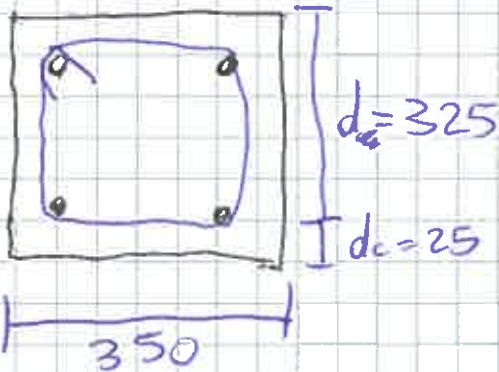
By: D100

Date: 29/5/2017

Checked by: _____

Date: 1/1

Design concrete column Pg 6



$$4 \text{ N12 Bars} = 440 \text{ mm}^2 A_{st}$$

$$1 \text{ N12 Bar} = 110 \text{ mm}^2 A_{st}$$

$$f_{sy} = 500 \text{ mpa}$$

$$\phi = 0.8$$

$$A_g = 350^2 \text{ mm}^2$$

$$A_{st} = 440 \text{ mm}^2$$

$$D = 350 \text{ mm}$$

$$d = 325 \text{ mm}$$

$$d_c = 350 \text{ mm}$$

$$d_c = 25 \text{ mm}$$

$$E_s = 200\,000$$

$$\gamma = 1.05 - 0.007 f_c$$

$$= 0.77$$

$$\text{condition} - 0.67 \leq \gamma \leq 0.85$$

$$\lambda_1 = 1 - 0.003 f_c$$

$$= 0.88$$

$$\text{condition } f_c = 50 \text{ mpa} = 0.85$$

$$\text{min cover} = 25 \text{ mm}$$

AS 3600

✓ - cl 2.2.1

+6-2-2.2

AS 3600
cl 10.6.25

DPC Hand Calculations

By: Dino

Date: 29/5/17

Checked by: _____

Date: 1/1

Design concrete column pg 7

Squash load

$$N_{uo} = \lambda \cdot F_c \cdot A_g + F_{sy} \cdot A_{st}$$

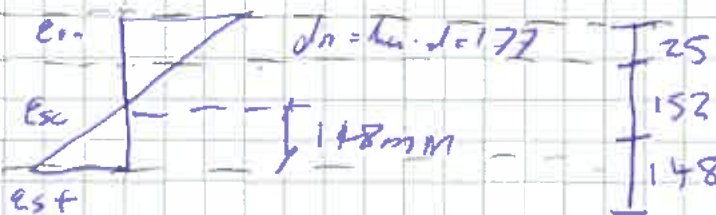
$$= 0.85 \times 40 \times 350^2 + 500 \times 440 \quad (\times 10^{-3} \text{ kN})$$

$$= 4385 \text{ kN}$$

AS 3600
cl 10.6.2.2

$$M_o = 0$$

Balanced failure $N_{ud} + M_{ud}$ ($K_u = 0.575$)



cl 10.6.2.5

$k_u = 0.575$
500 MPa
Bars

$$\epsilon_{sc} = \frac{177 - 25}{177} \times 0.003$$

$$= 0.00257 \neq 0.0025 \text{ (yield)}$$

$$C_s = A_{sc} \cdot E_s \cdot \epsilon_{sc}$$

$$= (2 \times 110) \times 200000 \times 0.0025$$

$$= 110 \text{ kN}$$

AS 3600
cl 6.2.2.5
 $\lambda \cdot d_1$

DPC Hand Calculations

By: Dino

Date: 29/5/17

Checked by: _____

Date: / /

Design of concrete column pg 8

$$\sum F = 0$$

$$\begin{aligned} T &= A_{st} \cdot f_{sy} \\ &= (2 \times 110) \times 500 \\ &= 110 \text{ kN} \end{aligned}$$

$$\begin{aligned} C_c &= 0.85 \cdot f_c \cdot \gamma \cdot k_u \cdot d \cdot b \\ &= 0.85 \cdot 40 \cdot 0.88 \cdot 0.545 \times 325 \times 350 \\ &= 1854 \text{ kN} \end{aligned}$$

$$\begin{aligned} N_{ub} &= C_c + C_s - T \\ &= 1854 + 110 - 110 \\ &= 1854 \text{ kN} \end{aligned}$$

First
principle

Take moment about steel

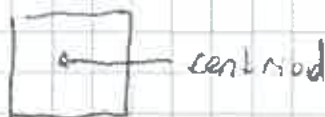
$$C_s \text{ lever arm} = 325 - 25 = 300$$

$$C_c \text{ lever arm} = 325 - \frac{1}{2} \times 0.88 \times 0.545 \times 325 = 247$$

$$N_{ub} \text{ lever arm} = h_{ub}$$

$$h_{ub} = \frac{(110 \times 0.3) + (1854 \times 0.247)}{1854}$$

$$= 264 \text{ mm}$$



$$v = \frac{350}{2} = 150$$

$$\begin{aligned} e &= h_{ub} - 150 \\ &= 115 \text{ mm} \end{aligned}$$

$$\frac{M_{ub}}{N_{ub} \cdot e}$$

$$\begin{aligned} &= N_{ub} \cdot e \\ &= 1854 \times 115 \\ &= 213 \text{ kN.m} \end{aligned}$$

DPC Hand Calculations

By: Dino

Date: 29/5/17

Checked by: _____

Date: 1/1

Design concrete column pg 9

De-compression point N_u, M_u ($k_u = 1$)



compression
steel
 $T=0$ C_s

$$k_{cs} = \frac{325 - 25}{325} \times 0.003 = 0.0027$$

$$\sum F > 0$$

$$\begin{aligned} C_c &= 0.85 \times f_c \times \gamma \times k_u \times d \times b \\ &= 0.85 \times 40 \times 0.88 \times 1 \times 325 \times 350 \\ &= 2977 \text{ kN} \end{aligned}$$

$$\begin{aligned} C_s &= A_{sc} \times \sigma_s \\ &= A_{sc} \times E \times \epsilon_{cs} \\ &= (2 \times 110) \times 200000 \times 0.0025 \\ &= 110 \text{ kN} \end{aligned}$$

$$\sum F; \quad N_u = 2977 + 110 = 3087 \text{ kN}$$

\sum moments about tensile steel

$$h_u = (C_c \times c \text{ lever arm}) + (C_s \times c_s \text{ lever arm})$$

$$\begin{aligned} h_u &= \frac{2977 \times 212}{3087} \\ &= 150 \text{ mm} \\ e &= 62 \text{ mm} \end{aligned}$$

$$\begin{aligned} M_u &= N_u \times e \\ &= 3087 \times 62 \\ &= 191 \text{ kN.m} \end{aligned}$$

DPC Hand Calculations

By: Diao

Date: 29/5/17

Checked by: _____

Date: / /

Designing concrete column pg 10

Pure bending M_{ud} ($N=0$)

Trial \rightarrow error - A.m to get $\Sigma C = \Sigma T$

Trial #1 - $h_u - d = 25 \text{ mm}$

$$C_c = 0.85 \cdot f'_c \cdot \gamma \cdot h_u \cdot d \cdot b$$

$$= 0.85 \cdot 40 \cdot 0.77 \cdot 30 \cdot 350$$

$$= 229 \text{ kN}$$

$$C_s = A_{st} \cdot f_{sy}$$

$$= 0$$

$$T = A_{st} \cdot f_{sy}$$

$$= 220$$

$$229 \approx 220$$

$$\therefore M_{ud} = \Sigma F \times \text{Moment arm}$$

$$= 72 \text{ kN}\cdot\text{m}$$

DPC Hand Calculations

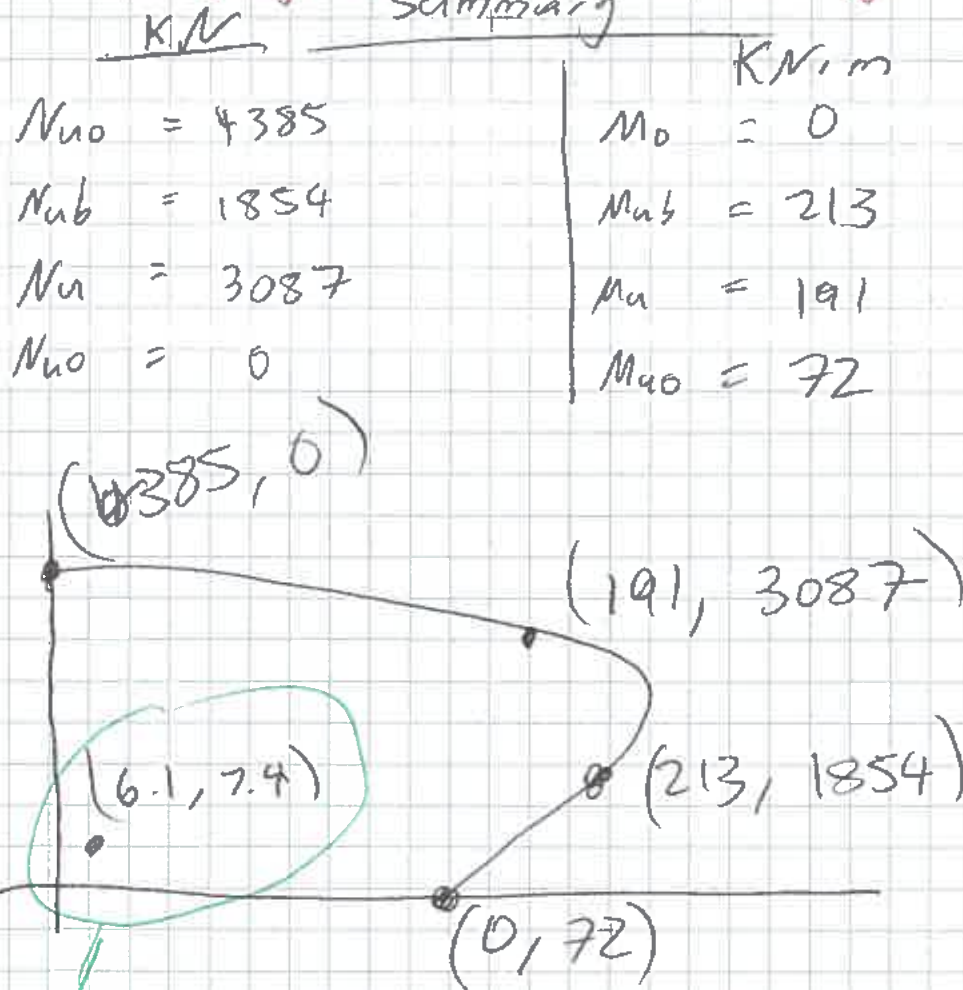
By: Dino

Date: 29/5/17

Checked by: _____

Date: / /

Designing concrete column pg 11



Good News our column is
in safe zone. Super safe
✓✓

DPC Hand Calculations

By: Dine

Date: 29/5/17

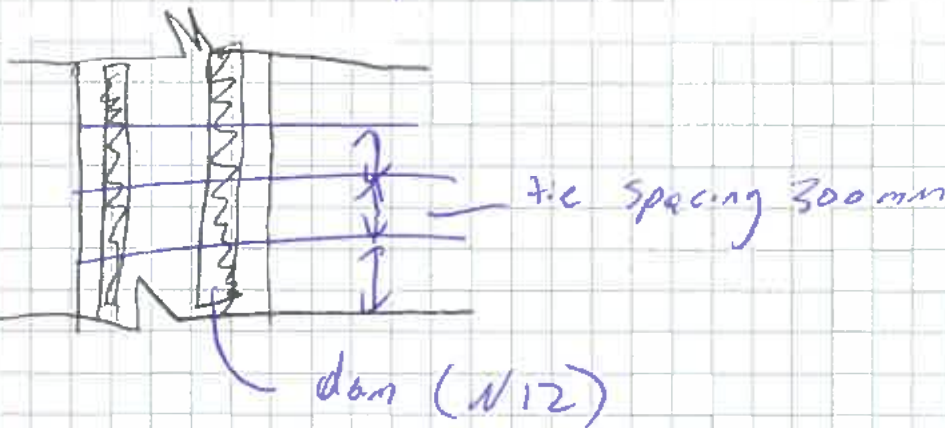
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Date: / /

Detailing Column Pg 12

Stirrup size

bar arrangement	tie size	spacing
single	R10 @ 150mm	N12 @ 300mm



HT - Hooked tie

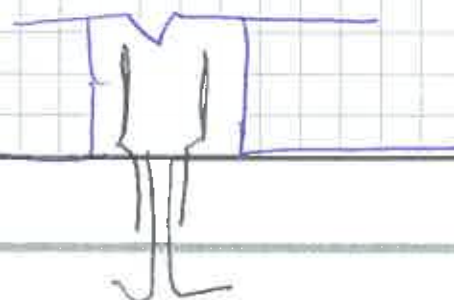
C = two 135° hooks

- rectangular
filament
pitch
by 2
135°
hooks



CC - cranked
column
bar

Eg



DPC Hand Calculations

By: Dino

Date: 29/5/17

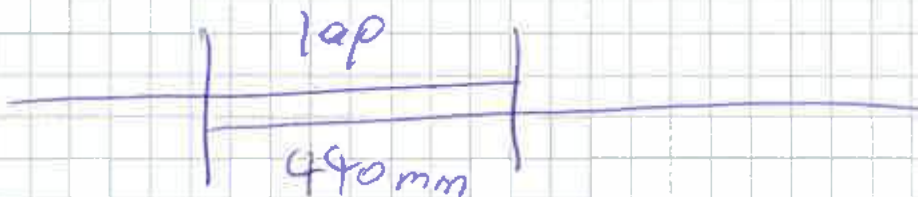
Checked by: _____

Date: / /

Detailing column pg 13

laps

$f'c$	L_{lap} for D500N (N12)
40mpa	440 mm



Detailing
booklet

embedment in slab / footing

Bar size	embedment
N12	440

DPC Hand Calculations

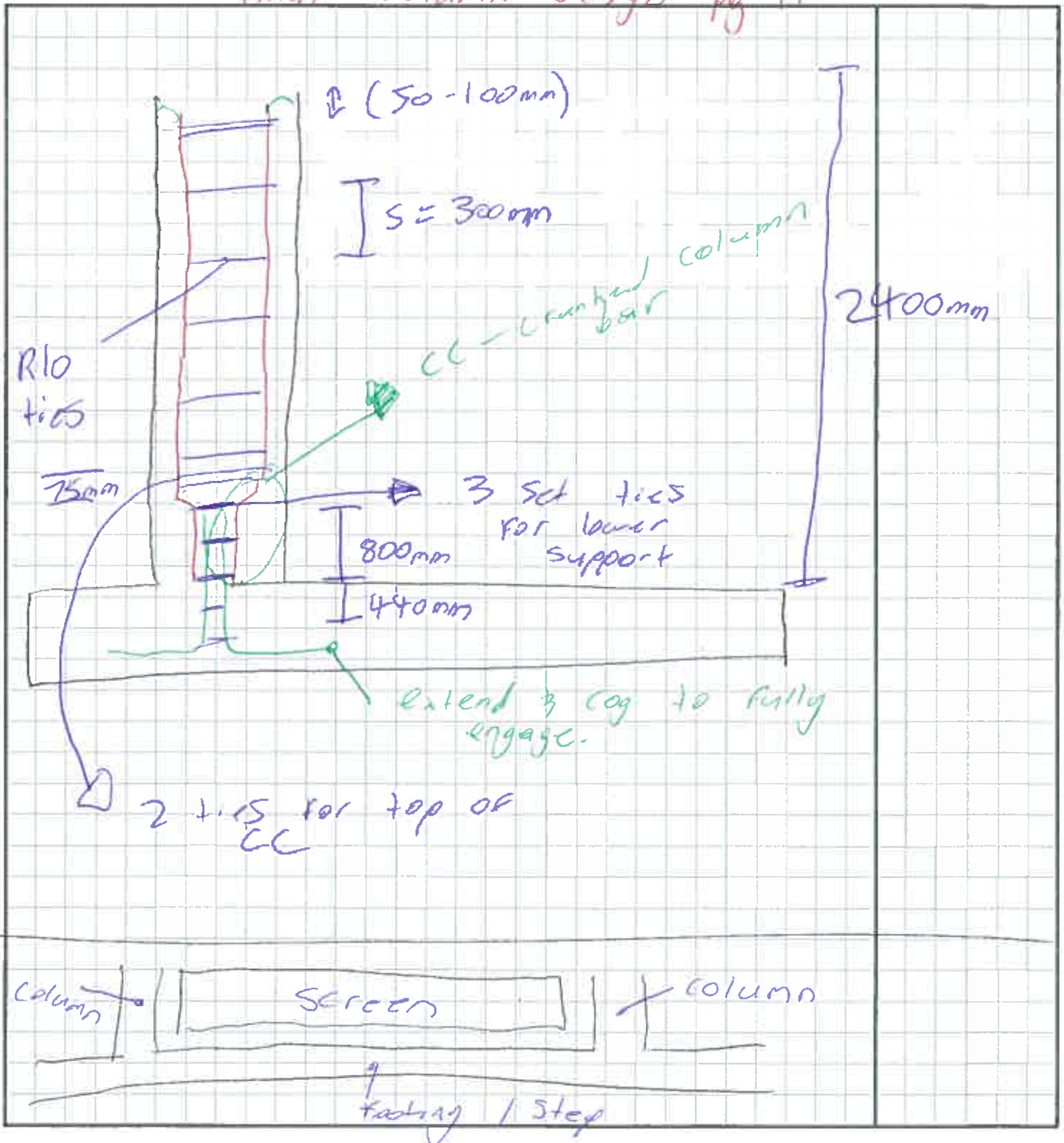
By: Dino

Date: 29/5/17

Checked by: _____

Date: 1/1

Final column design pg 14



APPENDIX J: SHELTER HAND CALCULATION

Date; 1/1

Section 4
to 4.3

DPC Hand Calculations

By: Dno

Date: 23/5/2017

Checked by: _____

Date: 1/1

Calculating wind loads Page 2

M_e

AS 1170.2
Section
4.2.2

$$M_e = M_h = 1 \quad \text{--- Assumption}$$

calculating $V_{sit, Q}$

$$V_{sit, Q} = V_R M_d (M_z, i + M_s M_e)$$

Direction i.e. North, South, etc

$$\begin{aligned} V_{sit, N} &= 45 \times 0.9 (0.83 \times 0.9 \times 1) = 30.25 \text{ m/s} \\ V_{sit, NE} &= 45 \times 0.8 (0.83 \times 0.9 \times 1) = 26.89 \text{ m/s} \\ V_{sit, E} &= 45 \times 0.8 (0.83 \times 0.9 \times 1) = 26.89 \text{ m/s} \\ V_{sit, SE} &= 45 \times 0.8 (0.83 \times 0.9 \times 1) = 26.89 \text{ m/s} \\ V_{sit, S} &= 45 \times 0.85 (0.83 \times 0.9 \times 1) = 28.57 \text{ m/s} \\ V_{sit, SW} &= 45 \times 0.95 (0.83 \times 0.9 \times 1) = 31.93 \text{ m/s} \\ V_{sit, W} &= 45 \times 1.00 (0.83 \times 0.9 \times 1) = 33.61 \text{ m/s} \\ V_{sit, NW} &= 45 \times 0.90 (0.83 \times 0.9 \times 1) = 30.25 \text{ m/s} \end{aligned}$$

Pressures

- Structure is between a building & a free standing structure, thus, using engineering judgement to use design steps in appendix D.

AS 1170.2
D1

$$C_{fig} = C_{p,n} k_p = 0.3$$

$$\underline{k_p = 0.85} \quad \underline{C_{p,n} = 1.2}$$

D1-4

D2

46 D2(A)

DPC Hand Calculations

By: Dino

Date: 23/5/2017

Checked by: _____

Date: 1/1

(calculating wind loads pg 3)

Internal pressures

$$P = (0.5 \times P_{air}) (V_{des})^2 C_{pi} C_{dgn}$$

$$P_{air} = 1.2$$

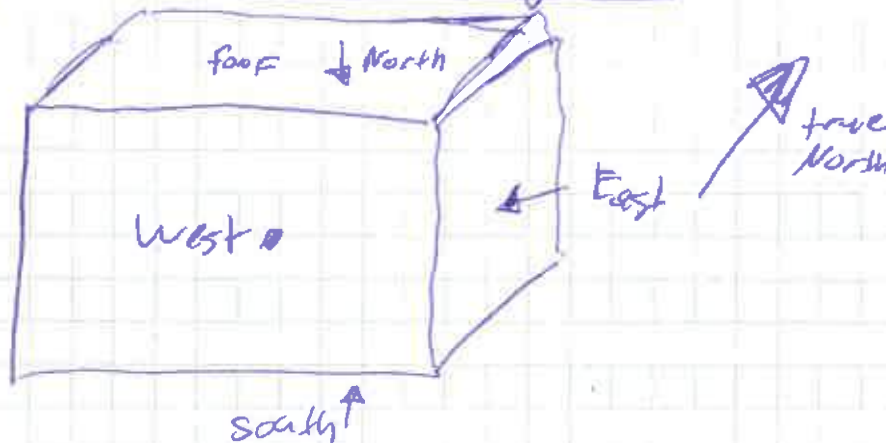
$$C_{dgn} > 1$$

N	=	164	pa
NE	=	130	pa
E	=	130	pa
SE	=	130	pa
S	=	150	pa
SW	=	189	pa
W	=	206	pa
NW	=	171	pa

⇒ Approx. round up.

AS 1170.2
Section 5

Building Sketch (not quite a building)



DPC Hand Calculations

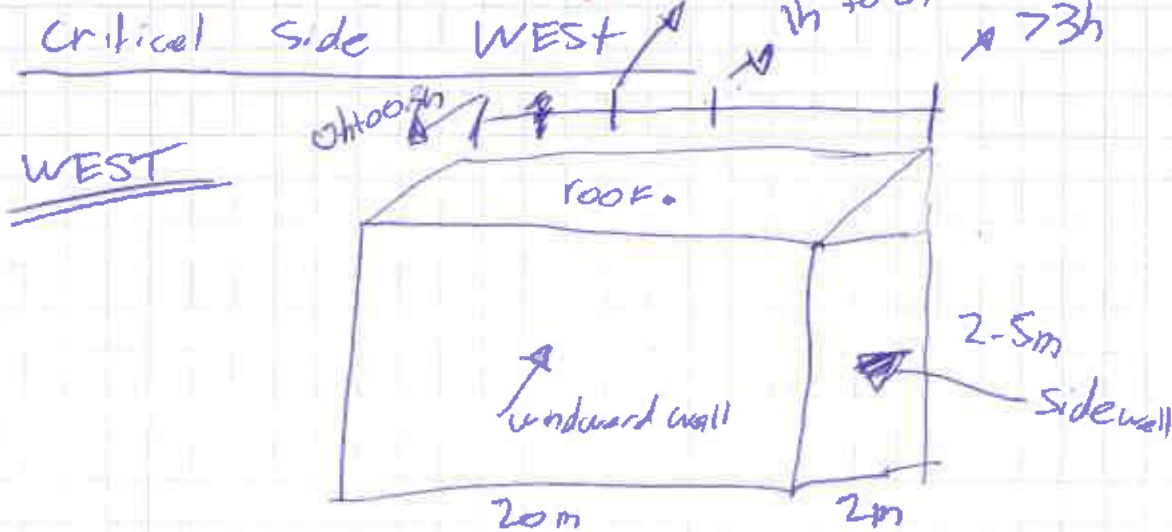
By: Dina

Date: 21/5/2017

Checked by: _____

Date: 1/1

Calculating wind load by



AS 170.2
Section
5.2

$$\frac{d}{b} < 1$$

windward

$$c_{pe} = 0.7$$

$$P = 480 \text{ pa}$$

leeward

$$c_{pe} = 0.3$$

$$P = -206 \text{ pa}$$

Sidewall

$$c_{pe} = 0.5$$

$$P = -350$$

Roof

$$c_{pe} = 0.9, 0.5, 0.3, 0.2$$

0h to 0.5h	= 0.9	= -620 pa
1h to 2h	= 0.5	= -344 pa
0.5h to 1h	= 0.3	= -206 pa
> 3h	= 0.2	= -137 pa

DPC Hand Calculations

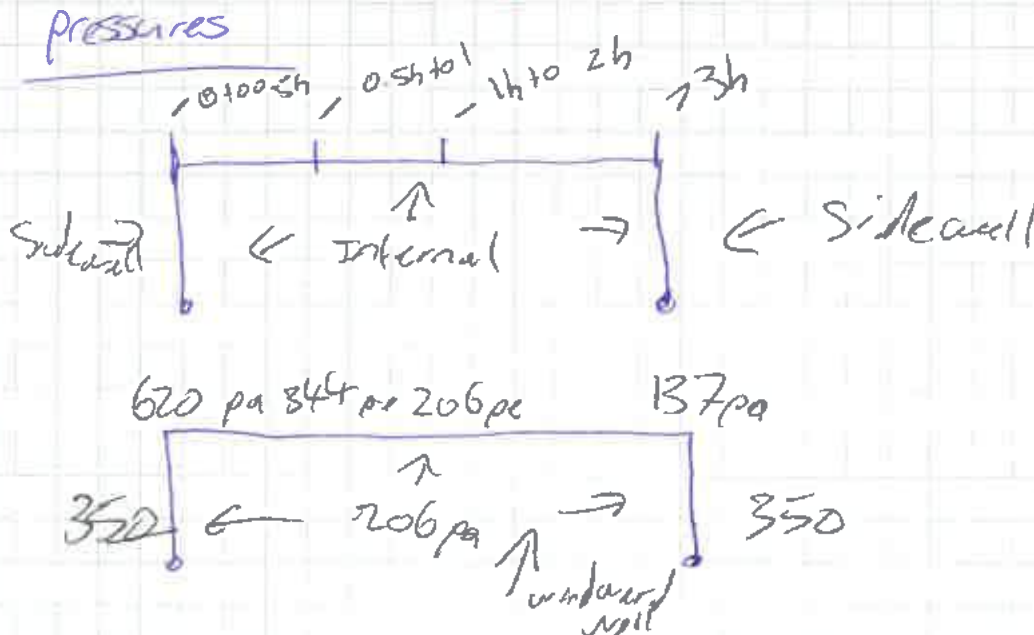
By: Dino

Date: 13/5/2012

Checked by: _____

Date: / /

Calculating wind load pg 5



A1170.2
Figure 52

max pressure from adding vertical pressures
is 826 pa

$$826 \text{ pa} \Rightarrow 0.82 \text{ kpa}$$

$$W_u = 0.82 \text{ kpa}$$

DPC Hand Calculations

By: Dino

Date: 26/5/2017

Checked by: _____

Date: 1/1

Calculating deadload. Pg 6

Deadload panel

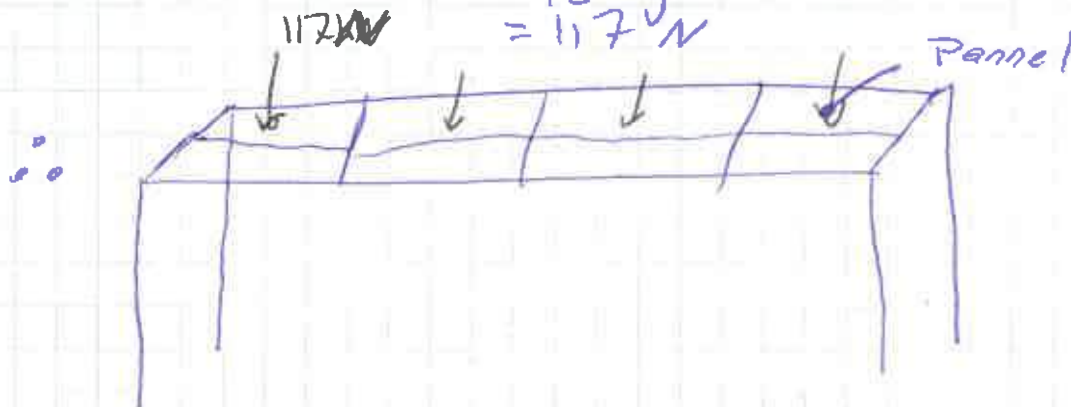


Density $1.2 \text{ g/cm}^3 = 1200 \text{ kg/m}^3$
Volume $= 0.01 \text{ m}^3$

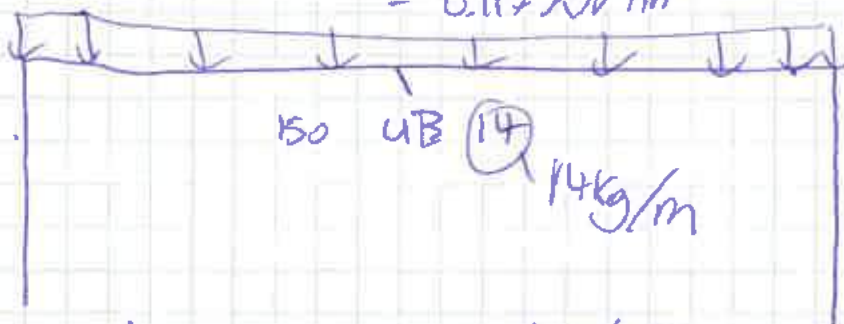
Sunlite
Specification
brochure
2017

10mm poly
carbonate
sheet

$$\begin{aligned} DL &= \text{Volume} \times \text{Density} \\ &= 1200 \times 0.01 \\ &= 12 \text{ kg} \\ &= 117 \text{ N} \end{aligned}$$



Convert to UDL $= 117 \text{ N/m}^2$
 $= 0.117 \text{ kN/m}^2$



$\therefore DL \text{ beam} = 0.14 \text{ kN/m}^2 = 0.14 \text{ kpa}$

DPC Hand Calculations

By: DinoDate: 26/5/2017

Checked by: _____

Date: / /

calculating DL & LL Pg 7

$$\text{Total DL} = 0.14 \text{ kpa} + 0.17 \text{ kpa} \\ = 0.31 \text{ kpa}$$

$$\text{LL} = 0.25 \text{ kpa}$$

$$\text{DL} = 1.2G + 1.5Q = 0.747 \text{ kpa}$$

$$\text{DL} = 0.9G + W_u = 1.099 \text{ kpa}$$

↑
CRITICAL

AS 1170.1

AS 1170.1
Section 4

cl 4.2.2

Summary

CRITICAL
Load
CASE

$$= 1.1 \text{ kpa (kN/m}^2\text{)}$$

or

$$1.1 \text{ kN/m}$$

DPC Hand Calculations

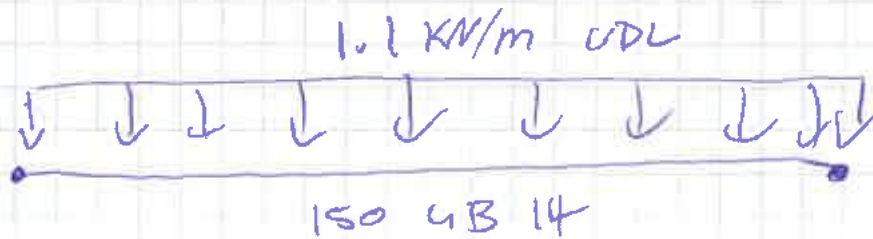
By: Dino

Date: 26/5/2017

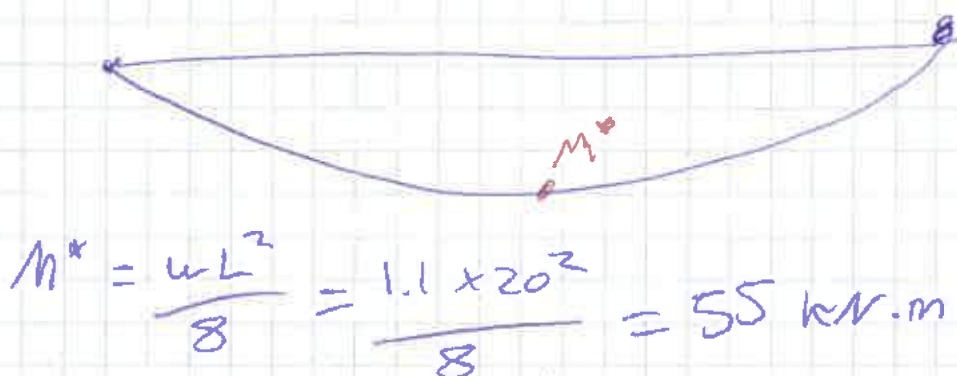
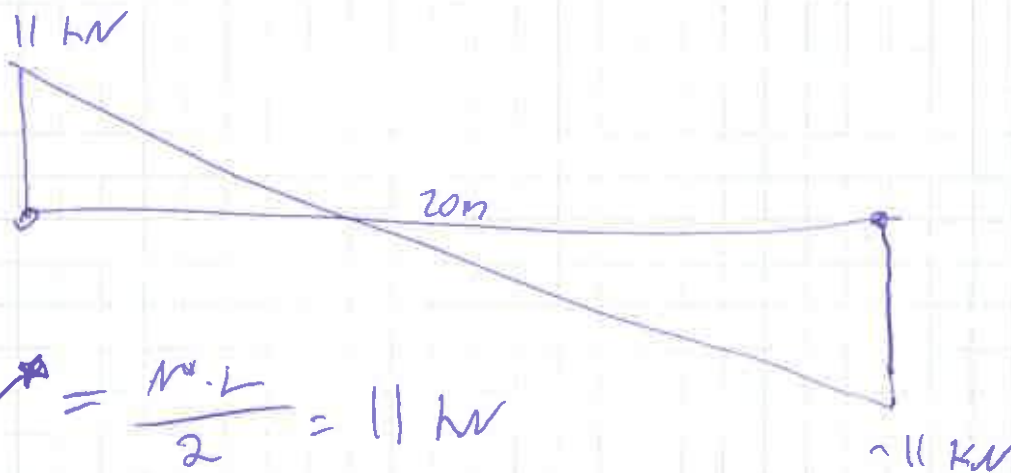
Checked by: _____

Date: / /

SE & BM Diagrams Pg 8



from
principles
I can't
do
steel &
timber



DPC Hand Calculations

By: Dino

Date: 26/5/2017

Checked by: _____

Date: / /

Check SF & BM Pg 9

For ISO UB 14

AS 4100
Section 5
cl 5.1

$$M^* \leq \phi M_{sx}$$

$$\begin{aligned} \phi M_{sx} &= f_y Z_e \phi \\ &= 320 \times 102 \times 10^3 \times 0.9 \\ &= 294 \text{ kNm} \end{aligned}$$

Z_e -
one steel
standard

$S_{SM} = 294 \text{ kNm} \checkmark$ ok for Bending

- Basically Smallest UB Scans
✓ way over designed
- however, we have an unlimited budget

$$V^* \leq \phi V_v$$

$$\begin{aligned} \phi V_v &= \phi V_a = \phi V_w \leq \phi = 0.6 \times f_y \times A_v \\ &\leq 441 \text{ kN} \checkmark \checkmark \text{ ok} \end{aligned}$$

AS 4100
cl 5.11.2

can stop
design
check here

$$\begin{aligned} \frac{\phi}{k_w} &\leq \frac{82}{\sqrt{J_y}} = 72 \\ 5.6 &\leq 72 \end{aligned}$$

\Rightarrow we use $V_v = V_w$

check for which V to use

will pass
combined
actions
by inspection

~~By~~ By Inspection ($V^* + M^*$ checks passes by so much)
ISO UB 14 is more than adequate for bench design.

USE ISO UB 14

DPC Hand Calculations

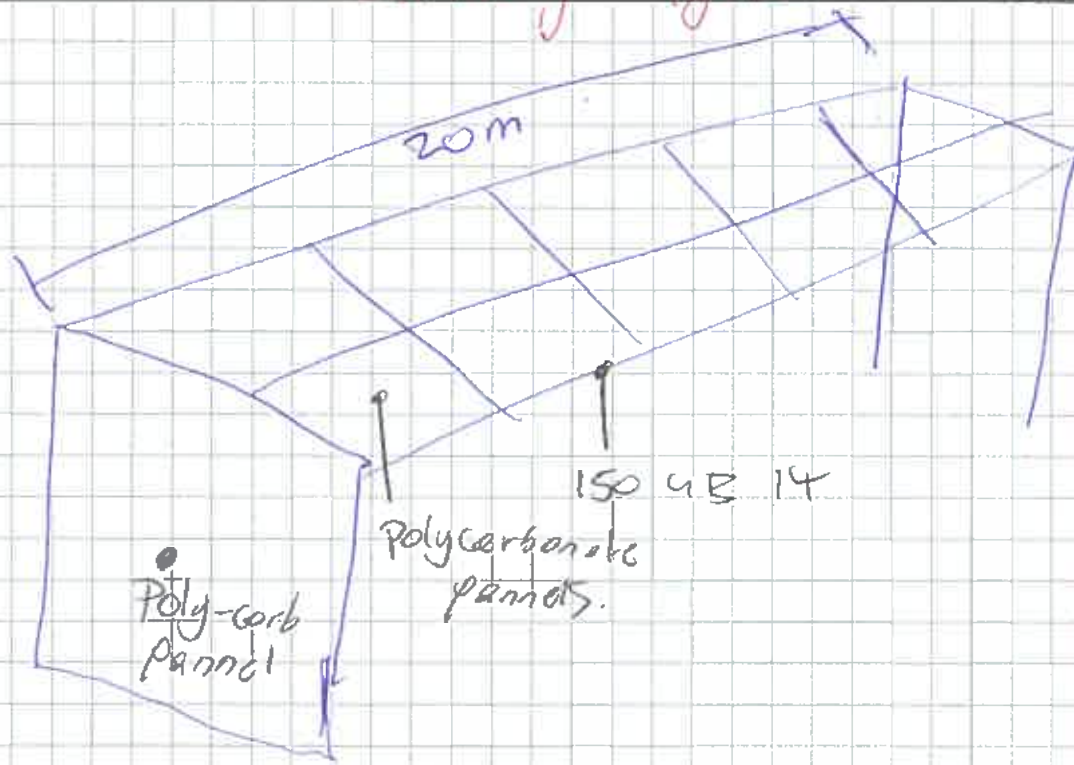
By: Dino

Date: 26/5/2017

Checked by: _____

Date: / /

Summary Pg 10



3 x
this in
a row.

Assumptions

- panels transfer all load to beams
- D.GW + W_s is critical
- The beam structure is symmetrical every 20m
- Panels are strong to withstand loads.

The beam structure is over designed if we use 150 UB 14 Beams. It passes bending & shear by a lot.

$$M^* \leq \phi M_{sx}$$

$$294 \text{ kNm} \leq 294 \text{ kNm}$$

$$V^* \leq \phi V$$

$$11 \leq 44$$

40 times
greater
than
what we
need