

A STUDY IN DESIGN AND COMPARISON OF REINFORCED, PRESTRESSED AND PARTIALLY PRESTRESSED PIER CAP OVER A RECTANGULAR HAMMERHEAD PIER

A Thesis Submitted in Fulfillment of the Requirement for the Award of the Degree of

MASTER OF ENGINEERING

in Civil Engineering/ Structures

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DECLARATION

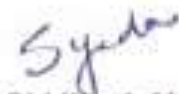
I, Arooshree, hereby declare that the work presented in this thesis entitled "A Study in Design and Comparison of Reinforced, Prestressed and Partially Prestressed Pier Cap over a Rectangular Hammerhead Pier" in fulfillment of the requirement for the award of degree of Master of Engineering (Structures) submitted at Department of Civil Engineering, Thapar Institute of Engineering & Technology (Deemed to be University), Patiala is an authentic record of work carried out under supervision of Dr. Prem Pal Bansal (Head of Department, Department of Civil Engineering, Thapar Institute of Engineering & Technology) and Mr. Siddharth Yadav (Senior Engineer, Bridges, AECOM Gurgaon) from July, 2017 to June, 2018. The matter presented in this has not been submitted either in part or full to any other university or institute for the award of any other degree.

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ABSTRACT

Nowadays, the designers in the field are required to design six lane structures like ROBs, RUBs and flyovers. Further, these structures need to be supported on a wider sub structure. In case of cross drainage structures, the provision of a wider sub structure can be feasible because of no traffic flow below it, but in case of grade separators like ROBs and flyovers, the space constriction and inability to find clear height, doesn't allow us to provide the required depth for the reinforced concrete Pier cap. To decrease the depth and to increase the moment and shear capacity, the scope of this thesis is to see the effect of prestressing or partial prestressing on those parameters.

An analytical method was used to develop spread sheets for design of Reinforced Concrete, Prestressed Concrete and Partially Prestressed Concrete Pier Caps as per the IS and IRC codes taking into account factors incorporating both serviceability and ultimate limit states.

Furthermore, a separate section of this thesis describes a comparative study of three different categories of prestressing i.e. no prestress, partial prestress and fully prestressed pier caps. The relationship of each of these to the serviceability parameters is considered. Finally, the cost comparison was done to calculate the most viable type of structure.

CONTENTS

DECLARATION	ii
ACKNOWLEDGEMENT	iii
ABSTRACT	iv
TABLE OF CONTENTS	v
LIST OF TABLES	ix
LIST OF FIGURES	xviii
LIST OF SYMBOLS	xix

	Page no.
CHAPTER 1 – INTRODUCTION	1 – 9
1.1 GENERAL.....	1
1.2 CLASSIFICATION OF PARTIALLY PRESTRESSED STRUCTURES.....	4
1.3 ADVANTAGES AND DISADVANTAGES OF PARTIAL PRESTRESSING.....	6
1.4 ADVANTAGES AND DISADVANTAGES OF PRESTRESSING.....	7
1.5 PURPOSE OF STUDY.....	8
1.6 OUTLINE OF THESIS.....	9
CHAPTER 2 – LITERATURE REVIEW	10-32
2.1 EARLY HISTORY.....	10
2.2 REVIEW OF PREVIOUS WORK.....	11
2.2.1 Hypothetical Tensile Stress.....	11
2.2.2 Partial Prestressing Ratio.....	12
2.2.3 Degree of Prestress.....	14
2.2.4 Review of Previous Research on Deflection.....	16
2.2.5 Moment vs Curvature.....	17
2.2.6 Present Design considerations for Partially Prestressed Concrete.....	18
2.2.7 Present Design Considerations for Reinforced Concrete.....	23
2.2.8 Present Design Considerations for Prestressed Concrete.....	29
2.2.9 Codes Used and Referred.....	32

CHAPTER 3 – METHODOLOGY

33-140

3.1. REINFORCED CONCRETE PIER CAP.....	33
3.2. DESIGN OF RCC PIER CAP SUPPORTING THREE GIRDER SYSTEM.....	34
3.2.1. Calculation of Forces.....	34
3.2.2. Design of Pier Cap.....	36
3.3. PRESTRESSED CONCRETE PIER CAP.....	40
3.4. DESIGN OF PSC PIER CAP SUPPORTING THREE GIRDER SYSTEM.....	41
3.4.1. Design of Pier Cap.....	41
3.5. PARTIALLY PRESTRESSED CONCRETE PIRE CAP.....	46
3.6. DESIGN OF PPSC PIER CAP SUPPORTING THREE GIRDER SYSTEM.....	47
3.6.1. Design of Pier Cap.....	47
3.7. DESIGN OF RCC PIER CAP SUPPORTING FOUR GIRDER SYSTEM.....	52
3.7.1. Calculation of Forces.....	52
3.7.2. Design of Pier Cap.....	55
3.8. DESIGN OF PSC PIER CAP SUPPORTING FOUR GIRDER SYSTEM.....	59
3.8.1. Calculation of Forces.....	59
3.8.2. Design of Pier Cap.....	59
3.9. DESIGN OF PPSC PIER CAP SUPPORTING FOUR GIRDER SYSTEM.....	65
3.9.1. Calculation of Forces.....	65
3.9.2. Design of Pier Cap.....	66
3.10. DESIGN OF RCC PIER CAP SUPPORTING FIVE GIRDER SYSTEM.....	73
3.10.1. Calculation of Forces.....	73
3.10.2. Design of Pier Cap.....	75
3.11. DESIGN OF PSC PIER CAP SUPPORTING FIVE GIRDER SYSTEM.....	79
3.11.1. Design of Pier Cap.....	79
3.12. DESIGN OF PPSC PIER CAP SUPPORTING FIVE GIRDER SYSTEM.....	84
3.12.1. Design of Pier Cap.....	84

3.13.	DESIGN OF RCC PIER CAP SUPPORTING SIX GIRDER SYSTEM.....	90
3.13.1.	Calculation of Forces.....	90
3.13.2.	Design of Pier Cap.....	92
3.14.	DESIGN OF PSC PIER CAP SUPPORTING SIX GIRDER SYSTEM.....	96
3.14.1.	Design of Pier Cap.....	96
3.15.	DESIGN OF PPSC PIER CAP SUPPORTING SIX GIRDER SYSTEM.....	101
3.15.1.	Design of Pier Cap.....	101
3.16.	DESIGN OF RCC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM.....	107
3.16.1.	Calculation of Forces.....	107
3.16.2.	Design of Pier Cap.....	109
3.17.	DESIGN OF PSC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM.....	113
3.17.1.	Design of Pier Cap.....	113
3.18.	DESIGN OF PPSC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM.....	118
3.18.1.	Design of Pier Cap.....	118
3.19.	DESIGN OF RCC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM.....	124
3.19.1.	Calculation of Forces.....	124
3.19.2.	Design of Pier Cap.....	126
3.20.	DESIGN OF PSC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM.....	130
3.20.1.	Design of Pier Cap.....	130
3.21.	DESIGN OF PPSC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM.....	134
3.21.1.	Design of Pier Cap.....	134

CHAPTER 4 – RESULTS

140-149

4.1	CRACK WIDTH	140
4.2	PARTIAL PRESTRESSING RATIO	141
4.3	DEPTH REQUIRED FOR MOST ECONOMICAL SECTION.....	142
4.4	SHEAR CAPACITY.....	143
4.4.1	At $L/2$	144
4.4.2	At d_{eff}	145

4.4.3	At face of Pier Shaft.....	146
4.5	COST COMPARISON.....	147
4.6	COST W.R.T. CRACK WIDTH.....	148
CHAPTER 5 – CONCLUSIONS		150
ANNEXURE 1.....		151
ANNEXURE 2.....		160
ANNEXURE 3.....		171
REFERNCES.....		174

LIST OF TABLES

Table 2.1	Hypothetical Flexural Tensile Stresses for Type 3 Members, IS 1343:2012.....	22
Table 2.2	Limiting Depth of Neutral axis for different Grades of Steel.....	26
Table 2.3	Non dimensional parameter K for different grades of steel.....	29
Table 3.1	Reactions on Bearings on Left Side for Three Girder System.....	34
Table 3.2	Reactions on Bearings on Right Side for Three Girder System.....	34
Table 3.3	Load factors for Design.....	35
Table 3.4	Summary of Bending Moments at Face of Shaft for Three Girder System.....	35
Table 3.5	Summary of Bending Moments at d eff from Face of Shaft for Three Girder System.....	36
Table 3.6	Summary of Bending Moments at half the Cantilever Length for Three Girder System.....	36
Table 3.7	Material Properties of RCC Pier Cap.....	36
Table 3.8	Summary of ULS Capacity Check for Three Girder System in RCC.....	37
Table 3.9	Summary of Stress Check for SLS Load Combination for Three Girder System in RCC.....	37
Table 3.10	Check for Crack Width for Three Girder System.....	38
Table 3.11	Check for Shear (ULS) for Three Girder System.....	38
Table 3.12	Shear Reinforcement provided at the Sections for Three Girder System.....	39
Table 3.13	Summary of Torsional Reinforcement at the sections for Three Girder System...	39
Table 3.14	Calculation of Cost of RCC Pier Cap for Three Girder System.....	40
Table 3.15	Material Properties of PSC Pier Cap.....	41
Table 3.16	Forces at Sections after considering Friction and Slip Losses for Three Girder System.....	42
Table 3.17	Stress Check with $0.9 \times$ Prestress for Three Girder System.....	42

Table 3.18	Stress Check with 1.1x Prestress for Three Girder System.....	43
Table 3.19	Summary of ULS Capacity Check for Three Girder System in PSC.....	44
Table 3.20	Check for Shear Requirement (ULS) for Three Girder System in PSC.....	44
Table 3.21	Shear Reinforcement at various Sections for Three Girder System in PSC.....	45
Table 3.22	Summary of Torsional Reinforcement at the sections for Three Girder System in PSC.....	45
Table 3.23	Calculation of Cost of PSC Pier Cap for Three Girder System.....	46
Table 3.24	Material Properties for PPSC Pier Cap.....	47
Table 3.25	Forces at Sections after considering Friction and Slip Losses for Three Girder System.....	48
Table 3.26	Calculation of Cost for a PPSC Pier Cap for Three Girder System.....	50
Table 3.27	Reactions on Bearings on Left Side for Four Girder System.....	52
Table 3.28	Reactions on Bearings on Right Side for Four Girder System.....	53
Table 3.29	Load factors for Design.....	53
Table 3.30	Summary of Bending Moments at Face of Shaft for Four Girder System.....	54
Table 3.31	Summary of Bending Moments at d_{eff} from Face of Shaft for Four Girder System.....	54
Table 3.32	Summary of Bending Moments at half the Cantilever Length for Four Girder System.....	54
Table 3.33	Material Properties of RCC Pier Cap.....	55
Table 3.34	Summary of ULS Capacity Check for Four Girder System in RCC.....	55
Table 3.35	Summary of Stress Check for SLS Load Combination for Four Girder System in RCC.....	56
Table 3.36	Check for Crack Width for Four Girder System.....	56
Table 3.37	Check for Shear (ULS) for Four Girder System.....	57
Table 3.38	Shear Reinforcement provided at the Sections for Four Girder System.....	57
Table 3.39	Summary of Torsional Reinforcement at the sections for Four Girder System...	58

Table 3.40	Calculation of Cost of RCC Pier Cap for Four Girder System.....	58
Table 3.41	Material Properties of PSC Pier Cap.....	60
Table 3.42	Forces at Sections after considering Friction and Slip Losses for Four Girder System.....	61
Table 3.43	Stress Check with 0.9 x Prestress for Four Girder System.....	62
Table 3.44	Stress Check with 1.1x Prestress for Four Girder System.....	62
Table 3.45	Summary of ULS Capacity Check for Four Girder System in PSC.....	63
Table 3.46	Check for Shear Requirement (ULS) for Four Girder System in PSC.....	63
Table 3.47	Shear Reinforcement at various Sections for Four Girder System in PSC.....	64
Table 3.48	Summary of Torsional Reinforcement at the sections for Four Girder System in PSC.....	64
Table 3.49	Calculation of Cost of PSC Pier Cap for Four Girder System.....	65
Table 3.50	Material Properties for PPSC Pier Cap.....	66
Table 3.51	Forces at Sections after considering Friction and Slip Losses for Four Girder System.....	67
Table 3.52	Summary of ULS check for Pier Cap for a Four Girder PPSC system.....	70
Table 3.53	Check for Shear Capacity (ULS) at various sections for a Four Girder PPSC system.....	70
Table 3.54	Shear Reinforcement at various Sections for a Four Girder PPSC system.....	71
Table 3.55	Summary of Torsional Reinforcement at the sections for a Four Girder PPSC system.....	71
Table 3.56	Calculation of Cost for a PPSC Pier Cap for Four Girder System.....	72
Table 3.57	Reactions on Bearings on Left Side for Five Girder System.....	73
Table 3.58	Reactions on Bearings on Right Side for Five Girder System.....	73
Table 3.59	Load factors for Design.....	74
Table 3.60	Summary of Bending Moments at Face of Shaft for Five Girder System.....	74

Table 3.61	Summary of Bending Moments at d eff from Face of Shaft for Five Girder System.....	75
Table 3.62	Summary of Bending Moments at half the Cantilever Length for Five Girder System.....	75
Table 3.63	Material Properties of RCC Pier Cap.....	75
Table 3.64	Summary of ULS Capacity Check for Five Girder System in RCC.....	76
Table 3.65	Summary of Stress Check for SLS Load Combination for Five Girder System in RCC.....	76
Table 3.66	Check for Crack Width for Five Girder System.....	77
Table 3.67	Check for Shear (ULS) for Five Girder System.....	77
Table 3.68	Shear Reinforcement provided at the Sections for Five Girder System.....	78
Table 3.69	Summary of Torsional Reinforcement at the sections for Five Girder System...	78
Table 3.70	Calculation of Cost of RCC Pier Cap for Five Girder System.....	79
Table 3.71	Material Properties of PSC Pier Cap.....	79
Table 3.72	Forces at Sections after considering Friction and Slip Losses for Five Girder System.....	80
Table 3.73	Stress Check with 0.9 x Prestress for Five Girder System.....	80
Table 3.74	Stress Check with 1.1x Prestress for Five Girder System.....	81
Table 3.75	Summary of ULS Capacity Check for Five Girder System in PSC.....	82
Table 3.76	Check for Shear Requirement (ULS) for Five Girder System in PSC.....	82
Table 3.77	Shear Reinforcement at various Sections for Five Girder System in PSC.....	83
Table 3.78	Summary of Torsional Reinforcement at the sections for Five Girder System in PSC.....	83
Table 3.79	Calculation of Cost of PSC Pier Cap for Five Girder System.....	84
Table 3.80	Material Properties for PPSC Pier Cap.....	84
Table 3.81	Forces at Sections after considering Friction and Slip Losses for Five Girder System.....	85

Table 3.82	Summary of ULS check for Pier Cap for a Five Girder PPSC system.....	87
Table 3.83	Check for Shear Capacity (ULS) at various sections for a Five Girder PPSC system.....	88
Table 3.84	Shear Reinforcement at various Sections for a Five Girder PPSC system.....	88
Table 3.85	Summary of Torsional Reinforcement at the sections for a Five Girder PPSC system.....	89
Table 3.86	Calculation of Cost for a PPSC Pier Cap for Five Girder System.....	89
Table 3.87	Reactions on Bearings on Left Side for Six Girder System.....	90
Table 3.88	Reactions on Bearings on Right Side for Six Girder System.....	90
Table 3.89	Load factors for Design.....	91
Table 3.90	Summary of Bending Moments at Face of Shaft for Six Girder System.....	91
Table 3.91	Summary of Bending Moments at d eff from Face of Shaft for Six Girder System.....	92
Table 3.92	Summary of Bending Moments at half the Cantilever Length for Six Girder System.....	92
Table 3.93	Material Properties of RCC Pier Cap.....	92
Table 3.94	Summary of ULS Capacity Check for Six Girder System in RCC.....	93
Table 3.95	Summary of Stress Check for SLS Load Combination for Six Girder System in RCC.....	93
Table 3.96	Check for Crack Width for Six Girder System.....	94
Table 3.97	Check for Shear (ULS) for Six Girder System.....	94
Table 3.98	Shear Reinforcement provided at the Sections for Six Girder System.....	95
Table 3.99	Summary of Torsional Reinforcement at the sections for Six Girder System.....	95
Table 3.100	Calculation of Cost of RCC Pier Cap for Six Girder System.....	96
Table 3.101	Material Properties of PSC Pier Cap.....	96
Table 3.102	Forces at Sections after considering Friction and Slip Losses for Six Girder System.....	97

Table 3.103	Stress Check with $0.9 \times$ Prestress for Six Girder System.....	97
Table 3.104	Stress Check with $1.1 \times$ Prestress for Six Girder System.....	98
Table 3.105	Summary of ULS Capacity Check for Six Girder System in PSC.....	99
Table 3.106	Check for Shear Requirement (ULS) for Six Girder System in PSC.....	99
Table 3.107	Shear Reinforcement at various Sections for Six Girder System in PSC.....	100
Table 3.108	Summary of Torsional Reinforcement at the sections for Six Girder System in PSC.....	100
Table 3.109	Calculation of Cost of PSC Pier Cap for Six Girder System.....	101
Table 3.110	Material Properties for PPSC Pier Cap.....	101
Table 3.111	Forces at Sections after considering Friction and Slip Losses for Six Girder System.....	102
Table 3.112	Summary of ULS check for Pier Cap for a Six Girder PPSC system.....	104
Table 3.113	Check for Shear Capacity (ULS) at various sections for a Six Girder PPSC system.....	105
Table 3.114	Shear Reinforcement at various Sections for a Six Girder PPSC system.....	105
Table 3.115	Summary of Torsional Reinforcement at the sections for a Six Girder PPSC system.....	106
Table 3.116	Calculation of Cost for a PPSC Pier Cap for Six Girder System.....	106
Table 3.117	Reactions on Bearings on Left Side for Seven Girder System.....	107
Table 3.118	Reactions on Bearings on Right Side for Seven Girder System.....	107
Table 3.119	Load factors for Design.....	108
Table 3.120	Summary of Bending Moments at Face of Shaft for Seven Girder System.....	108
Table 3.121	Summary of Bending Moments at d eff from Face of Shaft for Seven Girder System.....	109
Table 3.122	Summary of Bending Moments at half the Cantilever Length for Seven Girder System.....	109
Table 3.123	Material Properties of RCC Pier Cap.....	109

Table 3.124	Summary of ULS Capacity Check for Seven Girder System in RCC.....	110
Table 3.125	Summary of Stress Check for SLS Load Combination for Seven Girder System in RCC.....	110
Table 3.126	Check for Crack Width for Seven Girder System.....	111
Table 3.127	Check for Shear (ULS) for Seven Girder System.....	111
Table 3.128	Shear Reinforcement provided at the Sections for Seven Girder System.....	112
Table 3.129	Summary of Torsional Reinforcement at the sections for Seven Girder System.....	112
Table 3.130	Calculation of Cost of RCC Pier Cap for Seven Girder System.....	113
Table 3.131	Material Properties of PSC Pier Cap.....	113
Table 3.132	Forces at Sections after considering Friction and Slip Losses for Seven Girder System.....	114
Table 3.133	Stress Check with 0.9 x Prestress for Seven Girder System.....	114
Table 3.134	Stress Check with 1.1x Prestress for Seven Girder System.....	115
Table 3.135	Summary of ULS Capacity Check for Seven Girder System in PSC.....	116
Table 3.136	Check for Shear Requirement (ULS) for Seven Girder System in PSC.....	116
Table 3.137	Shear Reinforcement at various Sections for Seven Girder System in PSC.....	117
Table 3.138	Summary of Torsional Reinforcement at the sections for Seven Girder System in PSC.....	117
Table 3.139	Calculation of Cost of PSC Pier Cap for Seven Girder System.....	118
Table 3.140	Material Properties for PPSC Pier Cap.....	118
Table 3.141	Forces at Sections after considering Friction and Slip Losses for Seven Girder System.....	119
Table 3.142	Summary of ULS check for Pier Cap for a Seven Girder PPSC system.....	121
Table 3.143	Check for Shear Capacity (ULS) at various sections for a Seven Girder PPSC system.....	122
Table 3.144	Shear Reinforcement at various Sections for a Seven Girder PPSC system.....	122

Table 3.145	Summary of Torsional Reinforcement at the sections for a Seven Girder PPSC system.....	123
Table 3.146	Calculation of Cost for a PPSC Pier Cap for Seven Girder System.....	123
Table 3.147	Reactions on Bearings on Left Side for Eight Girder System.....	124
Table 3.148	Reactions on Bearings on Right Side for Eight Girder System.....	124
Table 3.149	Load factors for Design.....	125
Table 3.150	Summary of Bending Moments at Face of Shaft for Eight Girder System.....	125
Table 3.151	Summary of Bending Moments at d eff from Face of Shaft for Eight Girder System.....	126
Table 3.152	Summary of Bending Moments at half the Cantilever Length for Eight Girder System.....	126
Table 3.153	Material Properties of RCC Pier Cap.....	126
Table 3.154	Summary of ULS Capacity Check for Eight Girder System in RCC.....	127
Table 3.155	Summary of Stress Check for SLS Load Combination for Eight Girder System in RCC.....	127
Table 3.156	Check for Crack Width for Eight Girder System.....	128
Table 3.157	Check for Shear (ULS) for Eight Girder System.....	128
Table 3.158	Shear Reinforcement provided at the Sections for Eight Girder System.....	129
Table 3.159	Summary of Torsional Reinforcement at the sections for Eight Girder System.....	129
Table 3.160	Calculation of Cost of RCC Pier Cap for Eight Girder System.....	130
Table 3.161	Material Properties of PSC Pier Cap.....	130
Table 3.162	Forces at Sections after considering Friction and Slip Losses for Eight Girder System.....	131
Table 3.163	Stress Check with 0.9 x Prestress for Eight Girder System.....	131
Table 3.164	Stress Check with 1.1x Prestress for Eight Girder System.....	132
Table 3.155	Summary of ULS Capacity Check for Eight Girder System in PSC.....	133

Table 3.166	Check for Shear Requirement (ULS) for Eight Girder System in PSC.....	133
Table 3.167	Summary of Torsional Reinforcement at the sections for Eight Girder System in PSC.....	134
Table 3.168	Calculation of Cost of PSC Pier Cap for Eight Girder System.....	134
Table 3.169	Material Properties for PPSC Pier Cap.....	135
Table 3.170	Forces at Sections after considering Friction and Slip Losses for Eight Girder System.....	135
Table 3.171	Summary of ULS check for Pier Cap for a Eight Girder PPSC system.....	138
Table 3.172	Check for Shear Capacity (ULS) at various sections for a Eight Girder PPSC system.....	139
Table 3.173	Shear Reinforcement at various Sections for a Eight Girder PPSC system.....	139
Table 3.174	Summary of Torsional Reinforcement at the sections for a Eight Girder PPSC system.....	140
Table 3.175	Calculation of Cost for a PPSC Pier Cap for Eight Girder System.....	140
Table 4.1	Design Depth for most economical section for RCC, PSC and PPSC Pier Cap.	143
Table 4.2	Variation in Shear Capacity at $L/2$	145
Table 4.3	Variation in Shear Capacity at d_{eff}	146
Table 4.4	Variation in Shear Capacity at face of Pier Shaft	147
Table 4.5	Approximate Cost of RCC, PSC and PPSC Pier Caps	149
Table 4.6	Cost Comparison in PPSC Pier Cap for $w_k = 0.2\text{mm}$ and $w_k = 0.3\text{mm}$	147

LIST OF FIGURES

Figure 1.1	Formwork for a wide Pier Cap.....	2
Figure 1.2	Increased Height of Piers.....	2
Figure 1.3	Clear Height requirement in a Pier.....	3
Figure 1.4	Fully Prestressed Pier Cap.....	3
Figure 2.1	Moment Curvature Diagram, Karayannis et al, 2012.....	18
Figure 2.2	Depth Factors for Tensile Stresses for Type 3 Members, IS 1343: 2012.....	21
Figure 2.3	Behaviour of Rectangular Section at Ultimate Limit State in Flexure.....	23
Figure 2.4	Characteristic and design stress-strain curves for concrete in flexure.....	24
Figure 2.5	Characteristic and design stress-strain curves for Fe 250 grade mild steel.....	24
Figure 2.6	Characteristic and design stress-strain curves for Fe 415 grade.....	25
Figure 2.7	Concrete stress-block parameters in compression.....	27
Figure 2.8	Stress Distribution due to Eccentric Prestressing, Dead and Live loads.....	30
Figure 2.9	Stresses due to Prestress, Dead and Applied Loads.....	30
Figure 3.1	Plan and Elevation of a RCC Pier Cap for a Four Girder System.....	52
Figure 3.2	Plan and Elevation for PSC Pier Cap for Four Girder System.....	59
Figure 3.3	Cable Profile for a Pier Cap supporting a Four Girder System.....	61
Figure 3.4	Plan and Elevation for PPSC Pier Cap for Four Girder System.....	65
Figure 4.1	Crack width in RCC, PSC and PPSC Pier Cap for the most economical section.....	141
Figure 4.2	PPR for $w_k = 0.2\text{mm}$ and $w_k = 0.3\text{mm}$ in PPSC	142
Figure 4.3	Variation in maximum depth for RCC, PSC and PPSC Pier Cap	143
Figure 4.4	Variation in Shear Capacity at L/2 for RCC, PSC and PPSC Pier Cap	145
Figure 4.5	Variation in Shear Capacity at d_{eff} for RCC, PSC and PPSC Pier Cap	146
Figure 4.6	Variation in Shear Capacity at face for RCC, PSC and PPSC Pier Cap	147
Figure 4.7	Cost Comparison of RCC, PSC and PPSC w.r.t. number of girders.....	148
Figure 4.8	Percentage decrease in Cost w.r.t. RCC.....	149

LIST OF SYMBOLS AND ABBREVIATIONS

RCC	-	Reinforced Cement Concrete
PSC	-	Prestressed Concrete
PPSC	-	Partially Prestressed Concrete
PPR	-	Partial Prestressing Ratio
A_s	-	Area of untensioned steel
A_p	-	Area of tensioned steel
f_{ps}	-	Tensile Stress in the prestressing tendons
f_y	-	Yield Stress of ordinary tensile steel
M_{np}	-	Moment Capacity due to prestressing steel
M_n	-	Total moment capacity due to both prestressing steel and ordinary tensile steel
ω	-	Reinforcement Index
ω_p	-	Reinforcement Index in Prestressing Steel
d_s	-	distances from centroid of ordinary tensile steel to the extreme compression fibre
d_p	-	distances from centroid of prestressing tensile steel to the extreme compression fibre
f'_c	-	cylinder compressive strength of concrete
f'_y	-	yield stress of ordinary compressive steel
M_D	-	moment due to dead load
M_L	-	moment due to live load
M_{dec}	-	moment to which when the effective prestress is added produces zero concrete stress at the extreme fibre of the section
λ	-	degree of Prestress
ULS	-	Ultimate Limit State
SLS	-	Serviceability Limit State
w_k	-	crack width
ϵ_{cu}	-	maximum compressive strain
γ_c	-	partial safety factor in concrete
f_{ck}	-	characteristic strength of concrete

E_s	-	elastic modulus of steel
E_c	-	elastic modulus of concrete
NA	-	neutral axis
x_u	-	depth of neutral axis
$x_{u,max}$	-	maximum depth of neutral axis
d	-	effective depth of section
T_u	-	resultant force in tension
C_u	-	resultant force in compression
z	-	lever arm
A_{st}	-	area of untensioned steel
e	-	eccentricity of the prestressing force
P	-	prestressing force
Z_t	-	section modulus of the top fibre
Z_b	-	section modulus of the bottom fibre
A	-	cross section of the concrete member
r	-	radius of gyration
α_e	-	modular ratio
f_c	-	prestress in concrete at the level of steel
ϵ_{cs}	-	residual shrinkage strain
70RW	-	70R wheeled vehicle
SIDL	-	super imposed dead load
QP	-	Quasi Permanent
Class A3	-	three lanes of class A vehicle
σ_s	-	stress in steel
f_{ctm}	-	mean value of axial tensile strength of concrete
ρ_r	-	effective reinforcement ratio
$A_{c, eff}$	-	effective tension area that is generally the area of concrete surrounding the tension reinforcement
β_1	-	coefficient that takes account of bond properties
β_2	-	coefficient that takes account of duration of loading or repeated loading
k_1	-	coefficient that takes account of bond properties of bars

k_2	-	coefficient that takes account of the form of the strain distribution
$s_{r,max}$	-	maximum crack spacing
$f_{ct, eff}$	-	f_{ctm} (mean value of axial tensile strength of concrete)
w_{max}	-	maximum crack width
d_c	-	distance between edge and centre of the lowest bar
b	-	overall width of the member
h	-	overall height of the member
c	-	clear cover to the longitudinal reinforcement
φ	-	diameter of the longitudinal reinforcement
$\rho_{r, min}$	-	minimum value of effective reinforcement ratio
I_{cr}	-	moment of inertia of cracked section
Q	-	applied load

Chapter 1

INTRODUCTION

1.1. GENERAL

The development of reinforced concrete was based on the concept of combining two materials, with concrete which is strong in compression resisting the compressive stresses and steel resisting the tensile stresses. Prestressed concrete was later introduced by Fressyinet as a new material which put concrete into precompression by tensioning steel, so that it could resist both tensile and compressive stresses. He thus formulated the design criterion of permitting no tensile stress in concrete. Emperger and Abeles advocated a third method of combining concrete and steel in which limited tensile stresses or even hair cracks would be permitted in concrete, which Abeles termed as *Partially Prestressed Concrete* to distinguish it from Fully Prestressed Concrete.

Partially Prestressed Concrete, therefore, occupies the complete spectrum between the two extremes. The conventional reinforced concrete in which none of the reinforcement is prestressed represents one extreme while the other extreme, commonly known as fully prestressed concrete, consists of reinforcement all of which is prestressed.

Decades have gone by since the development of prestressed concrete by Fressyinet. More and more partially prestressed concrete structures have been built. Today, partial prestressing is accepted worldwide, albeit implicitly, and a number of prominent researchers have already drawn attention to its potential.

The increase in construction of flyovers and expressways over existing pathways require a construction of four to six lane structures supported on slender piers so that there is no space constriction on the traffic flow on the original way.

Due to the increasing loads that have to be transferred, there is an increase in shear stresses. To counter which, while giving a required clear height (or keeping the depth of the pier cap restricted), the pier cap is required to be prestressed.

Smaller pier caps require lesser obstruction of traffic while construction on the original pathway.

Furthermore, construction of metro lines requires a number of piers over a large distance. To facilitate a speedy construction of these, precast pier caps can be placed.



Figure1.1 Formwork for a wide Pier Cap, *google*



Figure 1.2 Increased Height of Piers, *google*

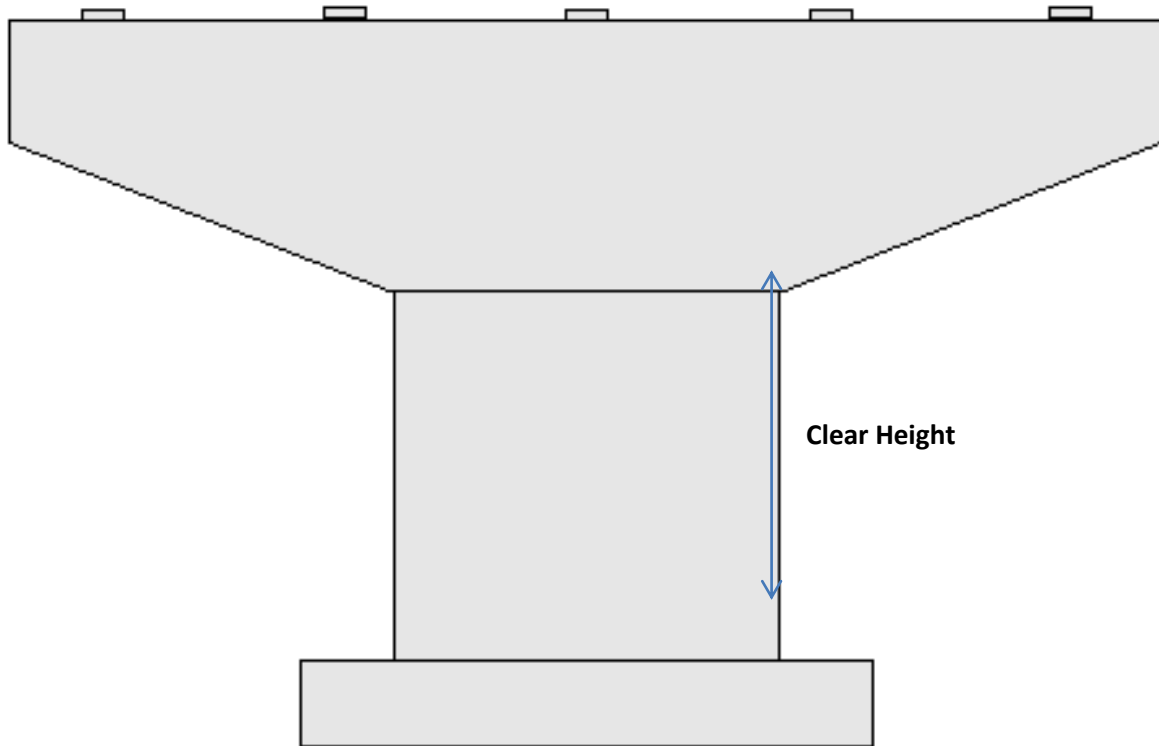


Figure 1.3 Clear Height requirement in a Pier



Figure1.4 Fully Prestressed Pier Cap, clicked by self

1.2. CLASSIFICATION OF PARTIALLY PRESTRESSED STRUCTURES

A number of various systems may be adopted to ascertain a classification of PPSC structures. Something that is collective to all is that the design of all of these is done considering the occurrence of certain cracks under service/working loads. Some variations come up within the technicalities, particularly in lieu of the standards, arrangement and distribution of the cables/reinforcement.

1.2.1. Mixed Reinforcement

In this type, at every section of the structure, the total reinforcement contains part of prestressed and an ordinary reinforced part. The prestressing is done in such a way that a part of tensile stresses under the working loads is removed by prestressing wires, strands or cables (e. g. tensile stresses owing to dead load and some of live load). The part of reinforcement that is not prestressed can be made of any grade of steel or can be even of prestressing quality. It is provided so that it can regulate the formation of cracks and control the deformations because of operating loads and to make sure that a desired safety margin against failure is provided.

1.2.2. Prestressing Reinforcement with Partial Prestress

In this all prestressing is done to all of the reinforcement that is provided. The area of reinforcement is calculated to make sure that a desired safety margin against failure is achieved. The only difference is that due to service loads, controlled cracks and some tensile stresses can occur in the member. Such a type is generally appropriate for manufactured pre tensioned members.

1.2.3. Composite Beams

These are a composite of prefabricated prestressed elements and attached to it are some concrete elements. There is no continuous distribution of stresses over a particular cross section. The parts of the composite beams that are prestressed might act as fully or partly prestressed components under the service loads. The concrete elements attached are principally not prestressed. A low bond stress acts at the interface in between the prestressed part and additional concrete part. A favourable distribution over a particular cross section can be expected, if the contact surfaces are roughened the increase the bond between the two.

1.2.4. Combined Systems

At times prestressed elements and concrete parts with normal reinforcement make up statically indeterminate parts like girders of a superstructure. A PPSC or even a full PSC system can be achieved by making use of post-tensioned cables. Examples of this combined system are girders with prestress in longitudinal direction, 2-D structures with prestress in only one direction, such as slabs etc.

There is an alternative classification that had been suggested by Joint Committee FIP-CEB. It used tensile stresses as a criterion and the analogous cracks in the:

Class I: There can be no cracking permissible under any or all conditions and stages of construction. Also, no tensile stresses can occur in concrete under any loading condition during the execution and in the service life of the structure. Therefore, absolutely no cracks openings should be there at least theoretically. Complete prestress should be applied to structures or members under the action of dynamic loads resulting in fatigue and/or to structures subjected to an extreme environment.

Class II: No tensile stresses should be produced by dead load and part of live load. Even because of the condition that extreme loads are acting, tensile strains in concrete are restricted such that cracks openings do not show. For instance, to the members of bridges or the structures which are not exposed to the full impact of the live loads, the partial prestressing can be applied to.

Class III: In this, cracks are allowed but the amount of crack width and the stress experienced by the reinforcement is restricted. Neglecting the tensile stresses experienced in concrete, the stresses under service loads are calculated for a cracked section.

This particular system is currently applicable in the field of execution. Class II and class I are currently in use as limited and full prestress respectively however Class III corresponds to the PPSC.

Another classification is given as per IS 1343:2012, Code of Practice for Prestressed Concrete based on the Limit State of Serviceability for Cracking.

Type 1: There will be no development of tensile stresses.

Type 2: There is an allowable value of tensile stresses to be developed but no noticeable cracking should be there.

Type 3: There is an allowable crack width in this type, however it must not affect the stability and appearance and/or durability of the member; the allowable limits of which would depend on the type of structure and the type of environment and will vary between wide limits and the calculation of exact maximum thickness of crack width is not possible.

It should also be noted that the flexural tensile and compressive stress at any cross section of the structure/member, both at transfer and under the most unfavourable combination of design loads, shall satisfy the criteria for the corresponding type of structure.

In most of the codes of practice currently in use, concrete structures are divided into numerous classes or types on the basis of either the nominal concrete tensile stresses at service load or crack width. In Current *British Code CP110 (6)*, the classification comprises of the following categories:

Class I: Structure in which no tensile stress is permitted in concrete under service load.

Class II: Structure in which limited tensile stresses, less than the concrete tensile strength (modulus of rupture) are permitted in concrete under service loads.

Class III: Structure in which cracks of limited widths are permitted.

Class IV: Conventional reinforced concrete which is without any prestressed reinforcement in which cracks upto a permissible width are permitted under service load.

Class II and Class III members, predominantly the latter, are denoted to as Partially Prestressed members.

1.3. ADVANTAGES AND DISADVANTAGES OF PARTIAL PRESTRESSING

The availability of a wider range of stresses at the critical points in partially prestressed concrete members, can lead to less prestressed steel than required for fully prestressed concrete and a

smaller total amount of steel than required for reinforced concrete. The primary factor in favour of partial prestressing is therefore, economy.

It is known that at the ultimate limit state there is basically no difference between ordinary reinforced, partially prestressed or fully prestressed concrete. The distinctive behaviour therefore occurs under the service load.

Other advantages of partial prestressing are as follows:

1.3.1. By prestressing a part of the reinforcement, the downward deflection, is normally smaller than ordinary reinforced concrete and the initial upward camber is usually lower than fully prestressed concrete. The ability to control deflections or upward cambers, by varying the level of prestress is of considerable value in cases where the camber of long-span prestressed elements might otherwise be a problem.

1.3.2. Non-prestressed reinforcement serves a dual purpose of contributing to the ultimate strength and controlling crack formation and crack width.

1.3.3. Cracks of a width which would otherwise be unpleasing and subject to corrosion risk can be controlled by prestressing a part of reinforcement.

1.3.4. The ductility of members at ultimate load is increased thereby avoiding sudden failure. This feature is of particular value in design for seismic conditions.

1.4. ADVANTAGES AND DISADVANTAGES OF PRESTRESSING

1.4.1. Advantages

- a) The members are lighter and slender as compared to their RC counterparts as high strength concrete is used.

- b) In fully prestressed members, the whole of the section is effective as there are no tensile stresses under working loads.
- c) Dead loads can be counter-balanced by eccentric prestressing in prestressed members.
- d) The durability of the structure/member is increased due to use of high strength concrete and absence of cracks.
- e) The construction of long span structures is conceivable because of substantial decrease in self-weight and are thus economical.
- f) Prefabricated members are possible.
- g) Lastly, prestressed concrete structure deflects visibly and appreciably before ultimate failure, therefore giving sufficient warning before collapse.

1.4.2. Disadvantages

- a) The availability of experienced designers and builders is difficult.
- b) The cost of initial equipment is very high.
- c) The availability of experienced consultants and engineers is scanty.
- d) Prestressed sections are brittle in nature.

1.5. PURPOSE OF STUDY

The present study was carried out to add to the existing knowledge of prestressed concrete and partially prestressed pier caps.

This research was carried out in two phases. The first was to study the design of all three categories of prestressing i.e. no prestress, partial prestress and fully prestressed pier caps. And the second phase was comparative studies of the three thereof, with crack width, shear capacity and cost as parameters.

In phase one, manual calculation was carried out by making excel spreadsheets. Further, various codes and specifications are examined to calculate the values of various serviceability

parameters. In phase two, a comparative study was carried out. The performance of all three was studied in relation to various parameters and finally a cost assessment was done.

Finally conclusions were drawn as to which type is most economic corresponding to a particular width of carriageway.

1.6. OUTLINE OF THESIS

The thesis is divided into five chapters. The general introduction and the review of previous research and current design recommendations consist of chapters one and two.

Chapter three is concerned with the design of reinforced concrete, prestressed concrete and partially prestressed concrete pier caps with posttensioned tendons.

Chapter four include the comparative study of the three categories with respect to some parameters.

The conclusion of this investigation described in chapter five and recommendations are made.

Chapter 2

LITERATURE REVIEW

2.1. EARLY HISTORY

The idea of prestressing concrete was not successfully applied until the introduction of high tensile strength wire in the mid-1920s; because of extensive losses due to creep. Freyssinet who was pioneering this new technique regarded prestressing as a means of eliminating undesirable cracks and allowing beam to function as homogeneous materials.

In the years that followed, Abeles(1) further developed and endorsed this new concept of prestressing. The term Partial Prestressing was first introduced by him, to distinguish this new concept of permitting tensile stress or even hair cracks under service loads, from that of Prestressing where no tensile stress is permitted. The idea of permitting tensile stresses in concrete under service load means that less prestressing force would be required and either all the reinforcement should be tensioned to a lesser degree or only a part of reinforcement to be tensioned. Abeles also advocated the use of high tensile strength wire as non prestressed steel

whilst maintaining the same ultimate resistance. He claimed that a saving could thus be achieved in the amount of steel required, even allowing for the high unit cost of high tensile wire. In ordinary reinforced concrete, this same high strength steel could be used to achieve the same ultimate strength of the beam but only in the expense of excessive deflection and crackwidth. And by applying a prestressing force, that excessive deflection and crack width could be reduced to an acceptable level.

This method of partial prestressing as suggested by Abeles(1) in 1942 was essentially applied around 1948. The entire reinforcement that is provided consisted of prestressing wires strands or cables and is also tensioned. Furthermore, the prestressing force doesn't provide a complete prestress and prestressing is done in such a way that there are a certain allowable tensile stresses under working loads and a certain permissible crack width also under working/service loads.

The first global dialogue on PPSC took place initially at the Congress of the "*Federation Internationale de la Precontrainte*" (FIP) in 1962. Consequently, a Joint Committee FIP-CEB (*Federation Internationale de la Precontrainte/Comite Europeen du Beton*) was formed in order to research the area between RCC and PSC. Since that first discussion theoretical and experimental studies have taken place in various countries. Specifically, a study "Enquete sur la Precontrainte Partielle" was carried out in 1965 in Belgium by R.Baus and V.Depaux. It characterised a worldwide survey of the then current status with respect to PPSC and contained relevant references.

Until the present-day, the codal provisions for most countries allow only full prestress or partial prestress while pertaining to the assumption that no crack can develop in the structure. Therefore, the calculation of stress under service loads is done considering an uncracked and homogenous section. On the other hand, some European countries allow PPSC under principally static loads, with tolerance for a permissible value of crack openings (crack width).

The inherent potential of PPSC and a global attention stimulated the International Association for Bridge and Structural Engineering (IABSE, *estd.* 1929) to be chosen as a subject matter for their IABSE congress in 1968. The main aim of this was to assemble the results and conclusions of the research already in progress and thereby promoting the application of PPSC.

2.2. REVIEW OF PREVIOUS WORK

2.2.1. Hypothetical Tensile Stress

This approach was initially proposed by Abeles (1). The hypothetical tensile stress is the stress developed if the concrete section were to remain uncracked. The resultant tensile stress is the difference between the bending stress at the tensile face, calculated by assuming an uncracked section, and the prestress after accounting for all the losses.

Further, the allowable hypothetical tensile stress is specified with reference to the following:

- a. The permissible crack width depending on the environment to which the member is likely to be subjected,
- b. The concrete strength,
- c. The distribution of the tendons,
- d. The percentage of non prestressed reinforcement and
- e. The depth of the section

In a more recent publication by Bennett (4), the design approach is demonstrated for partially prestressed concrete by using hypothetical tensile stress as an index for the level of prestressing. The design steps are summarised as follows:

- a. Select a suitable value of hypothetical tensile stress corresponding to the approximately to the degree of cracking permitted and then correspondingly minimum section modulus.
- b. Determine the concrete cross section and calculate its dimensional properties.
- c. Calculate the prestress required at the extreme top and bottom faces and the magnitude and location of the prestressing force. Also, calculate the number of tendons required.
- d. Calculate the amount of non prestressed reinforcement required to satisfy the ultimate limit state condition.

- e. Check the serviceability limit state conditions, if required.

2.2.2. Partial Prestressing Ratio (PPR)

For estimate the amount of prestressing that has been done in a structure with respect to full prestressing, many ratios have been used, mainly partial prestressing ratio, degree of prestress and the partial prestressing index. Naaman (3) recommended the use of Partial Prestressing Ratio to characterise the amount of partial prestress, because it allows a uniform behaviour of the ultimate flexural limit state for RCC, PSC and PPSC. The PPR for a beam with rectangular cross section is defined as:

$$\text{PPR} = \frac{M_{np}}{M_n} = \frac{A_p f_{ps} (d_p - 0.5a)}{A_p f_{ps} (d_p - 0.5a) + A_s f_y (d_s - 0.5a)} \quad (2.1)$$

where,

a corresponds to the depth of equivalent stress block ;

A_s and A_p are the areas of ordinary tensile steel and prestressing steel, respectively;

d_s and d_p are distances from centroids of ordinary tensile steel and prestressing steel, respectively to the extreme compression fibre;

f_{ps} is the tensile stress in prestressing tendons;

f_y is the yield stress of ordinary tensile steel;

M_{np} is the moment capacity due to prestressing steel; and

M_n is the total moment capacity due to both prestressing steel and ordinary tensile steel.

The reinforcement index ω for a rectangular section is usually used to estimate the total amount of flexural reinforcement is given as:

$$\omega = \omega_p + \frac{d_s}{d_p} (\omega_s - \omega_s') = \frac{A_s f_{ps}}{b d_p f'_c} + \frac{d_s}{d_p} \left(\frac{A_s f_y}{b d_s f'_c} - \frac{A'_s f'_y}{b d_s f'_c} \right) \quad (2.2)$$

where,

A'_s is the area of ordinary compressive steel;

b is the width of the beam;

f'_c is the cylinder compressive strength of concrete; and

f_y is the yield stress of ordinary compressive steel.

The value of PPR is zero for an ordinary reinforced beam ($A_p = 0$) and at the other extreme when the member is fully prestressed, it is unity ($A_s = 0$). According to the definition, PPR is always one for fully prestressed members (or for members without non prestressed reinforcement) independently of whether or not cracking is assumed under service load. Thus, a fully prestressed member in which cracking is allowed also has a PPR value of one.

Besides being useful as an index of level of prestress, the PPR also serves as an extra equation in the analysis of a cracked section. In order to satisfy the ultimate moment criterion, there are generally two unknown quantities in the dimensioning of ordinary reinforced concrete and fully prestressed concrete i.e. A_s and A_p . As two equations of equilibrium (force and moment) are available in the ultimate load, the two unknowns can be easily determined. However, three unknowns exist in a partially prestressed section i.e. A_s , A_p and the depth of the neutral axis. Hence, a third equation is essential and the PPR can be easily selected to provide the required equation.

The design procedure based on ultimate strength conditions suggested by Naaman (3) can be summarised in the following steps:

- a. Select the concrete section and the material properties.
- b. Choose a suitable Partial Prestressing Ratio.
- c. Calculate the required ultimate moment of resistance M_u and determine the effective reinforcement ratio. For a rectangular section where d is the level of total reinforcement, the effective reinforcement ratio is given as:

$$\frac{A_p f_{pu} + A_s f_y}{b d f_{cu}} = \frac{M_u}{0.87 b d f_{cu} \left(d - \frac{x}{2} \right)} \quad (2.3)$$

d. Another equation is obtained from the equilibrium of forces at ultimate load.

$$0.36 f_{cu} b x = 0.87 (A_p f_{pu} + A_s f_y) \quad (2.4)$$

- e. Areas of tendons and the non-prestressed steel can be calculated by solving the equations in steps b, c and d.
- f. Checking the section at serviceability limit state. The stresses in both steel and concrete are checked. The deflection and crack width are also calculated. The procedure is repeated with a different value of PPR, if the serviceability requirements are not met.

3.

2.2.3. Degree of Prestress

The degree of prestress λ is defined by Bachmann(5) as:

$$\lambda = \frac{M_{dec}}{M_D + M_L} \quad (2.5)$$

where,

M_{dec} corresponds to that moment to which when the effective prestress is added produces zero concrete stress at the extreme fibre of the section.

M_D is the moment due to dead load; and

M_L is the moment due to live load.

It is well known that the main advantage in prestressing a member lies in its superior structural behaviour under service load. At the ultimate state, however, there is no considerable difference in structural behaviour between a PSC, RCC and a PPSC structure. Hence, Bachmann(5) suggested an explanation which was based on the service load behaviour of the member. The degree of prestress as stated above gives a

degree of that proportion of the total service moment under which the maximum tensile fibre stress is zero.

Like PPR, the value of Degree of Prestress is zero for ordinary reinforced concrete (RCC) while its value is always one if the section is fully prestressed (PSC) without exception. However, a fully prestressed section designed to develop tensile stress under service load, will be classified as fully prestressed according to the definition of PPR but not according to the Degree of Prestress. On the other hand, for a prestressed concrete member reinforced with both types of steel and designed to be under permanent compression, Degree of Prestress will be one while PPR will be less than one.

Bachmann(5) suggested that the design method for PPSC should allow a even transition between RCC and PSC design, and should be a combined procedure, applicable to all classes of concrete structures. A general outline of the proposed design structure based on serviceability state is as follows:

- a. Choose a minimum degree of prestressing based on design requirements. Keeping the minimum cost and area of steel as the main factor. The decompression moment for which the section is to be prestressed is then calculated.
- b. With a preselected section, calculate the required prestress force which together with the decompression moment (as explained above), at the extreme bottom fibre produces zero. Then the necessary area of prestressed reinforcement can be found.
- c. Then the area of extra ordinary steel is calculated to satisfy the ultimate conditions.
- d. The stresses in concrete under the working loads are calculated and the values are checked against permissible loads.

The main advantage of using Degree of Prestress over Partial Prestressing Ratio as a definition for partially prestressed concrete is its reference to the serviceability state

which helps the designer to foresee whether the amount of prestress will, for example, be sufficient to cover a certain proportion of the total service load. To calculate a minimum total A_{st} for benchmark, Bachmann(5) suggested a value of 0.6 for a rectangular section.

Considering the relative cost of the prestressed to non prestressed steel to be approximately 3:1, Leonhardt (6) found that for the most economical design, the degree of prestress ω must be within the range of 0.45 to 0.6 subjected to the kind of section.

2.2.4. Review of Previous Research on Deflection

One of the main advantages of prestress is its ability to control deflection, but the lack of understanding of its behaviour under the service load has limited its application. While the elastic theory for prediction of deflection of ordinary reinforced and fully prestressed members has been well established, the analytical method for calculation of deflection in partially prestressed members is still developing.

Amin Ghali(14) stated that the deflection is calculated by the superimposing a number of loadings in which the loads are applied when the concrete has different ages. Instance, at stage t_1 , only the prestress force and a partial dead load (balanced by the initial prestress) are introduced, like the self-weight. The SIDL is applied at a later stage. Further, to calculate the deflection at an even later stage, the effects of shrinkage and creep of concrete and relaxation of steel, amongst other losses need to be accounted for.

The above losses lead to a total loss in prestress in the prestressed wire or cable and a gradual development of stress (generally compressive) in the non-prestressed steel. In other words, because the resultant stresses in a section are constant, the change in the forces in any of the two kinds on reinforcement (ordinary or prestressed) results in an equal and opposite change in the forces on concrete.

Bennett and Dave (7) tested forty rectangular beams in which the amount of reinforcement and the level of prestress were varied. The tests represented a full range between fully prestressed and conventionally reinforced concrete.

They observed a typical behaviour of partially prestressed concrete beams. After cracking, the load-deflection curves for all the beams with the same amount of reinforcement had about the same increased slope although higher cracking loads were naturally observed in beams with higher prestress.

Branson and Sheikh (13), carried out more tests on partially prestressed concrete beams. Twelve concrete beams with varied level of prestress and reinforcement were tested, to study the effect of both quality and quantity of the non-tensioned steel on the behaviour of beams.

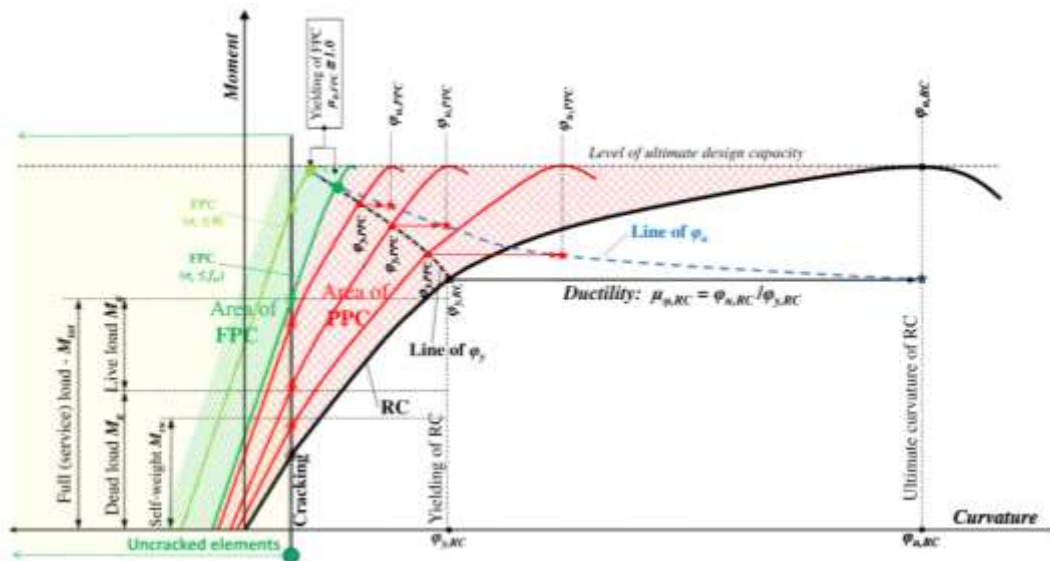
They observed that the deflections of the beams with the non prestressed steel was considerably less than the deflections under the corresponding loads of identical nature as compared to beams with small amounts of prestressed steel.

Abeles (1) conducted various tests to calculate deflection but the only concern was that he advocated the use of high tensile steel of the same quality as prestressed tendons for the ordinary reinforcement as well.

2.2.5. Moment vs Curvature

Karayannis (2) gave the typical behaviour curves (moment versus curvature) of reinforced concrete (RC), fully prestressed concrete (PSC) and partially prestressed concrete (PPSC) flexural members with equal ultimate capacity level are shown. Under the total load, there is no crack shown by the fully prestressed concrete elements (green) and it is likely to show brittle failure. On the other hand, a limited amount of cracking is allowed to be appear in partially prestressed concrete elements under dead load (red) and it is expected to exhibit a less brittle failure depending on the partial prestressing

ratio that is used. Lastly, the typical RCC section that is likely to crack under service loads (solid black line) show excellent post yielding or ductile behaviour.



4.

Figure 2.1 Moment Curvature Diagram, *Karayannis et al, 2012*

2.2.6. Present Design considerations for Partially Prestressed Concrete

- a. *Safety* suggest that the probability of the partial or full collapse of a structure is low and this is not only for the normal loads but also stands true for the abnormal loads like earthquake or wind loads. Structural integrity is another part of safety. The main aim here is to nullify the chance of a collapse.
- b. *Serviceability* suggests acceptable conduct of a structure under service loads without any effect to the durability or appearance or distress to the user due to say excess cracks, deflections or vibrations. Some other concerns which come with respect to serviceability are impermeability, acoustic insulation, thermal insulation and durability.

It is not necessary that a design satisfying safety conditions will also satisfy the serviceability conditions. As an instance, a very thin but reinforced concrete slab can be safe with respect to collapse by providing enough reinforcement (Safety), but it may

have extreme deflections, or cracks or permeability thus making the exposed steel susceptible to corrosion. Hence, it does not satisfy the serviceability conditions.

2.2.6.1. *Limit State Method (LSM)*

The philosophy of Limit State Method of Design (LSM) represents a definite advancement over the traditional design philosophies. The LSM philosophy uses a multiple safety factor format which attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads, by considering all possible 'limit states'. The selection of the various multiple safety factors is supposed to have a sound probabilistic basis, involving the separate consideration of different kinds of failure, types of materials and types of loads.

- a. **Ultimate limit state** (or 'limit states of collapse'), it deals with fracture, sliding, overturning, fatigue, buckling, strength and therefore safety.
- b. **Serviceability limit state**, which deal with discomfort to use and/or failure, initiated by excessive cracks, deflection, leaking, vibration and therefore loss of durability.

2.2.6.2. IS 1343: 2012

The IS 1343:2012 (30), Code of Practice for Prestressed Concrete has in its latest revised code (2012) incorporated the provision of intermediate degrees of prestress (partial prestress). As a result, the Code covers three types of structures, the types being classified with respect to the permissible tensile stress in concrete.

IS 1343: 2012 mentions three types of structures that is:

- a. For Type 1: There will be no tensile stress
- b. For Type 2: There will be tensile stresses less than the allowable limit but no visible cracking.
- c. Type 3: There is an allowable crack width in this type, however it must not affect the stability and appearance and/or durability of the member; the allowable limits of which would depend on the type of structure and the type of environment and will vary between wide limits and the calculation of exact maximum thickness of crack width is not possible.

LIMIT FOR CRACK WIDTH

As a guide, the reasonable limits are given as, 0.1mm for members which are exposed to especially aggressive environments such as the severe category and 0.2mm for all other members.

LIMIT FOR DEFLECTION

- a. For Type 1 and Type 2 Structures, within the first 24 hours of removal of superimposed dead load, the structure should recover at least 85 percent of the deflection and more than 90 percent after 72 hours of removal.
- b. For Type 3 Structures, within the first 24 hours, of the superimposed dead load, the structure should recover at least 75 percent of the deflection and more than 80 percent after 72 hours of removal or else the structure will be deemed as unacceptable.

LIMIT FOR TENSILE STRESSES

- 1. For Type 1 members: There should be tensile stress.
- 2. For Type 2 members: The tensile stress shall not exceed 3N/mm^2 (permissible). However, when a part of service load is temporary, this stress can be exceeded by 1.5N/mm^2 .

3. For Type 3 members: For members in which cracking is permitted, it may be assumed that the section is uncracked and hypothetical tensile stresses exist at the maximum size of cracks. The hypothetical tensile stresses for use in these calculations for members for wither pre tensioned or post tensioned are given in Table 2.1 modified by the coefficients given in Fig 2.2.

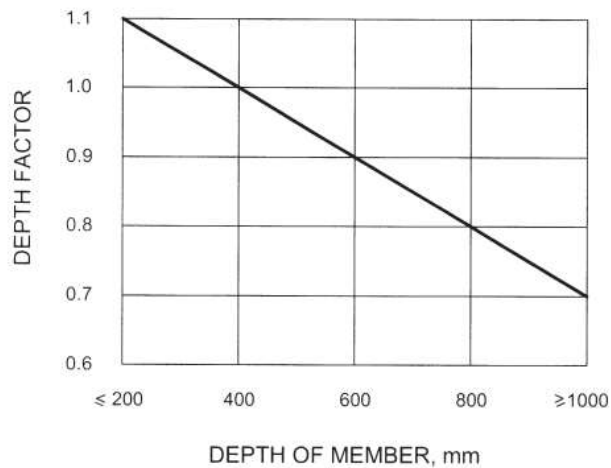


Figure 2.2 Depth Factors for Tensile Stresses for Type 3 Members, *IS 1343: 2012*

Table 2.1 Hypothetical Flexural Tensile Stresses for Type 3 Members, *IS 1343:2012*

Stress of Concrete for Grade	M 50 & Above						
	M 45	4.4	5.4	4.4	5.4	5.8	6.8
	M 40	4.1	5.0	4.1	5.0	5.3	6.3
	M 35	,	,	3.6	4.4	,	,

S.No	Type of Tendons	Limiting Crack Width	
		(mm)	M 30
(i)	Pre-tensioned Tendons	0.1	'
		0.2	'
(ii)	Grouted Post-tensioned Tendons	0.1	3.2
		0.2	3.8
(iii)	Pre-tensioned tendons distributed in the tensile zone and positioned close to the tension face of concrete	0.1	'
		0.2	'

2.2.7. Present Design Considerations for Reinforced Concrete

2.2.7.1. Assumptions

At the ultimate limit state of flexure, the basic assumptions which are considered in the analysis, Cl. 38.1, is 456:2000, (31) are as follows:

- Plane sections normal to the beam axis remain plane after bending, i.e., in an initially straight beam, strain varies linearly over the depth of the section.
- The maximum compressive strain in concrete (at the outermost fibre) ϵ_{cu} shall be taken as 0.0035. This is so, because regardless of whether the beam is under-reinforced or over-reinforced, collapse always occurs by the crushing of concrete.

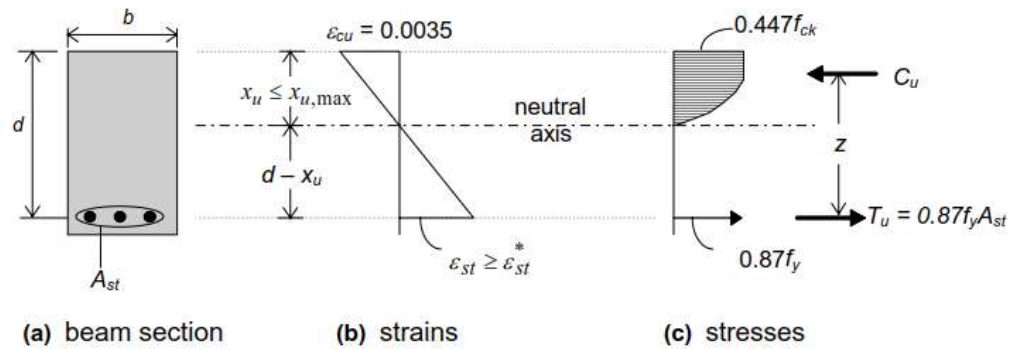


Figure 2.3 Behaviour of Rectangular Section at Ultimate Limit State in Flexure, IS 456:2000

- c. The design stress–strain curve of concrete in flexural compression is as shown. The partial safety factor $\gamma_c = 1.5$ is to be considered.

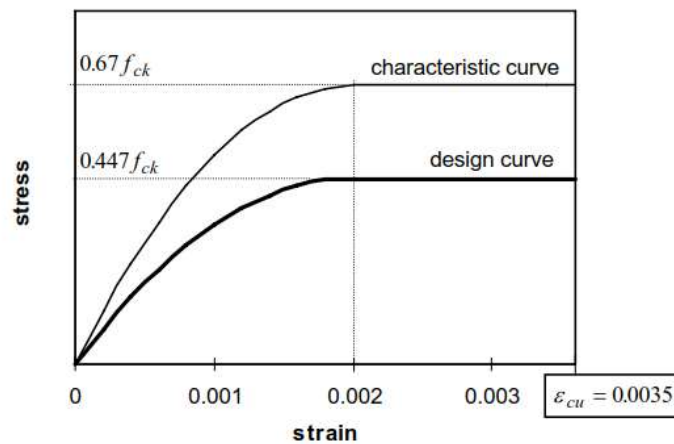


Figure 2.4 Characteristic and design stress-strain curves for concrete in flexure

- d. The tensile strength of the concrete is ignored.
- e. The design stress–strain curves for mild steel and cold-worked bars are as shown below. The partial safety factor $\gamma_s = 1.15$ is to be considered.

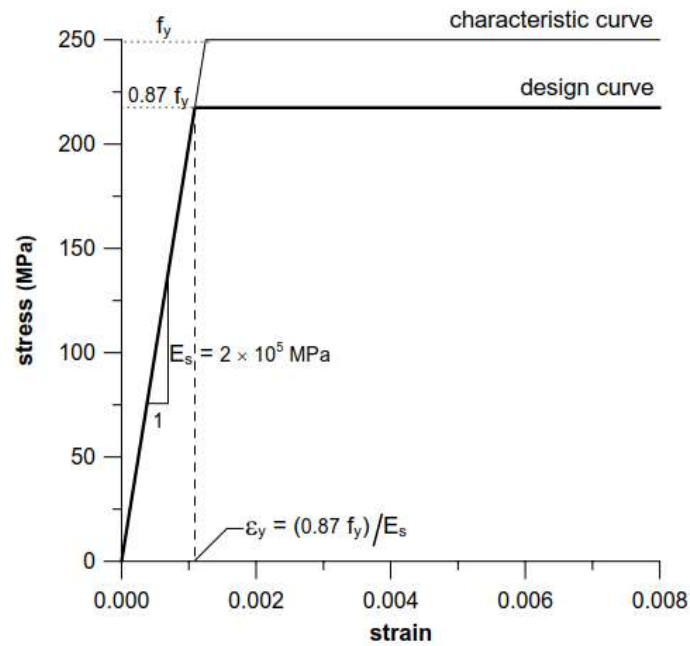


Figure 2.5 Characteristic and design stress-strain curves for Fe 250 grade mild steel, IS 456:2000

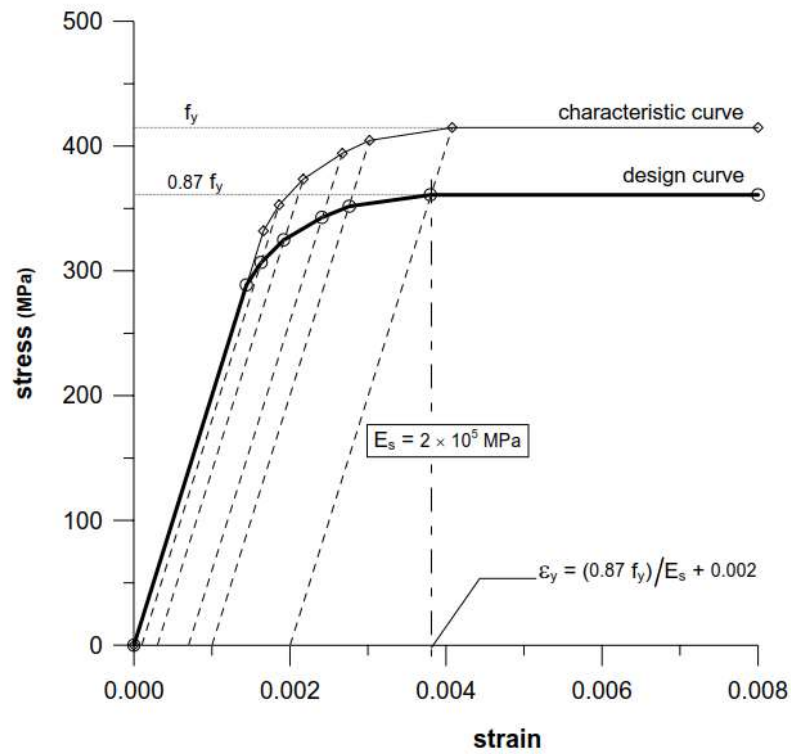


Figure 2.6 Characteristic and design stress-strain curves for Fe 415 grade, IS 456:2000

- f. The strain ϵ_y in the tension reinforcement (at its centroid) at the ultimate limit state shall not be less than ϵ_{st} defined as:

$$\epsilon_y = 0.002 + \left(\frac{0.87 f_y}{E_s} \right) \quad (2.6)$$

2.2.7.2. Limiting Depth of Neutral Axis

An equation for the depth of the NA at the ultimate limit state can be found from the strain diagram based on the assumptions given in the code. Considering similar triangles,

$$\frac{x_u}{d} = \frac{0.0035}{0.0035 + \epsilon_{st}} \quad (2.7)$$

According to the last assumption as given above, there is a limiting (maximum) value of the neutral axis depth $x_{u,max}$. This is calculated as

$$\frac{x_{u,max}}{d} = \frac{0.0035}{0.0055 + 0.87 f_y / E_s} \quad (2.8)$$

The values of $x_{u,max}/d$ for different grades of steel, obtained by making use of the above equation are as follows:

Table 2.2 Limiting Depth of Neutral axis for different Grades of Steel, *calculated*

Steel Grade	Fe 250	Fe 415	Fe 500
$x_{u,max}/d$	0.5313	0.4791	0.4560

The limiting depth of neutral axis $x_{u,max}$ corresponds to the balanced section, i.e. a section that is expected to outcome in a balanced failure at the ultimate limit state in flexure. If the neutral axis depth x_u is less than $x_{u,max}$, then the section is *under-reinforced* (resulting in a tension failure); whereas if x_u exceeds $x_{u,max}$, it is *over-reinforced* (resulting in a compression failure).

2.2.7.3. Analysis of Singly Reinforced Rectangular Sections

- a. Analysis of a given RCC section at the ultimate limit state of flexure implies the calculation of the *ultimate moment of resistance* M_{uR} of the section.

$$M_{uR} = C_u \cdot z = T_u \cdot z \quad (2.9)$$

Where T_u and C_u are the resultant forces in tension and compression respectively, and z is the lever arm.

For $x_u < x_{u,max}$

$$T_u = 0.87 f_y A_{st} \quad (2.10)$$

- b. CONCRETE STRESS BLOCK IN COMPRESSION

Analysis of the concrete stress block in compression is done to determine the magnitude of C_u and its line of action. As ultimate failure of a reinforced concrete beam in flexure occurs by the crushing of concrete, for both under-reinforced and over-reinforced beams, therefore, in both cases, the shape of the compressive stress distribution (stress block) at failure will be as below:

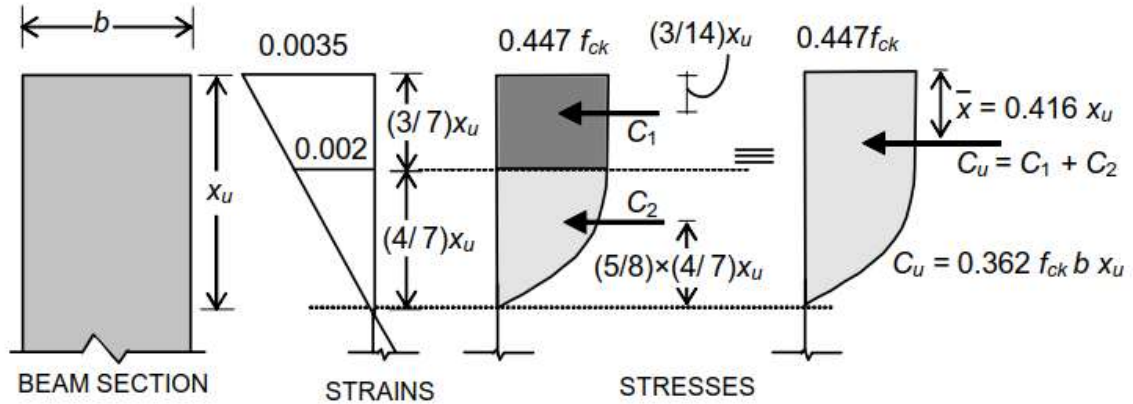


Figure 2.7 Concrete stress-block parameters in compression

The value of C_u can be calculated because we know that the compressive stress in concrete is uniform with the value of $0.447 f_{ck}$ till a depth of $3x_u/7$, and then it varies parabolically over a depth of $4x_u/7$ to zero at the depth of NA.

For a rectangular section with width b ,

$$C_u = 0.447 f_{ck} b \left(\frac{3 x_u}{7} \right) + \frac{2}{3} \times \left(\frac{4 x_u}{7} \right) \quad (2.11)$$

$$C_u = 0.362 f_{ck} b x_u \quad (2.12)$$

c. Depth of Neutral Axis

In a particular section, the depth of NA should be in such a way that $T_u = C_u$, substantiating the equilibrium of forces. Equating $T_u = C_u$ we get,

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} \quad (2.13)$$

It should be noted that the resultant x_u obtained should be less than $x_{u,max}$.

For the other condition i.e $x_u > x_{u,max}$, suggesting that at the ULS condition, the steel could not have yielded yet, and the steel stress cannot be taken as $0.87f_y$. Hence the above equation is not valid. The correct position of the NA is obtained by a hit and trial method when the steel has not yielded; called *Strain-Compatibility method*.

d. Ultimate Moment of Resistance

The ultimate resistance M_{uR} of a beam cross section can be obtained by using the above equations. For a singly reinforced rectangular section, the lever arm z is given by

$$z = d - 0.416x_u \quad (2.14)$$

Accordingly, in terms of compressive strength,

$$M_{uR} = 0.362f_{ck}bx_u(d - 0.416x_u) \quad (2.15)$$

And in terms of steel tensile stress, for $x_u < x_{u,max}$

$$M_{uR} = 0.87f_yA_{st}(d - 0.416x_u) \quad (2.16)$$

e. Limiting Moment of Resistance

The limiting moment of resistance for a given section, (according to the Cl. G-1.1, IS 456:2000) gives the following expression:

$$M_{u,lim} = 0.362 f_{ck} b x_{u,max} (d - 0.416 x_{u,max})$$

(2.17)

$$\frac{M_{u,lim}}{f_{ck} b d^2} = 0.362 \frac{x_{u,max}}{d} \left(1 - 0.416 \frac{x_{u,max}}{d}\right) \equiv K$$

(2.18)

Table 2.3 Non dimensional parameter K for different grades of steel

Steel Grade	Fe 250	Fe 415	Fe 500
K	0.1498	0.1389	0.1338

2.2.8. Present Design Considerations for Prestressed Concrete

2.2.8.3. Analysis of Prestress

a. Resultant Stresses at a Section

The resultant stresses at the top and bottom fibres of concrete at any section is given as

$$f_{sup} = \left(\frac{P}{A} - \frac{Pe}{Z_t}\right) + \left(\frac{M_g}{Z_t}\right) + \left(\frac{M_q}{Z_t}\right)$$

(2.19)

$$f_{inf} = \left(\frac{P}{A} + \frac{Pe}{Z_b}\right) - \left(\frac{M_g}{Z_b}\right) - \left(\frac{M_q}{Z_b}\right)$$

(2.20)

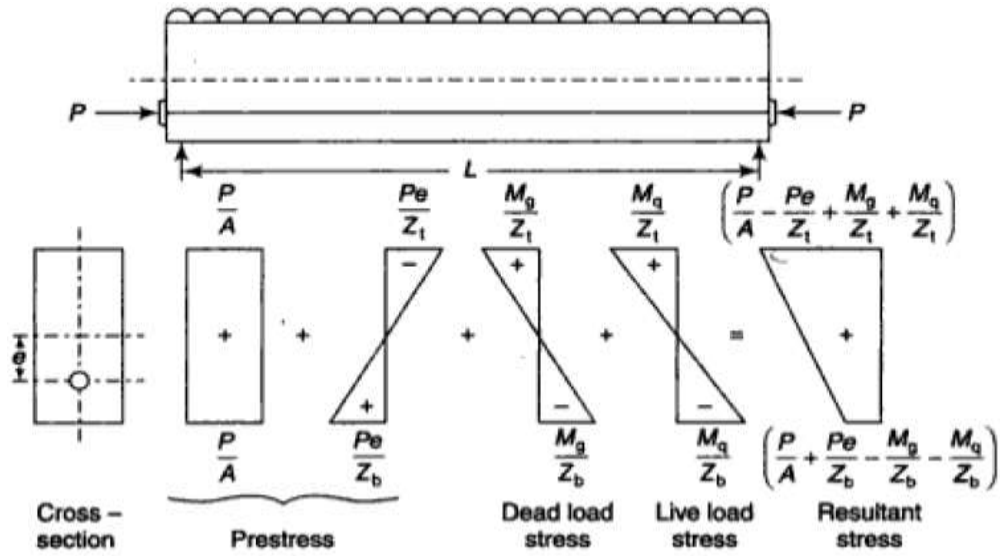


Figure 2.8 Stress Distribution due to Eccentric Prestressing, Dead and Live loads (22)

b. Minimum Section Modulus

Certain governing inequalities help calculate the minimum section moduli for the expected moments. These inequalities are as follows:

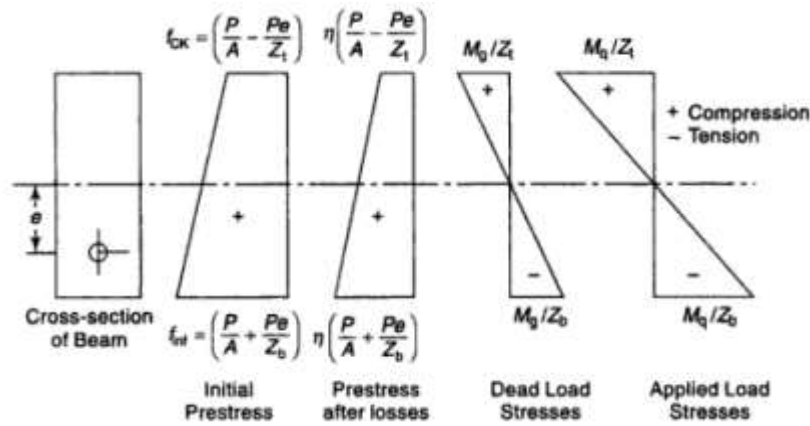


Figure 2.9 Stresses due to Prestress, Dead and Applied Loads (22)

At Transfer,

Top Fibres

$$\left(f_{\text{sup}} + \frac{M_g}{Z_t}\right) \geq f_{\text{tt}}$$

(2.21)

Bottom Fibres

$$\left(f_{\text{inf}} + \frac{M_g}{Z_b}\right) \geq f_{\text{ct}} \quad (2.22)$$

At Service Loads,

Top Fibres

$$\left(\eta f_{\text{sup}} + \frac{M_g}{Z_t} + \frac{M_q}{Z_t}\right) \leq f_{\text{cw}} \quad (2.23)$$

Bottom Fibres

$$\left(\eta f_{\text{inf}} - \frac{M_g}{Z_b} - \frac{M_q}{Z_b}\right) \geq f_{\text{tw}} \quad (2.24)$$

From equations (2.21) and (2.23) we get,

$$\left(\frac{M_q + (1 - \eta)M_g}{Z_t}\right) \leq (f_{\text{cw}} - \eta f_{\text{tt}}) \leq f_{\text{tr}} \quad (2.25)$$

From equations (2.22) and (2.24) we get,

$$\left(\frac{M_q + (1 - \eta)M_g}{Z_b}\right) \leq (\eta f_{\text{ct}} - f_{\text{tw}}) \leq f_{\text{br}} \quad (2.26)$$

Here, f_{tr} and f_{br} are ranges of stresses at top and bottom fibres respectively.

The formulae for the required section moduli are expressed as:

$$Z_t \geq \left[\frac{M_q + (1 - \eta) M_g}{f_{tr}} \right] \quad (2.27)$$

$$Z_b \geq \left[\frac{M_q + (1 - \eta) M_g}{f_{br}} \right] \quad (2.28)$$

c. Prestressing Force

Usually the section provided is greater than the minimum indicated by the section modulus above. The minimum prestressing force will be calculated by selecting the maximum tensile prestress at the top fibre and the minimum compressive prestress at the bottom fibre.

$$P = \frac{A(f_{inf}Z_b + f_{sup}Z_t)}{(Z_t + Z_b)} \quad (2.29)$$

And the corresponding maximum eccentricity is given by:

$$e = \frac{Z_t Z_b (f_{inf} - f_{sup})}{A(f_{sup}Z_t - f_{inf}Z_b)} \quad (2.30)$$

2.2.9. Codes Used and Referred

- a. IRC 112: 2012
- b. IS 1343:2012
- c. IRC 6: 2016
- d. ACI 318: 2008

Chapter 3

METHODOLOGY AND MODELLING

3.22. REINFORCED CONCRETE PIER CAP

5. Pier caps for girder arrangement from 3 girders to 8 girders (deck slab spanning from 9.5m to 24.5m) were designed for the basis of this research.

6.

7. The design of reinforced concrete pier cap is done in the traditional way using the limit state method.

8.

9. a/d ratio is calculated to decide whether the pier cap has to be designed as a cantilever or if it has to be designed as a corbel. If $a/d < 1$, it is designed as a corbel. Otherwise it is designed as a cantilever.

10.

11. The moment at three locations is calculated that is at the face of the pier shaft, at d effective from the face of the pier shaft and at half the cantilever portion.

12.

13. The span of the flyover is 34.5m and the depth of the girder is kept fixed at 2.3m to calculate the dead load of the girder and the thickness of deck slab is kept as 0.220m.

14.

15. The reactions from maximum moment and maximum reaction case are considered to calculate the worse possible moments for the design of pier cap.

16.

17. The check for moment, stresses, shear force, crack width is done using suitable modification factors for ULS and SLS.

18.

19. The total loads considered are self-weight of the pier cap, dead load of the superstructure, superimposed dead load i.e. surfacing and crash barrier, the live load which includes the impact factor (70RW, 70RW + Class A and 3 lanes of Class A).

20.

3.2. DESIGN OF RCC PIER CAP SUPPORTING THREE GIRDER SYSTEM

3.2.1. Calculation of Forces

21.

3.2.1.1. The depth of pier cap is assumed. In this case the depth assumed was 1.30m.

22.

3.2.1.2. The distribution of live loads on all three girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 1 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3
70RW	27.263	14.877	2.492
70RW+A	28.796	24.140	19.483
Class A3	20.146	27.788	35.429
Maximum Moment Case			
70RW	53.234	29.050	4.866
70RW+A	52.749	42.913	33.077
Class A3	30.152	41.589	53.026

Table 3. 2 Reactions on Bearings on Right Side

Maximum Reaction Case	R1	R2	R3
70RW	28.567	16.181	3.796
70RW+A	27.553	23.097	18.642
Class A3	13.106	20.748	28.389
Maximum Moment Case			
70RW	0.000	0.000	0.000
70RW+A	0.000	0.000	0.000
Class A3	0.000	0.000	0.000

3.2.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed (practically).

3.2.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 3 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.2.1.5. The final forces are calculated at the face of the pier shaft, at the distance deff from the face of the pier shaft and at a distance of cantilever/2 from the face of the pier shaft.

3.2.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 4 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	872.208	896.0
SLS RARE	596.842	610.810
SLS QP	510.520	516.895

Table 3. 5 Summary of Bending Moments at d eff from Face of Shaft

At deff from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	443.417	351.951
SLS RARE	302.011	239.719
SLS QP	255.053	202.458

Table 3. 6 Summary of Bending Moments at half the Cantilever Length

At L/2		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	440.255	134.558
SLS RARE	299.668	88.131
SLS QP	252.710	66.153

3.2.2. Design of Pier Cap

3.2.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 7 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
F _{yd}	0.6f _y
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.2.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 8 Summary of ULS Capacity Check

Section	M _{ED} (t-m)	A _{st} required (mm ²)	A _{st} provided (mm ²)	Diameter (mm)	Number
At face of Shaft	896.0	21502.69	22580	25	46
At deff from face	351.951	13919.81	22580	25	46
At L/2	134.558	7994.65	22580	25	46

3.2.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 9 Summary of Stress Check for SLS Load Combination

Section	Combina- -tion	Compressi ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss- ible Tensile stress (MPa)
At face of Shaft	SLS Rare	11.425	16.800	250.586	300
	SLS QP	7.089	12.600	213.283	300
At deff from face	SLS Rare	7.886	16.800	144.757	300
	SLS QP	4.645	12.600	115.779	300
At L/2	SLS Rare	3.876	16.800	67.414	300
	SLS QP	1.772	12.600	41.716	300

3.2.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which as are follows:

Table 3. 10 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.205	0.300
At deff from face	0.099	0.300
At L/2	0.036	0.300

3.2.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 11 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	172.38	872.208	Shear rein. Required
At deff from face	146.98	443.417	Shear rein. Required
At L/2	140.29	440.255	Shear rein. Required

3.2.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 12 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm^2)	Ast provided (mm^2)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	872.208	27919.21	28148.67	14	16	100
At deff from face	443.417	20141.23	28148.67	14	16	100
At L/2	440.255	21957.00	28148.67	14	16	100

3.2.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 13 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	139.413	683.397	2261.947	12	2	100
At deff from face	139.413	683.397	2261.947	12	2	100
At L/2	139.413	683.397	2261.947	12	2	100

3.2.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 14 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	27.472 m ³	8000	219776
Steel	4148.053 kg	50	207403
Total			427179

3.3. PRESTRESSED CONCRETE PIER CAP

23. Again, Prestressed Pier caps for girder arrangement from 3 girders to 8 girders (deck slab spanning from 9.5m to 24.5m) were designed for the basis of this research.
- 24.
25. The moment at three locations is calculated that is at the face of the pier shaft, at d effective from the face of the pier shaft and at the half cantilever distance.
- 26.
27. The span of the flyover is 34.5m and the depth of the girder is kept fixed at 2.3m to calculate the dead load of the girder and the thickness of deck slab is kept as 0.220m.
- 28.
29. The reactions from maximum moment and maximum reaction case are considered to calculate the worse possible moments for the design of pier cap.
- 30.
31. The check for moment, stresses, shear force, crack width is done using suiTable modification factors for ULS and SLS.
- 32.
33. The total loads considered are self-weight of the pier cap, dead load of the superstructure, superimposed dead load i.e. surfacing and crash barrier, the live load which includes the impact factor (70RW, 70RW + Class A and 3 lanes of Class A).
- 34.
35. A suitable profile of the cable is chosen and losses due to friction, slip are considered.
- 36.
37. Time dependant losses like creep, shrinkage and relaxation losses are also considered during design.
- 38.

3.4. DESIGN OF PSC PIER CAP SUPPORTING THREE GIRDER SYSTEM

3.4.1. Design of PSC Pier Cap

3.4.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 15 Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one cable	2660 mm ²
Diameter of duct	107 mm
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.4.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 16 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F_h (t)	F_v (t)
At l/2	330.2	0.267	329.1	-27.4
at d eff from left	325.6	0.227	325.6	-5.6
at face of pier shaft	323.7	0.225	323.7	0.0

3.4.1.3. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 17 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	449.3	429.4	417.9
	Bottom	122.6	-84.8	-105.5
Stress after laying Superstructure	Top	883.1	525.6	246.6
	Bottom	231.1	286.5	362.4
Stress during Service at Infinity	Top	799.3	431.5	165.9
	Bottom	210.1	305.2	386.9

3.4.1.4. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 18 Stress Check with 1.1x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	550.9	534.8	517.7
	Bottom	148.0	-107.7	-135.8
Stress after laying Superstructure	Top	1074.7	743.5	434.6
	Bottom	279.0	293.2	305.1
Stress during Service at Infinity	Top	948.3	516.3	197.0
	Bottom	247.4	271.5	342.4

3.4.1.5. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 19 Summary of ULS Capacity Check

Section	M _{ED} (t-m)	Ast provided (mm ²)	Ultimate Moment Capacity (t-m)
At face of Shaft	896.0	11760	1856.6
At deff from face	610.810	11760	1291.4
At L/2	516.895	11760	761.7

3.4.1.6. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 20 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	610.5	872.208	Shear rein. Required
At deff from face	505.5	596.842	Shear rein. Required
At L/2	443.1	510.520	Shear rein. Required

3.4.1.7. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 21 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	872.208	16	16	100	1364
At deff from face	596.842	16	16	100	949
At L/2	510.520	16	16	100	560

3.4.1.8. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 22 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.25	0.42	12	2	100
At deff from face	1.14	0.42	12	2	100
At L/2	1.30	0.42	12	2	100

3.4.1.9. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 23 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	24.48 m ³	8000	195840
Tensioned Steel	347.755 kg	140	48685.7
Untensioned Steel	2939.577 kg	50	146978.9
Total			496213.0

3.5. PARTIALLY PRESTRESSED CONCRETE PIER CAP

Partially prestressed Pier caps are also designed for girder arrangement from 3 girders to 8 girders (deck slab spanning from 9.5m to 24.5m) was designed for the basis of this research. The aim is to find the most suitable arrangement for most economic section.

A certain value of crack width is permitted while design of partially prestressed pier caps.

The span of the flyover is 34.5m and the depth of the girder is kept fixed at 2.3m to calculate the dead load of the girder and the thickness of deck slab is kept as 0.220m.

The reactions from maximum moment and maximum reaction case are considered to calculate the worse possible moments for the design of pier cap.

The check for moment, stresses, shear force, crack width is done using suitable modification factors for ULS and SLS.

The total loads considered are self-weight of the pier cap, dead load of the superstructure, superimposed dead load i.e. surfacing and crash barrier, the live load which includes the impact factor (70RW, 70RW + Class A and 3 lanes of Class A).

A suitable profile of the cable is chosen and losses due to friction, slip are considered. This along with un-tensioned steel is used in the pier cap design.

Time dependant losses like creep, shrinkage and relaxation losses are also considered during design.

3.6. DESIGN OF PPSC PIER CAP SUPPORTING THREE GIRDER SYSTEM

3.6.1. Design of PSC Pier Cap

3.6.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 24 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
Diameter of duct	107 mm
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.6.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 25 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	330.2	0.267	329.1	-27.4
at d eff from left	325.6	0.227	325.6	-5.6
at face of pier shaft	323.7	0.225	323.7	0.0

3.6.1.3. Assume a value of crack width according to IRC 112:2011.

$w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

3.6.1.4. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

Minimum value of steel = $\max\{7017 \text{ and } 5012\}$

= 7017 mm^2

3.6.1.5. Assume value of A_s which is greater than $A_{s \min}$

Assumed $20\phi\text{-}33 \text{ nos}$ bars. Therefore $A_s = 10367 \text{ mm}^2$

3.6.1.6. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = 516.895 t-m .

3.6.1.7. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}}$$

(5.2)

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{\text{ceff}} = 2.5b(h - d) \quad \text{for beams}$$

(5.3)

$$A_{\text{ceff}} = 561000 \text{ mm}^2$$

$$\rho_r = 0.0185$$

3.6.1.8. Calculate the value of $\rho_{r \text{ min}}$ to compare the value of ρ_r .

$\rho_{r \text{ min}}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{r \text{ min}}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{r \text{ min}}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0$$

(5.4)

where, $k_1 = 0.8$

$$k_2 = 0.5$$

$$k_3 = 3.4$$

$$k_4 = 0.425$$

$$k_t = 0.4$$

Therefore,

$$\rho_{r \text{ min}} = 0.018$$

$$\rho_r > \rho_{r \text{ min}} \quad (\text{OK})$$

3.6.1.9. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{s_{\text{max}}} + k_t f_{ctm} \left(\frac{A_{\text{ceff}}}{A_s} + \frac{E_s}{E_{cm}} \right)$$

(5.5)

$$\text{Solving, we get,} \quad \sigma_s = 209 < 300 \text{ MPa} \quad (\text{OK})$$

3.6.1.10. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2}x^3 + \frac{3bd}{2}x^2 + \frac{3\alpha_e M}{2\sigma_s}x - \frac{3\alpha_e Md}{\sigma_s} = 0 \quad (5.6)$$

Solving, we get, $x = 262 \text{ mm}$

3.6.1.11. Calculation of $A_{p \max}$

The maximum area of A_p is calculated to find the upper limit of the prestressing steel.

$$A_{p \max} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.7)$$

$$A_{p \max} = 13116 \text{ mm}^2$$

3.6.1.12. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.8)$$

$$F_p = 2378 \text{ kN}$$

3.6.1.13. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

In this case, 2 cables with 19 strands were required to cater to this F_p after taking into consideration all losses.

3.6.1.14. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel both tensioned and untensioned, summary of which is as follows:

Table 3. 26 Calculation of Cost for a PPSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	24.48 m ³	8000	195840
Tensioned Steel	211.008 kg	140	29541.12
Untensioned Steel	4784.138 kg	50	239206.9
Total			464588

3.7. DESIGN OF RCC PIER CAP SUPPORTING FOUR GIRDER SYSTEM

3.7.1. Calculation of Forces

3.7.1.1. The depth of pier cap is assumed. In this case the depth assumed was 1.50m.

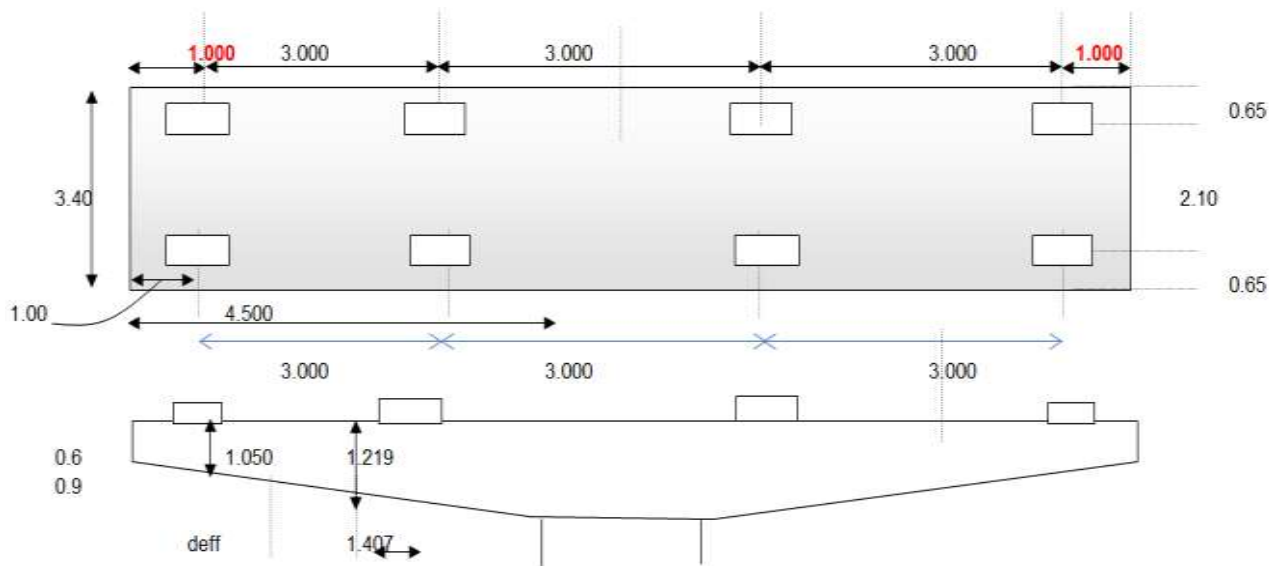


Figure 3. 1 Plan and Elevation of a Pier Cap for a Four Girder System

3.7.1.2. The distribution of live loads on all four girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 27 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3	L4
70RW	25.284	15.867	6.449	-2.968
70RW+A	31.762	22.657	13.553	4.448
Class A3	28.760	23.480	18.201	12.921
Maximum Moment Case				
70RW	49.370	30.982	12.593	-5.795
70RW+A	57.397	40.589	23.781	6.972
Class A3	43.045	35.143	27.241	19.339

Table 3.28 Reactions on Bearings on Right Side

Maximum Reaction Case	R1	R2	R3	R4
70RW	26.262	16.845	7.427	-1.990
70RW+A	30.390	21.679	12.967	4.256
Class A3	23.480	18.200	12.921	7.641
Maximum Moment Case				
70RW	0.000	0.000	0.000	0.000
70RW+A	0.000	0.000	0.000	0.000
Class A3	0.000	0.000	0.000	0.000

3.7.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed (practically).

3.7.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 29 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.7.1.5. The final forces are calculated at the face of the pier shaft, at the distance d_{eff} from the face of the pier shaft and at a distance of $cantilever/2$ from the face of the pier shaft.

3.7.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 30 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	783.503	1607.673
SLS RARE	529.858	1079.117
SLS QP	441.118	879.368

Table 3. 31 Summary of Bending Moments at d_{eff} from Face of Shaft

At d_{eff} from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	761.768	838.726
SLS RARE	513.607	560.920
SLS QP	424.867	452.491

Table 3. 32 Summary of Bending Moments at half the Cantilever Length

At L/2		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	400.789	496.145
SLS RARE	268.049	331.475
SLS QP	216.256	266.734

3.7.2. Design of Pier Cap

3.7.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 33 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
F _{yd}	0.6f _y
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.7.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 34 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Number
At face of Shaft	1607.673	34244	37306	25	76
At deff from face	838.726	25324	37306	25	76
At L/2	496.145	20312	37306	25	76

3.7.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 35 Summary of Stress Check for SLS Load Combination

Section	Combina- -tion	Compressi ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss- ible Tensile stress (MPa)
At face of Shaft	SLS Rare	13.119	16.800	232.292	300
	SLS QP	7.884	12.600	191.877	300
At deff from face	SLS Rare	10.279	16.800	160.021	300
	SLS QP	5.903	12.600	125.004	300
At L/2	SLS Rare	8.334	16.800	117.622	300
	SLS QP	4.583	12.600	87.537	300

3.7.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which are as follows:

Table 3. 36 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.161	0.300
At deff from face	0.085	0.300
At L/2	0.059	0.300

3.7.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 37 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	220.90	783.503	Shear rein. Required
At deff from face	196.52	411.768	Shear rein. Required
At L/2	180.78	400.789	Shear rein. Required

3.7.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 38 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	783.503	22327.03	24127.43	16	12	100
At deff from face	411.768	15690.34	24127.43	16	12	100
At L/2	400.789	18048.63	24127.43	16	12	100

3.7.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 39 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	242.129	1186.909	2261.947	12	2	100
At deff from face	242.129	1186.909	2261.947	12	2	100
At L/2	242.129	1186.909	2261.947	12	2	100

3.7.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 40 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	42.636 m ³	8000	341088
Steel	6383.83 kg	50	319191
Total			660279

3.8. DESIGN OF PSC PIER CAP SUPPORTING FOUR GIRDER SYSTEM

3.8.1. Calculation of Forces

3.8.1.1. The depth of pier cap is assumed. In this case the depth assumed was 1.50m.

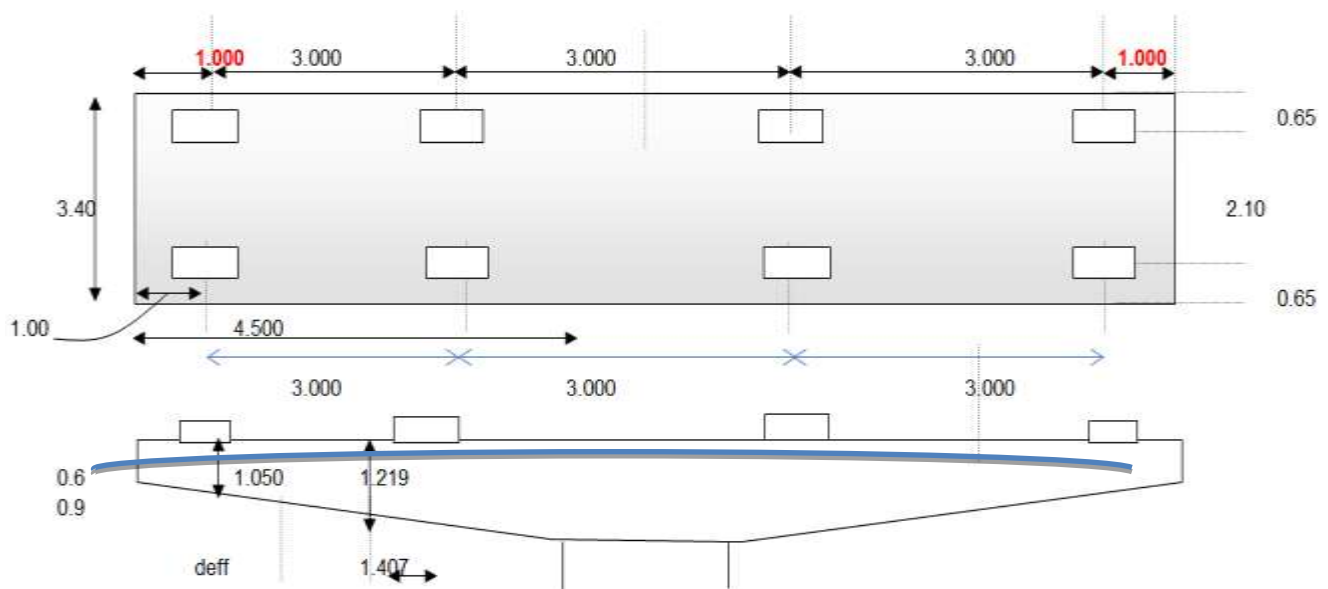


Figure 3. 2 Plan and Elevation for PSC Pier Cap for Four Girder System

3.8.1.2. The distribution of live loads on all four girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated.

3.8.2. Design of PSC Pier Cap

3.8.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 41. Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
Diameter of duct	107 mm
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.8.2.2. Assumed Profile of Cables

The assumed profile of cables for a Pier Cap supporting a four girder system is as follows:

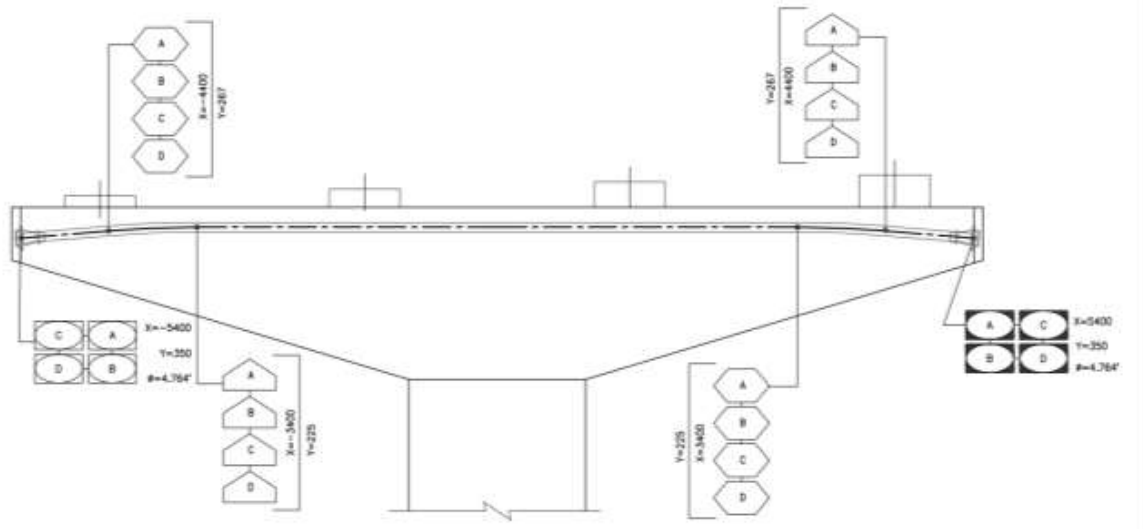


Figure 3. 3 Cable Profile for a Pier Cap supporting a Four Girder System

3.8.2.3. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 42 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	341.8	0.267	340.6	-28.4
at d eff from left	335.3	0.225	335.3	0.0
at face of pier shaft	334.2	0.225	334.2	0.0

3.8.2.4. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 43 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	489.7	471.7	382.5
	Bottom	-73.3	-118.6	-96.5
Stress after laying Superstructure	Top	1177.7	818.9	487.7
	Bottom	-71.7	119.1	271.9
Stress during Service at Infinity	Top	996.1	640.6	333.2
	Bottom	-36.7	174.2	326.7

3.8.2.5. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 44 Stress Check with 1.1x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	396.0	378.9	302.0
	Bottom	-55.3	-90.0	-68.0
Stress after laying Superstructure	Top	933.6	577.1	277.9
	Bottom	-24.7	193.7	346.4
Stress during Service at Infinity	Top	807.7	454.1	171.4
	Bottom	0.5	231.7	384.1

3.8.2.6. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 45 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	1607.7	14560	3006
At deff from face	838.7	14560	2342.8
At L/2	496.8	14560	1846.8

3.8.2.7. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 46 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	610.5	783.503	Shear rein. Required
At deff from face	505.5	761.607	Shear rein. Required
At L/2	443.1	400.789	Provide minimum

3.8.2.8. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 47 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	783.503	16	12	130	1029.36
At deff from face	761.607	16	12	130	802.25
At L/2	400.789	16	12	130	632.42

3.8.2.9. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 48 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.74	0.42	12	2	130
At deff from face	1.09	0.42	12	2	130
At L/2	1.44	0.42	12	2	130

3.8.2.10. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 49 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	42.636 m ³	8000	341088
Tensioned Steel	628.63 kg	140	88007
Untensioned Steel	3336.03 kg	50	166801
Total			595897

3.9. DESIGN OF PPSC PIER CAP SUPPORTING FOUR GIRDER SYSTEM

3.9.1. Calculation of Forces

3.9.1.1. The depth of pier cap is assumed. In this case the depth assumed was 1.50m.

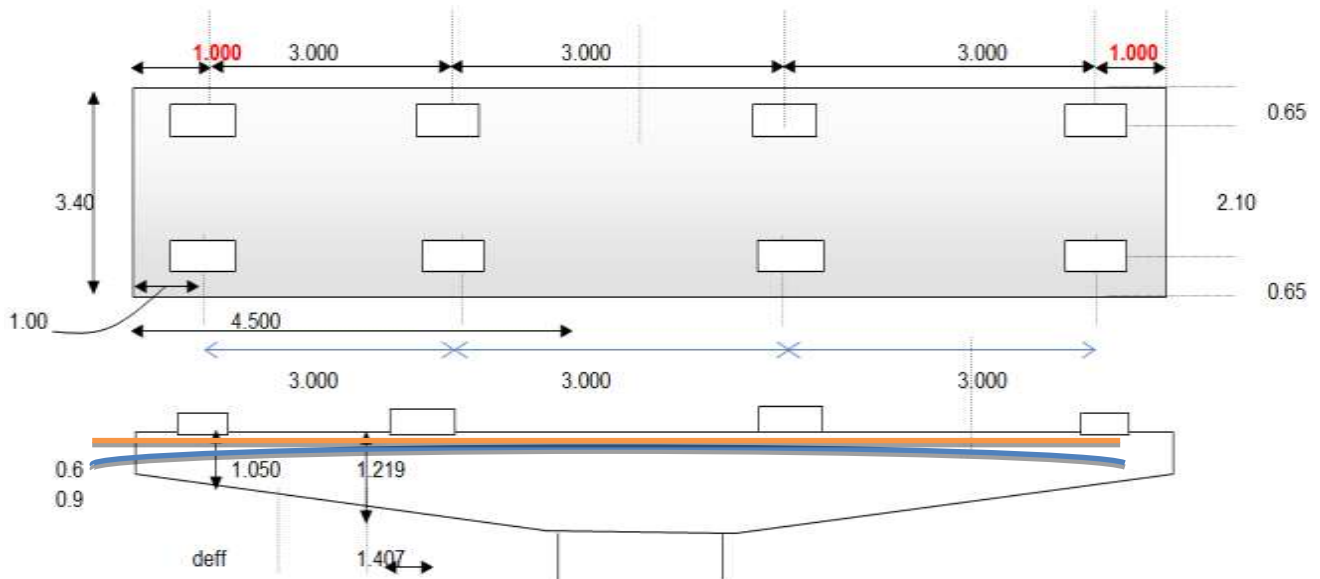


Figure 3. 4 Plan and Elevation for PPSC Pier Cap for Four Girder System

3.9.1.2. The distribution of live loads on all four girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated.

3.9.2. Design of PSC Pier Cap

3.9.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 50 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
Diameter of duct	107 mm
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.9.2.2. Assumed Profile of Cables

The following figure shows the assumed profile of the cables:

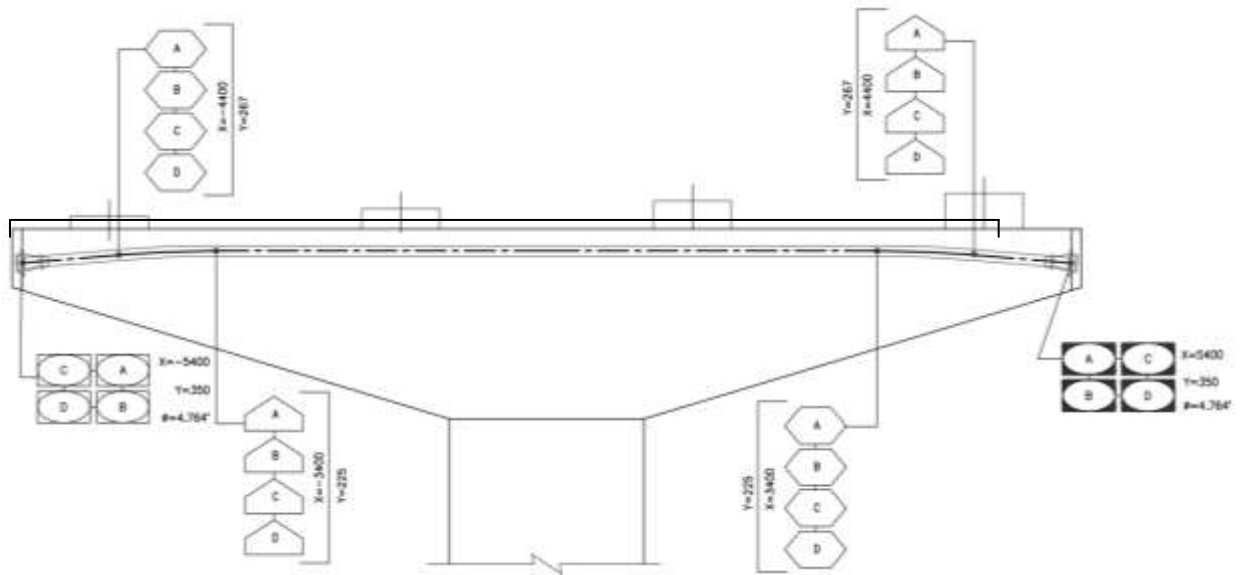


Figure 3. 5 Assumed Profile of the cables for PPSC Pier Cap for a Four Girder System

3.9.2.3. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 51 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	341.8	0.267	340.6	-28.4
at d eff from left	335.3	0.225	335.3	0.0
at face of pier shaft	334.2	0.225	334.2	0.0

- 3.9.2.4. Assume a value of crack width according to IRC 112:2011.
 $w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

- 3.9.2.5. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

$$\begin{aligned}\text{Minimum value of steel} &= \max\{8858 \text{ and } 6327\} \\ &= 8858 \text{ mm}^2\end{aligned}$$

- 3.9.2.6. Assume value of A_s which is greater than $A_{s \min}$
Assumed $25\phi\text{-}24$ nos bars. Therefore $A_s = 11781 \text{ mm}^2$

- 3.9.2.7. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = 1079.1 t-m .

- 3.9.2.8. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}}$$

(5.2)

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{ceff} = 2.5b(h - d) \quad \text{for beams}$$

(5.3)

$$A_{ceff} = 582250 \text{ mm}^2$$

$$\rho_r = 0.0202$$

3.9.2.9. Calculate the value of $\rho_{r \min}$ to compare the value of ρ_r .

$\rho_{r \min}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{r \min}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{r \min}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0 \quad (5.4)$$

where, $k_1 = 0.8$

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

$k_t = 0.4$

Therefore,

$$\rho_{r \min} = 0.0196$$

$$\rho_r > \rho_{r \min} \quad (\text{OK})$$

3.9.2.10. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{s_{r \max}} + k_t f_{ctm} \left(\frac{A_{c, \text{eff}}}{A_s} + \frac{E_s}{E_{cm}} \right) \quad (5.5)$$

Solving, we get, $\sigma_s = 193 < 300 \text{ MPa} \quad (\text{OK})$

3.9.2.11. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2} x^3 + \frac{3bd}{2} x^2 + \frac{3\alpha_e M}{2\sigma_s} x - \frac{3\alpha_e M d}{\sigma_s} = 0 \quad (5.6)$$

Solving, we get, $x = 388 \text{ mm}$

3.9.2.12. Calculation of $A_{p \max}$

The maximum area of A_p is calculated to find the upper limit of the

prestressing steel.

$$A_{pmax} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.7)$$

$$A_{pmax} = 31209 \text{ mm}^2$$

3.9.2.13. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.8)$$

$$F_p = 6016 \text{ kN}$$

3.9.2.14. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

3.9.2.15. Check for Ultimate Moment Capacity (ULS)

The ultimate moment capacity is calculated by taking into account both the A_s and A_p .

Table 3. 52 Summary of ULS check for the Pier Cap

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	1607.7	17101	3530.6
At deff from face	838.7	17101	2631.3
At L/2	496.8	17101	1830.1

3.9.2.16. Shear Capacity of the section (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 53 Check for Shear Capacity (ULS) at various sections

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	578.7	783.503	Shear rein. Required
At deff from face	474.1	761.607	Shear rein. Required
At L/2	411.5	400.789	Provide minimum

3.9.2.17. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 54 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	783.503	16	12	130	1029.36
At deff from face	761.607	16	12	130	802.25
At L/2	400.789	16	12	130	632.42

3.9.2.18. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 55 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.74	0.42	12	2	130
At deff from face	1.09	0.42	12	2	130
At L/2	1.44	0.42	12	2	130

3.9.2.19. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 56 Calculation of Cost of PPSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	36.652 m ³	8000	293216
Tensioned Steel	265.95 kg	140	37234
Untensioned Steel	4907.71 kg	50	245385
Total			575835

3.10. DESIGN OF RCC PIER CAP SUPPORTING FIVE GIRDER SYSTEM

3.10.1. Calculation of Forces

3.10.1.1. The depth of pier cap is assumed. In this case the depth assumed was 1.70m.

39.

3.10.1.2. The distribution of live loads on all five girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 57 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3	L4	L5
70RW	22.807	15.867	8.926	1.986	-4.954
70RW+A	30.831	22.657	14.484	6.311	-1.863
Class A3	30.288	23.480	16.673	9.865	3.057
Maximum Moment Case					
70RW	44.534	30.982	17.430	3.878	-9.674
70RW+A	55.430	40.589	25.748	10.907	-3.934
Class A3	45.332	35.143	24.954	14.764	4.575

Table 3.58 Reactions on Bearings on Right Side

Maximum Reaction Case	R1	R2	R3	R4	R5
70RW	23.590	16.649	9.709	2.769	-4.172
70RW+A	29.499	21.679	13.858	6.038	-1.782
Class A3	26.064	19.256	12.449	5.641	-1.167
Maximum Moment Case					
70RW	0.000	0.000	0.000	0.000	0.000
70RW+A	0.000	0.000	0.000	0.000	0.000
Class A3	0.000	0.000	0.000	0.000	0.000

3.10.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed.

3.10.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 59 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.10.1.5. The final forces are calculated at the face of the pier shaft, at the distance d_{eff} from the face of the pier shaft and at a distance of $cantilever/2$ from the face of the pier shaft.

3.10.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 60 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	738.8	2550.049
SLS RARE	497.408	1704.820
SLS QP	410.187	1379.555

Table 3. 61 Summary of Bending Moments at d eff from Face of Shaft

At deff from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	710.102	1464.152
SLS RARE	476.150	974.123
SLS QP	388.929	788.062

Table 3. 62 Summary of Bending Moments at half the Cantilever Length

At L/2		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	374.122	727.588
SLS RARE	248.993	482.686
SLS QP	198.719	382.137

3.10.2. Design of Pier Cap

3.10.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 63 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
Fyd	$0.6f_y$
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.10.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 64 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Number
At face of Shaft	738.8	46277.59	49863	32	62
At deff from face	497.408	34474.77	49863	32	62
At L/2	410.187	19708.00	49863	32	62

3.10.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 65 Summary of Stress Check for SLS Load Combination

Section	Combina- -tion	Compressi ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss- ible Tensile stress (MPa)
At face of Shaft	SLS Rare	14.930	16.800	238.847	300
	SLS QP	9.113	12.600	200.327	300
At deff from face	SLS Rare	11.885	16.800	171.442	300
	SLS QP	7.087	12.600	139.821	300
At L/2	SLS Rare	8.149	16.800	104.681	300
	SLS QP	4.781	12.600	82.679	300

3.10.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which are as follows:

Table 3. 66 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.180	0.300
At deff from face	0.109	0.300
At L/2	0.058	0.300

3.10.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 67 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	260.29	738.8	Shear rein. Required
At deff from face	236.25	710.102	Shear rein. Required
At L/2	213.55	374.122	Shear rein. Required

3.10.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 68 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	738.8	18524.12	21652.82	16	14	130
At deff from face	710.102	21422.27	21652.82	16	14	130
At L/2	374.122	13778.20	21652.82	16	14	130

3.10.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 69 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	224.556	1186.909	1739.96	12	2	130
At deff from face	224.556	1186.909	1739.96	12	2	130
At L/2	224.556	1186.909	1739.96	12	2	130

3.10.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 70 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	60.68 m ³	8000	486880
Steel	10541.48 kg	50	527074.1
Total			1013955

3.11. DESIGN OF PSC PIER CAP SUPPORTING FIVE GIRDER SYSTEM

3.11.1. Design of PSC Pier Cap

3.11.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 71. Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.11.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 72 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	341.3	0.500	336.7	-56.1
at d eff from left	341.6	0.417	337.0	-56.2
at face of pier shaft	342.0	0.333	337.3	-56.2

3.11.1.3. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 73 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	400.5	398.6	395.6
	Bottom	187.7	3.6	-74.8
Stress after laying Superstructure	Top	668.2	439.4	336.3
	Bottom	512.2	500.9	493.6
Stress during Service at Infinity	Top	567.5	312.4	202.0
	Bottom	478.7	516.8	537.8

3.11.1.4. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 74 Stress Check with 1.1x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	498.5	463.8	382.5
	Bottom	220.3	-10.4	-112.8
Stress after laying Superstructure	Top	858.0	678.6	589.1
	Bottom	575.5	522.2	424.9
Stress during Service at Infinity	Top	713.9	496.8	396.9
	Bottom	527.4	542.9	573.9

3.11.1.5. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 75 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	2550	21840	4833.2
At deff from face	1464	21840	3597.8
At L/2	239	21840	2475.5

3.11.1.6. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 76 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	1031.4	710.5	Shear rein. Required
At deff from face	710.1	610.7	Shear rein. Required
At L/2	374.1	522.7	Provide minimum

3.11.1.7. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 77 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	710.5	16	10	100	1087
At deff from face	610.7	16	10	100	809
At L/2	522.7	16	10	100	557

3.11.1.8. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 78 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.54	0.42	12	2	110
At deff from face	0.74	0.42	12	2	110
At L/2	1.02	0.42	12	2	110

3.11.1.9. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 79 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	60.52 m ³	8000	484160
Tensioned Steel	1200.108 kg	140	168015.1
Untensioned Steel	4679.885 kg	50	233994.3
Total			886170

3.12. DESIGN OF PPSC PIER CAP SUPPORTING FIVE GIRDER SYSTEM

3.12.1. Design of PSC Pier Cap

3.12.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are:

Table 3. 80 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³

3.12.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 81 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	341.3	0.500	336.7	-56.1
at d eff from left	341.6	0.417	337.0	-56.2
at face of pier shaft	342.0	0.333	337.3	-56.2

3.12.1.3. Assume a value of crack width according to IRC 112:2011.

$w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

3.12.1.4. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

$$\begin{aligned}\text{Minimum value of steel} &= \max\{10096 \text{ and } 7211\} \\ &= 10096 \text{ mm}^2\end{aligned}$$

3.12.1.5. Assume value of A_s which is greater than $A_{s \min}$

Assumed $25\phi\text{-}24 \text{ nos}$ bars. Therefore $A_s = 11781 \text{ mm}^2$

3.12.1.6. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = 1704.8 t-m .

3.12.1.7. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}}$$

(5.2)

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{ceff} = 2.5b(h - d) \quad \text{for beams}$$

(5.3)

$$A_{ceff} = 582250 \text{ mm}^2$$

$$\rho_r = 0.0202$$

3.12.1.8. Calculate the value of $\rho_{r \min}$ to compare the value of ρ_r .

$\rho_{r \min}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{r \min}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{r \min}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0$$

(5.4)

where, $k_1 = 0.8$

$$k_2 = 0.5$$

$$k_3 = 3.4$$

$$k_4 = 0.425$$

$$k_t = 0.4$$

Therefore,

$$\rho_{r \min} = 0.0196$$

$$\rho_r > \rho_{r \min} \quad (\text{OK})$$

3.12.1.9. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{s_{r \max}} + k_t f_{ctm} \left(\frac{A_{c,eff}}{A_s} + \frac{E_s}{E_{cm}} \right)$$

(5.5)

Solving, we get, $\sigma_s = 193 < 300 \text{ MPa} \quad (\text{OK})$

3.12.1.10. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2} x^3 + \frac{3bd}{2} x^2 + \frac{3\alpha_e M}{2\sigma_s} x - \frac{3\alpha_e M d}{\sigma_s} = 0$$

(5.6)

Solving, we get, $x = 482 \text{ mm}$

3.12.1.11. Calculation of $A_{p \max}$

The maximum area of A_p is calculated to find the upper limit of the prestressing steel.

$$A_{p \max} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s$$

(5.7)

$$A_{p \max} = 48343 \text{ mm}^2$$

3.12.1.12. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s$$

(5.8)

$$F_p = 9320 \text{ kN}$$

3.12.1.13. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

3.12.1.14. Check for Ultimate Moment Capacity (ULS)

The ultimate moment capacity is calculated by taking into account both the A_s and A_p .

Table 3. 82 Summary of ULS check for the Pier Cap

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	2550	19901	3767.8
At deff from face	1434	19901	2613.9
At L/2	239	19901	1407.9

3.12.1.15. Shear Capacity of the section (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 83 Check for Shear Capacity (ULS) at various sections

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	663.7	710.5	Shear rein. Required
At deff from face	564.7	610.7	Shear rein. Required
At L/2	477.5	522.7	Shear rein. Required

3.12.1.16. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 84 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	710.5	16	10	110	1087
At deff from face	610.7	16	10	110	809
At L/2	522.7	16	10	110	557

3.12.1.17. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 85 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.54	0.42	12	2	110
At deff from face	0.74	0.42	12	2	110
At L/2	1.02	0.42	12	2	110

3.12.1.18. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 86 Calculation of Cost of PPSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	47.6 m ³	8000	380800
Tensioned Steel	569.282 kg	140	79699.48
Untensioned Steel	6411.362 kg	50	327860.5
Total			788360

3.13. DESIGN OF RCC PIER CAP SUPPORTING SIX GIRDER SYSTEM

3.13.1. Calculation of Forces

3.13.1.1. The depth of pier cap is assumed. In this case the depth assumed was 2.00m.

40.

3.13.1.2. The distribution of live loads on all six girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 87 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3	L4	L5	L6
70RW	20.541	15.300	10.059	4.818	-0.423	-5.664
70RW+A	28.919	22.179	15.440	8.700	1.961	-4.779
Class A3	29.574	23.302	17.030	10.758	4.486	-1.786
Maximum Moment Case						
70RW	40.110	29.876	19.642	9.408	-0.826	-11.060
70RW+A	51.854	39.695	27.536	15.377	3.218	-8.941
Class A3	44.263	34.876	25.488	16.101	6.714	-2.674

Table 3.88 Reactions on Bearings on Right Side

Maximum Reaction Case	R1	R2	R3	R4	R5	R6
70RW	21.193	15.952	10.711	5.470	0.229	-5.012
70RW+A	27.670	21.221	14.773	8.324	1.876	-4.573
Class A3	26.054	19.782	13.510	7.238	0.966	-5.306
Maximum Moment Case						
70RW	0.000	0.000	0.000	0.000	0.000	0.000
70RW+A	0.000	0.000	0.000	0.000	0.000	0.000
Class A3	0.000	0.000	0.000	0.000	0.000	0.000

3.13.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed (practically).

3.13.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 89 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.13.1.5. The final forces are calculated at the face of the pier shaft, at the distance d_{eff} from the face of the pier shaft and at a distance of $cantilever/2$ from the face of the pier shaft.

3.13.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 90 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	994.335	3648.148
SLS RARE	672.451	2440.331
SLS QP	563.949	1994.635

Table 3. 91 Summary of Bending Moments at d_{eff} from Face of Shaft

At d_{eff} from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	685.979	2176.172
SLS RARE	460.112	1449.061
SLS QP	376.788	1173.936

Table 3. 92 Summary of Bending Moments at half the Cantilever Length

At $L/2$		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	362.390	944.935
SLS RARE	241.781	626.648
SLS QP	194.624	496.965

3.13.2. Design of Pier Cap

3.13.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 93 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
F _{yd}	0.6f _y
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.13.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 94 Summary of ULS Capacity Check

Section	M _{ED} (t-m)	A _{st} required (mm ²)	A _{st} provided (mm ²)	Diameter (mm)	Number
At face of Shaft	3648.148	55637.81	61123	32	76
At deff from face	2176.172	40384.42	61123	32	76
At L/2	944.935	20099.52	61123	32	76

3.13.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 95 Summary of Stress Check for SLS Load Combination

Section	Combina-tion	Compressi-ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss-ible Tensile stress (MPa)
At face of Shaft	SLS Rare	14.957	16.800	235.968	300
	SLS QP	9.245	12.600	199.265	300
At deff from face	SLS Rare	11.759	16.800	168.901	300
	SLS QP	7.117	12.600	138.744	300
At L/2	SLS Rare	6.946	16.800	89.347	300
	SLS QP	4.125	12.600	71.416	300

3.13.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which as are follows:

Table 3. 96 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.170	0.300
At deff from face	0.105	0.300
At L/2	0.046	0.300

3.13.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 97 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	305.72	994.335	Shear rein. Required
At deff from face	280.61	685.979	Shear rein. Required
At L/2	254.44	362.390	Shear rein. Required

3.13.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 98 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	994.335	20859.48	21934.03	16	12	110
At deff from face	685.979	16946.92	21934.03	16	12	110
At L/2	362.390	10808.91	21934.03	16	12	110

3.13.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 99 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	279.052	1367.901	2056.315	12	2	110
At deff from face	279.052	1367.901	2056.315	12	2	110
At L/2	279.052	1367.901	2056.315	12	2	110

3.13.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 100 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	93.704 m ³	8000	749632
Steel	14135.41 kg	50	706771
Total			1456403

3.14. DESIGN OF PSC PIER CAP SUPPORTING SIX GIRDER SYSTEM

3.14.1. Design of PSC Pier Cap

3.14.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 101. Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.14.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 102 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F_h (t)	F_v (t)
At l/2	336.1	0.400	329.6	-65.9
at d eff from left	338.2	0.300	338.2	0
at face of pier shaft	337.0	0.300	337.0	0

3.14.1.3. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 103 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	514.5	503.6	413.3
	Bottom	-27.7	-60.5	-103.1
Stress after laying Superstructure	Top	1052.7	669.5	351.3
	Bottom	119.0	256.6	443.1
Stress during Service at Infinity	Top	905.8	506.4	205.0
	Bottom	40.6	222.6	384.9

3.14.1.4. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 104 Stress Check with 1.1x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	627.4	649.3	534.7
	Bottom	-45.9	-103.5	-146.7
Stress after laying Superstructure	Top	1323.3	964.4	616.7
	Bottom	348.9	161.3	-20.8
Stress during Service at Infinity	Top	1111.0	423.5	387.0
	Bottom	10.4	387.0	620.2

3.14.1.5. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 105 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	Ast provided (mm ²)	Ultimate Moment Capacity (t-m)
At face of Shaft	3648.148	24640	6782.7
At deff from face	2176.172	24640	5673.2
At L/2	944.935	24640	4189.3

3.14.1.6. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 106 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	831.1	994.335	Shear rein. Required
At deff from face	727.3	685.979	Provide minimum
At L/2	623.3	362.390	Provide minimum

3.14.1.7. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 107 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final shear Capacity
At face of Shaft	710.5	16	10	140	1062
At deff from face	610.7	16	10	140	888
At L/2	522.7	16	10	140	656

3.14.1.8. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 108 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.48	0.42	12	2	140
At deff from face	0.62	0.42	12	2	140
At L/2	0.85	0.42	12	2	140

3.14.1.9. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 109 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	87.55 m ³	8000	700400
Tensioned Steel	1644.104 kg	140	230174.6
Untensioned Steel	5317.286 kg	50	265864.3
Total			1196439

3.15. DESIGN OF PPSC PIER CAP SUPPORTING SIX GIRDER SYSTEM

3.15.1. Design of PSC Pier Cap

3.15.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are:

Table 3. 110 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³

3.15.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 111 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	336.1	0.400	329.6	-65.9
at d eff from left	338.2	0.300	338.2	0
at face of pier shaft	337.0	0.300	337.0	0

3.15.1.3. Assume a value of crack width according to IRC 112:2011.

$w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

3.15.1.4. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

$$\begin{aligned}\text{Minimum value of steel} &= \max\{11312 \text{ and } 8080\} \\ &= 11312 \text{ mm}^2\end{aligned}$$

3.15.1.5. Assume value of A_s which is greater than $A_{s \min}$

Assumed $32\phi\text{-}18$ nos bars. Therefore $A_s = 14476 \text{ mm}^2$

3.15.1.6. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = **2416.4 t-m**.

3.15.1.7. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}} \quad (5.2)$$

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{ceff} = 2.5b(h - d) \quad \text{for beams} \quad (5.3)$$

$$A_{ceff} = \mathbf{612000 \text{ mm}^2}$$

$$\rho_r = \mathbf{0.0240}$$

3.15.1.8. Calculate the value of $\rho_{r \min}$ to compare the value of ρ_r .

$\rho_{r \min}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{r \min}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{r \min}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0 \quad (5.4)$$

where, $k_1 = 0.8$

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

$k_t = 0.4$

Therefore,

$$\rho_{r \min} = 0.022$$

$$\rho_r > \rho_{r \min} \quad (\mathbf{OK})$$

3.15.1.9. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{S_{rmax}} + k_t f_{ctm} \left(\frac{A_{c,eff}}{A_s} + \frac{E_s}{E_{cm}} \right) \quad (5.5)$$

Solving, we get, $\sigma_s = 176 < 300 \text{ MPa}$ (OK)

3.15.1.10. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2}x^3 + \frac{3bd}{2}x^2 + \frac{3\alpha_e M}{2\sigma_s}x - \frac{3\alpha_e Md}{\sigma_s} = 0 \quad (5.6)$$

Solving, we get, $x = 591 \text{ mm}$

3.15.1.11. Calculation of $A_{p \max}$

The maximum area of A_p is calculated to find the upper limit of the prestressing steel.

$$A_{pmax} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.7)$$

$$A_{p \max} = 69471 \text{ mm}^2$$

3.15.1.12. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.8)$$

$$F_p = 12260 \text{ kN}$$

3.15.1.13. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

3.15.1.14. Check for Ultimate Moment Capacity (ULS)

The ultimate moment capacity is calculated by taking into account both the A_s and A_p .

Table 3. 112 Summary of ULS check for the Pier Cap

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	3648.148	25396.5	6579.7
At deff from face	2176.172	25396.5	5496.5
At L/2	944.935	25396.5	3274.3

3.15.1.15. Shear Capacity of the section (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 113 Check for Shear Capacity (ULS) at various sections

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	748.1	994.335	Shear rein. Required
At deff from face	650.1	685.979	Shear rein. Required
At L/2	542.4	362.390	Provide min.

3.15.1.16. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 114 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final Shear Capacity (t)
At face of Shaft	994.335	16	10	140	1000
At deff from face	685.979	16	10	140	835
At L/2	362.390	16	10	140	593

3.15.1.17. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 115 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.52	0.42	12	2	140
At deff from face	0.68	0.42	12	2	140
At L/2	0.97	0.42	12	2	140

3.15.1.18. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 116 Calculation of Cost of PPSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	81.77 m ³	8000	654160
Tensioned Steel	728.637 kg	140	102009
Untensioned Steel	6985.28 kg	50	654160
Total			1105433

3.16. DESIGN OF RCC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM

3.16.1. Calculation of Forces

3.16.1.1. The depth of pier cap is assumed. In this case the depth assumed was 2.20m.

3.16.1.2. The distribution of live loads on all seven girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 117 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3	L4	L5	L6	L7
70RW	18.594	14.521	10.449	6.376	2.303	-1.769	-5.842
70RW+A	26.862	21.357	15.851	10.346	4.840	-0.665	-6.171
Class A3	28.135	22.726	17.318	11.909	6.500	1.092	-4.317
Maximum Moment Case							
70RW	36.307	28.355	20.402	12.450	4.498	-3.455	-11.407
70RW+A	48.086	38.188	28.290	18.391	8.493	-1.405	-11.303
Class A3	42.109	34.014	25.919	17.824	9.729	1.634	-6.461

Table 3.118 Reactions on Bearings on Right Side

Maximum Reaction Case	R1	R2	R3	R4	R5	R6	R7
70RW	19.153	15.080	11.008	6.935	2.862	-1.210	-5.283
70RW+A	25.702	20.434	15.166	9.899	4.631	-0.636	-5.904
Class A3	25.118	19.709	14.300	8.892	3.483	-1.926	-7.334
Maximum Moment Case							
70RW	0.000	0.000	0.000	0.000	0.000	0.000	0.000
70RW+A	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Class A3	0.000	0.000	0.000	0.000	0.000	0.000	0.000

3.16.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed (practically).

3.16.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 119 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.16.1.5. The final forces are calculated at the face of the pier shaft, at the distance d_{eff} from the face of the pier shaft and at a distance of cantilever/2 from the face of the pier shaft.

3.16.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 120 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	971.523	4844.672
SLS RARE	657.443	3245.744
SLS QP	552.966	2669.493

Table 3. 121 Summary of Bending Moments at d_{eff} from Face of Shaft

At d_{eff} from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	669.723	2885.512
SLS RARE	450.300	1923.317
SLS QP	371.671	1563.569

Table 3. 122 Summary of Bending Moments at half the Cantilever Length

At L/2		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	552.524	793.518
SLS RARE	380.254	505.660
SLS QP	336.451	385.697

3.16.2. Design of Pier Cap

3.16.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 123 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
F _{yd}	0.6f _y
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.16.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 124 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Number
At face of Shaft	4844.672	66118.87	72382	32	90
At deff from face	2885.512	47708.97	72382	32	90
At L/2	793.518	19560.74	72382	32	90

3.16.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 125 Summary of Stress Check for SLS Load Combination

Section	Combina- -tion	Compressi ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss- ible Tensile stress (MPa)
At face of Shaft	SLS Rare	15.790	16.800	239.485	300
	SLS QP	9.910	12.600	204.542	300
At deff from face	SLS Rare	12.298	16.800	169.998	300
	SLS QP	7.566	12.600	141.366	300
At L/2	SLS Rare	4.892	16.800	59.385	300
	SLS QP	2.699	12.600	43.842	300

3.16.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which are as follows:

Table 3. 126 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.169	0.300
At deff from face	0.105	0.300
At L/2	0.027	0.300

3.16.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 127 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	341.66	971.523	Shear rein. Required
At deff from face	313.81	669.723	Shear rein. Required
At L/2	279.60	552.524	Shear rein. Required

3.16.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 128 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	971.523	18379.29	24746.08	16	16	130
At deff from face	669.723	14865.51	24746.08	16	16	130
At L/2	552.524	15442.03	24746.08	16	16	130

3.16.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 129 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	261.046	1279.637	1740	12	2	130
At deff from face	261.046	1279.637	1740	12	2	130
At L/2	261.046	1279.637	1740	12	2	130

3.16.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 130 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	110.262 m ³	8000	882096
Steel	20207.6 kg	50	1010381
Total			1892477

3.17. DESIGN OF PSC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM

3.17.1. Design of PSC Pier Cap

3.17.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 131. Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.17.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 132 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F_h (t)	F_v (t)
At l/2	345.7	0.500	343.9	-34.4
at d eff from left	346.0	0.300	346.0	0.0
at face of pier shaft	344.6	0.300	344.6	0.0

3.17.1.3. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 133 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	461.2	409.8	355.3
	Bottom	55.6	-26.8	-78.7
Stress after laying Superstructure	Top	1033.3	760.5	394.3
	Bottom	184.4	240.4	465.4
Stress during Service at Infinity	Top	870.4	562.0	516.9
	Bottom	189.5	308.8	531.3

3.17.1.4. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 134 Stress Check with 1.1x Prestress

Section		At L/2 (t/m ²)	At deff from face (t/m ²)	At face of Shaft (t/m ²)
Stage				
Stress after 1st stage of prestressing	Top	590.7	516.3	471.2
	Bottom	52.2	-69.8	-123.3
Stress after laying Superstructure	Top	1305.4	1091.1	690.3
	Bottom	126.5	175.9	355.4
Stress during Service at Infinity	Top	1078.5	814.8	443.2
	Bottom	183.0	221.7	447.2

3.17.1.5. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 135 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	4844.672	28700	8829.8
At deff from face	2885.512	28700	7422.8
At L/2	793.518	28700	4879.6

3.17.1.6. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 136 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	920.8	971.523	Shear rein. Required
At deff from face	807.7	669.723	Provide minimum
At L/2	676.7	552.524	Provide minimum

3.17.1.7. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 137 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final Shear Capacity (t)
At face of Shaft	971.523	12	12	120	975
At deff from face	669.723	12	12	120	786
At L/2	552.524	12	12	120	556

3.17.1.8. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 138 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.39	0.42	12	2	120
At deff from face	0.51	0.42	12	2	120
At L/2	0.73	0.42	12	2	120

3.17.1.9. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 139 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	109.82 m ³	8000	878560
Tensioned Steel	4505.9 kg	140	630826
Untensioned Steel	5469.57 kg	50	273478
Total			1742864

3.18. DESIGN OF PPSC PIER CAP SUPPORTING SEVEN GIRDER SYSTEM

3.18.1. Design of PSC Pier Cap

3.18.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are:

Table 3. 140 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.18.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 141 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	345.7	0.500	343.9	-34.4
at d eff from left	346.0	0.300	346.0	0.0
at face of pier shaft	344.6	0.300	344.6	0.0

3.18.1.3. Assume a value of crack width according to IRC 112:2011.

$w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

3.18.1.4. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

Minimum value of steel = $\max\{11930 \text{ and } 8522\}$

$$= 11930 \text{ mm}^2$$

3.18.1.5. Assume value of A_s which is greater than $A_{s \min}$

Assumed $32\phi\text{-}30$ nos bars. Therefore $A_s = 24127 \text{ mm}^2$

3.18.1.6. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = 2412.7 t-m .

3.18.1.7. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}}$$

(5.2)

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{ceff} = 2.5b(h - d) \text{ for beams}$$

(5.3)

$$A_{ceff} = 612000 \text{ mm}^2$$

$$\rho_r = 0.039$$

3.18.1.8. Calculate the value of $\rho_{r \min}$ to compare the value of ρ_r .

$\rho_{r \min}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{rmin}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{rmin}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0 \quad (5.4)$$

where, $k_1 = 0.8$

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

$k_t = 0.4$

Therefore,

$\rho_{rmin} = 0.0216$

$\rho_r > \rho_{rmin}$ (OK)

3.18.1.9. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{s_{rmax}} + k_t f_{ctm} \left(\frac{A_{c,eff}}{A_s} + \frac{E_s}{E_{cm}} \right) \quad (5.5)$$

Solving, we get, $\sigma_s = 190 < 300$ MPa (OK)

3.18.1.10. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2}x^3 + \frac{3bd}{2}x^2 + \frac{3\alpha_e M}{2\sigma_s}x - \frac{3\alpha_e M d}{\sigma_s} = 0 \quad (5.6)$$

Solving, we get, $x = 651$ mm

3.18.1.11. Calculation of A_{pmax}

The maximum area of A_p is calculated to find the upper limit of the prestressing steel.

$$A_{pmax} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.7)$$

$A_{pmax} = 74557 \text{ mm}^2$

3.18.1.12. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.8)$$

$$F_p = 14129 \text{ kN}$$

3.18.1.13. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

3.18.1.14. Check for Ultimate Moment Capacity (ULS)

The ultimate moment capacity is calculated by taking into account both the A_s and A_p .

Table 3. 142 Summary of ULS check for the Pier Cap

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	4844.672	35887.4	8476.3
At deff from face	2885.512	35887.4	6586.9
At L/2	793.518	35887.4	1713.8

3.18.1.15. Shear Capacity of the section (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 143 Check for Shear Capacity (ULS) at various sections

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	789.4	971.523	Shear rein. Required
At deff from face	695.4	669.723	Provide min.
At L/2	565.7	552.524	Provide min.

3.18.1.16. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 144 Shear Reinforcement at various Sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final Shear Capacity (t)
At face of Shaft	971.523	16	8	120	991
At deff from face	669.723	16	8	120	884
At L/2	552.524	16	8	120	564

3.18.1.17. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 145 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.46	0.42	12	2	120
At deff from face	0.59	0.42	12	2	120
At L/2	0.88	0.42	12	2	120

3.18.1.18. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 146 Calculation of Cost of PPSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	99.28 m ³	8000	794240
Tensioned Steel	1846.32 kg	140	258485
Untensioned Steel	10004.9 kg	50	500244
Total			1552969

3.19. DESIGN OF RCC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM

3.19.1. Calculation of Forces

3.19.1.1. The depth of pier cap is assumed. In this case the depth assumed was 2.40m.

3.19.1.2. The distribution of live loads on all eight girders was calculated using the Courbon's method and the loads on all girders due the maximum moment case and the maximum reaction case were calculated. The values were as follows:

Table 3. 147 Reactions on Bearings on Left Side

Maximum Reaction Case	L1	L2	L3	L4	L5	L6	L7	L8
70RW	16.942	13.695	10.449	7.202	3.956	0.709	-2.537	-5.784
70RW+A	24.916	20.384	15.851	11.319	6.786	2.254	-2.279	-6.811
Class A3	26.514	21.916	17.318	12.719	8.121	3.523	-1.075	-5.673
Maximum Moment Case								
70RW	33.081	26.742	20.402	14.063	7.724	1.385	-4.954	-11.293
70RW+A	44.552	36.421	28.290	20.158	12.027	3.895	-4.236	-12.368
Class A3	39.683	32.801	25.919	19.037	12.155	5.273	-1.609	-8.491

Table 3.148 Reactions on Bearings on Right Side

[illegible]

3.19.1.3. Next, the dead load of the superstructure is calculated, since the depth of the surfacing, deck slab, girder is already assumed and or designed (practically).

3.19.1.4. The dead load and the live load calculated is multiplied by their respective load factors based on whether they are ULS, SLS rare, SLS QP (and of course multiplication of impact factor too in case of live load). The following load factors are used:

Table 3. 149 Load factors for Design

LOAD	ULS	SLS Load Factor	
		Rare	QP
Self-Weight	1.35	1	1
Superstructure Dead Load	1.35	1	1
SIDL Without Surfacing	1.35	1	1
SIDL With Surfacing	1.75	1	1
Live Load With Impact	1.5	0.75	0

3.19.1.5. The final forces are calculated at the face of the pier shaft, at the distance deff from the face of the pier shaft and at a distance of cantilever/2 from the face of the pier shaft.

3.19.1.6. The summary of worst case moments in all the cases is as follows:

Table 3. 150 Summary of Bending Moments at Face of Shaft

At the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	959.220	6042.039
SLS RARE	657.993	4058.603
SLS QP	558.276	3366.098

Table 3. 151 Summary of Bending Moments at d eff from Face of Shaft

At d eff from the face of the Shaft		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	670.006	3597.234
SLS RARE	452.777	2405.052
SLS QP	378.908	1972.617

Table 3. 152 Summary of Bending Moments at half the Cantilever Length

At L/2		
	SHEAR FORCE (t)	MOMENT (t-m)
ULS	539.227	1659.246
SLS RARE	371.908	1104.043
SLS QP	331.278	889.817

3.19.2. Design of Pier Cap

3.19.2.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 153 Material Properties of RCC Pier Cap

Grade of Concrete	M35
Grade of Reinforcement	Fe500
F _{yd}	0.6f _y
Clear Cover	40mm
Modulus of Elasticity of steel	200 GPa
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 50000/t

3.19.2.2. ULS Capacity Check

The check for maximum capacity of the member (ULS) check was done at the three locations and the summary is as follows:

Table 3. 154 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Number
At face of Shaft	6042.039	74472.33	80425	32	100
At deff from face	4058.603	52823.10	80425	32	100
At L/2	3366.098	29843.83	80425	32	100

3.19.2.3. Stress Check for SLS Load Combination

The stress coming in the member at the three locations was checked with respect the maximum values in the SLS Rare and SLS Quasi-Permanent cases with the result summary as follows:

Table 3. 155 Summary of Stress Check for SLS Load Combination

Section	Combina- -tion	Compressi ve stress (MPa)	Permissible Compressive stress (MPa)	Tensile stress (MPa)	Permiss- ible Tensile stress (MPa)
At face of Shaft	SLS Rare	16.331	16.800	245.965	300
	SLS QP	10.369	12.600	212.068	300
At deff from face	SLS Rare	12.411	16.800	171.662	300
	SLS QP	7.749	12.600	144.666	300
At L/2	SLS Rare	8.370	16.800	101.328	300
	SLS QP	5.101	12.600	82.510	300

3.19.2.4. Crack Width Check for SLS (QP)

According to the IRC 112: 2011, the crack width has to be checked only in the Quasi- Permanent case and that too excluding the effects of live load. The results of which are as follows:

Table 3. 156 Check for Crack Width

Section	Crack width at section (mm)	Permissible Crack Width (mm)
At face of Shaft	0.173	0.300
At deff from face	0.107	0.300
At L/2	0.048	0.300

3.19.2.5. Check For Shear (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 157 Check for Shear (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	372.09	969.220	Shear rein. Required
At deff from face	343.95	670.006	Shear rein. Required
At L/2	305.25	539.227	Shear rein. Required

3.19.2.6. Provision of Shear Reinforcement

In case of inadequate shear capacity, shear reinforcement is provided, summary of which for the three sections is as follows:

Table 3. 158 Shear Reinforcement provided at the Sections

Section	V_{ED} (t)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	969.220	16667.88	17233.88	16	12	140
At deff from face	670.006	13330.30	17233.88	16	12	140
At L/2	539.227	13591.71	17233.88	16	12	140

3.19.2.7. Design Reinforcement due to Torsion Only

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 159 Summary of Torsional Reinforcement at the sections

Section	T_{ED} (t-m)	Ast required (mm ²)	Ast provided (mm ²)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	244.200	1197.060	1615.676	12	2	140
At deff from face	244.200	1197.060	1615.676	12	2	140
At L/2	244.200	1197.060	1615.676	12	2	140

3.19.2.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 160 Calculation of Cost of RCC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	138.176 m ³	8000	1105408
Steel	24992.46 kg	50	1249623
Total			2355031

3.20. DESIGN OF PSC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM

3.20.1. Design of PSC Pier Cap

3.20.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are as follows:

Table 3. 161. Material Properties of PSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
First Stage Prestressing	7 days
Second Stage Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.20.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 162 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F_h (t)	F_v (t)
At l/2	337.8	0.550	334.1	-50.1
at d eff from left	344.6	0.400	344.6	0
at face of pier shaft	343.1	0.400	343.1	0

3.20.1.3. Stress Check with 0.9 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 0.9 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 163 Stress Check with 0.9 x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	366.9	336.6	240.2
	Bottom	13.3	64.2	76.4
Stress after laying Superstructure	Top	1120.9	778.0	403.9
	Bottom	194.0	323.7	552.9
Stress during Service at Infinity	Top	944.5	569.4	214.0
	Bottom	199.0	386.1	616.2

3.20.1.4. Stress Check with 1.1 x Prestress

According to IRC 112:2011, the stresses at the top and bottom of the member are considered by taking 1.1 times prestress to take into account any losses that might incur. The summary of the calculated losses at the various stages in the structure at various locations is as follows:

Table 3. 164 Stress Check with 1.1x Prestress

Section		At L/2 (t/m²)	At deff from face (t/m²)	At face of Shaft (t/m²)
Stage				
Stress after 1st stage of prestressing	Top	446.1	431.2	340.1
	Bottom	73.4	27.0	-23.3
Stress after laying Superstructure	Top	1415.1	1111.9	716.0
	Bottom	185.6	221.4	448.8
Stress during Service at Infinity	Top	1166.9	828.3	450.3
	Bottom	192.7	308.6	537.4

3.20.1.5. Check for Ultimate Limit State

After the section is designed and the prestressing force and the number of tendons have been ascertained, the section is checked considering a zero value of prestress and the entire amount of the prestressing steel as ordinary steel, to factor a case in which the prestressing force becomes zero or the tendons loosen proving no prestress at all. The summary of the check for Ultimate Limit State at the various sections is as follows:

Table 3. 165 Summary of ULS Capacity Check

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	6042.039	34860.0	11289.4
At deff from face	4058.603	34860.0	9561.4
At L/2	3366.098	34860.0	6941.4

3.20.1.6. Shear Capacity of the Section (ULS)

A shear capacity check is also undertaken to check whether shear reinforcement is required of minimum reinforcement is sufficient by incorporating the increase in the shear capacity because of the compressive force (by incorporating a factor of N_{ED}) and the summary is as follows:

Table 3. 166 Check for Shear Requirement (ULS)

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	1016.0	969.220	Provide minimum
At deff from face	901.6	670.006	Provide minimum
At L/2	750.0	539.227	Provide minimum

3.20.1.7. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 167 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.33	0.42	12	2	140
At deff from face	0.41	0.42	12	2	140
At L/2	0.30	0.42	12	2	140

3.20.1.8. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Table 3. 168 Calculation of Cost of PSC Pier Cap

Item	Quantity	Unit Cost	Total Cost
Concrete	137.7 m ³	8000	1101600
Tensioned Steel	3134.35 kg	140	438809
Untensioned Steel	7000.84 kg	50	350042
Total			1890450

3.21. DESIGN OF PPSC PIER CAP SUPPORTING EIGHT GIRDER SYSTEM

3.21.1. Design of PSC Pier Cap

3.21.1.1. Material Properties Used

The material properties used for the design of RCC pier cap are:

Table 3. 169 Material Properties for PPSC Pier Cap

Grade of Concrete	M50
Type of Cables	19K15
Stage of Prestressing	28 days
Area of one strand	140 mm ²
Area of one cable	2660 mm ²
Diameter of duct	107 mm
F _{ck} at first stage of prestressing	35MPa
F _{ck} at second stage of prestressing	50MPa
Number of sections checked	3
Mean value of axial tensile strength of concrete f _{ctm}	3.5 MPa
Characteristic Yield strength of Reinforcement	500 MPa
$\alpha_s = E_s/E_{cm}$	5.7
Prestressing force per cable	495.3t
Jacking force	371.5t
Cost of Concrete	Rs 8000/m ³
Cost of Steel	Rs 1400000/t

3.21.1.2. Summary of Prestressing Forces after Friction and Slip

The two instantaneous losses are friction and slip and the force at various locations is considered after factoring in the friction and slip losses, summary of which is as follows:

Table 3. 170 Forces at Sections after considering Friction and Slip Losses

SECTION	F (t)	CG (m)	F _h (t)	F _v (t)
At l/2	337.8	0.550	334.1	-50.1
at d eff from left	344.6	0.400	344.6	0
at face of pier shaft	343.1	0.400	343.1	0

3.21.1.3. Assume a value of crack width according to IRC 112:2011.

$w_k = 0.2 \text{ mm}$ for Prestressed members (exposure class- moderate)

3.21.1.4. Calculate the minimum value of A_s (untensioned steel).

$$A_{s \min} = \max\left\{0.26 \frac{f_{ctm}}{f_{yk}} b_t d, 0.0013 b_t d\right\}$$

(5.1)

$$\begin{aligned} \text{Minimum value of steel} &= \max\{14406 \text{ and } 10290\} \\ &= 14406 \text{ mm}^2 \end{aligned}$$

3.21.1.5. Assume value of A_s which is greater than $A_{s \min}$

Assumed $32\phi\text{-}20$ nos bars. Therefore $A_s = 16085 \text{ mm}^2$

3.21.1.6. IRC112:2011 mentions that the crack width has to be checked in the SLS frequent combination case. Hence the design moment = 4058.6 t-m .

3.21.1.7. Calculate Effective Reinforcement Ratio.

$$\rho_r = \frac{A_s}{A_{ceff}}$$

(5.2)

Where, A_{ceff} = Effective area of concrete in tension surrounding the reinforcement.

$$A_{ceff} = 2.5b(h - d) \quad \text{for beams}$$

(5.3)

$$A_{ceff} = 612000 \text{ mm}^2$$

$$\rho_r = 0.0263$$

3.21.1.8. Calculate the value of $\rho_{r \min}$ to compare the value of ρ_r .

$\rho_{r \min}$ is calculated by solving the following equation.

$$k_1 k_2 k_4 \phi \left(\frac{1}{\rho_{r \min}} \right)^2 + (k_3 c + k_1 k_2 k_4 \phi \alpha_e) \left(\frac{1}{\rho_{r \min}} \right) + \left(k_3 c \alpha_e - \frac{w_k E_s}{1.5 k_t f_{ctm}} \right) = 0$$

(5.4)

where, $k_1 = 0.8$

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

$k_t = 0.4$

Therefore,

$$\rho_{r \min} = 0.0216$$

$$\rho_r > \rho_{r \min} \quad (\text{OK})$$

3.21.1.9. Calculation of Stress in Steel (σ_s)

$$\sigma_s = \frac{w_k E_s}{s_{r \max}} + k_t f_{ctm} \left(\frac{A_{c,eff}}{A_s} + \frac{E_s}{E_{cm}} \right)$$

(5.5)

$$\text{Solving, we get,} \quad \sigma_s = 178 < 300 \text{ MPa} \quad (\text{OK})$$

3.21.1.10. To calculate the neutral axis depth, x we solve the following equation

$$-\frac{b}{2} x^3 + \frac{3bd}{2} x^2 + \frac{3\alpha_e M}{2\sigma_s} x - \frac{3\alpha_e M d}{\sigma_s} = 0$$

(5.6)

$$\text{Solving, we get,} \quad x = 761 \text{ mm}$$

3.21.1.11. Calculation of $A_{p \max}$

The maximum area of A_p is calculated to find the upper limit of the prestressing steel.

$$A_{p\max} = \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.7)$$

$$A_{p \max} = 93904 \text{ mm}^2$$

3.21.1.12. Calculation of Prestressing force F_p

$$F_p = \sigma_s \left(\frac{bx^2}{(2\alpha_e(d-x))} \right) - A_s \quad (5.8)$$

$$F_p = 16705 \text{ kN}$$

3.21.1.13. Corresponding to this F_p , a desired number of strands with a desired prestressing force is used.

In this case, 4 cables with 19 strands were required to cater to this F_p after taking into consideration all losses.

3.21.1.14. Check for Ultimate Moment Capacity (ULS)

The ultimate moment capacity is calculated by taking into account both the A_s and A_p .

Table 3. 171 Summary of ULS check for the Pier Cap

Section	M_{ED} (t-m)	A_{st} provided (mm^2)	Ultimate Moment Capacity (t-m)
At face of Shaft	6042.039	29805.0	9652.4
At deff from face	4058.603	29805.0	8174.9
At L/2	3366.098	29805.0	4798.8

3.21.1.15. Shear Capacity of the section (ULS)

According to the IRC 112: 2011, the Shear Capacity is checked in the ULS case and shear reinforcement is provided in case of inadequate shear capacity. The check for shear is demonstrated as follows:

Table 3. 172 Check for Shear Capacity (ULS) at various sections

Section	Shear Capacity (t)	V_{ED} (t)	Check
At face of Shaft	945.7	969.220	Shear rein. Required
At deff from face	831.8	670.006	Provide min.
At L/2	683.6	539.227	Provide min.

3.21.1.16. Calculation of Shear Reinforcement

The shear reinforcement if required is calculated at the various sections, summary of which is as follows:

Table 3. 173 Shear Reinforcement at various sections

Section	V_{ED} (t)	Diameter (mm)	Legs	Spacing (mm)	Final Shear Capacity (t)
At face of Shaft	969.220	16	8	140	1000
At deff from face	670.006	16	8	140	847
At L/2	539.227	16	8	140	575

3.21.1.17. Check for Torsion

According to the IRC 112: 2011, the check for torsional stress is carried out and reinforcement adequacy is checked. If adequate minimum reinforcement is provided, else, reinforcement is calculated. Summary of which is as follows:

Table 3. 174 Summary of Torsional Reinforcement at the sections

Section	Torsional Stress (MPa)	Permissible Stress (MPa)	Diameter (mm)	Legs	Spacing (mm)
At face of Shaft	0.33	0.42	12	2	140
At deff from face	0.41	0.42	12	2	140
At L/2	0.60	0.42	12	2	140

3.21.1.18. Calculation of Cost

The final calculation of cost is done considering the approximate per unit cost of concrete and steel, summary of which is as follows:

Item	Quantity	Unit Cost	Total Cost
Concrete	137.7 m ³	8000	1101600
Tensioned Steel	1238.57 kg	140	173400
Untensioned Steel	7000.84 kg	50	350042
Total			1625042

Table 3. 175 Calculation of Cost of PPSC Pier Cap

Chapter 4

RESULTS AND DISCUSSIONS

The following results can be gathered from the quantification and design of pier cap that we have done in the previous chapter. Various parameters like shear capacity, crack width, cost etc. are studied and results are summarised. Also, it is to be noted that the results for cost are for one single pier cap and not for an entire structure.

4.7 CRACK WIDTH

There is a permissible limit of crack width for the type of condition an RCC structure is exposed to given in IRC 112: 2012. And also, a PSC structure is generally made to have a zero crackwidth. Contrary to this, a PPSC structure is made on the basis of providing a limited crackwidth in the structure. It is based on the serviceability aspect of design. This reduces cost and increase the overall economy.

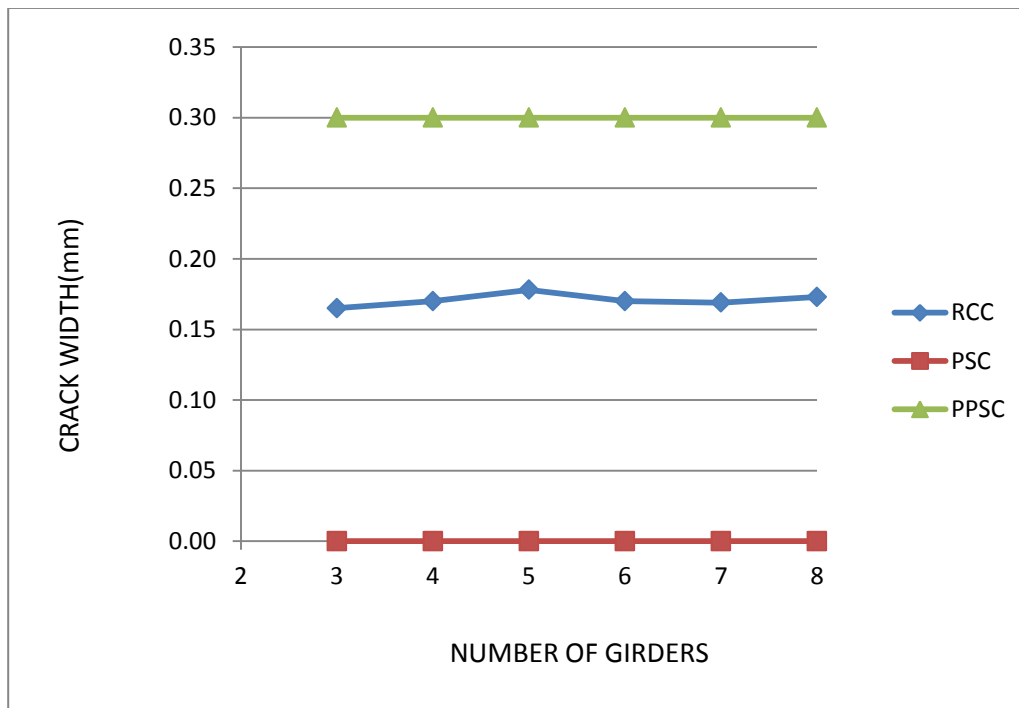


Figure 4.1 Crack width in RCC, PSC and PPSC Pier Cap for the most economical section

4.8 PARTIAL PRESTRESSING RATIO

Karayannis suggested that for a low imposed external bending moment a low value of PPR is obtained (lightly prestressed section) and that for a higher value of external moment a higher value of PPR is obtained.

This is verified by our design as when the loads increase, the PPR also increases. Further, he suggested that when the PPR ratio increases 0.9 and lies between 0.9 to 1.0, it is better to go for a fully prestressed section. In our cases, the PPR has never exceeded a PPR of 0.9, but it can be deduced that for a 9 or exceeding girder system, full prestress will be a better design.

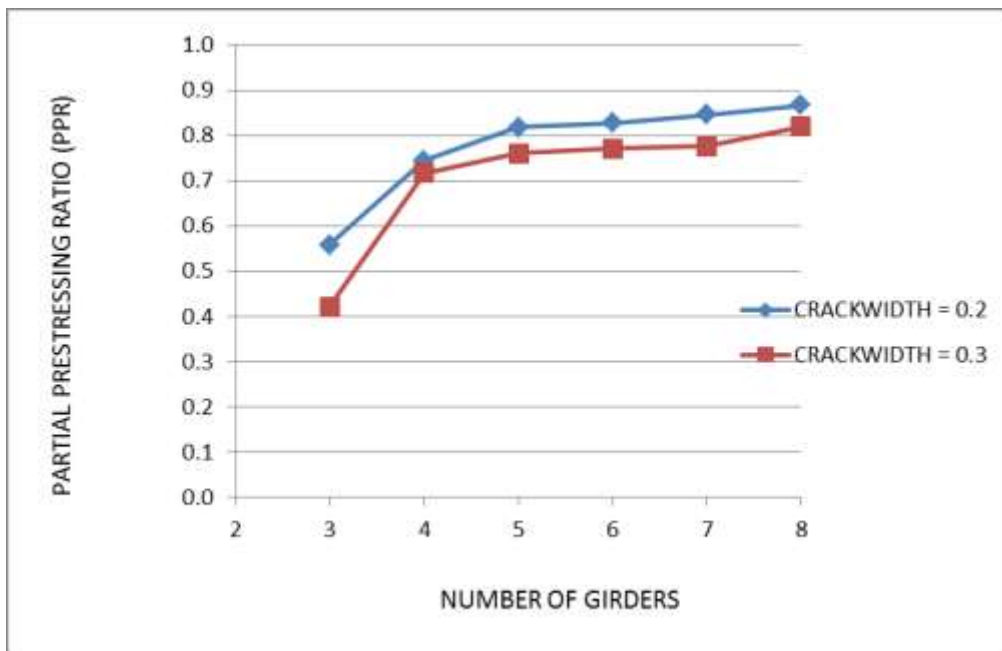


Figure 4.2 PPR for $w_k = 0.2\text{mm}$ and $w_k = 0.3\text{mm}$ in PPSC

4.9 DEPTH REQUIRED FOR MOST ECONOMICAL SECTION

The design depth for the most economical section for all the three designs and girder system is given.

Table 4.1 Design Depth for most economical section for RCC, PSC and PPSC Pier Cap

	RCC	PSC	PPSC
No.of Girders	Depth (m)		
3	1.30	1.20	1.20
4	1.50	1.30	1.30
5	1.70	1.40	1.35
6	2.10	2.00	1.90
7	2.20	2.20	2.00
8	2.40	2.40	2.20

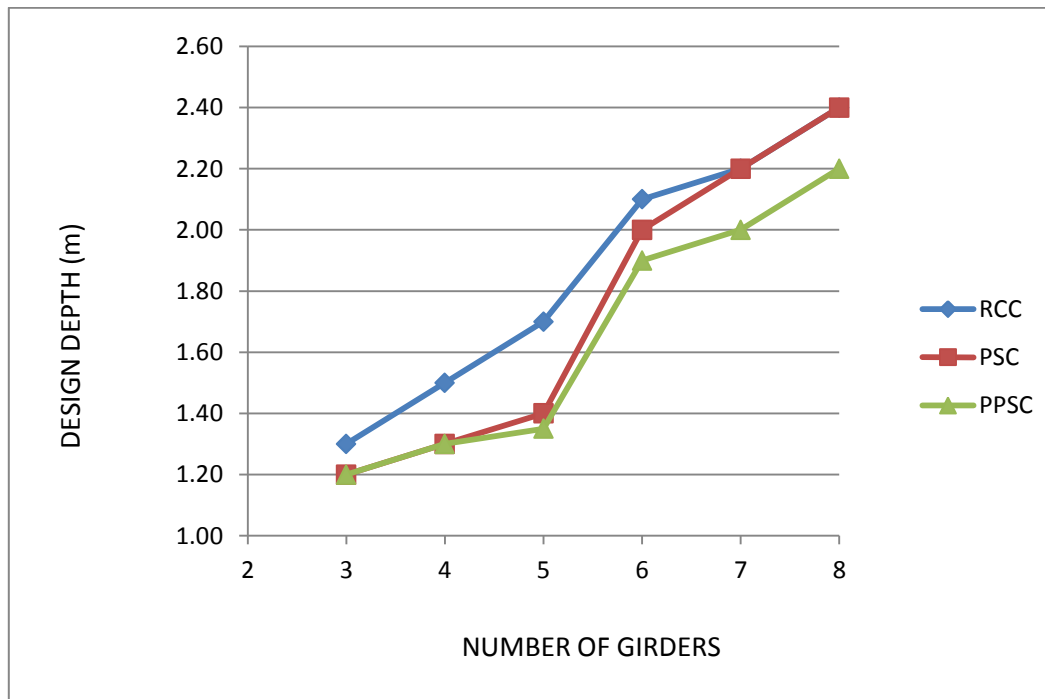


Figure 4.3 Variation in maximum depth for RCC, PSC and PPSC Pier Cap

It should be noted that the depth for both fully prestressed and partially prestressed member is lower as compared to the RCC section in the systems with lesser number of girders.

But contrary to the trend as the number of girders increase, due to the increasing loads, the prestressing force in a full prestressed structures increases. This leads to an increase in the compressive stresses in concrete. To counter this, the depth of section has to be increased. Hence, towards the end the depth of PSC is increased.

However, the depth of pier caps designed by partial prestressing, the depth of pier cap is minimum and hence most clear room can be provided as discussed initially in this research.

4.10 SHEAR CAPACITY

It is evident from the graphs and values below that the shear capacity of a member increases substantially when a prestressing force is applied to it. In this case we have taken three sections that is at the face of the pier shaft, at d_{eff} and at a distance $L/2$.

It is clear that there is almost 2 to 3 times increase in the shear capacity of each of the section due to the axial compressive force that is prestressing force. IRC 112: 2011 refers to this force as N_{ED} , that is the shear forces from external loads and prestressing forces.

The third deduction that can be made is that although cost has considerably decreased in the Partial prestressed section as compared to fully prestressed section, there is not a very substantial deduction in the shear capacity of the section.

4.10.1 Shear Capacity at half of cantilever portion

Table 4.2 Variation in Shear Capacity at L/2

No. Of Girders	RCC (t)	PSC (t)	PPSC (t)
3	155.68	321.1	297.1
4	199.96	443.1	411.5
5	235.22	522.7	477.5
6	281.44	623.3	578.7
7	307.74	671.9	618.9
8	339.14	750.0	683.6

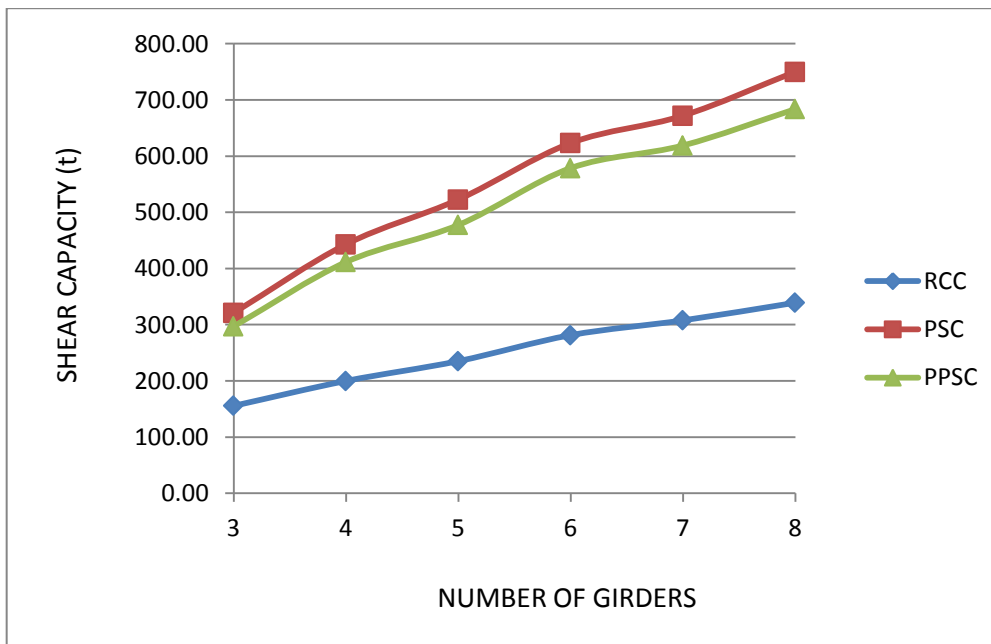


Figure 4.4 Variation in Shear Capacity at L/2 for RCC, PSC and PPSC Pier Cap

4.10.2 Shear capacity at effective depth

Table 4.3 Variation in Shear Capacity at d eff

No. Of Girders	RCC (t)	PSC(t)	PPSC(t)
3	163.11	399.9	375.7
4	217.38	505.5	474.1
5	260.23	610.7	564.7
6	310.39	727.3	680.9
7	345.39	801.4	747.9
8	382.14	901.6	831.8

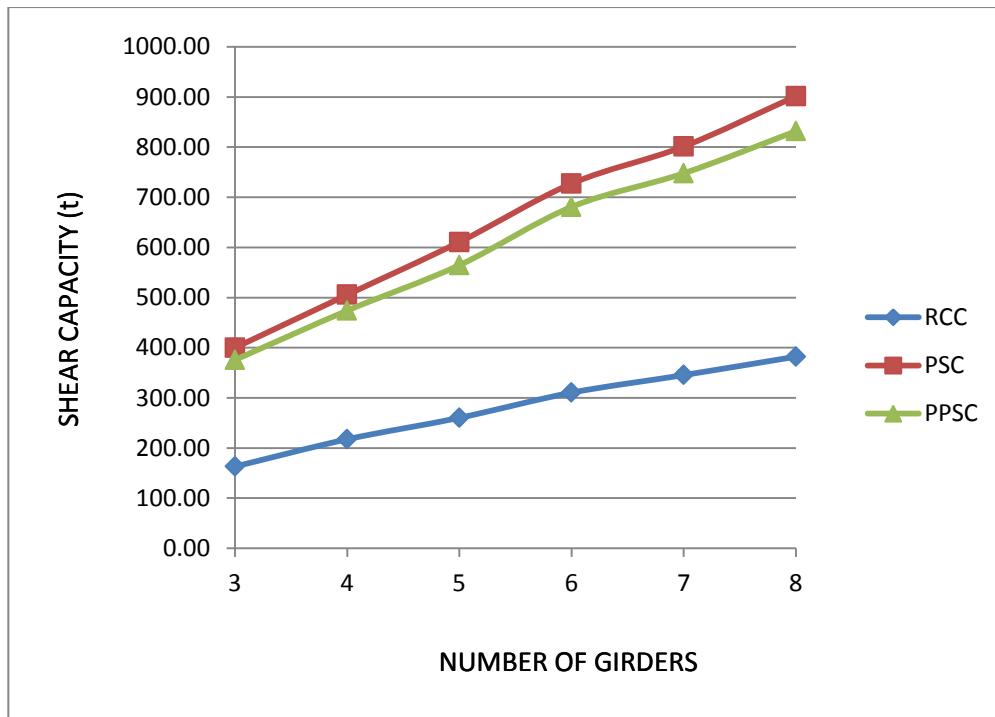


Figure 4.5 Variation in Shear Capacity at d eff for RCC, PSC and PPSC Pier Cap

4.3.3 Shear Capacity at the face of pier shaft

Table 4.4 Variation in Shear Capacity at face of Pier Shaft

No. Of Girders	RCC	PSC	PPSC
3	191.29	519.8	495.4
4	244.35	610.5	578.7
5	286.70	710.5	663.7
6	338.17	831.1	784.3
7	376.04	914.4	860.5
8	413.40	1016.0	945.7

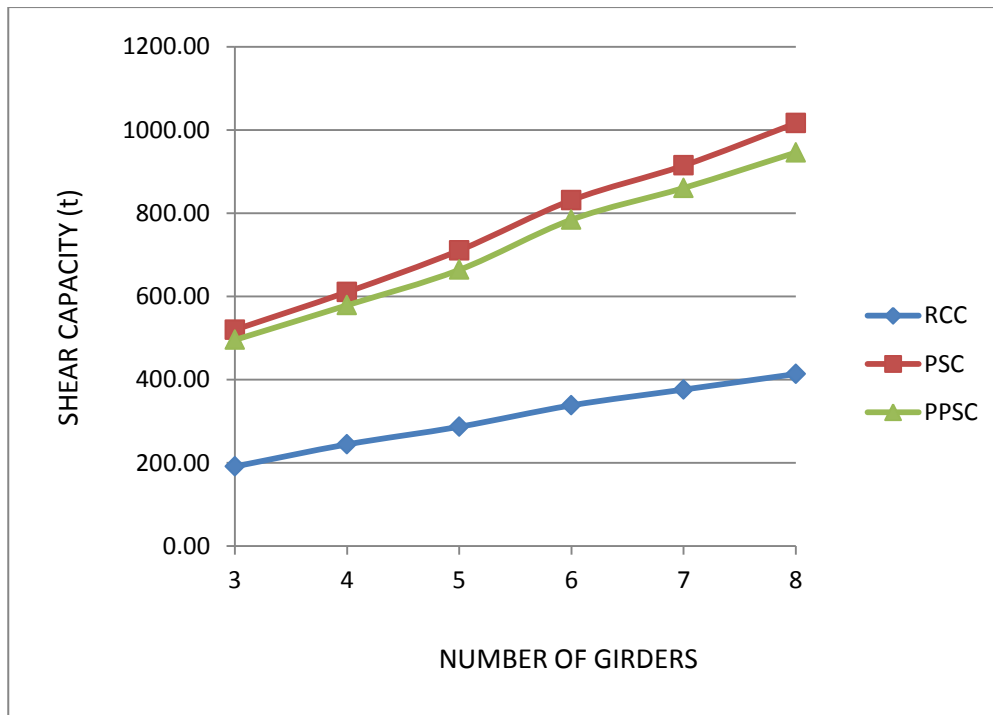


Figure 4.6 Variation in Shear Capacity at face for RCC, PSC and PPSC Pier Cap

4.11 COST COMPARISON

There is a significant increase in shear capacity of the section as the prestress increases and thus the sections are slender. Slender sections for larger spans help reduce the cost of concrete used in the member.

It is clear from the graph that as the width of pier cap increases (or as the span of carriageway increases), the RCC sections become uneconomical. A combination of untensioned and tensioned reinforcement gives the most economical section of the pier cap.

The graph shows that for the same girder system, the RCC design is cheaper only for three girder system. The difference is not as substantial for the four girder system. But beyond that there is almost a clear picture.



Figure 4.7 Cost Comparison of RCC, PSC and PPSC w.r.t. number of girders



Figure 4.8 Percentage decrease in Cost w.r.t. RCC

The graph shows an increasing trend in gap between the two systems as the span of pier cap increases, whether it's between PSC and PCC or PPSC and RCC.

Table 4.5 Approximate Cost of RCC, PSC and PPSC Pier Caps

Girder System	RCC (Rs)	PSC (Rs)	PPSC (Rs)
3	427178	496213	464588
4	660279	595897	575835
5	1013954	855788	788360
6	1456402	1196439	1105433
7	1892477	1742864	1552968
8	2355030	1890450	1625042

4.12 COST W.R.T CRACK WIDTH

Following are the difference in cost of Partially Prestressed systems which have different crack widths. It is natural that for a greater crack width that is of 0.3mm, the requirement of untensioned steel and prestressing force will be less. Hence we see a reduced cost in structures which allow a greater crack width.

The following table shows the difference in costs which might not seem substantial but is of the range 5 to 10 percent on an average.

Table 4.6 Cost Comparison in PPSC Pier Cap for $w_k = 0.2\text{mm}$ and $w_k = 0.3\text{mm}$

Crack width	Cost (Rs)	
	0.2 (mm)	0.3 (mm)
3	464588.0	391048.0
4	575835.0	551084.0
5	810899.0	781068.0
6	1105433.0	1093255.0
7	1552968.0	1512765.0
8	1625042.0	1574358.0

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