AN INDEPENDENT DESIGN CHECK OF THE PIER AT VIADUCT ON FEDERAL ROUTE FT180/001/40 WEST PORT – NORTH PORT, SELANGOR DARUL EHSAN

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FINAL REPORT

VOLUME II

REPORT ON DESIGN REVIEW

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VOLUME II

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1. INTRODUCTION

1.1. General

Kumpulan IKRAM Sdn Bhd (hereinafter referred to as "IKRAM") was commissioned by Jabatan Kerja Raya (hereinafter referred to as "JKR") to provide an independent design review and rehabilitation proposal for eight (8) nos. of pier which were reported cracks by JKR at viaduct on Federal Route FT180/001/40 West Port – North Port, Selangor.

Table 1. List of Affected Piers

PIER		PIER Type
ID	DES	
P-10A	Inverted "L"	P1-C
P-11A	Inverted "L"	P1-C
P-12A	Inverted "L"	P1-C
P-13B	Inverted "L"	P1-C
P-14B	Inverted "L"	P1-C
P-15B	Inverted "L"	P1-C
P-25	"T"	P1-A
P-33	"T"	P1-A

The scope of work includes the following:-

i. To carry out independent design review on the affected pier and crosshead structures which were designed and built in 1999 in accordance with the design version of bridge design codes i.e. BS 5400 and BD 37/88;

ii. To carry out detailed condition surveys, crack mappings and material testing on the affected piers;

iii. To propose rehabilitation or strengthening work design for the affected piers and crosshead.

1.2. Scope Of This Document

This document presents the assumptions, methodology and results of the independent design review. The desk-top study is based on the as-built drawings, design basis and information made available at the time for this design review. Three (3) piers, namely P11a, P25 and P33 have been selected for this design review.

2. REFERENCE DOCUMENTS, UNITS and ABBREVIATIONS

2.1. Design Codes and Standards

BS 5400-4:1990 – Part 4: Code of Practice for design of concrete bridges

Departmental Standard BD 37/88 – Loads for Highway Bridges

2.2. Design References

Ref #1: Strut-and-Tie Model for Deep Beam Design – A practical exercise using Appendix A of the 2002 ACI Building Code, *James K. Wight and Gustavo J. Parra-Montesinos*

Ref #2: Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. Appendix A – Strut-and-Tie Models

2.3. System of Units

The following units are used in the report presentation;

- *Length = m*
- *Tension / Compression Force = kN*
- *Tension / Compression Stress = N/mm^2*
- *Bending Moment = kN.m*
- *Shear Force = kN*
- *Section Ultimate Capacity (Bending) = kN.m*
- *Section Ultimate Capacity (Axial) = kN*
- *Crack Width = mm*
- *Reinforcement Area = mm^2*

2.4. Abbreviations

The following abbreviations are used in the report presentation;

- *FEM – Finite Element Model*
- *IKRAM - Kumpulan Ikram Sdn Bhd*
- *JKR – Jabatan Kerja Raya*
- *KEL – Knife Edge Load*
- *LHS – Left Hand Side*
- *MTAL – Medium Term Assessment Loading*
- *RHS – Right Hand Side*
- *STM – Strut and Tie Model*
- *SLS – Serviceability Limit State*
- *UDL – Uniform Distributed Load*
- *ULS – Ultimate Limit State*

3. INFORMATIONS SUPPLIED

The following reports and as-built drawings are made available for this study

3.1. Reports

RPT #1: "Laporan Pemeriksaan 8 Bilangan 'Pier' dan 'Crosshead' Jejambat FT180/001/40 di Laluan Pelabuhan Barat – Pelabuhan Utara, Daerah Klang, Selangor" by Bahagian Forensik (Struktur & Jambatan) Cawangan Kejuruteraan Awam, Struktur & Jambatan, Ibu Pejabat JKR Malaysia dated 22nd October 2010

3.2. As Built Drawings

Table 2. List of As Built Drawings

Item	Drawing Title	Drawing No.
1	Typical Cross Section From CH360 to CH550	KPKR/J/R/129653/1/AM13
2	Typical Cross Section From CH720 to CH850	KPKR/J/R/129653/1/AM15
3	General Layout (1)	KPKR/J/R/129653/1/ST1A
4	General Layout (2)	KPKR/J/R/129653/1/ST2A
5	General Layout (3)	KPKR/J/R/129653/1/ST3A
6	Piles Layout (1)	KPKR/J/R/129653/1/ST9
7	Piles Layout (2)	KPKR/J/R/129653/1/ST10
8	Pier Type P1-A – Concrete	KPKR/J/R/129653/1/ST17B
9	Pier Type P1-C – Concrete	KPKR/J/R/129653/1/ST19A
10	Pier Type P1-A – Reinforcement	KPKR/J/R/129653/1/ST23
11	Pier Type P1-C – Reinforcement	KPKR/J/R/129653/1/ST25
12	Precast Prestressed M-Beam	KPKR/J/R/129653/1/ST28A
13	Deck Slab (1)	KPKR/J/R/129653/1/ST32B
14	Deck Slab (2)	KPKR/J/R/129653/1/ST33B
15	Deck Slab (3)	KPKR/J/R/129653/1/ST34B
16	Elastomeric Bearing	KPKR/J/R/129653/1/ST12A
17	Standard Expansion Joints	KPKR/J/R/129653/1/ST13A

4. DESIGN REVIEW METHODOLOGY

4.1. Approach

There are eight (8) piers which were reported cracks by JKR. The details of the affected structures are summarized as below:-

Table 3. The Affected Pier Information (8 Nos.)

PIER		Pier Type	Pier dia. (m)	Cross-head Depth (m)	Piling	SPAN LENGTH / PRECAST BEAM TYPE	
ID	TYPE					LHS (m)	RHS (m)
P-10A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M
P-11A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M
P-12A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M
P-13B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.17m / 2^n-UM + 6^n-M	21.85m / 2^n-UM + 6^n-M
P-14B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.85m / 2^n-UM + 6^n-M	21.83m / 2^n-UM + 6^n-M
P-15B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.83m / 2^n-UM + 6^n-M	21.78m / 2^n-UM + 6^n-M
P-25	"T"	P1-A	3.0	3.5	6-Ø1200	35.05m / 12^n-U	28.05m / 2^n-UM + 14^n-M
P-33	"T"	P1-A	3.0	3.5	6-Ø1200	28.05m / 2^n-UM + 14^n-M	28.05m / 2^n-UM + 14^n-M

After examining their span configuration and structural form, P-11A, P-25 and P-33 have been selected for this review study.

Based on the as built drawings and design basis / criteria made available for this study, 3D analytical models were built for P-11A, P-25 and P-33 to investigate the maximum induced forces acting on the affected piers and crossheads. The forces are checked against two (2) limit stage conditions; Serviceability Limit State (SLS) and Ultimate Limit State (ULS). The sectional axial and bending capacities are computed and checked against the maximum forces from the analysis under ULS condition. The crack widths are computed based on the maximum force from the analysis under SLS condition.

Strut and Tie models (STM) and Finite Element Analysis (FEM) are performed to investigate the tension tie forces and localized tensile stresses distribution of the crossheads.

4.2. Loading

The following loads have been considered in this design review.

- Dead load
- Superimposed dead load
- Traffic live loads
- Wind load

4.2.1 Dead Load

Unit weight of precast post tensioned girder and in-situ deck slab is taken as 25.0kN/m^3.

- Precast M10 girder = 11.42kN/m
- Precast UM10 girder = 13.58kN/m
- Precast "U" girder = 18.66kN/m (for left span of P25 only)
- 160mm in-situ deck slab = 3.84kN/m^2

4.2.2 Superimposed Dead Load (SDL)

Unit weight of secondary RC element is taken as 24.0kN/m^3 and premix is taken as 22.6kN/m^3.

- 50mm premix = 1.13kN/m^2
- RC Parapet = 5.88kN/m (Edge)
  = 11.75kN/m (Central)

4.2.3 Traffic Live Load

Traffic live loads adopted for the study are as follows:-

- HA_{UDL} + KEL
- HA_{UDL} + 30 units HB (BD 37/88 cl. 6.3)
- 45 units HB (Unguided)
- JKR SV20 (Guided along centerline of carriageway)
- JKR Medium Term Axle Load (MTAL)

The traffic live loads used in accordance with BD 37/88 for 3 notional lanes are as follows:-

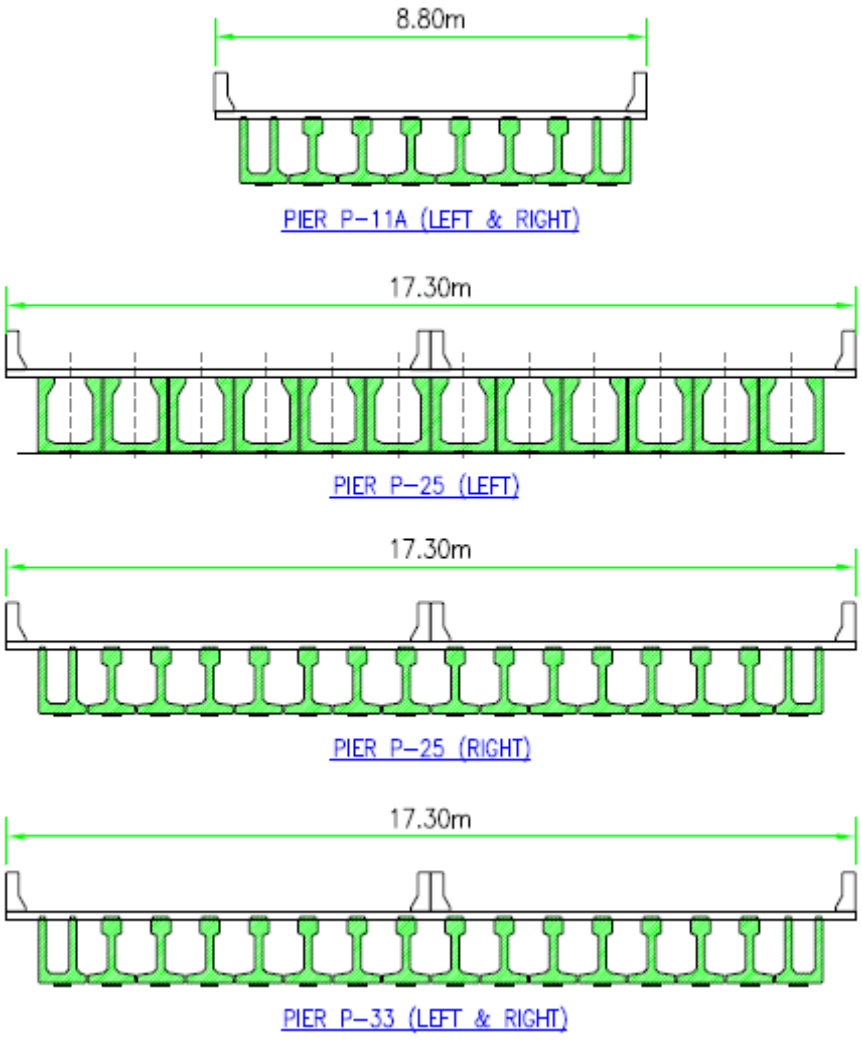


Figure 1. Typical section of bridge deck

Table 4. BD 37/88 (3 Notional Lanes) Vehicular Load Tabulation

Pier Type	Carriageway Width (m)	Notional Lane Width (m)	Loaded Length (m)	ULD (per lane)		KEL (kN)
				kN/m	kN/m^2	
P1-C (P-11A)	8.00	2.667	28.0	36.04	13.52	120.0
P1-A (P-25 Left)	7.85	2.617	35.0	31.03	11.86	120.0
P1-A (P-25 Right)	7.85	2.617	28.0	36.04	13.77	120.0
P1-A (P-33)	7.85	2.617	28.0	36.04	13.77	120.0

In accordance with *cl. 3.2.9.1* of BD 37/88, "*In the absence of raised kerbs it is the width between safety fences, less the amount of set-back required for these fences, being not less than 0.6m or more than 1.0m from the traffic face of each fence*". Based on this clause, a separate loading criterion is established for 2 notional lanes.

The traffic live load used in accordance with BD 37/88 for two (2) notional lanes is as follows:-

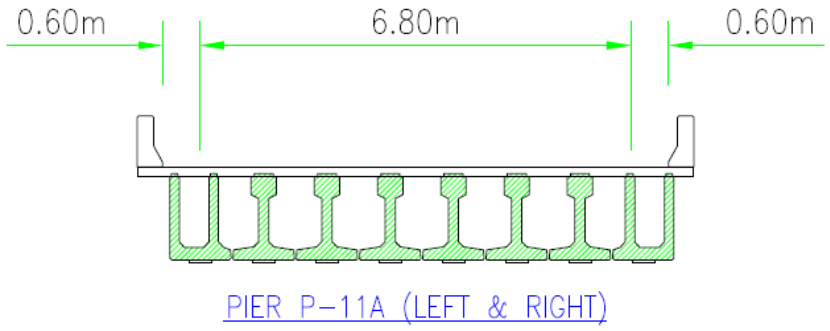


Figure 2. BD 37/88 (2 Notional Lanes) configuration

Table 5. BD 37/88 (2 Notional Lanes) load distribution

Pier Type	Carriageway Width (m)	Notional Lane Width (m)	Loaded Length (m)	ULD (per lane)		KEL (kN)
				kN/m	kN/m^2	
P1-C (P-11A)	6.80	3.400	28.0	36.04	10.60	120.0

The traffic live load used in accordance with JKR MTAL for three (3) fixed 2.5m wide notional lanes is as follows:-

Table 6. JKR MTAL (3 Notional Lanes) load distribution

Pier Type	Carriageway Width (m)	Notional Lane Width (m)	Loaded Length (m)	ULD (per lane)		KEL (kN)
				kN/m	kN/m^2	
P1-C (P-11A)	8.00	2.500	28.0	27.00	10.80	100.0

*Remaining 0.5m width loaded with 5.0 kN/m^2

The following Table (7) summarized types of the traffic live load applied in this study.

Table 7. Summary of traffic live load analysis

Pier Type	BD 37/88		SV20	JKR MTAL (3 Notional Lanes)
	3 Notional Lanes	2 Notional Lanes		
P1-C (P-11A)	√	√	√	√
P1-A (P-25)	√		√	
P1-A (P-33)	√		√	

4.2.4 Wind Load

Transverse wind load is checked against Pier P-11A. The nominal transverse wind load is derived based on a maximum 3-sec wind gust speed of 32 m/s.

4.3. Material Strength

4.3.1 Concrete

Concrete grade adopted for the study are as follows;

- Precast post tensioned beams = C50
- Reinforced concrete crossheads & piers = C40

4.3.2 Reinforcement

Reinforcement bar adopted for the study is Type 2 deformed bar with minimum yield strength, f_y = 460N/mm^2.

4.4. Load Combinations

The applied load factors γ_{fl} and γ_{f3} for SLS and ULS shall be as follows:-

Table 8. SLS and ULS load factors

No.	Load Case	Combination 1		γ_{f3} ULS
		γ_{fL}		
		SLS	ULS	
1	SW	1.00	1.15	1.10
2	Deck Slab	1.00	1.15	1.10
3	SDL (Parapet)	1.00	1.20	1.10
4	Premix	1.20	1.75	1.10
5	HA+KEL	1.20	1.50	1.10
6	HA+HB30	1.10	1.30	1.10
7	HB45	1.10	1.30	1.10
8	SV20	1.10	1.30	1.10
9	MTAL	1.20	1.50	1.10

4.5. Ultimate Limit State (ULS) - Sectional Capacity Check

The ultimate capacity for the affected piers and crossheads are computed based on the as-built drawings made available for this study. These capacities are checked against the maximum load effects acting on the pier in Ultimate Limit State (ULS) to assess the adequacy of the design.

4.6. Serviceability Limit State (SLS) - Crack Width Check

The crack width for the affected piers and crossheads are computed based on the as-built drawings made available for this study. The crack widths are checked using the maximum force derived from load case HA+KEL in Serviceability Limit State (SLS). The allowable crack width shall be less than 0.25mm in accordance with *BS 5400 cl. 4.1.1.1.*

Table 9. BS 5400-4:1990 "Table 1 – Design Crack Widths *(cl. 4.1.1.1)*

Severe Concrete surfaces exposed to: driving rain or alternate wetting and drying	 Wall and structure supports remote from the carriageway Bridge deck soffits Buried parts of structures	0.25

5. DESCRIPTION OF STRUCTURES

Based on the as-built information made available for this study, the basic data of the affected piers and crossheads used for the analysis are summarized as follows;

Table 10. Pier description and configuration

PIER		Pier Type	Pier dia. (m)	Cross-head Depth (m)	Piling	SPAN LENGTH / PRECAST BEAM TYPE		H	
ID	TYPE					LHS (m)	RHS (m)	(m)	
P-10A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M	10.451	*EJ*
P-11A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M	10.472	
P-12A	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.05m / 2^n-UM + 6^n-M	28.05m / 2^n-UM + 6^n-M	10.497	*Fixed*
P-13B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	28.17m / 2^n-UM + 6^n-M	21.85m / 2^n-UM + 6^n-M	11.087	
P-14B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.85m / 2^n-UM + 6^n-M	21.83m / 2^n-UM + 6^n-M	11.087	*EJ*
P-15B	Inverted "L"	P1-C	2.5	2.5	4-Ø1200	21.83m / 2^n-UM + 6^n-M	21.78m / 2^n-UM + 6^n-M	10.573	
P-25	"T"	P1-A	3.0	3.5	6-Ø1200	35.05m / 12^n-U	28.05m / 2^n-UM + 14^n-M	8.961	*EJ*
P-33	"T"	P1-A	3.0	3.5	6-Ø1200	28.05m / 2^n-UM + 14^n-M	28.05m / 2^n-UM + 14^n-M	8.961	*EJ*

The working capacity of the Ø1200mm bored pile is 6,000kN.

After examining the span configuration and structural form for the eight (8) cracked piers, P-11A, P-25 and P-33 have been selected for this review study.

5.1. Pier P-11A (Type P1-C)

P-11A is an inverted “L” shape pier supporting 28.05m long precast girders on the left and right. Each span consists of 6 nos. of precast M10 girder and 2 nos. of precast UM10 edge girder and supported by 4 nos. of 1200mm dia. bored pile. The working capacity of the 1200mm dia. bored pile is 6,000kN.

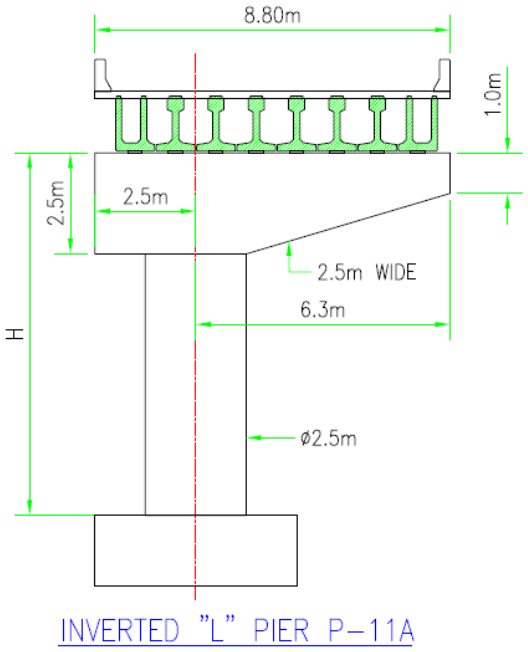


Figure 3. Pier P-11A (Type P1-C)

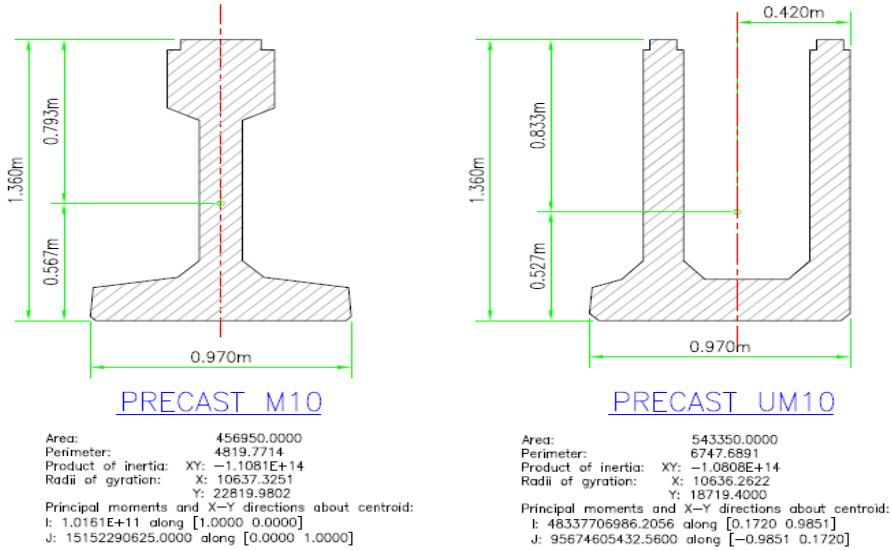


Figure 4. Pier P-11A (Type P1-C) precast beam

5.2. Pier P-25 (Type P1-A)

P-25 is a "T" shape pier supporting 35.05m span on the left and 28.05m span on the right. The left span consists of 12 nos. of precast U girder and the right span consists of 14 nos. of precast M10 girder and 2 nos. of precast UM10 edge girder and supported by 6 nos. of 1200mm dia. bored pile. The working capacity of the 1200mm dia. bored pile is 6,000kN.

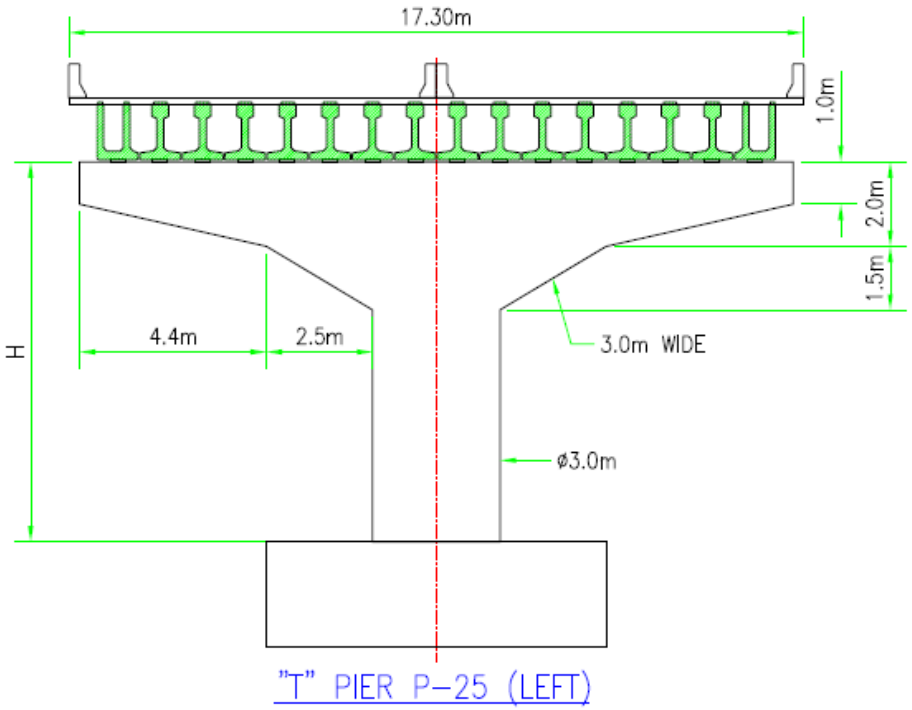


Figure 5. Pier P-25 (Type P1-A) – Left span

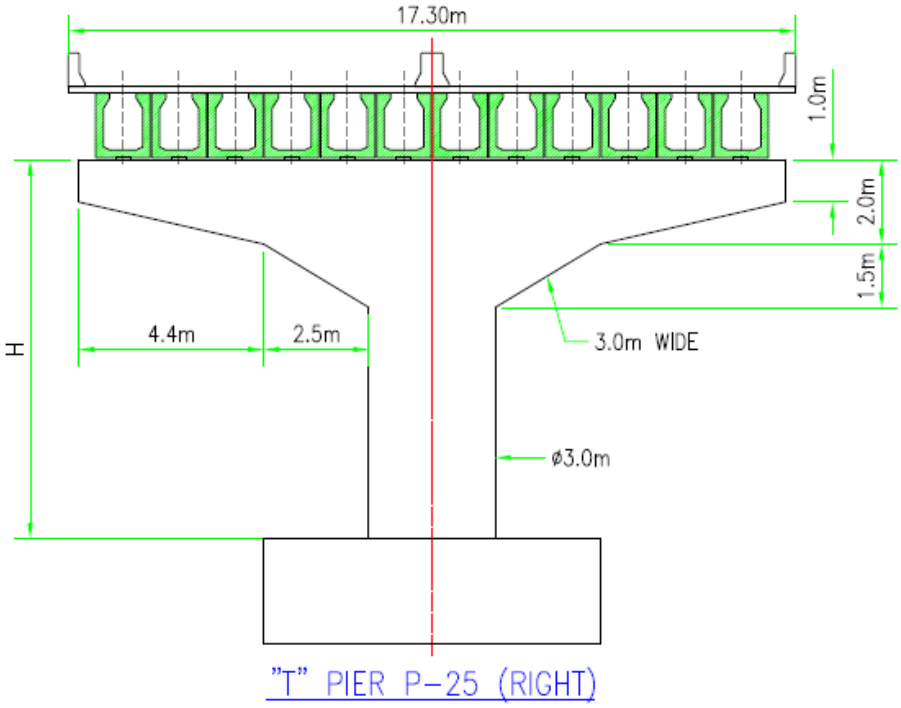


Figure 6. Pier P-25 (Type P1-A) – Right span

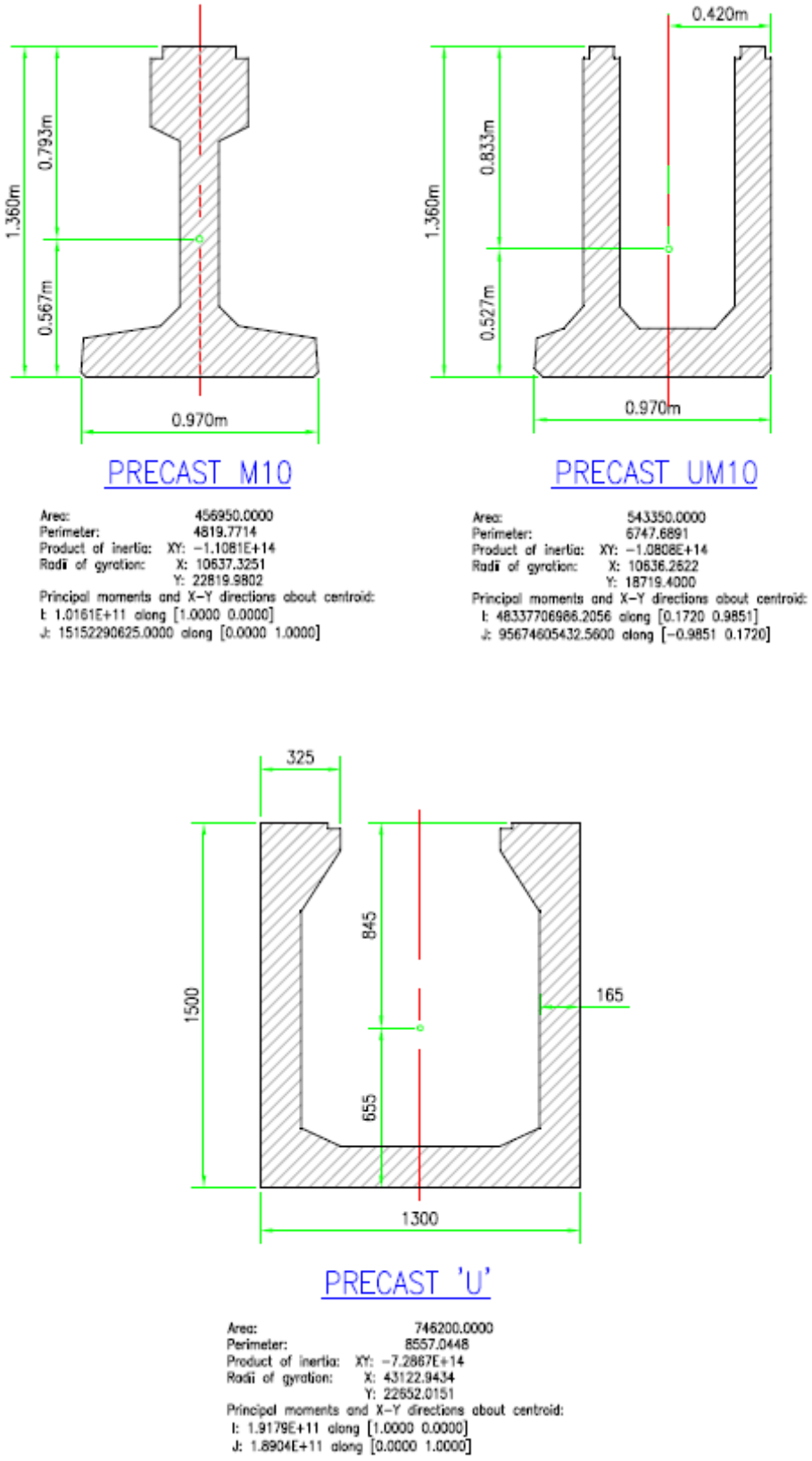


Figure 7. Pier P-25 (Type P1-A) precast beam section properties

5.3. Pier P-33 (Type P1-A)

P-33 is a “T” shape pier supporting 28.05m span on the left and right. Each span consists of 14 nos. of precast M10 beam and 2 nos. of precast UM10 beam and supported by 6 nos. of 1200mm dia. bored pile. The working capacity of the 1200mm dia. bored pile is 6,000kN.

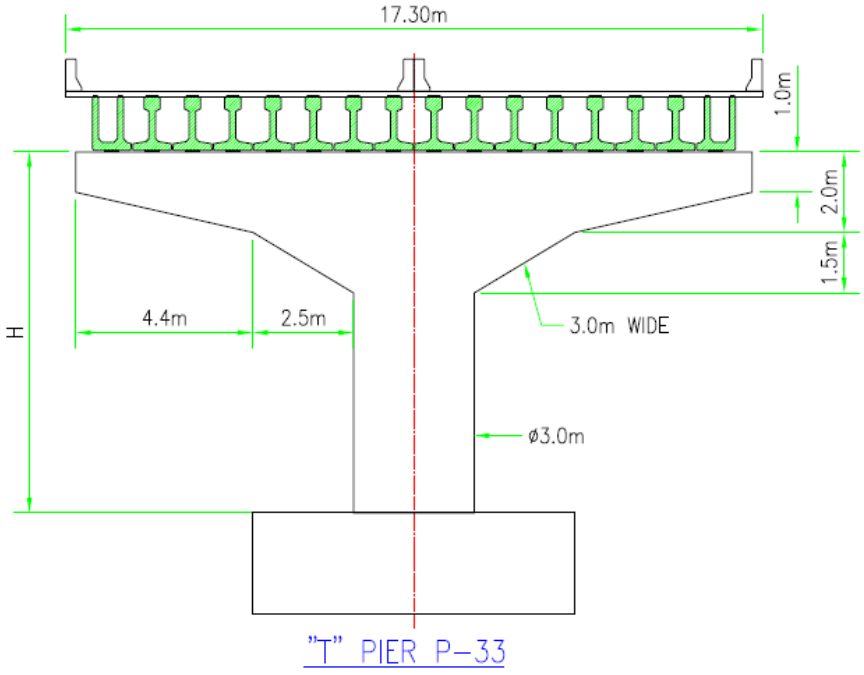


Figure 8. Pier P-33 (Type P1-A)

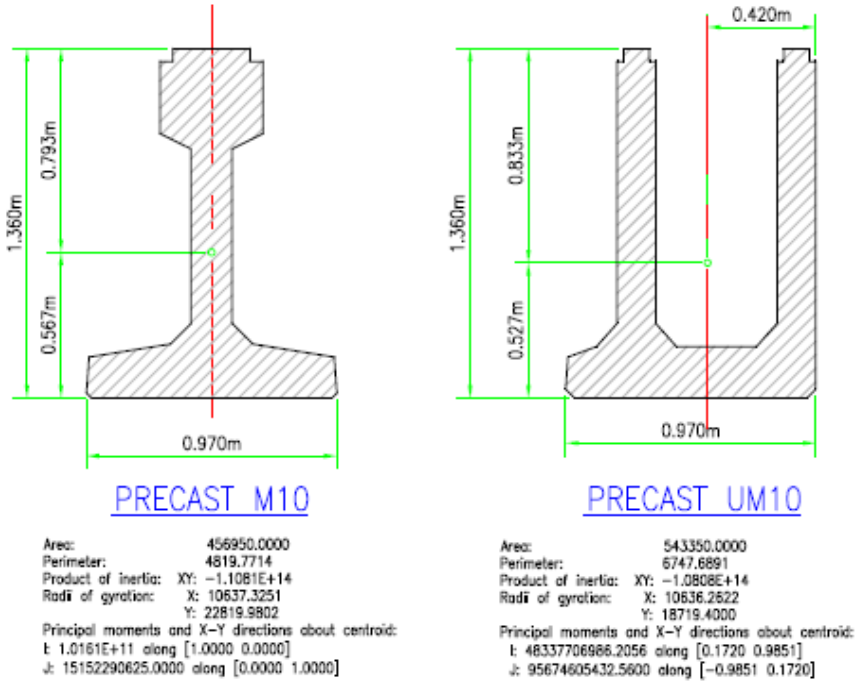


Figure 9. Pier P-33 (Type P1-A) precast beam section properties

6. 3D ANALYTICAL MODELS

6.1. Description of Analytical Model

a) 3D analytical models are established using analysis software StaadPro 2007 to determine the maximum combined load effects on the affected piers and crossheads under the service load conditions.

   Three (3) independent models are built for this study, i.e;

   - P-11A (Type P1-C) – Inverted "L" shape pier
   - P-25 (Type P1-A) – "T" shape pier
   - P-33 (Type P1-A) – "T" shape pier

b) 2D STM's (Strut and Tie Model) are built to check the shear critical structure or the D-regions of deep hammer pier type concrete structure.

   Two (2) independent STM models are built for this study, i.e;

   - P-11A (Type P1-C) – Inverted "L" shape pier
   - P-33 (Type P1-A) – "T" shape pier

c) 2D FEM's (Finite Element Model) are performed to investigate the localized stresses on the piers and crossheads.

   Two (2) independent FEM models are built for this study, i.e;

   - P-11A (Type P1-C) – Inverted "L" shape pier
   - P-33 (Type P1-A) – "T" shape pier

6.2. Pier P-11A (Type P1-C)

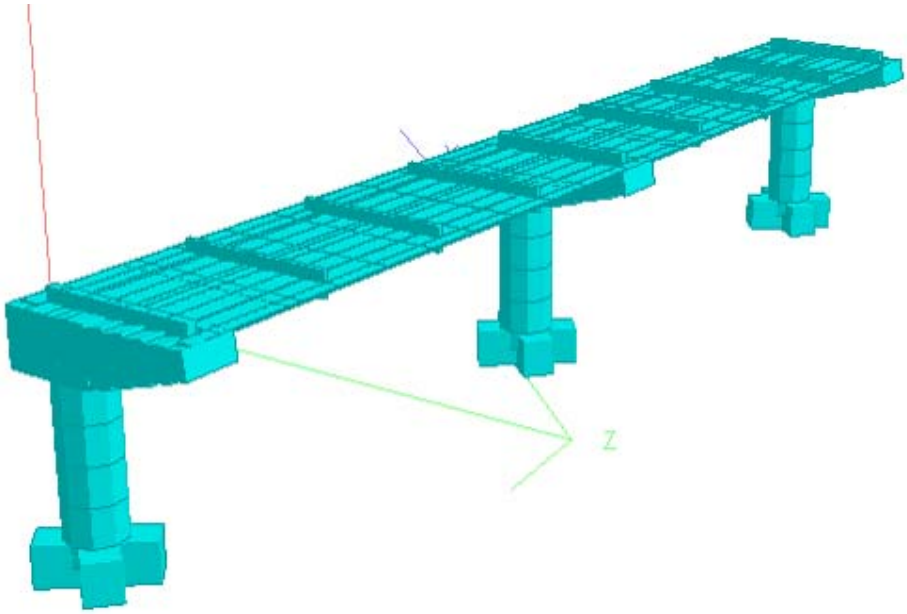


Figure 10. *P-11A model*

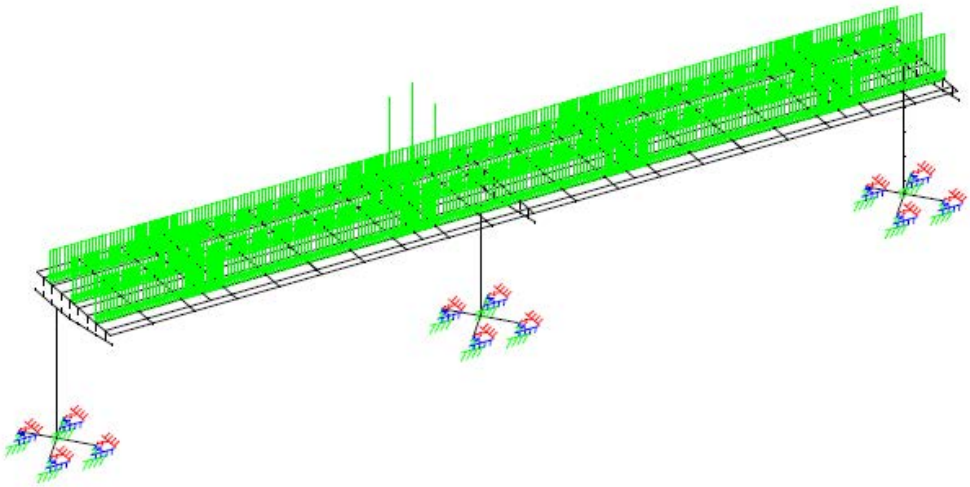


Figure 11. *P-11A model – HA+KEL*

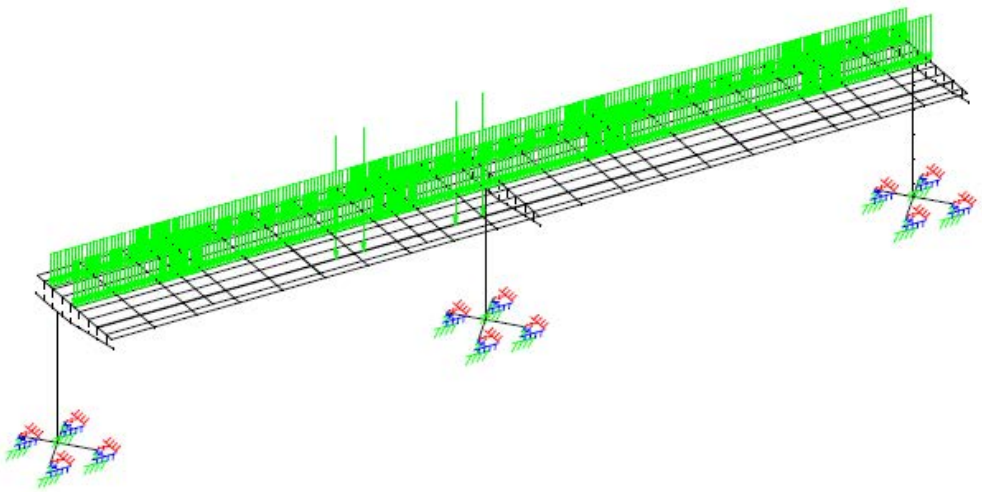


Figure 12*. P-11A model – Combined HB30 + HA*

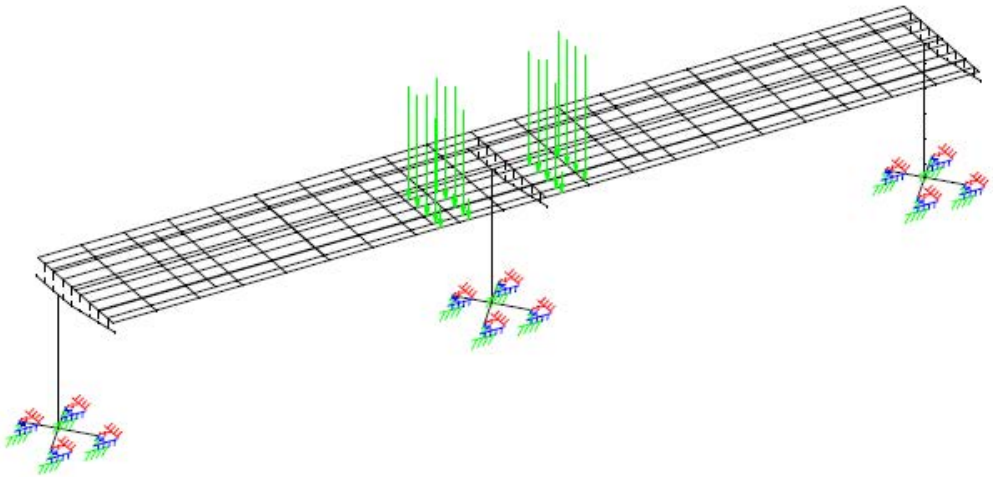


Figure 13*. P-11A model – HB45 Moving Load*

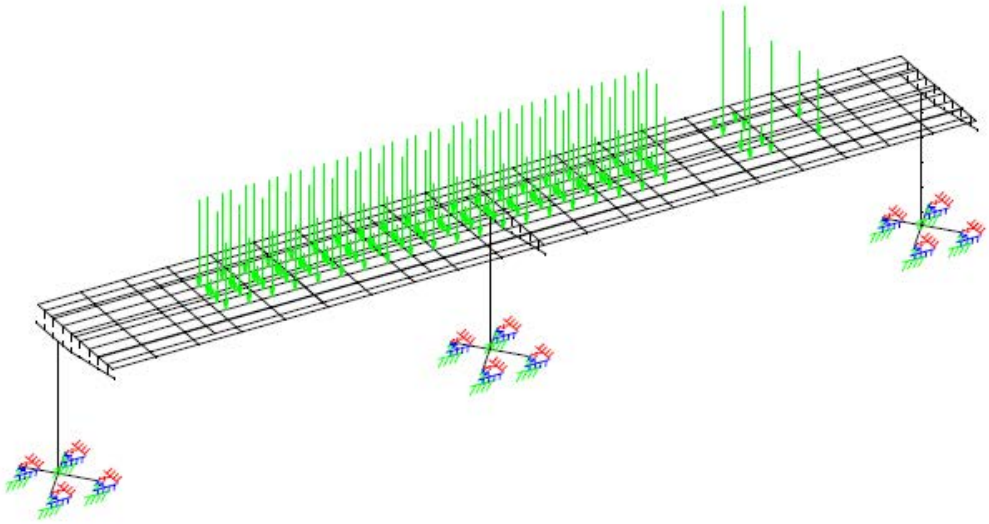


Figure 14*. P-11A model – SV20 Moving Load*

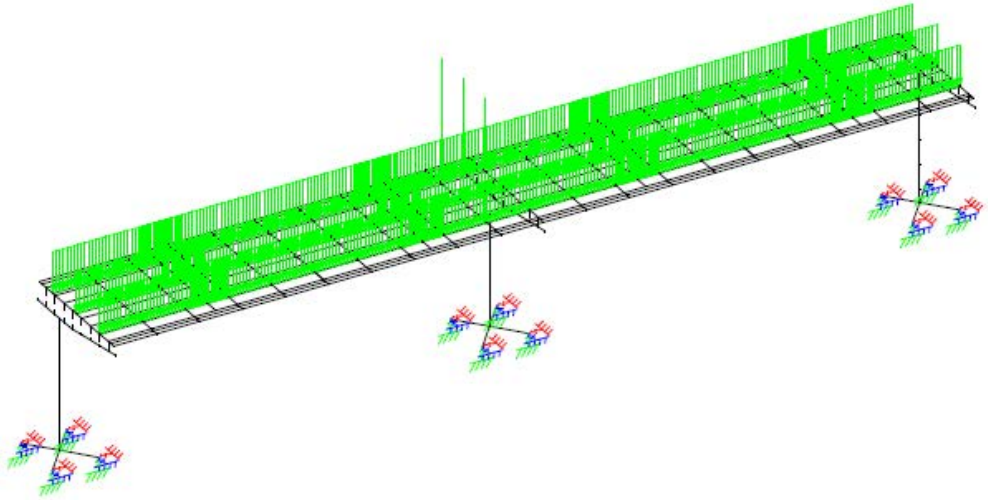


Figure 15*. P-11A model – JKR MTAL*

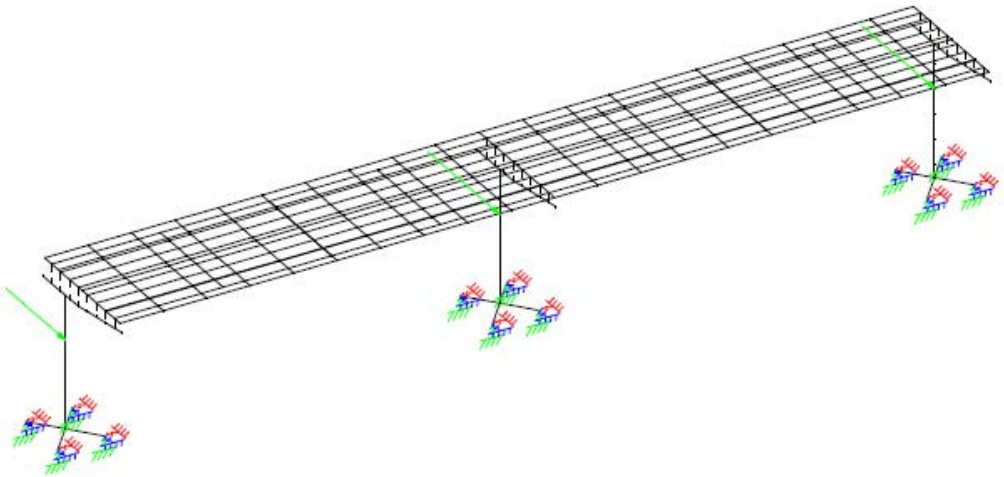


Figure 16*. P-11A model – Transverse Wind Load*

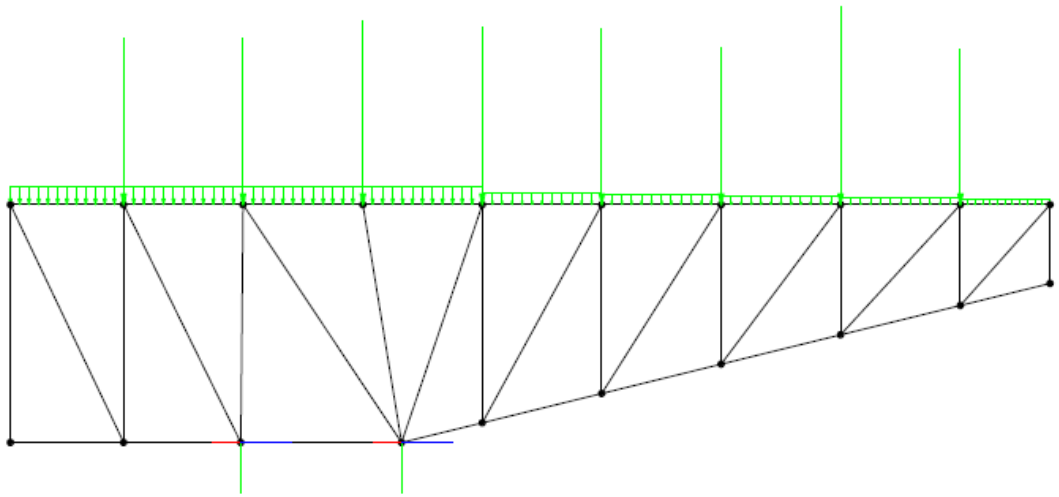


Figure 17*. P-11A model – STM Model*

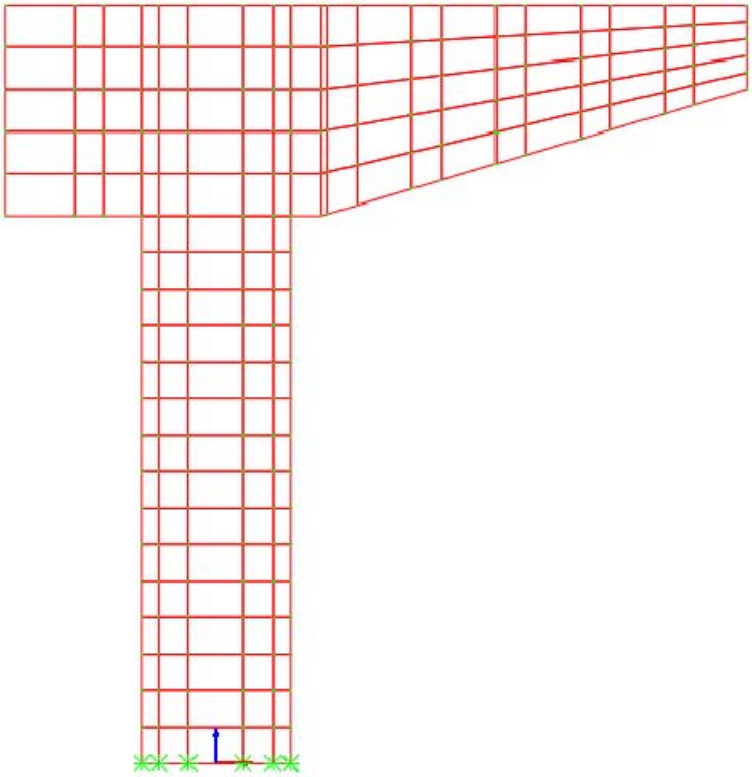


Figure 18*. P-11A model – FEM Model*

6.3. Pier P-25 (Type P1-A)

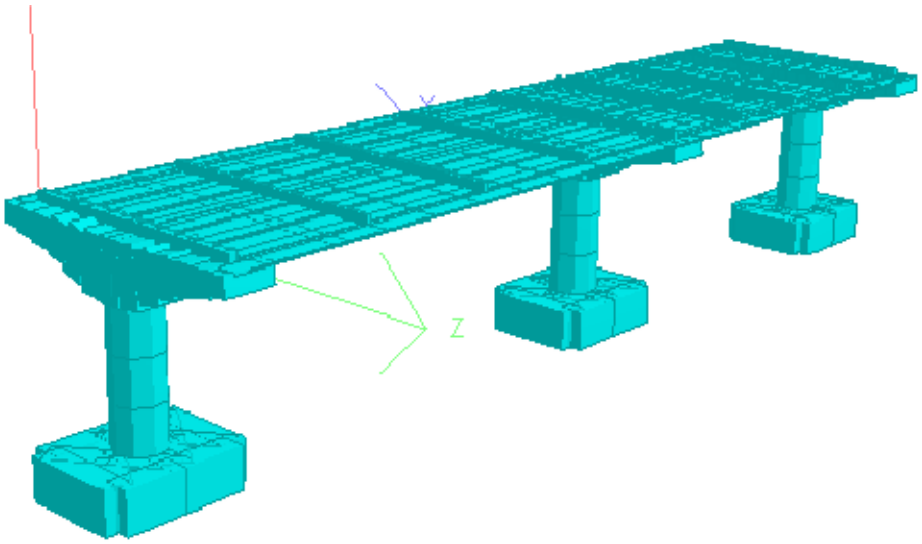


Figure 19*. P-25 model*

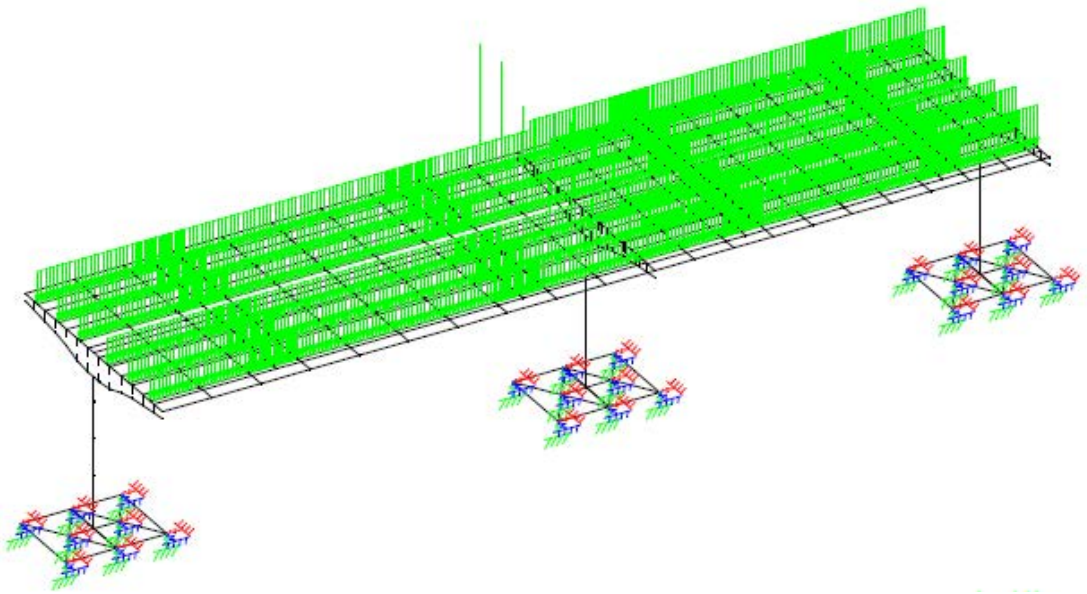


Figure 20*. P-25 model – HA+KEL*

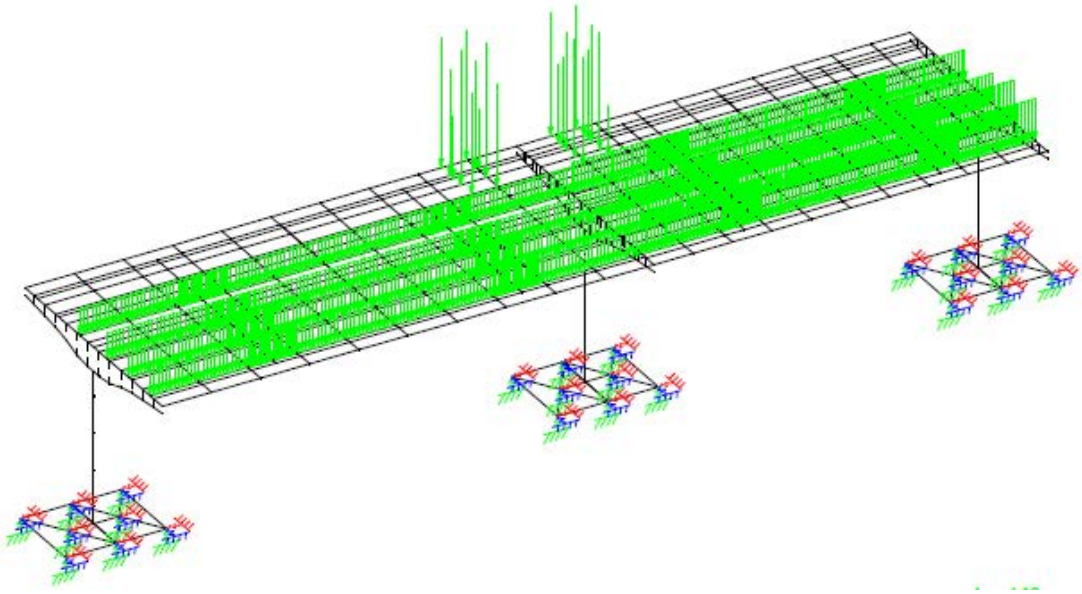


Figure 21*. P-25 model – Combined HA + HB30*

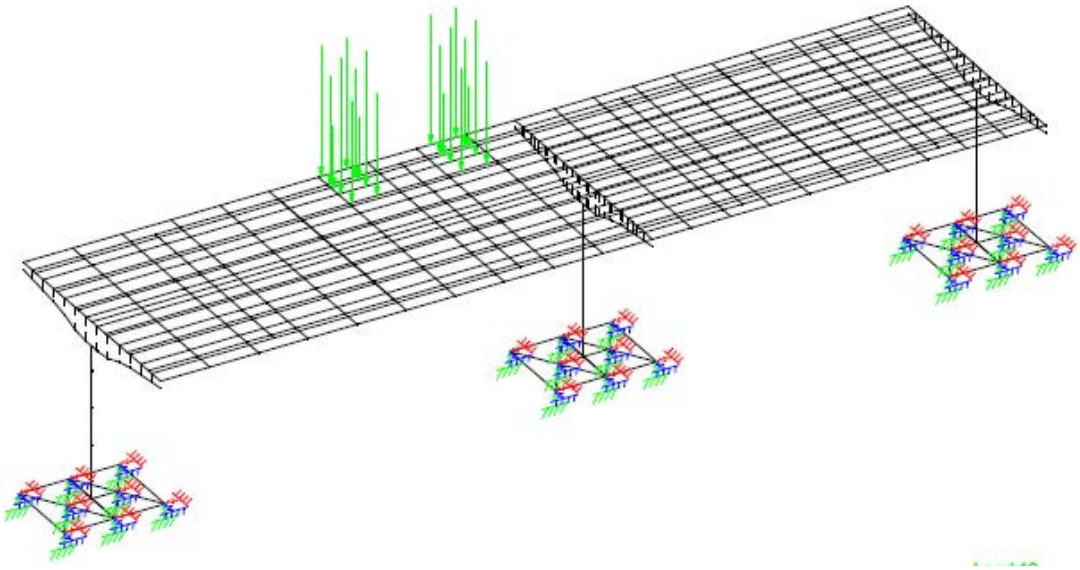


Figure 22*. P-25 model – HB45 Moving Load*

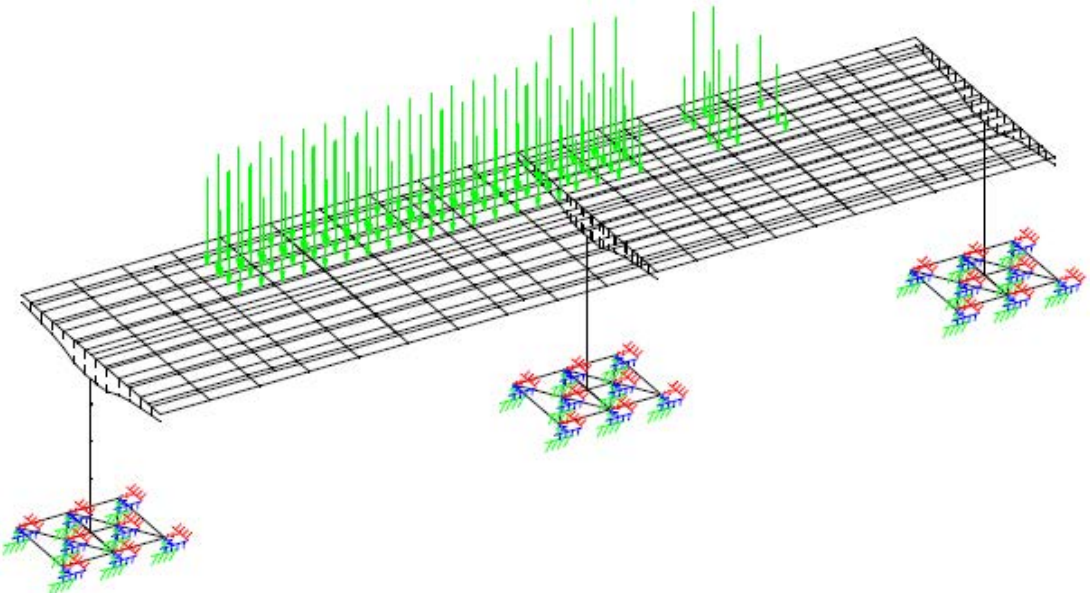


Figure 23*. P-25 model – SV20 Moving Load*

6.4. Pier P-33 (Type P1-A)

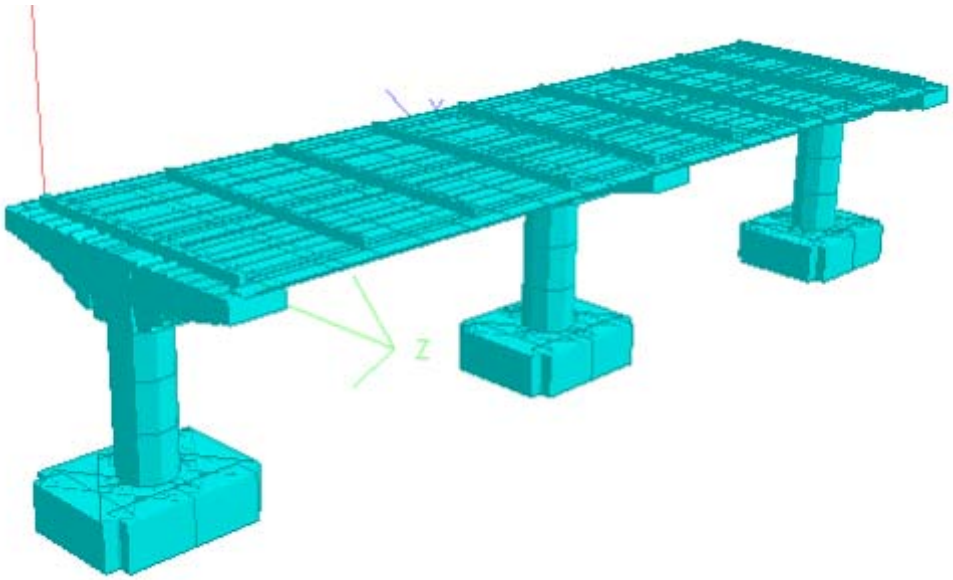


Figure 24. *P-33 Model*

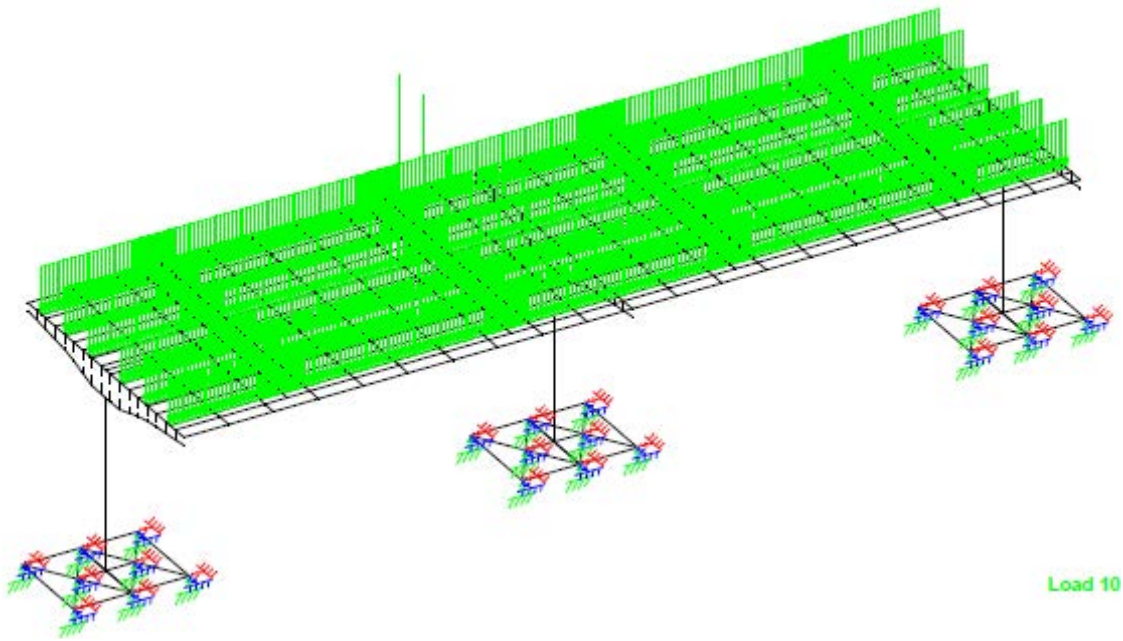


Figure 25. *P-33 Model - HA+KEL*

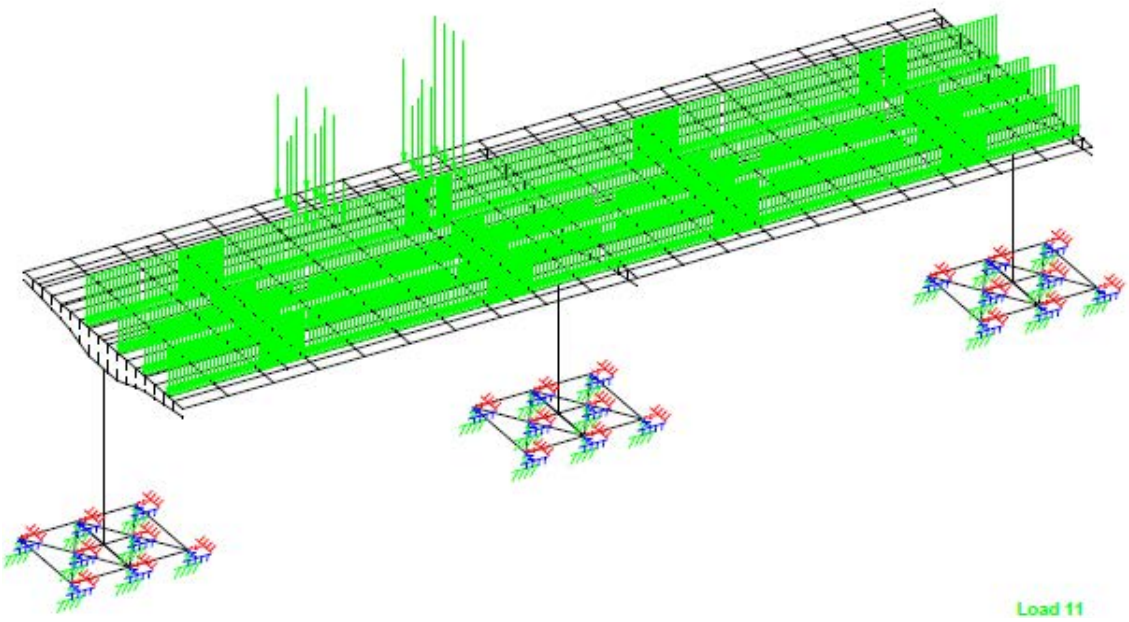


Figure 26. *P-33 Model – Combined HA + HB30*

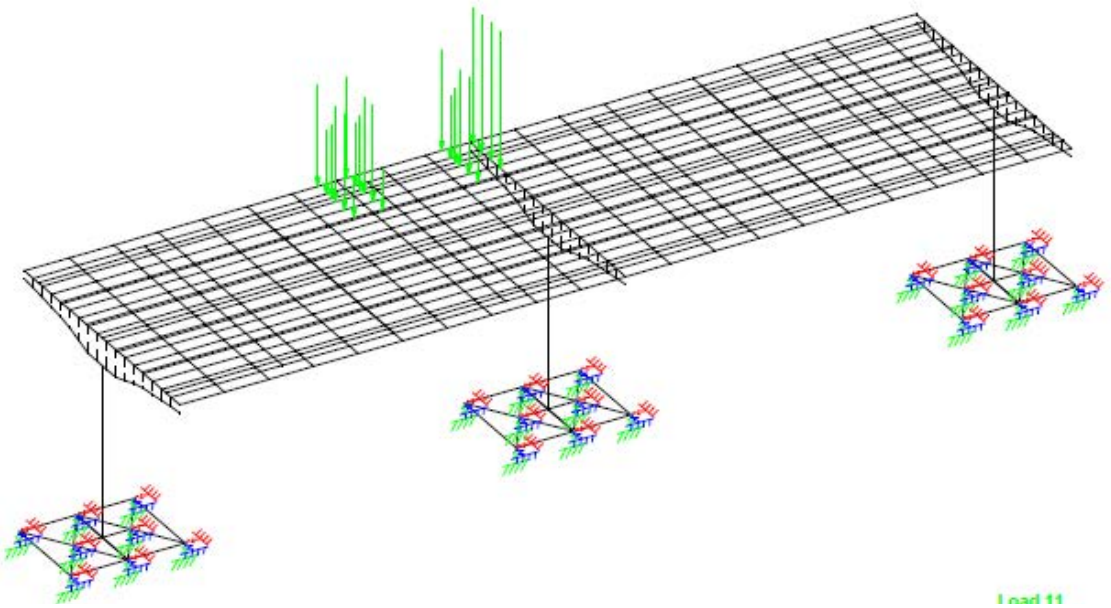


Figure 27. *P-33 Model – HB45 Moving Load*

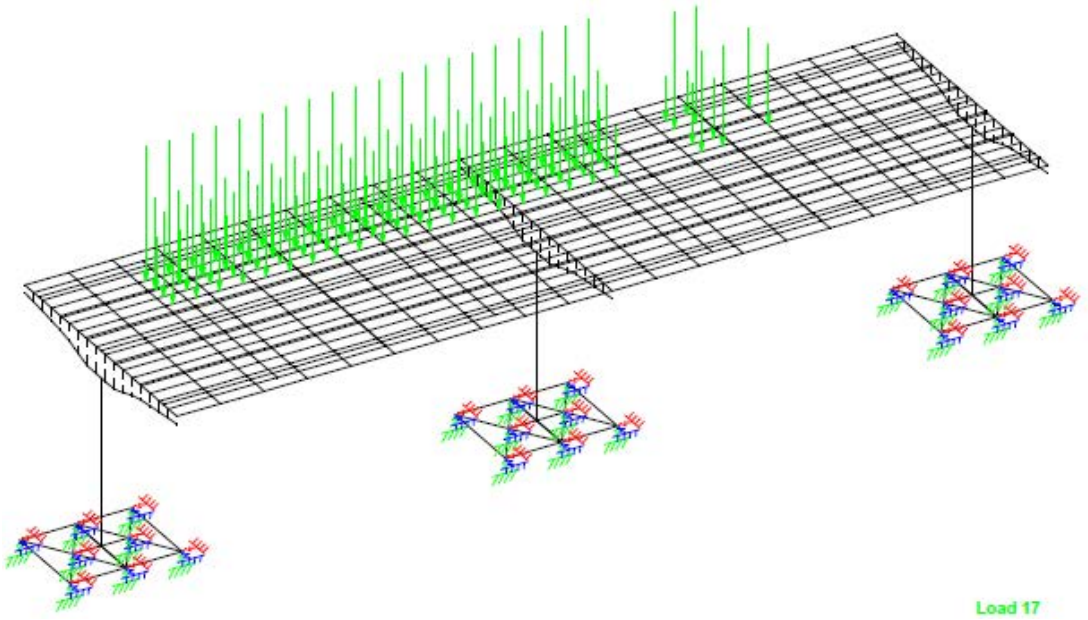


Figure 28*. P-33 Model – SV20 Moving Load*

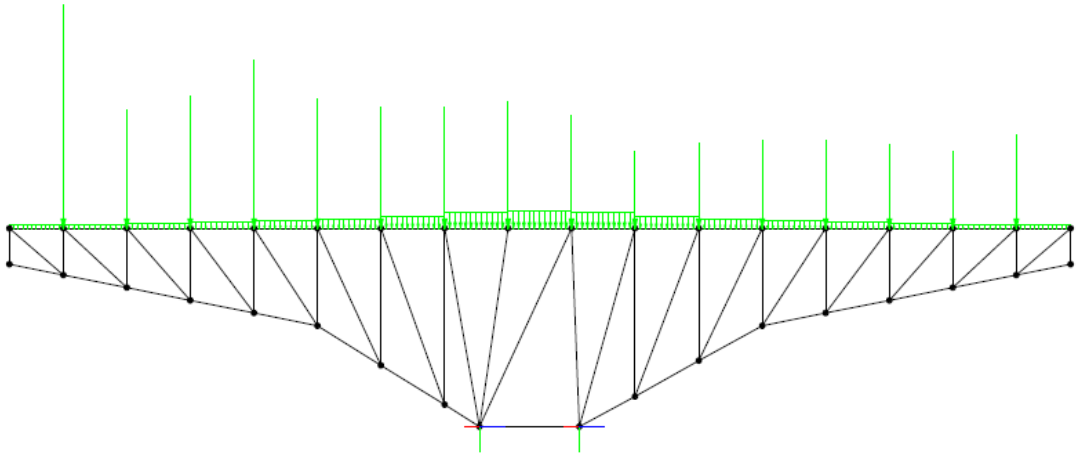


Figure 29*. P-33 Model – STM Model*

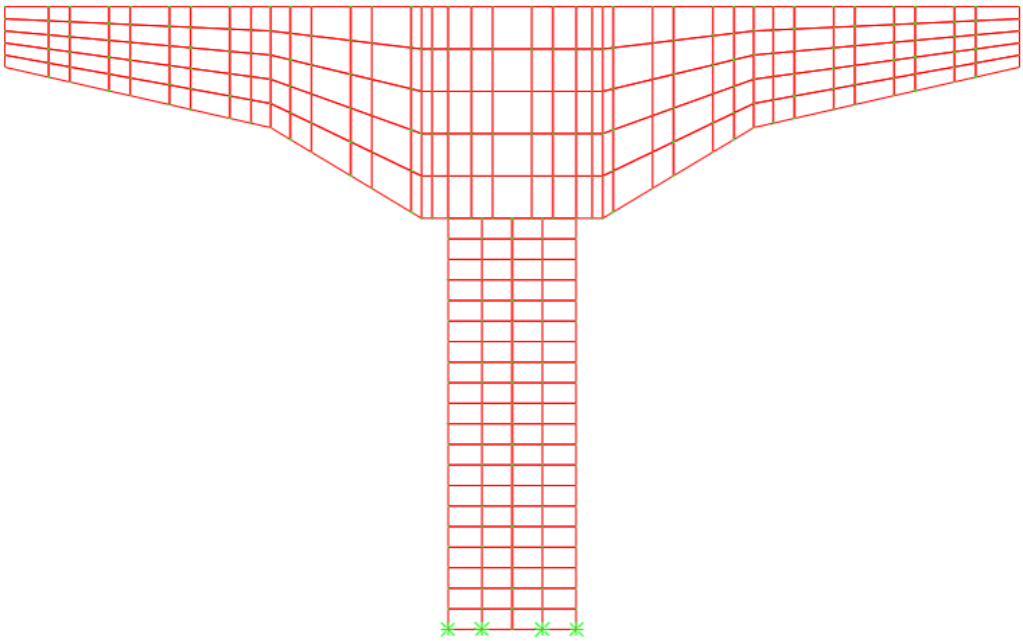


Figure 30. *P-33 Model – FEM Model*

7. DESIGN REVIEW FOR PIER P-11A (TYPE P1-C)

From the 3D analysis, the load effects under each load case can be obtained for P-11A.

7.1. Pier Column Check for Pier P-11A

The member forces for pier column are presented below for various load combinations. The design checks for pier column members under ULS and SLS are performed.

7.1.1 Analysis Results for Pier P-11A Column

The maximum design forces at pier column base are tabulated.

7.1.1.1 BD 37/88 (3 Notional Lanes)

Table 11. P-11A pier force – BD 37/88 (3 Notional Lanes)

No.	Load Case	N (kN)	M (kN.m)	Combination 1 γ_{fL}		γ_{f3}
				SLS	ULS	ULS
1	SW	5073	7192	1.00	1.15	1.10
2	Deck Slab	968	1900	1.00	1.15	1.10
3	SDL (Parapet)	549	782	1.00	1.20	1.10
4	Premix	285	559	1.20	1.75	1.10
5	HA+KEL	2574	5493	1.20	1.50	1.10
6	HA+HB30	2488	5225	1.10	1.30	1.10
7	HB45	1551	6140	1.10	1.30	1.10
8	SV20	3004	5551	1.10	1.30	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms, crosshead and column*

Table 12. P-11A pier force load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)	N_g (kN)	M_g (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	10020	17137	6931	10545	6592
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	9667	16292	6931	10545	5747
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	8637	17300	6931	10545	6755
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	10236	16652	6931	10545	6107

ULS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	13161	22674
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	12471	21081
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	11132	22391
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	13210	21549

7.1.1.2 BD 37/88 (2 Notional Lanes)

Table 13. P-11A pier force – BD 37/88 (2 Notional Lanes)

No.	Load Case	N (kN)	M (kN.m)	Combination 1		γ_{f3}
				γ_{fL}		
				SLS	ULS	ULS
1	SW	5073	7192	1.00	1.15	1.10
2	Deck Slab	968	1900	1.00	1.15	1.10
3	SDL (Parapet)	549	782	1.00	1.20	1.10
4	Premix	285	559	1.20	1.75	1.10
5	HA+KEL	2169	4039	1.20	1.50	1.10
6	HA+HB30	2002	4186	1.10	1.30	1.10
7	HB45	1549	5983	1.10	1.30	1.10
8	SV20	3004	5550	1.10	1.30	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms, crosshead and column*

Table 14. P-11A pier force load combination – BD 37/88 (2 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)	N_g (kN)	M_g (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	9534	15392	6931	10545	4847
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	9133	15150	6931	10545	4605
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	8635	17127	6931	10545	6582
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	10236	16649	6931	10545	6105

ULS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	12492	20274
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	11777	19596
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	11129	22166
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	13210	21546

7.1.1.3 JKR MTAL (3 Notional Lanes)

Table 15. P-11A pier force – JKR MTAL (3 Notional Lanes)

No.	Load Case	N (kN)	M (kN.m)	Combination 1 γ_{fL}		γ_{f3}
				SLS	ULS	ULS
1	SW	5073	7192	1.00	1.15	1.10
2	Deck Slab	968	1900	1.00	1.15	1.10
3	SDL (Parapet)	549	782	1.00	1.20	1.10
4	Premix	285	559	1.20	1.75	1.10
5	MTAL (UDL+KEL)	2572	4788	1.20	1.50	1.10
6	MTAL (5.0kPa)	70	131	1.20	1.50	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms, crosshead and column*

Table 16. P-11A pier force load combination – JKR MTAL (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)	N_g (kN)	M_g (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (MTAL)	10101	16448	6931	10545	5903

ULS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (MTAL)	13273	21726

7.1.1.4 Transverse Wind Load with BD 37/88 (3 Notional Lanes)

Table 17. P-11A pier force – BD 37/88 (Transverse wind load)

No.	Load Case	N (kN)	M (kN.m)	Combination 2		
				γ_{fL}		γ_{f3}
				SLS	ULS	ULS
1	SW	5073	7192	1.00	1.15	1.10
2	Deck Slab	968	1900	1.00	1.15	1.10
3	SDL (Parapet)	549	782	1.00	1.20	1.10
4	Premix	285	559	1.20	1.75	1.10
5	HA+KEL	2574	5493	1.00	1.25	1.10
6	HA+HB30	2488	5225	1.00	1.10	1.10
7	HB45	1551	6140	1.00	1.10	1.10
8	SV20	3004	5551	1.00	1.10	1.10
9	WIND	0	39	1.00	1.10	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms, crosshead and column*

Table 18. P-11A pier force load combination – BD 37/88 (Transverse wind load)

SLS Design to Load Combination 2

Case #	Load Combination	N (kN)	M (kN.m)
SLS1C2	(SW+Deck Slab+SDL+Premix) + (HA+KEL)		
SLS2C2	(SW+Deck Slab+SDL+Premix) + (HA+HB30)		
SLS3C2	(SW+Deck Slab+SDL+Premix) + (HB45)		
SLS4C2	(SW+Deck Slab+SDL+Premix) + (SV20)		

ULS Design to Load Combination 2

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C2	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	12453	21210
ULS2C2	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	11924	19979
ULS3C2	(SW+Deck Slab+SDL+Premix) + (HB45)	10791	21087
ULS4C2	(SW+Deck Slab+SDL+Premix) + (SV20)	12549	20374

7.1.2 Sectional Capacity Check (ULS) for P-11A Column

The pier section capacity is calculated based on the following as-built parameters;

- *Ø2500mm, f_{cu}=40MPa, 50-T32*

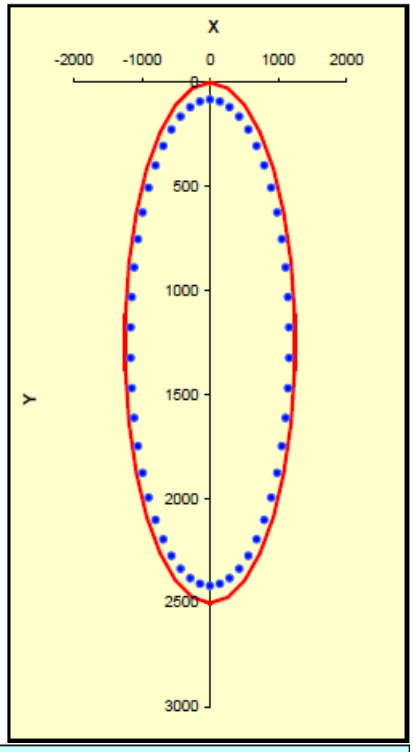
From the following design checks, all the applied ULS forces lies within the P-M interaction capacity envelope; hence it can be conclude that the existing design of pier column P-11A is adequate at ULS.

7.1.2.1 BD 37/88 (3 Notional Lanes)

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE P1 -C (3 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	4,872,930	mm²	: Area of concrete section
ΣAs=	40,212	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.83	%	
Ap/Ag=	0.00	%	
Pu,max=	90,360	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

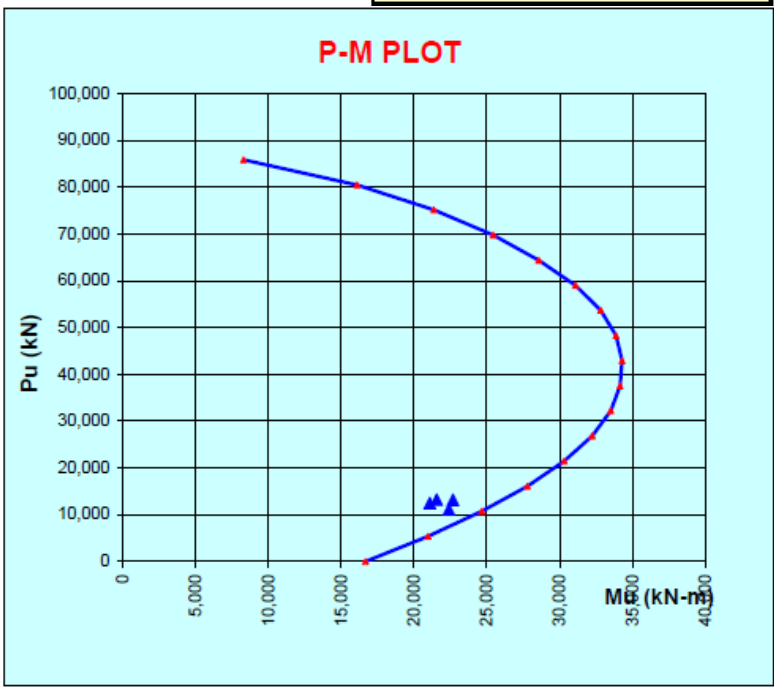
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	4,800,834	mm²	: Area of concrete in compression
Fc=	-76,813	kN	: Force on concrete
Fs=	-9,029	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.306E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	2389.04	mm	
Pu=	85,842	kN	: Ultimate Axial Force (+) compression
Mu=	8,274	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	16,658
2	0.06	5,365	20,959
3	0.12	10,730	24,664
4	0.18	16,095	27,782
5	0.24	21,461	30,308
6	0.30	26,826	32,235
7	0.36	32,191	33,522
8	0.42	37,556	34,155
9	0.48	42,921	34,279
10	0.53	48,286	33,872
11	0.59	53,651	32,821
12	0.65	59,017	31,076
13	0.71	64,382	28,564
14	0.77	69,747	25,424
15	0.83	75,112	21,362
16	0.89	80,477	16,089
17	0.95	85,842	8,274

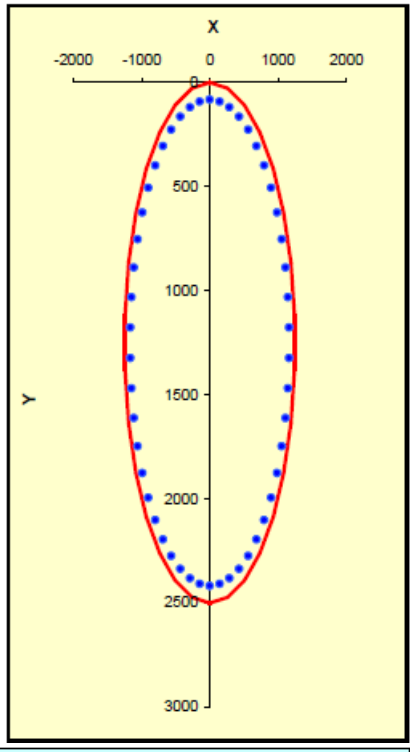


7.1.2.2 BD 37/88 (2 Notional Lanes)

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE P1 -C (2 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	4,872,930	mm²	: Area of concrete section
ΣAs=	40,212	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.83	%	
Ap/Ag=	0.00	%	
Pu,max=	90,360	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

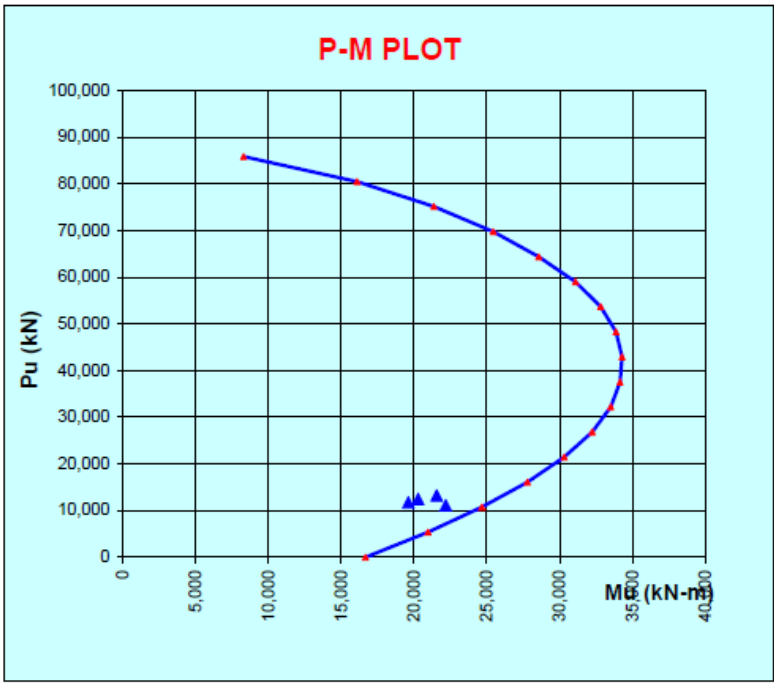
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	4,800,834	mm²	: Area of concrete in compression
Fc=	-76,813	kN	: Force on concrete
Fs=	-9,029	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.306E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	2389.04	mm	
Pu=	85,842	kN	: Ultimate Axial Force (+) compression
Mu=	8,274	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	16,658
2	0.06	5,365	20,959
3	0.12	10,730	24,664
4	0.18	16,095	27,782
5	0.24	21,461	30,308
6	0.30	26,826	32,235
7	0.36	32,191	33,522
8	0.42	37,556	34,155
9	0.48	42,921	34,279
10	0.53	48,286	33,872
11	0.59	53,651	32,821
12	0.65	59,017	31,076
13	0.71	64,382	28,564
14	0.77	69,747	25,424
15	0.83	75,112	21,362
16	0.89	80,477	16,089
17	0.95	85,842	8,274

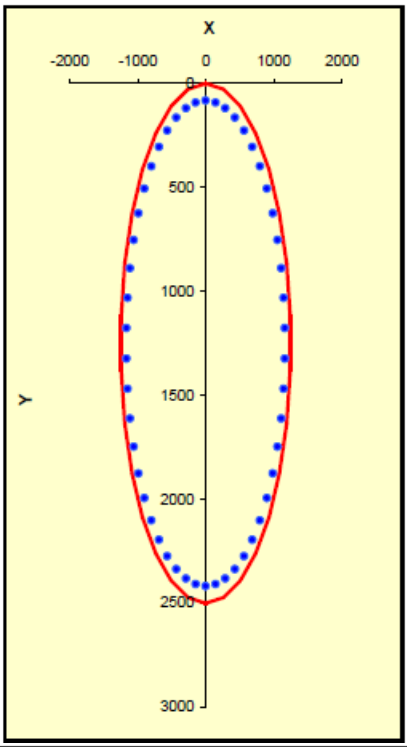


7.1.2.3 JKR MTAL (3 Notional Lanes)

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE P1 -C (JKR MTAL Loading Criteria)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	4,872,930	mm²	: Area of concrete section
ΣAs=	40,212	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.83	%	
Ap/Ag=	0.00	%	
Pu,max=	90,360	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	4,800,834	mm²	: Area of concrete in compression
Fc=	-76,813	kN	: Force on concrete
Fs=	-9,029	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.306E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	2389.04	mm	
Pu=	85,842	kN	: Ultimate Axial Force (+) compression
Mu=	8,274	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	16,658
2	0.06	5,365	20,959
3	0.12	10,730	24,664
4	0.18	16,095	27,782
5	0.24	21,461	30,308
6	0.30	26,826	32,235
7	0.36	32,191	33,522
8	0.42	37,556	34,155
9	0.48	42,921	34,279
10	0.53	48,286	33,872
11	0.59	53,651	32,821
12	0.65	59,017	31,076
13	0.71	64,382	28,564
14	0.77	69,747	25,424
15	0.83	75,112	21,362
16	0.89	80,477	16,089
17	0.95	85,842	8,274

7.1.2.4 Transverse Wind Load for P-11A Column

It is found that the ultimate pier induced force for Load Combination (2) under transverse wind load condition is less critical compared to Load Combination (1). Therefore, no further check would be required.

7.1.3 Crack Width Check (SLS) for P-11A Column

The pier crack width is calculated based on the following as-built parameters

Ø2500mm, f_{cu}=40mPa, 50-T32

From the following crack width checks, the computed crack widths for pier column P-11A (Type P1-C) are summarized as follows;

Table 19. Summary of P-11A SLS crack width check

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-C (P-11A)	***0.416***	***0.370***	***0.398***

Hence, the existing design of Pier P-11A exceeds the allowable crack width of 0.25mm for all 3 cases.

7.1.3.1 BD 37/88 (3 Notional Lanes)

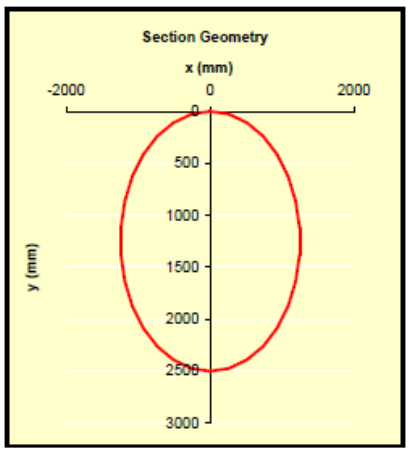
TITTLE : Pier P11A Type P1-C SLS1C1 (3 Notional Lanes)

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



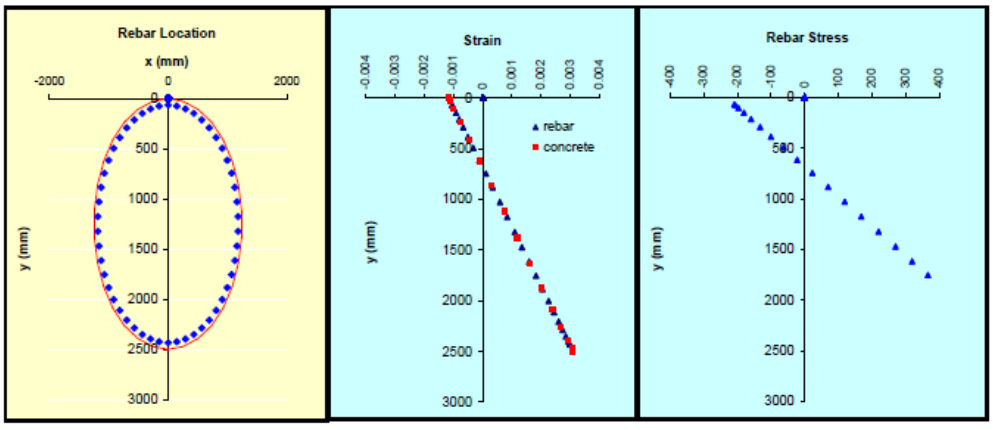
Neutral Axis (Elastic Analysis)

*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

ΣAc =	1,067,112	mm^2	: Area of concrete
ΣAs =	518,869	mm^2	: Transformed area of rebar
ΣA =	1,585,981	mm^2	: Gross area
Err(x) =	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn =	679.53	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	-6931	kN	: Permanent Axial Force; (-) compression
M_g =	10545	kN-m	: Permanent moment
M_q =	6592	kN-m	: Live load moment
M_s =	17137	kN-m	: Applied SLS moment
h =	2500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	50	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	2.82E-03		: Average strain at point considered
ε_o =	-2.82E-04		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	0.00E+00		: Strain due to tension stiffening effect
(1-Mq/Mg) =	3.75E-01		



Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	66	32	50	-0.00116	-2.82E-04	0	-1.44E-03	uncracked
0	0	0	0	0	0	-0.00116	-2.82E-04	0	-1.44E-03	uncracked
0	0	0	0	0	0	-0.00116	-2.82E-04	0	-1.44E-03	uncracked
0	0	0	0	0	0	-0.00116	-2.82E-04	0	-1.44E-03	uncracked
0	0	0	0	0	0	-0.00116	-2.82E-04	0	-1.44E-03	uncracked
0	2500	0	2434	32	50	0.003104	-2.82E-04	0.00E+00	2.82E-03	0.416

7.1.3.2 BD 37/88 (2 Notional Lanes)

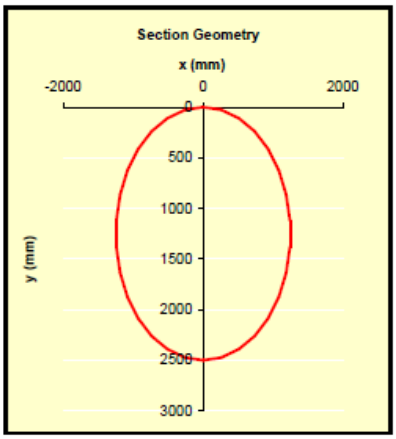
TITTLE : Pier P11A Type P1-C SLS1C1 (2 Notional Lanes)

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



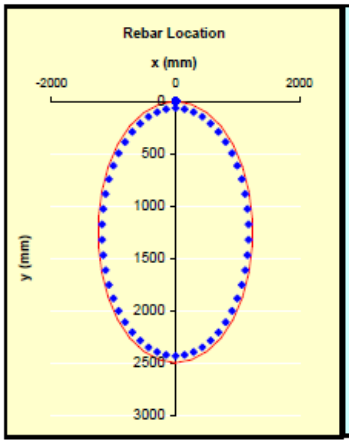
Neutral Axis (Elastic Analysis)

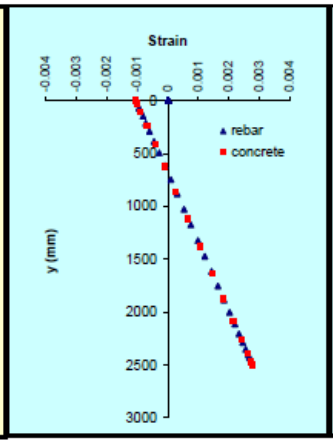
*Neutral Axis, Yn =Σ(A*Yi)/ΣA*

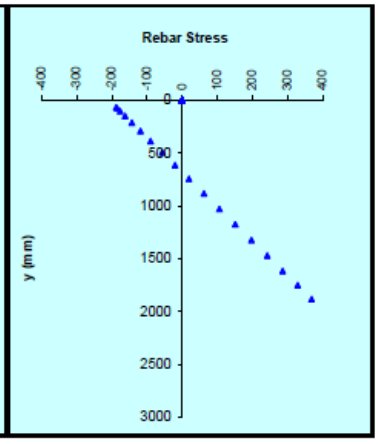
ΣAc=	1,067,112	mm^2	: Area of concrete
ΣAs=	518,869	mm^2	: Transformed area of rebar
ΣA=	1,585,981	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	679.53	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	-6931	kN	: Permanent Axial Force; (-) compression
M_g =	10545	kN-m	: Permanent moment
M_q =	4847	kN-m	: Live load moment
M_s =	15392	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	50	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	2.51E-03		: Average strain at point considered
ε_o =	-2.82E-04		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	0.00E+00		: Strain due to tension stiffening effect
(1-Mq/Mg) =	0.540350877		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	ε_{sm}	ε_m	W_{max} (mm)
0	0	0	66	32	50	-0.00104	-2.82E-04	0	-1.32E-03	uncracked
0	0	0	0	0	0	-0.00104	-2.82E-04	0	-1.32E-03	uncracked
0	0	0	0	0	0	-0.00104	-2.82E-04	0	-1.32E-03	uncracked
0	0	0	0	0	0	-0.00104	-2.82E-04	0	-1.32E-03	uncracked
0	0	0	0	0	0	-0.00104	-2.82E-04	0	-1.32E-03	uncracked
0	2500	0	2434	32	50	0.002788	-2.82E-04	0.00E+00	2.51E-03	0.370

7.1.3.3 JKR MTAL (3 Notional Lanes)

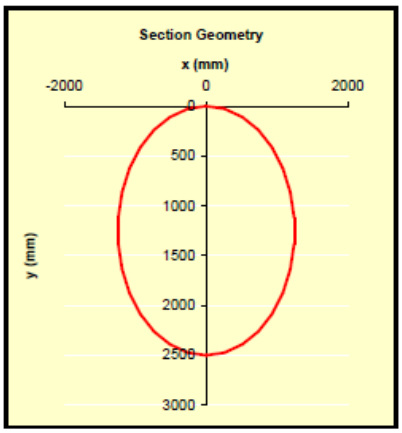
TITTLE : Pier P11A Type P1-C SLS1C1 (JKR MTAL Criteria)

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m²	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm²	: Steel Yield Strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



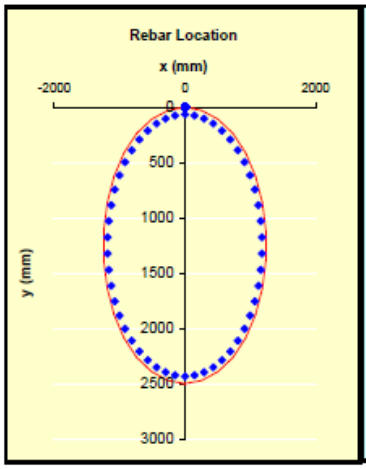
Neutral Axis (Elastic Analysis)

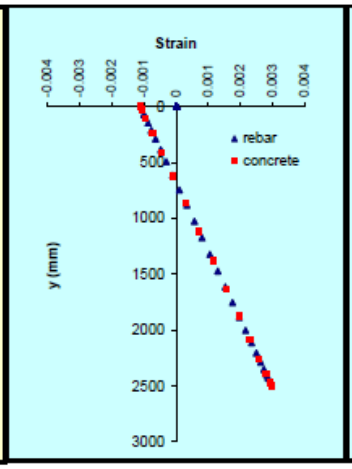
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

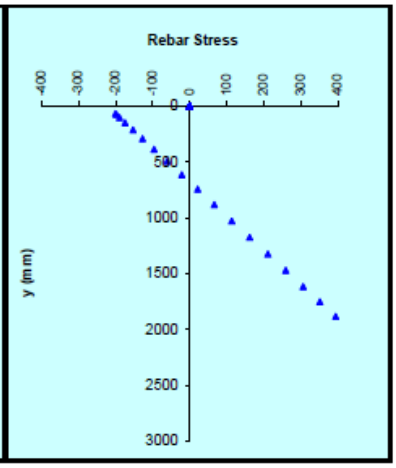
ΣAc=	1,067,112	mm²	: Area of concrete
ΣAs=	518,869	mm²	: Transformed area of rebar
ΣA=	1,585,981	mm²	: Gross area
Err(x)=	0.0		RE_ITERATE : Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	679.53	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	-6931	kN	: Permanent Axial Force; (-) compression
M_g =	10545	kN-m	: Permanent moment
M_q =	5903	kN-m	: Live load moment
M_s =	16448	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	50	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	2.70E-03		: Average strain at point considered
ε_o =	-2.82E-04		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	0.00E+00		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.40E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	66	32	50	-0.00111	-2.82E-04	0	-1.39E-03	uncracked
0	0	0	0	0	0	-0.00111	-2.82E-04	0	-1.39E-03	uncracked
0	0	0	0	0	0	-0.00111	-2.82E-04	0	-1.39E-03	uncracked
0	0	0	0	0	0	-0.00111	-2.82E-04	0	-1.39E-03	uncracked
0	0	0	0	0	0	-0.00111	-2.82E-04	0	-1.39E-03	uncracked
0	2500	0	2434	32	50	0.002979	-2.82E-04	0.00E+00	2.70E-03	0.398

7.2. Crosshead Check for Pier P-11A

The member forces of crosshead are presented below for various load combinations. The design checks for crosshead members under ULS and SLS are performed based on the following as-built drawing.

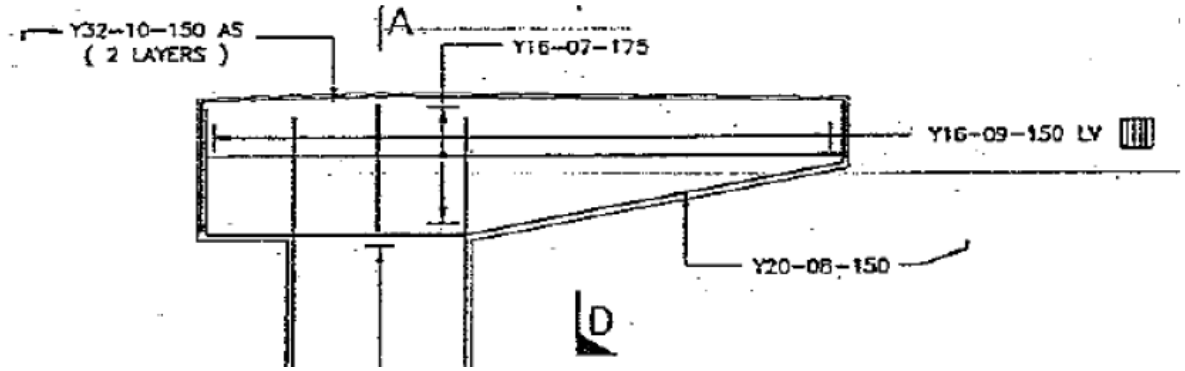


Figure 31. P-11A As-built crosshead reinforcement

7.2.1 Analysis Results for Pier P-11A Crosshead

The maximum design forces for crosshead are tabulated for various cases.

7.2.1.1 BD 37/88 (3 Notional Lanes)

Table 20. P-11A crosshead moment – BD 37/88 (3 Notional Lanes)

No.	Load Case	Mmax (kN.m)	Combination 1		
			γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	8324	1.00	1.15	1.10
2	Deck Slab	2142	1.00	1.15	1.10
3	SDL (Parapet)	1168	1.00	1.20	1.10
4	Premix	630	1.20	1.75	1.10
5	HA+KEL	5945	1.20	1.50	1.10
6	HA+HB30	5762	1.10	1.30	1.10
7	HB45	5989	1.10	1.30	1.10
8	SV20	5907	1.10	1.30	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 21. P-11A crosshead moment load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	M (kN.m)	M_q (kN.m)	M_g (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	19525	12391	7134
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	18729	12391	6338
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	18979	12391	6588
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	18889	12391	6498

ULS Design to Load Combination 1

Case #	Load Combination	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	25805
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	24235
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	24560
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	24443

Table 22. P-11A crosshead shear @ 2.5m depth – BD 37/88 (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1		
			γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	2352	1.00	1.15	1.10
2	Deck Slab	591	1.00	1.15	1.10
3	SDL (Parapet)	273	1.00	1.20	1.10
4	Premix	174	1.20	1.75	1.10
5	HA+KEL	1713	1.20	1.50	1.10
6	HA+HB30	1685	1.10	1.30	1.10
7	HB45	1476	1.10	1.30	1.10
8	SV20	1970	1.10	1.30	1.10

*StaadPro member 30105

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 23. P-11A crosshead shear @ 2.5m depth load combination – BD 37/88 (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	7245
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	6828
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	6529
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	7235

7.2.1.2 BD 37/88 (2 Notional Lanes)

Table 24. P-11A crosshead moment – BD 37/88 (2 Notional Lanes)

No.	Load Case	Mmax (kN.m)	Combination 1		
			γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	8324	1.00	1.15	1.10
2	Deck Slab	2142	1.00	1.15	1.10
3	SDL (Parapet)	1168	1.00	1.20	1.10
4	Premix	630	1.20	1.75	1.10
5	HA+KEL	4396	1.20	1.50	1.10
6	HA+HB30	4581	1.10	1.30	1.10
7	HB45	5838	1.10	1.30	1.10
8	SV20	5906	1.10	1.30	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 25. P-11A crosshead moment load combination – BD 37/88 (2 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	M (kN.m)	M_g (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	17666	12391	5275
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	17431	12391	5039
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	18814	12391	6422
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	18888	12391	6497

ULS Design to Load Combination 1

Case #	Load Combination	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	23249
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	22547
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	24345
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	24442

Table 26. P-11A crosshead shear @ 2.5m depth – BD 37/88 (2 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1		
			γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	2352	1.00	1.15	1.10
2	Deck Slab	591	1.00	1.15	1.10
3	SDL (Parapet)	273	1.00	1.20	1.10
4	Premix	174	1.20	1.75	1.10
5	HA+KEL	1246	1.20	1.50	1.10
6	HA+HB30	1268	1.10	1.30	1.10
7	HB45	1465	1.10	1.30	1.10
8	SV20	1967	1.10	1.30	1.10

*StaadPro member 30105

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 27. P-11A crosshead shear @ 2.5m depth load combination – BD 37/88 (2 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	6475
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	6231
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	6514
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	7231

7.2.1.3 JKR MTAL (3 Notional Lanes)

Table 28. P-11A crosshead moment – JKR MTAL (3 Notional Lanes)

No.	Load Case	Mmax (kN.m)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	8324	1.00	1.15	1.10
2	Deck Slab	2142	1.00	1.15	1.10
3	SDL (Parapet)	1168	1.00	1.20	1.10
4	Premix	630	1.20	1.75	1.10
5	MTAL (UDL+KEL)	5394	1.20	1.50	1.10
6	MTAL (5.0kPa)	168	1.20	1.50	1.10

**SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 29. P-11A crosshead moment load combination – JKR MTAL (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	M (kN.m)	M_q (kN.m)	M_g (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (MTAL)	19065	12391	6674

ULS Design to Load Combination 1

Case #	Load Combination	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (MTAL)	25173

Table 30. P-11A crosshead shear @ 2.5m depth – JKR MTAL (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	2352	1.00	1.15	1.10
2	Deck Slab	591	1.00	1.15	1.10
3	SDL (Parapet)	273	1.00	1.20	1.10
4	Premix	174	1.20	1.75	1.10
5	MTAL (UDL+KEL)	1591	1.20	1.50	1.10
6	MTAL (5.0kPa)	39	1.20	1.50	1.10

*StaadPro member 30105

*SW includes 6 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead

Table 31. P-11A crosshead shear @ 2.5m depth load combination – JKR MTAL (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (MTAL)	7107

7.2.2 Section Capacity Check (ULS) for P-11A Crosshead

The crosshead is checked for its moment and shear capacity under Ultimate Limit State (ULS).

The crosshead section capacity is calculated based on the following as-built information:-

Crosshead P-11A (*Type P1-C*)

- *Width = 2500mm, Depth = 2500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (2 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

7.2.2.1 Ultimate Moment Capacity Check for P-11A Crosshead

The computed crosshead ultimate moment capacities for Pier P-11A (Type P1-C) are computed and compared with the ULS applied moments.

Table 32. Summary of P-11A crosshead ULS moment capacity check

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	*24,439*	26,069	25,805	1.06
BD 37/88 (2 Notional Lanes)	*24,439*	26,069	24,442	1.00
JKR MTAL (3 Notional Lanes)	*24,439*	26,069	25,173	1.03

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The applied moments exceed the P-M interaction envelopes when the ultimate moment capacity is analyzed without taking into consideration the side reinforcement. However, when the side reinforcement is incorporated in the ultimate capacity calculation, the applied moments are within the P-M interaction envelope. Hence the existing crosshead moment capacity design for P-11a (Type P1-C) is marginally pass at ULS.

The detailed computations of the sectional moment capacities are presented below.

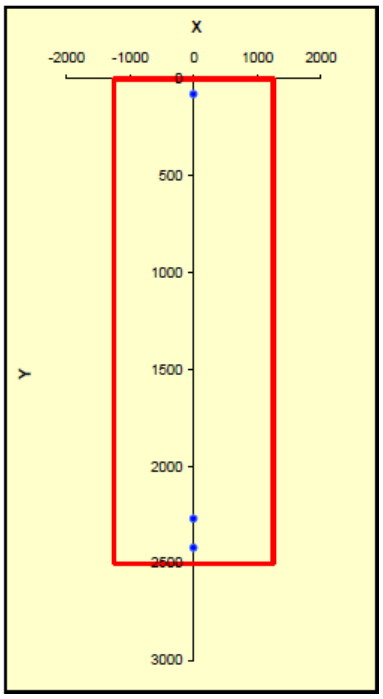
7.2.2.1.1 BD 37/88 (3 Notional Lanes)

**Without Side* Reinforcement

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P11A TYPE 1-C WITHOUT SIDE BARS (3 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	32,685	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.52	%	
Ap/Ag=	0.00	%	
Pu,max=	110,074	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

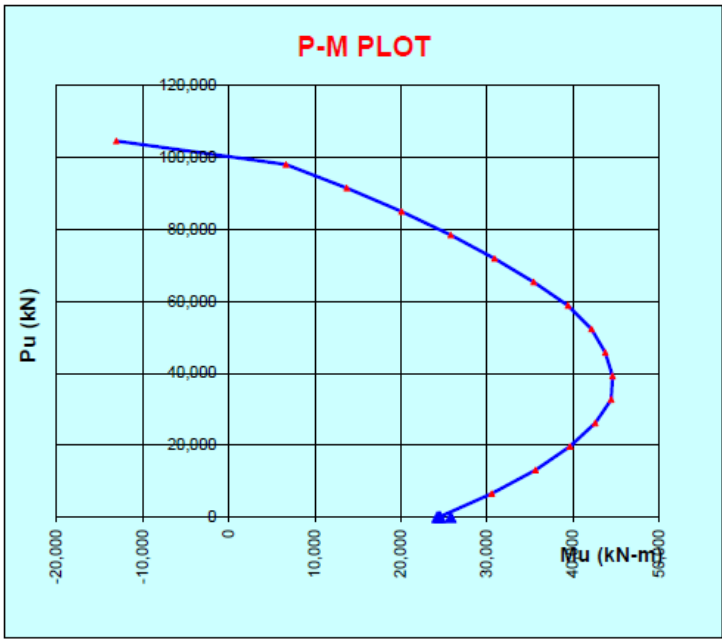
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	572,346	mm²	: Area of concrete in compression
Fc=	-9,158	kN	: Force on concrete
Fs=	9,158	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	228.94	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	24,439	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	24,439
2	0.06	6,536	30,579
3	0.12	13,071	35,650
4	0.18	19,607	39,654
5	0.24	26,142	42,589
6	0.30	32,678	44,457
7	0.36	39,214	44,611
8	0.42	45,749	43,758
9	0.48	52,285	42,155
10	0.53	58,821	39,418
11	0.59	65,356	35,445
12	0.65	71,892	30,884
13	0.71	78,427	25,785
14	0.77	84,963	20,085
15	0.83	91,499	13,728
16	0.89	98,034	6,662
17	0.95	104,570	-13,061

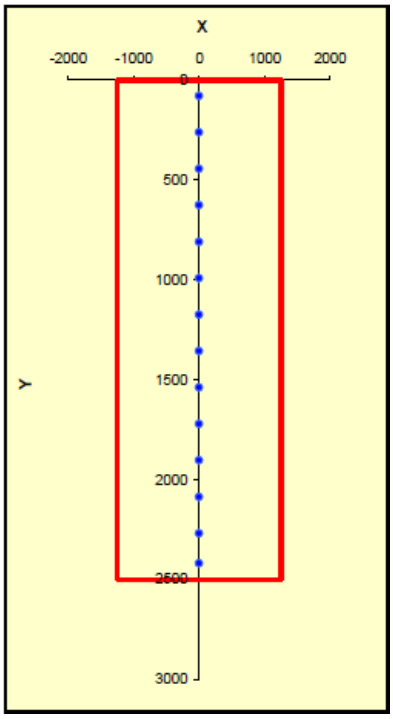


* *With Side* Reinforcement *T16-175 (Both Sides)*

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P11A TYPE 1 -C (WITH SIDE BARS) 3 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	37,108	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.59	%	
Ap/Ag=	0.00	%	
Pu,max=	111,437	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

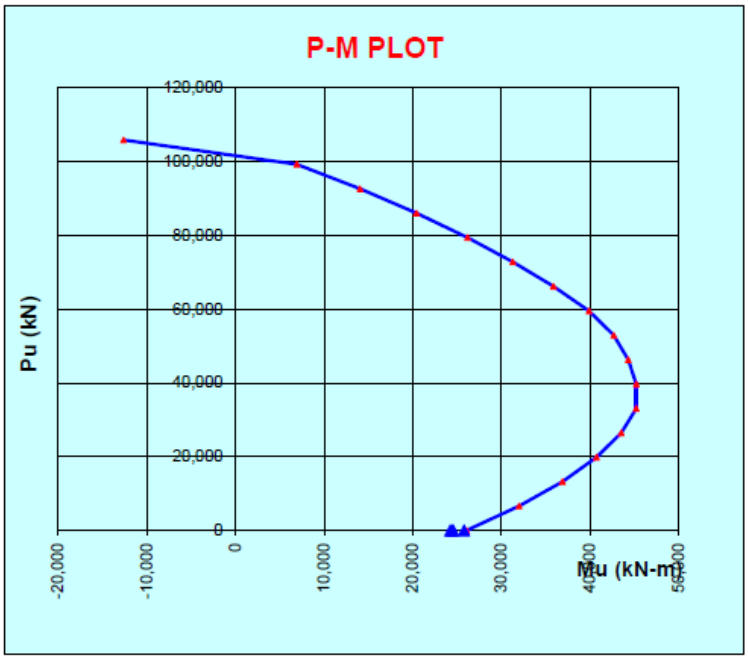
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	670,998	mm²	: Area of concrete in compression
Fc=	-10,736	kN	: Force on concrete
Fs=	10,736	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	268.40	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	26,069	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	26,069
2	0.06	6,617	32,021
3	0.12	13,233	36,914
4	0.18	19,850	40,749
5	0.24	26,466	43,530
6	0.30	33,083	45,242
7	0.36	39,699	45,242
8	0.42	46,316	44,331
9	0.48	52,932	42,678
10	0.53	59,549	39,889
11	0.59	66,166	35,887
12	0.65	72,782	31,306
13	0.71	79,399	26,176
14	0.77	86,015	20,442
15	0.83	92,632	14,046
16	0.89	99,248	6,927
17	0.95	105,865	-12,615



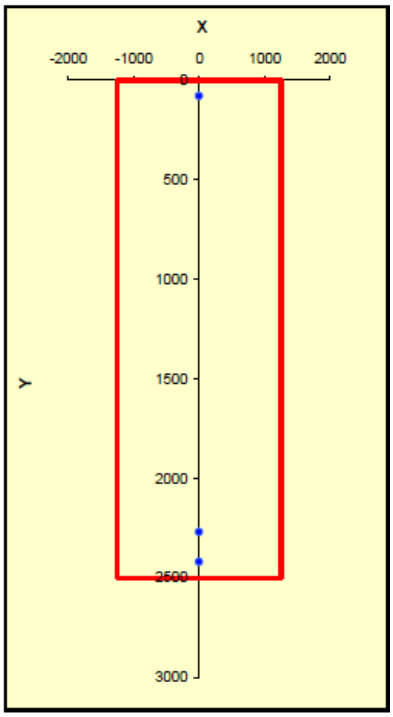
7.2.2.1.2 BD 37/88 (2 Notional Lanes)

** Without Side* Reinforcement

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE 1 -C WITHOUT SIDE BARS (2 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	32,685	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.52	%	
Ap/Ag=	0.00	%	
Pu,max=	110,074	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

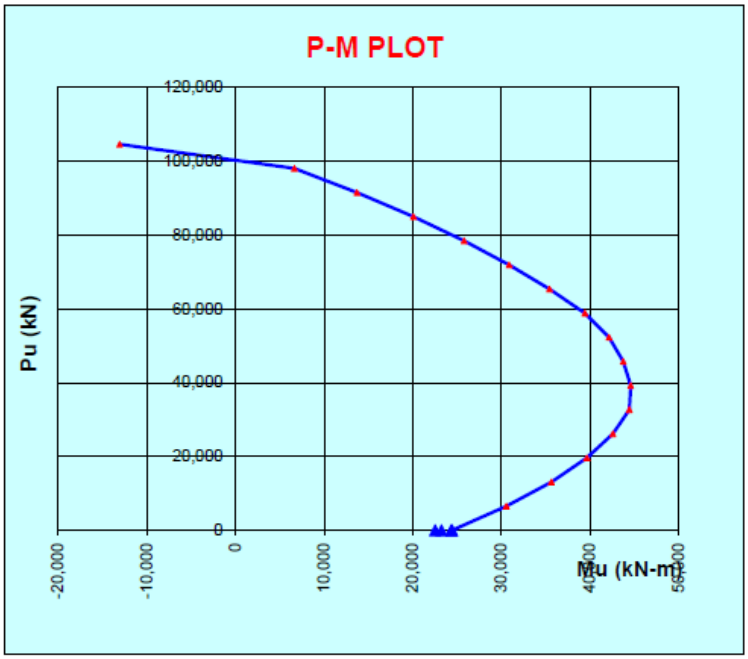
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	572,346	mm²	: Area of concrete in compression
Fc=	-9,158	kN	: Force on concrete
Fs=	9,158	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	228.94	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	24,439	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	24,439
2	0.06	6,536	30,579
3	0.12	13,071	35,650
4	0.18	19,607	39,654
5	0.24	26,142	42,589
6	0.30	32,678	44,457
7	0.36	39,214	44,611
8	0.42	45,749	43,758
9	0.48	52,285	42,155
10	0.53	58,821	39,418
11	0.59	65,356	35,445
12	0.65	71,892	30,884
13	0.71	78,427	25,785
14	0.77	84,963	20,085
15	0.83	91,499	13,728
16	0.89	98,034	6,662
17	0.95	104,570	-13,061

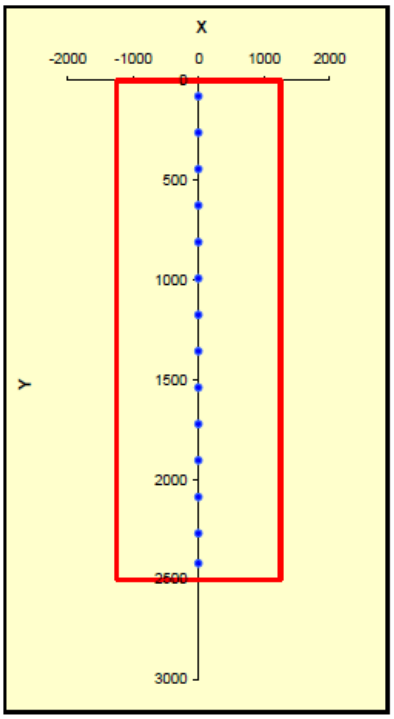


* *With Side* Reinforcement *T16-175 (Both Sides)*

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE 1 -C (WITH SIDE BARS) 2 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	37,108	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.59	%	
Ap/Ag=	0.00	%	
Pu,max=	111,437	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

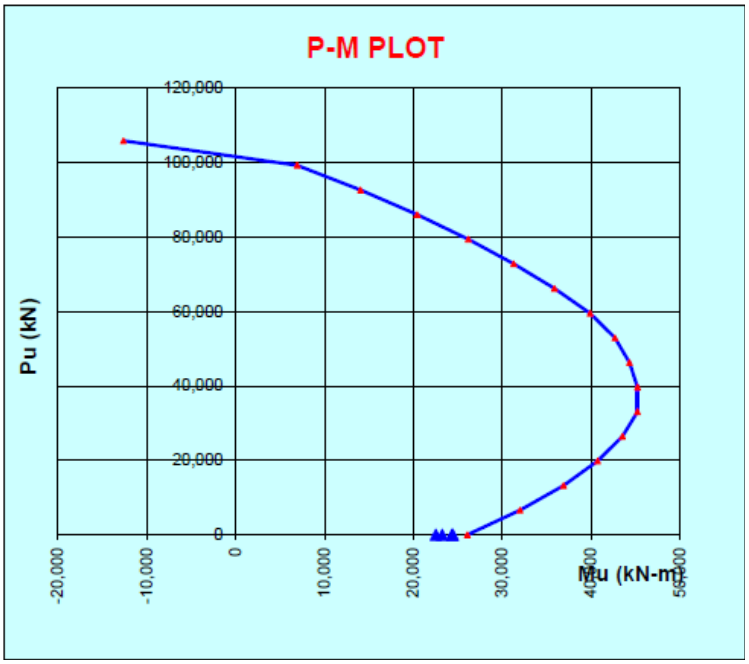
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	670,998	mm²	: Area of concrete in compression
Fc=	-10,736	kN	: Force on concrete
Fs=	10,736	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	268.40	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	26,069	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	26,069
2	0.06	6,617	32,021
3	0.12	13,233	36,914
4	0.18	19,850	40,749
5	0.24	26,466	43,530
6	0.30	33,083	45,242
7	0.36	39,699	45,242
8	0.42	46,316	44,331
9	0.48	52,932	42,678
10	0.53	59,549	39,889
11	0.59	66,166	35,887
12	0.65	72,782	31,306
13	0.71	79,399	26,176
14	0.77	86,015	20,442
15	0.83	92,632	14,046
16	0.89	99,248	6,927
17	0.95	105,865	-12,615



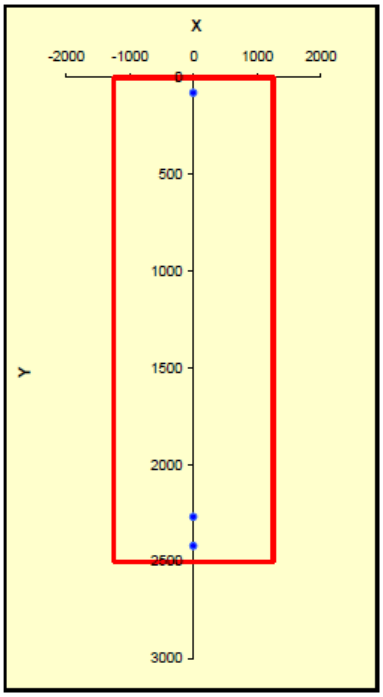
7.2.2.1.3 JKR MTAL (3 Notional Lanes)

** Without Side Reinforcement*

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE 1 -C WITHOUT SIDE BARS (JKR MTAL CRITERIA)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	32,685	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.52	%	
Ap/Ag=	0.00	%	
Pu,max=	110,074	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

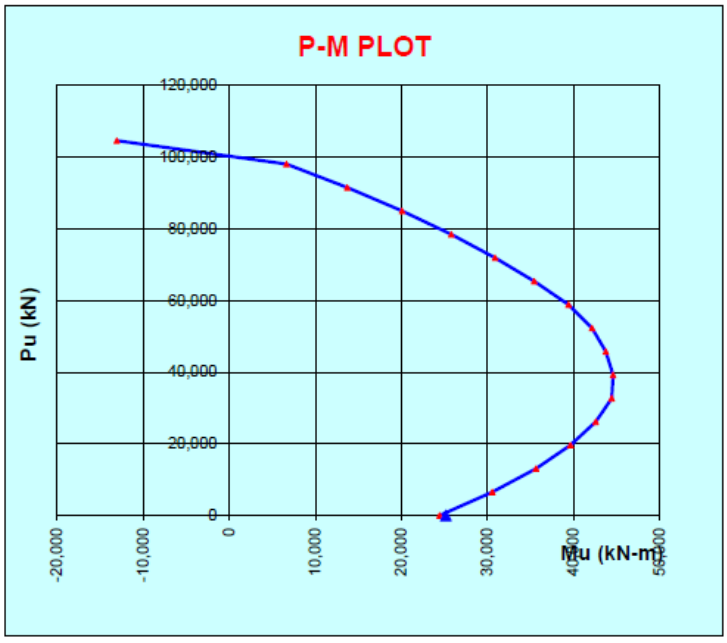
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	572,346	mm²	: Area of concrete in compression
Fc=	-9,158	kN	: Force on concrete
Fs=	9,158	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	228.94	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	24,439	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	24,439
2	0.06	6,536	30,579
3	0.12	13,071	35,650
4	0.18	19,607	39,654
5	0.24	26,142	42,589
6	0.30	32,678	44,457
7	0.36	39,214	44,611
8	0.42	45,749	43,758
9	0.48	52,285	42,155
10	0.53	58,821	39,418
11	0.59	65,356	35,445
12	0.65	71,892	30,884
13	0.71	78,427	25,785
14	0.77	84,963	20,085
15	0.83	91,499	13,728
16	0.89	98,034	6,662
17	0.95	104,570	-13,061

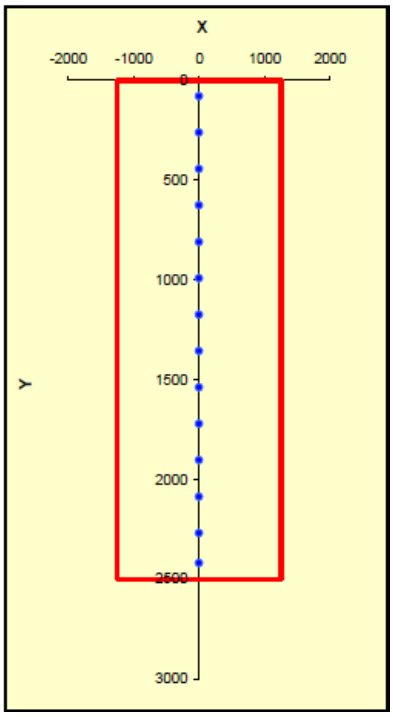


* *With Side* Reinforcement *T16-175 (Both Sides)*

Ultimate Section Capacity BS5400

Design Information: PIER P11A TYPE 1 -C (WITH SIDE BARS) JKR MTAL CRITERIA

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1250.000	mm	: Centroid of section
ΣAg=	6,250,000	mm²	: Area of concrete section
ΣAs=	37,108	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.59	%	
Ap/Ag=	0.00	%	
Pu,max=	111,437	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

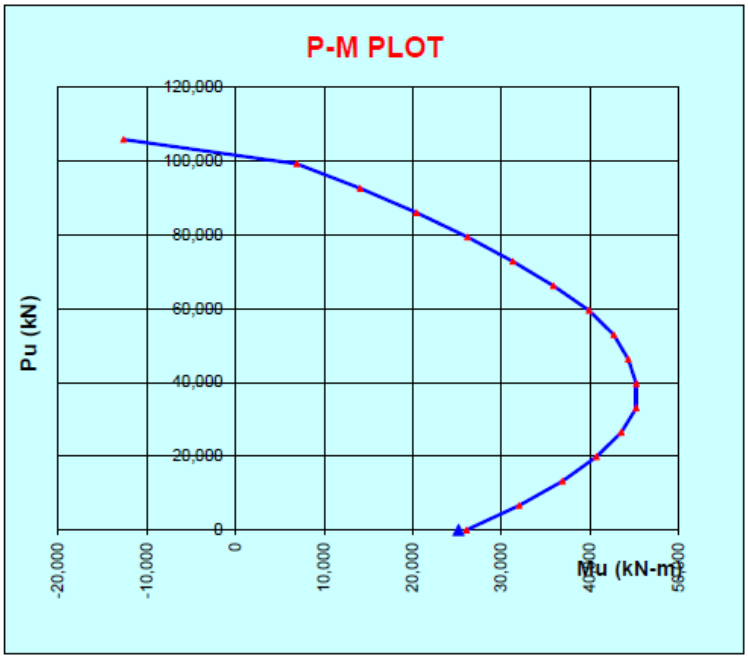
Neutral Axis, Yn, where **Σ(Fc+Fp+Fs+Pu)=0**

ΣAc=	670,998	mm²	: Area of concrete in compression
Fc=	-10,736	kN	: Force on concrete
Fs=	10,736	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	268.40	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	26,069	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	26,069
2	0.06	6,617	32,021
3	0.12	13,233	36,914
4	0.18	19,850	40,749
5	0.24	26,466	43,530
6	0.30	33,083	45,242
7	0.36	39,699	45,242
8	0.42	46,316	44,331
9	0.48	52,932	42,678
10	0.53	59,549	39,889
11	0.59	66,166	35,887
12	0.65	72,782	31,306
13	0.71	79,399	26,176
14	0.77	86,015	20,442
15	0.83	92,632	14,046
16	0.89	99,248	6,927
17	0.95	105,865	-12,615



7.2.2.2 Ultimate Shear Capacity Check for P-11A Crosshead

The shear link required for crosshead of Pier P-11A (Type P1-C) under ULS is computed and compared to the shear link provided.

Table 33. Summary of P-11A crosshead ULS shear force capacity check

BD 37/88 (3 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.72	8.04	0.96
ULS2C1	7.28	8.04	0.90
ULS3C1	6.96	8.04	0.87
ULS4C1	7.71	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

BD 37/88 (2 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	6.90	8.04	0.86
ULS2C1	6.64	8.04	0.83
ULS3C1	6.94	8.04	0.86
ULS4C1	7.70	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

JKR MTAL @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.57	8.04	0.94

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Based on the above, the shear link provided in the as-built drawings is more than the requirement. Hence, the existing crosshead shear design for P-11A (Type P1-C) is adequate at ULS.

The detailed computations of the sectional shear capacities are presented as below.

7.2.2.2.1 BD 37/88 (3 Notional Lanes)

***ULS1C1**

Element ID = **P-11A Crosshead (ULS1C1 - BD 37/88 3 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	7,245	kN				
v	=	1.24	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
$A_{s,prov}$	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.72					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

***ULS2C1**

Element ID = **P-11A Crosshead (ULS2C1 - BD 37/88 3 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	6,828	kN				
v	=	1.17	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.28					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS3C1

Element ID = **P-11A Crosshead (ULS3C1 - BD 37/88 3 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	2,500	mm			
d	=	2,343	mm			
V_{ult}	=	6,529	kN			
v	=	1.11	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.700				
A_s	=	27,336	mm^2	(2 layers of 17T32)		
v_c	=	0.57	N/mm^2	$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	6.96				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

*ULS4C1

Element ID = **P-11A Crosshead (ULS4C1 - SV20) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	2,500	mm			
d	=	2,343	mm			
V_{ult}	=	7,235	kN			
v	=	1.24	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.700				
A_s	=	27,336	mm^2	(2 layers of 17T32)		
v_c	=	0.57	N/mm^2	$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	7.71				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

7.2.2.2.2 BD 37/88 (2 Notional Lanes)

**ULS1C1*

Element ID = **P-11A Crosshead (ULS1C1 - BD 37/88 2 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	6,475	kN				
v	=	1.11	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.90					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

**ULS2C1*

Element ID = **P-11A Crosshead (ULS2C1 - BD 37/88 2 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	6,231	kN				
v	=	1.06	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.64					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS3C1

Element ID = **P-11A Crosshead (ULS3C1 - BD 37/88 2 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	6,514	kN				
v	=	1.11	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.94					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS4C1

Element ID = **P-11A Crosshead (ULS4C1 - BD 37/88 2 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	2,500	mm				
d	=	2,343	mm				
V_{ult}	=	7,231	kN				
v	=	1.23	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	27,336	mm^2	(2 layers of 17T32)			
v_c	=	0.57	N/mm^2		$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.70					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

7.2.2.2.3 JKR MTAL (3 Notional Lanes)

ULS1C1

Element ID = **P-11A Crosshead (ULS1C1 - JKR MTAL 3 Notional Lanes) @ 2.5m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	2,500	mm			
d	=	2,343	mm			
V_{ult}	=	7,107	kN			
v	=	1.21	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.700				
A_s	=	27,336	mm^2	(2 layers of 17T32)		
v_c	=	0.57	N/mm^2	$\xi_s v_c$	=	0.40
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	7.57				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

7.2.3 Crack Width Check (SLS) for Pier P-11A Crosshead

The crosshead crack width is calculated based on the following as-built parameters:-

Crosshead P-11A (*Type P1-C*)

- *Width = 2500mm, Depth = 2500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (2 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

The computed crosshead crack widths for Pier P-11A (Type P1-C) are summarized as follows:-

Table 34. Summary of P-11A crosshead SLS crack width check

Loading Criteria	Crack Width (mm)	
	Without Sidebar	With Sidebar
BD 37/88 (3 Notional Lanes)	***0.270***	***0.255***
BD 37/88 (2 Notional Lanes)	0.196	0.181
JKR MTAL (3 Notional Lanes)	***0.252***	0.238

The computed crack widths for 3 notional lanes under BD37/88 and JKR MTAL loading criteria without taking side reinforcement into consideration are 0.270mm and 0.252mm, which exceed the allowable crack width of 0.250mm. When side reinforcement is incorporated into the design check, the computed crack width under BD 37/88 is 0.255mm which still exceeds the allowable limit. However, under JKR MTAL is 0.238mm which is less than the allowable limit.

Under BD 37/88 2 notional lanes criteria, both the design checks with and without side reinforcement are 0.181mm and 0.196mm respectively which are less than the allowable crack width of 0.250mm.

The detailed computation of the crack widths are presented below.

7.2.3.1 BD 37/88 (3 Notional Lanes)

** Without Side Reinforcement*

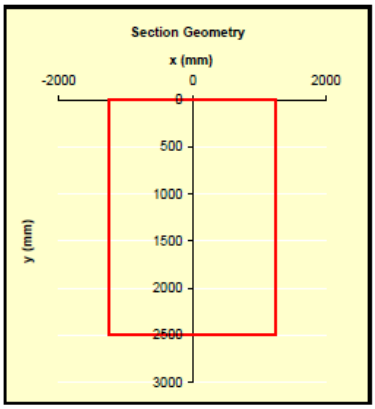
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITHOUT SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m²	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm²	: Steel Yield Strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



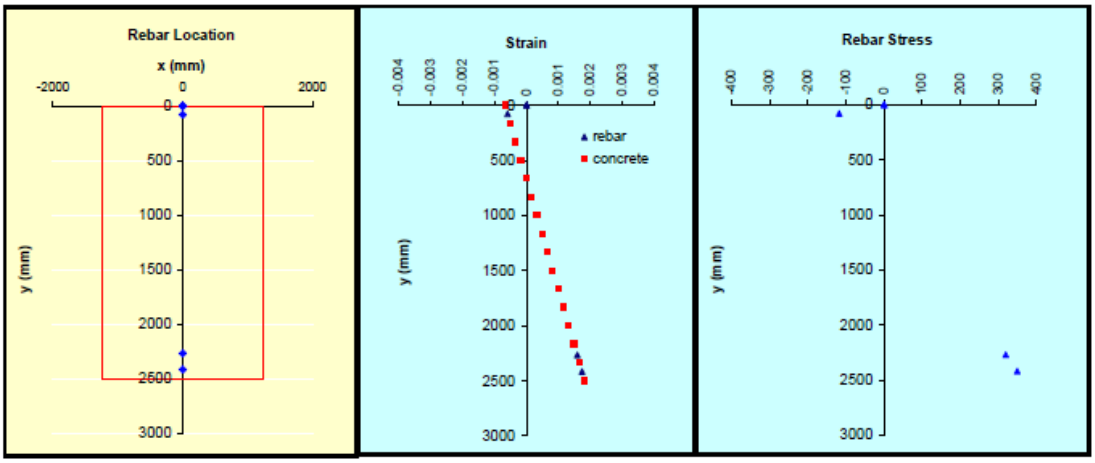
Neutral Axis (Elastic Analysis)

*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

ΣAc =	1,661,123	mm²	: Area of concrete
ΣAs =	421,744	mm²	: Transformed area of rebar
ΣA =	2,082,867	mm²	: Gross area
Err(x) =	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn =	664.45	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	7134	kN-m	: Live load moment
M_s =	19525	kN-m	: Applied SLS moment
h =	2500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.41E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-4.22E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.24E-01		



Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-6.62E-04	0.00E+00	0.00E+00	-6.62E-04	uncracked
0	0	0	0	0	0	-6.62E-04	0.00E+00	0.00E+00	-6.62E-04	uncracked
0	0	0	0	0	0	-6.62E-04	0.00E+00	0.00E+00	-6.62E-04	uncracked
0	0	0	0	0	0	-6.62E-04	0.00E+00	0.00E+00	-6.62E-04	uncracked
0	0	0	0	0	0	-6.62E-04	0.00E+00	0.00E+00	-6.62E-04	uncracked
0	2500	0	2418	32	66	1.83E-03	0.00E+00	-4.22E-04	1.41E-03	0.270

With Side Reinforcement T16-175 (Both Sides)

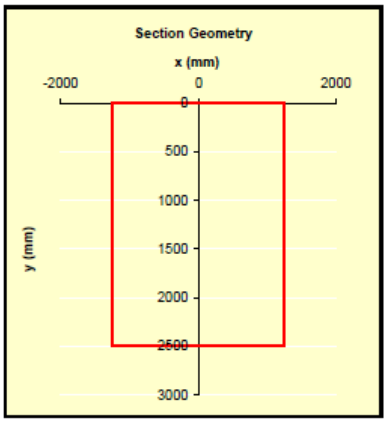
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITH SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



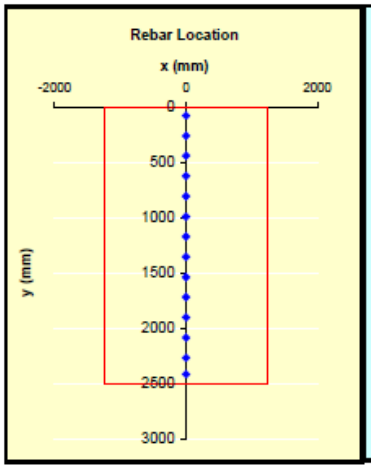
Neutral Axis (Elastic Analysis)

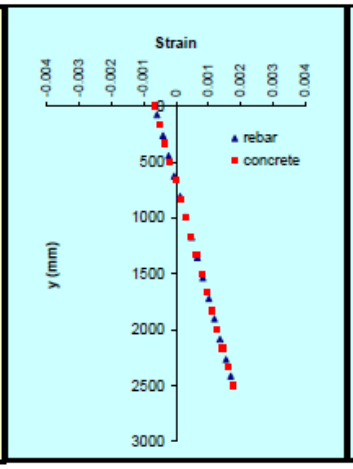
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

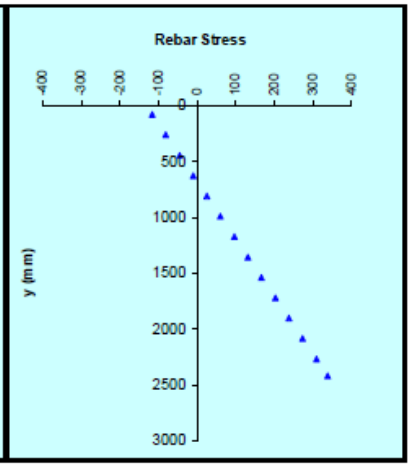
ΣAc=	1,694,808	mm^2	: Area of concrete
ΣAs=	478,819	mm^2	: Transformed area of rebar
ΣA=	2,173,627	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	677.92	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	7134	kN-m	: Live load moment
M_s =	19525	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.33E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-4.36E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.24E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{sum.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-6.58E-04	0.00E+00	0.00E+00	-6.58E-04	uncracked
0	0	0	0	0	0	-6.58E-04	0.00E+00	0.00E+00	-6.58E-04	uncracked
0	0	0	0	0	0	-6.58E-04	0.00E+00	0.00E+00	-6.58E-04	uncracked
0	0	0	0	0	0	-6.58E-04	0.00E+00	0.00E+00	-6.58E-04	uncracked
0	0	0	0	0	0	-6.58E-04	0.00E+00	0.00E+00	-6.58E-04	uncracked
0	2500	0	2418	32	66	1.77E-03	0.00E+00	-4.36E-04	1.33E-03	0.255

7.2.3.2 BD 37/88 (2 Notional Lanes)

** Without Side Reinforcement*

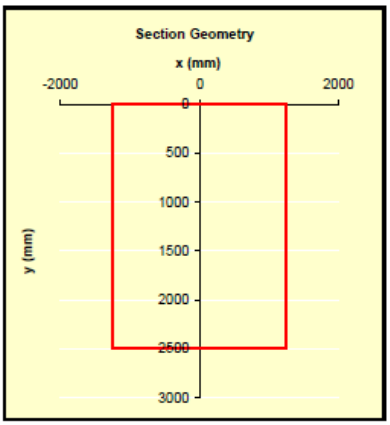
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITHOUT SIDEBAR) 2 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



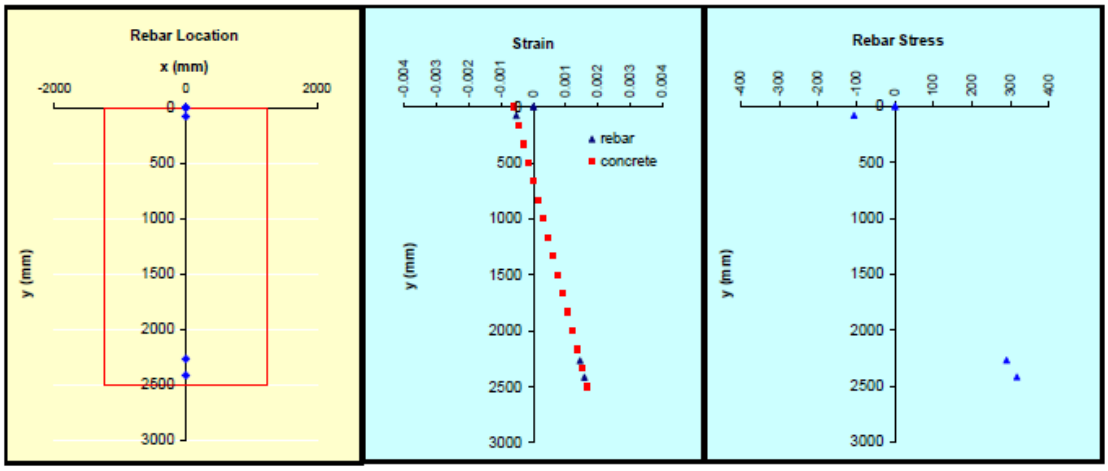
Neutral Axis (Elastic Analysis)

*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

ΣAc=	1,661,123	mm^2	: Area of concrete
ΣAs=	421,744	mm^2	: Transformed area of rebar
ΣA=	2,082,867	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	664.45	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	5275	kN-m	: Live load moment
M_s =	17666	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.02E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-6.31E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	5.74E-01		



Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.99E-04	0.00E+00	0.00E+00	-5.99E-04	uncracked
0	0	0	0	0	0	-5.99E-04	0.00E+00	0.00E+00	-5.99E-04	uncracked
0	0	0	0	0	0	-5.99E-04	0.00E+00	0.00E+00	-5.99E-04	uncracked
0	0	0	0	0	0	-5.99E-04	0.00E+00	0.00E+00	-5.99E-04	uncracked
0	0	0	0	0	0	-5.99E-04	0.00E+00	0.00E+00	-5.99E-04	uncracked
0	2500	0	2418	32	66	1.65E-03	0.00E+00	-6.31E-04	1.02E-03	0.196

With Side Reinforcement T16-175 (Both Sides)

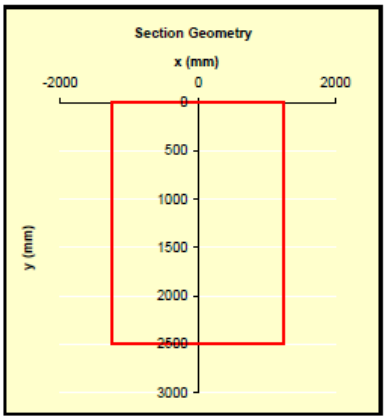
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITH SIDEBAR) 2 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



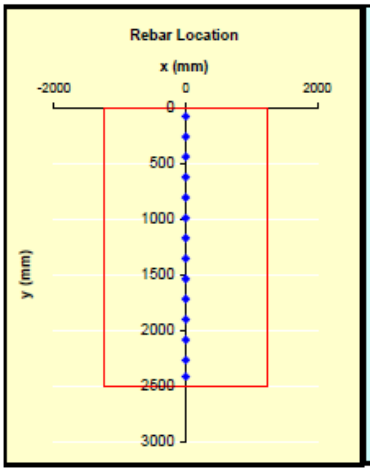
Neutral Axis (Elastic Analysis)

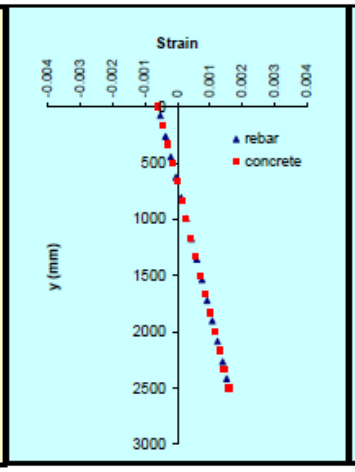
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

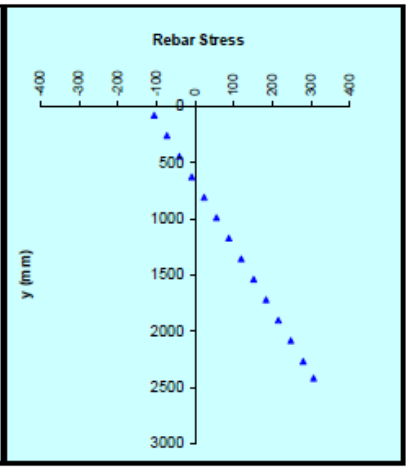
ΣAc=	1,694,808	mm^2	: Area of concrete
ΣAs=	478,819	mm^2	: Transformed area of rebar
ΣA=	2,173,627	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	677.92	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	5275	kN-m	: Live load moment
M_s =	17666	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	9.48E-04		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-6.53E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	5.74E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{suff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.95E-04	0.00E+00	0.00E+00	-5.95E-04	uncracked
0	0	0	0	0	0	-5.95E-04	0.00E+00	0.00E+00	-5.95E-04	uncracked
0	0	0	0	0	0	-5.95E-04	0.00E+00	0.00E+00	-5.95E-04	uncracked
0	0	0	0	0	0	-5.95E-04	0.00E+00	0.00E+00	-5.95E-04	uncracked
0	0	0	0	0	0	-5.95E-04	0.00E+00	0.00E+00	-5.95E-04	uncracked
0	2500	0	2418	32	66	1.60E-03	0.00E+00	-6.53E-04	9.48E-04	0.181

7.2.3.3 JKR MTAL (3 Notional Lanes)

** Without Side Reinforcement*

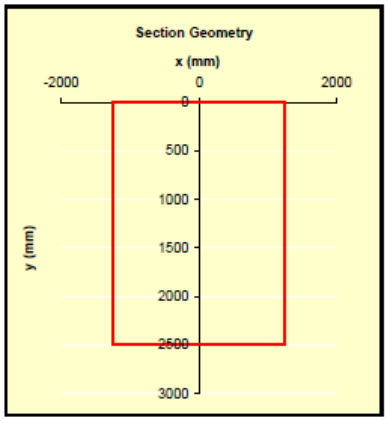
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITHOUT SIDEBAR) JKR MTAL Criteria

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



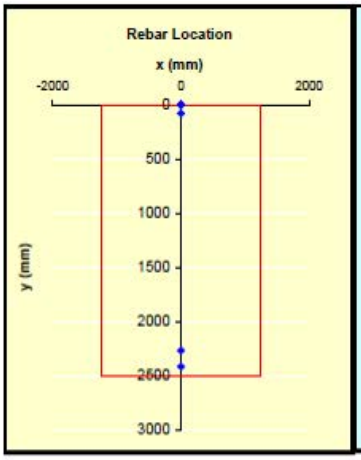
Neutral Axis (Elastic Analysis)

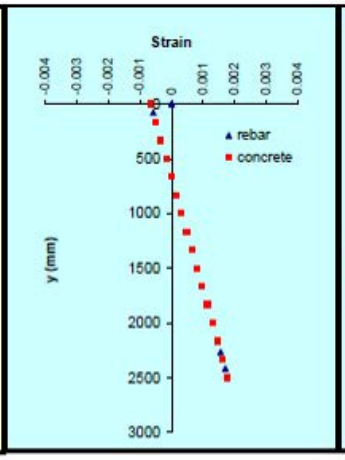
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

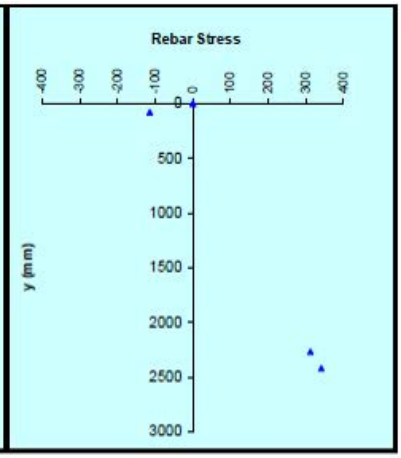
ΣAc=	1,661,123	mm^2	: Area of concrete
ΣAs=	421,744	mm^2	: Transformed area of rebar
ΣA=	2,082,867	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	664.45	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	6674	kN-m	: Live load moment
M_s =	19065	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.32E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-4.70E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.61E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-6.46E-04	0.00E+00	0.00E+00	-6.46E-04	uncracked
0	0	0	0	0	0	-6.46E-04	0.00E+00	0.00E+00	-6.46E-04	uncracked
0	0	0	0	0	0	-6.46E-04	0.00E+00	0.00E+00	-6.46E-04	uncracked
0	0	0	0	0	0	-6.46E-04	0.00E+00	0.00E+00	-6.46E-04	uncracked
0	0	0	0	0	0	-6.46E-04	0.00E+00	0.00E+00	-6.46E-04	uncracked
0	2500	0	2418	32	66	1.79E-03	0.00E+00	-4.70E-04	1.32E-03	0.252

**With Side* Reinforcement *T16-175 (Both Sides)*

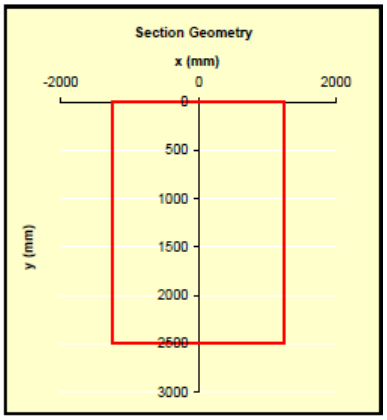
TITTLE : Crosshead P11A Type P1-C SLS1C1 (WITH SIDEBAR) JKR MTAL Criteria

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



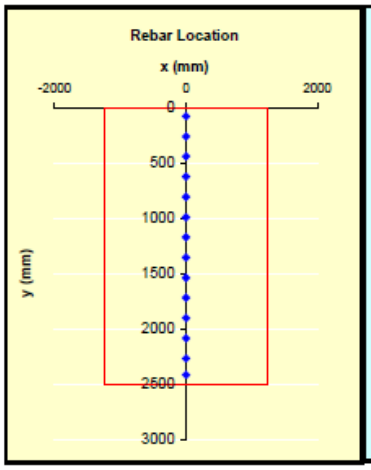
Neutral Axis (Elastic Analysis)

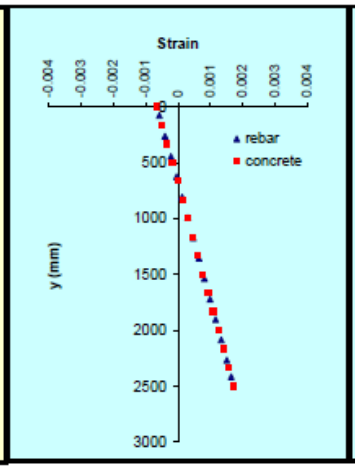
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

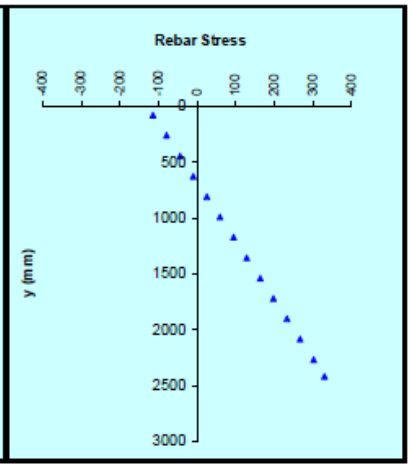
ΣAc=	1,694,808	mm^2	: Area of concrete
ΣAs=	478,819	mm^2	: Transformed area of rebar
ΣA=	2,173,627	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	677.92	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	12391	kN-m	: Permanent moment
M_q =	6674	kN-m	: Live load moment
M_s =	19065	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.24E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-4.86E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.61E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-6.43E-04	0.00E+00	0.00E+00	-6.43E-04	uncracked
0	0	0	0	0	0	-6.43E-04	0.00E+00	0.00E+00	-6.43E-04	uncracked
0	0	0	0	0	0	-6.43E-04	0.00E+00	0.00E+00	-6.43E-04	uncracked
0	0	0	0	0	0	-6.43E-04	0.00E+00	0.00E+00	-6.43E-04	uncracked
0	0	0	0	0	0	-6.43E-04	0.00E+00	0.00E+00	-6.43E-04	uncracked
0	2500	0	2418	32	66	1.73E-03	0.00E+00	-4.86E-04	1.24E-03	0.238

7.3. Strut and Tie Analysis (STM) for Pier P-11A Crosshead

The STM model is based on BD 37/88 three (3) notional lanes loading criteria for Ultimate Limit State under Load Combination (1).

Summary of maximum bearing force based on BD 37/88 (3 Notional Lanes)

No.	Load Case	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)	Combination 1 γ_{fL} SLS	Combination 1 γ_{fL} ULS	γ_{f3} ULS
1	SW	310	358	449	347	364	370	375	300	1.00	1.15	1.10
2	Deck Slab	124	113	140	115	119	121	121	115	1.00	1.15	1.10
3	SDL (Parapet)	270	***-38***	43	38	40	37	14	145	1.00	1.20	1.10
4	Premix	36	33	41	34	35	36	36	34	1.20	1.75	1.10
5	HA+KEL	172	386	301	389	358	267	468	232	1.20	1.50	1.10
6	HA+HB30	276	247	279	330	399	358	418	181	1.10	1.30	1.10
7	HB45	***-129***	84	119	111	266	324	304	472	1.10	1.30	1.10
8	SV20	115	366	552	560	568	439	390	15	1.10	1.30	1.10

SLS Design to Load Combination 1

Case #	Load Combination	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	955	936	1043	1007	995	890	1115	880
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	1051	744	988	903	1004	964	1014	800
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	606	565	813	662	857	926	888	1120
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	875	875	1289	1156	1190	1052	982	618

ULS Design to Load Combination 1

Case #	Load Combination	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	1260	1246	1378	1341	1323	1178	1487	1165
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	1370	963	1280	1170	1302	1249	1314	1041
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	791	730	1052	858	1111	1201	1150	1457
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	1141	1133	1670	1499	1543	1365	1273	804

Note : N_8 is located at the tip of the cantilever (furthest from the pier)

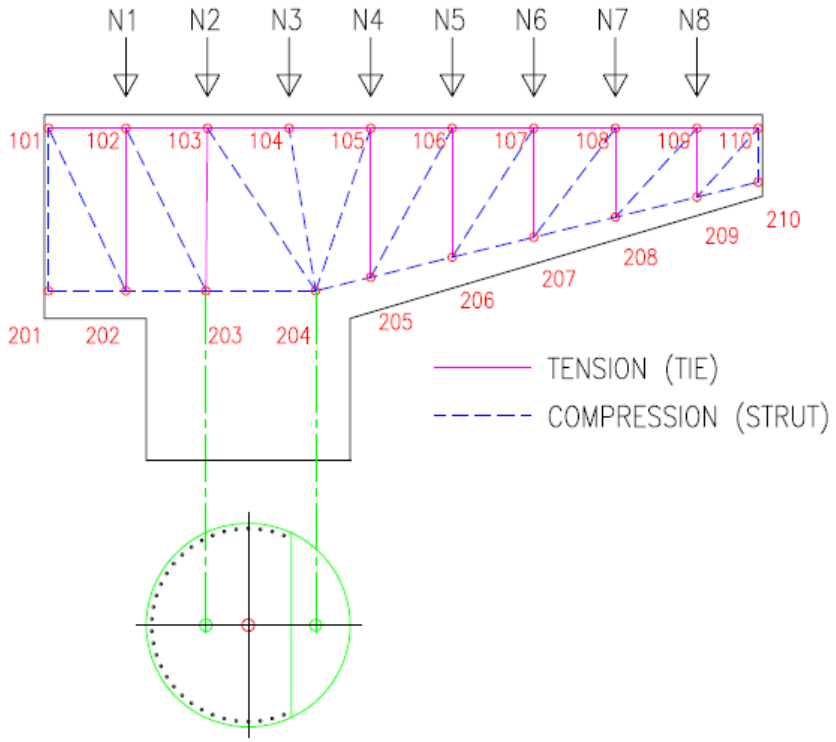


Figure 32. *P-11A STM Analysis Model*

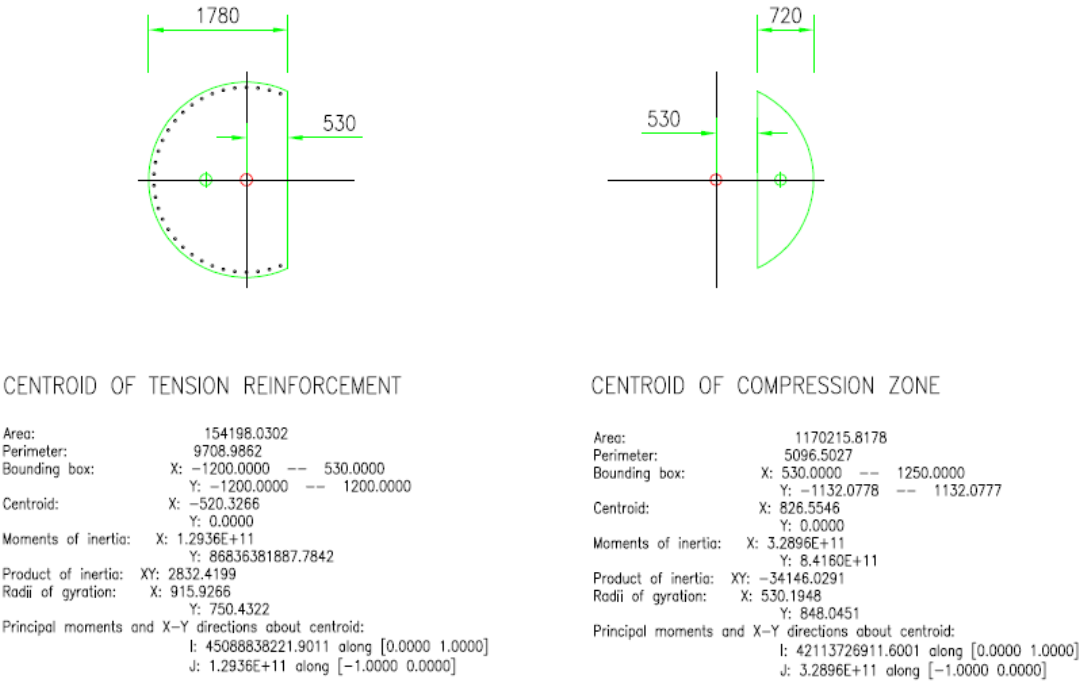


Figure 33. *Pier P-11A Tension and Compression Zone based on ULS1C1*

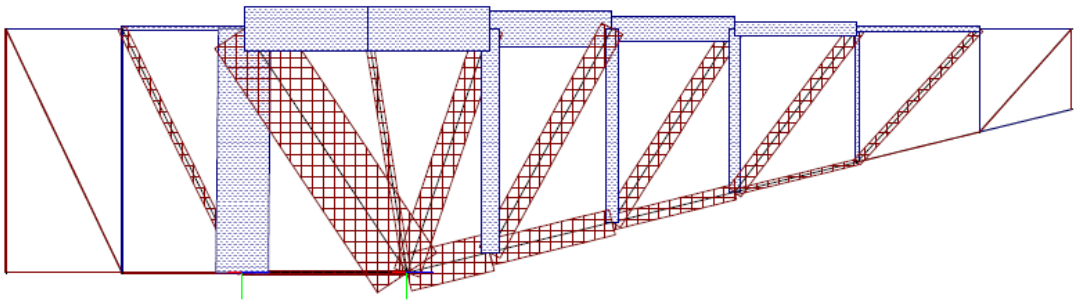


Figure 34. *P-11A STM Axial Force Diagram (Blue = Tension, Red = Compression)*

The support for the STM model is modelled based on the centroid of the tension reinforcement zone and concrete compression zone as shown in Figure 33.

The tie tension forces obtained from the analysis are checked as follows;

Top Tension Tie *(104-105)* **Check**

fy =	460	Mpa	
As,prov =	27,336	mm^2	(2 x 17T32 top reinforcement)
Tu =	9,190	kN	
As,req =	$Tu/0.87f_y$		
=	22,964	mm^2	
Remarks :	As,prov > As,req	O.K!	

Vertical Tension Tie *(103-203)* **Check**

fy =	460	Mpa	
As,prov =	25,728	mm^2	(32T32 column main reinforcement inside 1780mm tension zone)
	14,311	mm^2	(3T16-150 links in 1780mm tension zone)
	40,039	mm^2	
Tu =	10,685	kN	
As,req =	26,700	mm^2	
Remarks :	As,prov > As,req	O.K!	

Vertical Tension Tie *(105-205)* **Check**

fy =	460	Mpa	
As,prov =	0	mm^2	
	8,040	mm^2	(3T16-150 links in 1000mm tension zone)
	8,040	mm^2	
Tu =	3,666	kN	
As,req =	**9,161**	mm^2	
Remarks :	As,prov < As,req	FAILED!	

The check shows that the top reinforcement and column reinforcement in the tension zone provided are sufficient to cater for the tension forces. However, the shear links provided is insufficient to resist the tension forces.

The bottom compression strut forces obtained from the analysis are checked as follows;

Bottom Compression Strut *(204-205)* **Check**

fcu =	40	Mpa	
b =	2,500	mm	(Crosshead width)
d =	680	mm	(Crosshead compression zone)
Su =	7,673	kN	
σ =	**4.51**	Mpa	
Remarks :	< 0.4fcu	O.K	

Based on the check, the concrete stress calculated is 4.51 N/mm^2, which is less than $0.4f_{cu}$; 16.0 N/mm^2. Thus, the bottom strut concrete compression stress is within the concrete strength limit.

The diagonal compression strut is checked as follows;

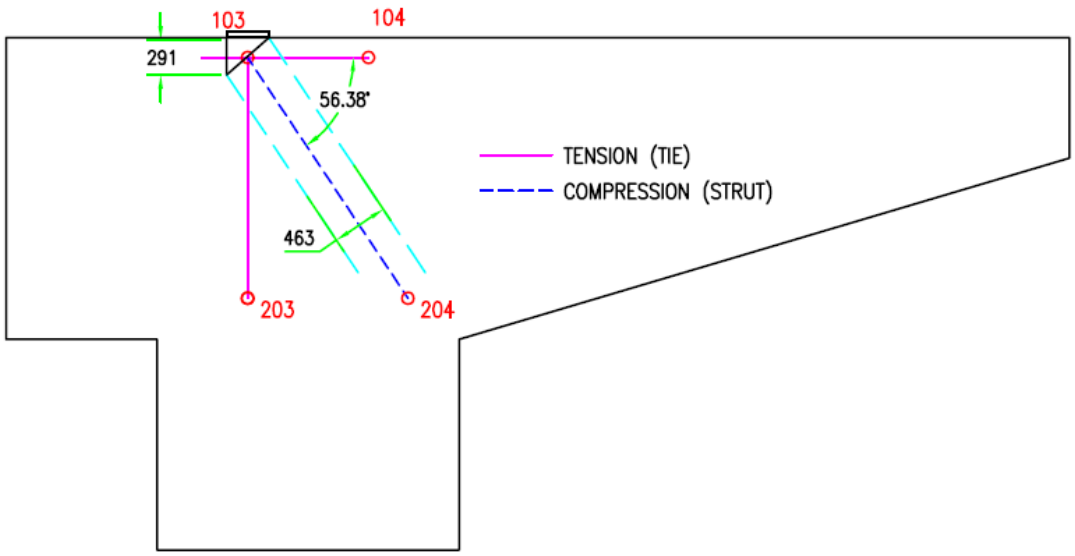


Figure 35. P-11A diagonal strut check

Diagonal Strut *(103-204)* Check based on ACI

Effective Compressive Strength for Node *103*

β_n =	0.6	(CTT)	
f'_c =	40	Mpa	*(cube strength)*
	4,640	psi	
$f_{ce(103)}$ =	$0.85\beta_n f'_c$		*(eq. A-8)*
=	2.37	ksi	
=	16.32	Mpa	

Calculate Width of Tie *103-104*

ϕ =	0.85		*(cl. C.9.3.2.6)*
b_w =	2,500	mm	
	98.5	in.	
F =	8,937	kN	
	2,009	k	

$w_{(103-104)} = F / \phi(b_w)f_{cu}$

=	10.1	in.
=	257.6	mm

Effective Compressive Strength for Strut $_{103\text{-}204}$

β_s =	1.00		*(cl. A.3.2.1)*
f'_c =	40	Mpa	*(cube strength)*
	4,640	psi	*(cylinder strength)*
$f_{ce(103\text{-}204)}$ =	$0.85\beta_s f'_c$		*(eq. A-3)*
=	3.94	ksi	
=	27.20	Mpa	

Check Strut $_{103\text{-}204}$ **Capacity**

$w_{s(103\text{-}204)}$ =	452	mm	*(measured from drawing)*
	17.8	in.	
$\phi F_{ns(103\text{-}204)}$ =	$\phi f_{ce} w_{s(103\text{-}204)} b_w$		*(eq. A-2)*
=	5,876	k	
=	26,137	kN	> Su = 14,532 kN
	O.K!		

Based on the checking, the diagonal compression strut width is measured to be 463 mm. The maximum ultimate compression strut force from the analysis is 14,532 kN, which is lower than the calculated capacity of 26,137 kN. Therefore, the diagonal compression strut capacity satisfies the ultimate limit force from the analysis.

7.4. Finite Element Analysis (FEM) for P-11A

The FEM model was based on BD 37/88 3 notional lanes loading criteria for Serviceability Limit State Load Combination 1.

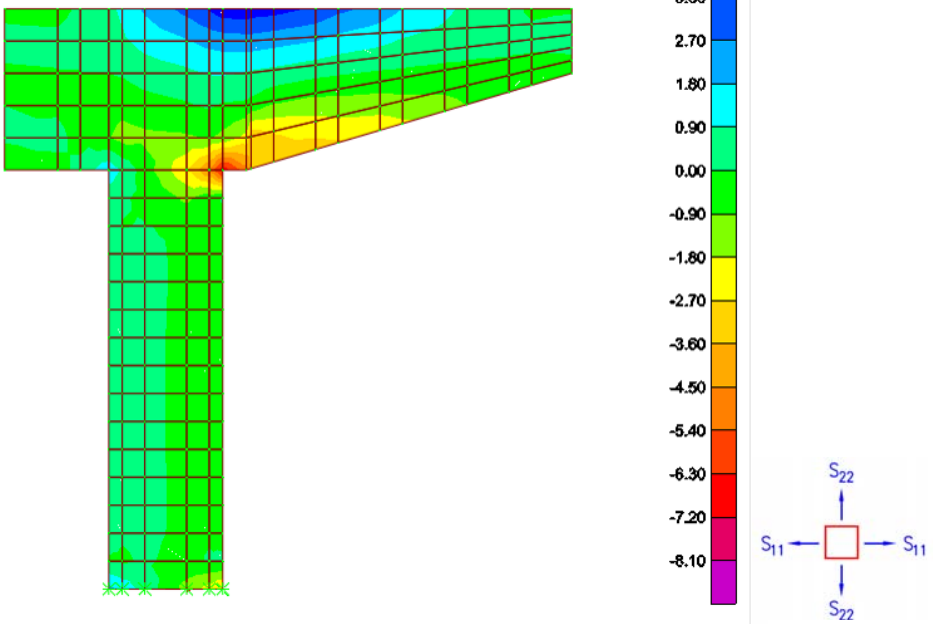


Figure 36. *P-11A S11 stress diagram*

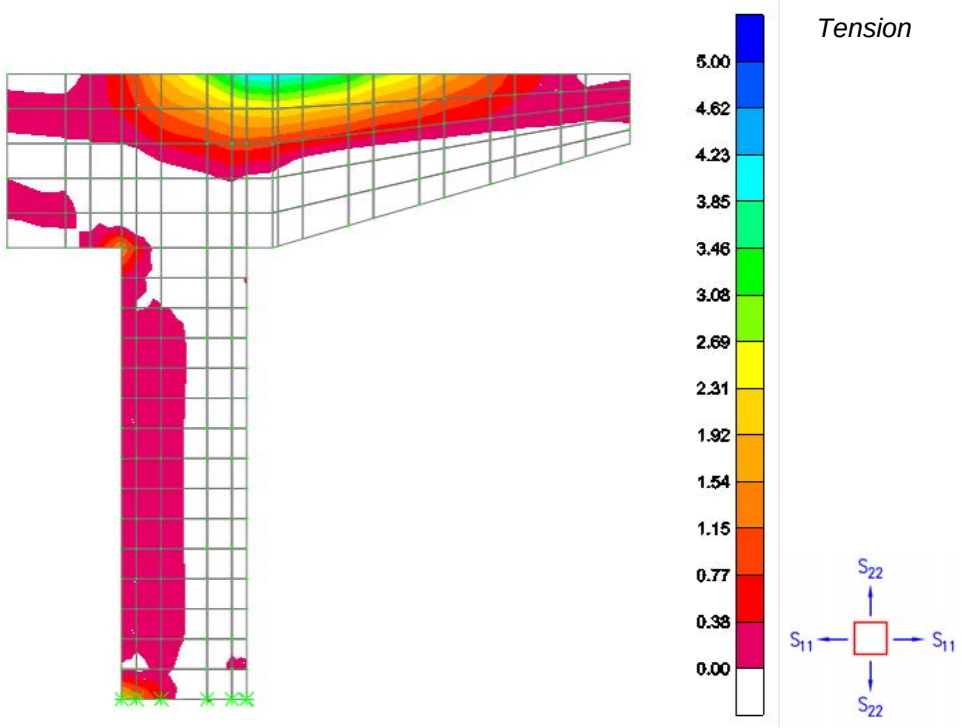


Figure 37. *P-11A S11 tension stress diagram*

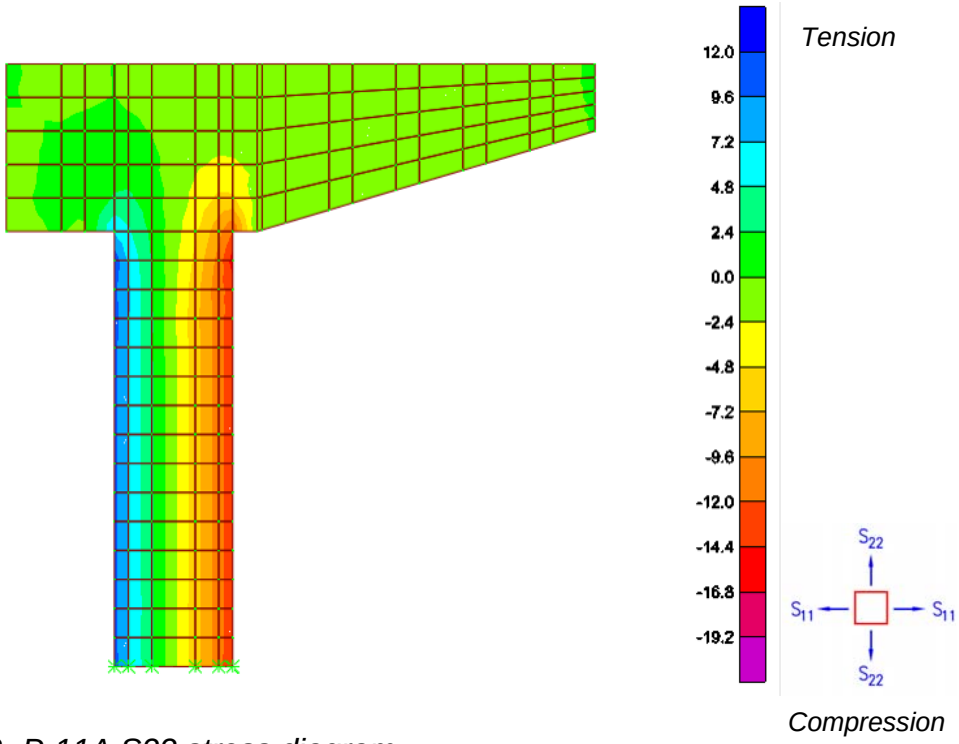


Figure 38. *P-11A S22 stress diagram*

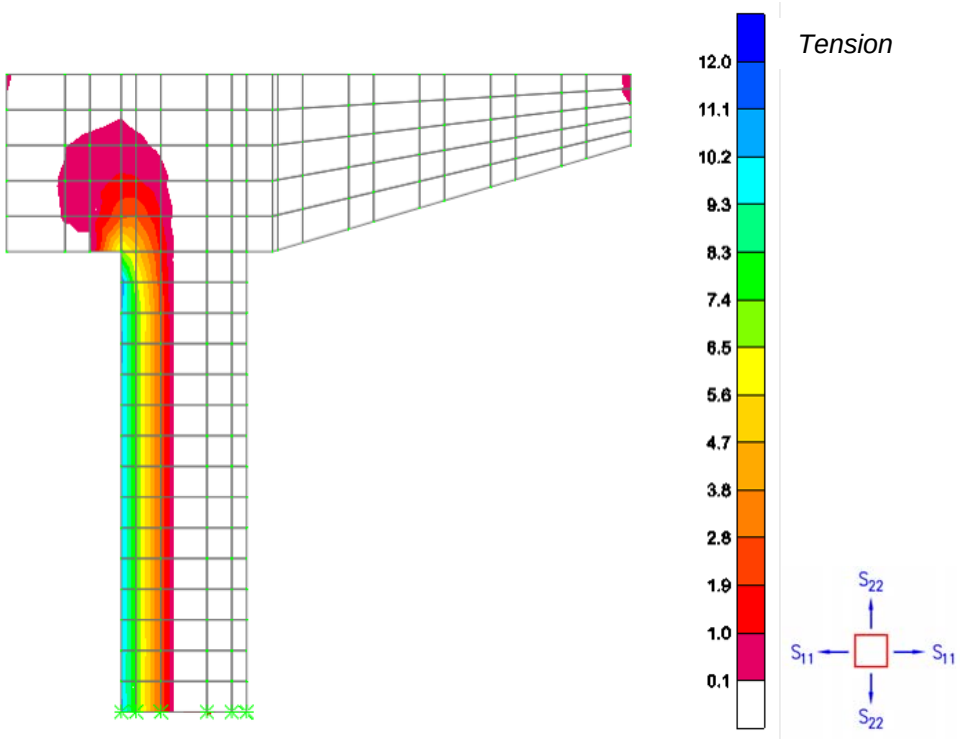


Figure 39. *P-11A S22 tension stress diagram*

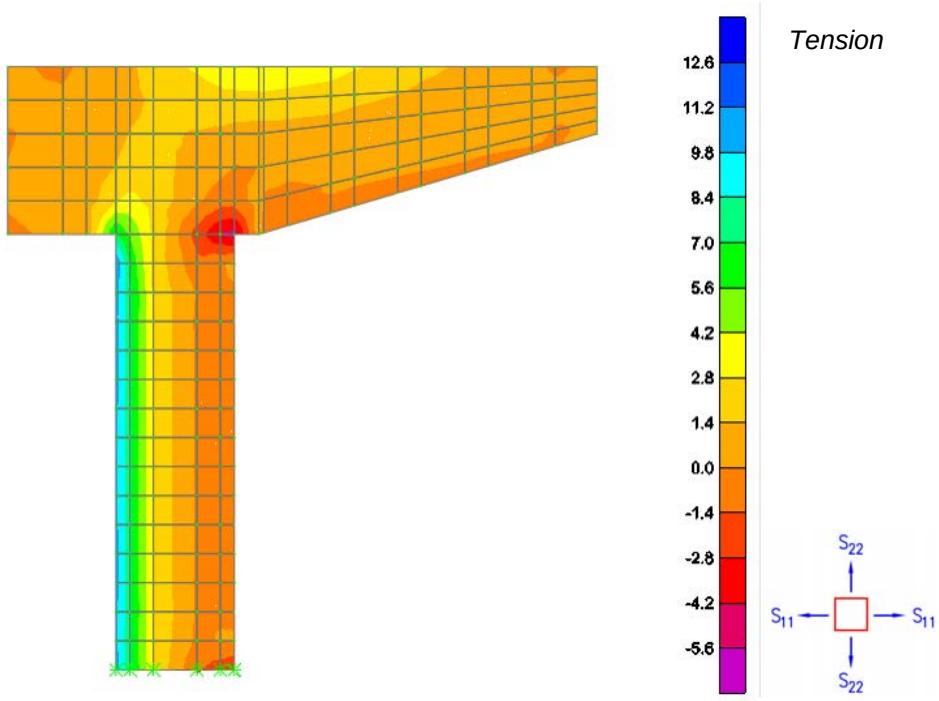


Figure 40. *P-11A SMAX stress diagram*

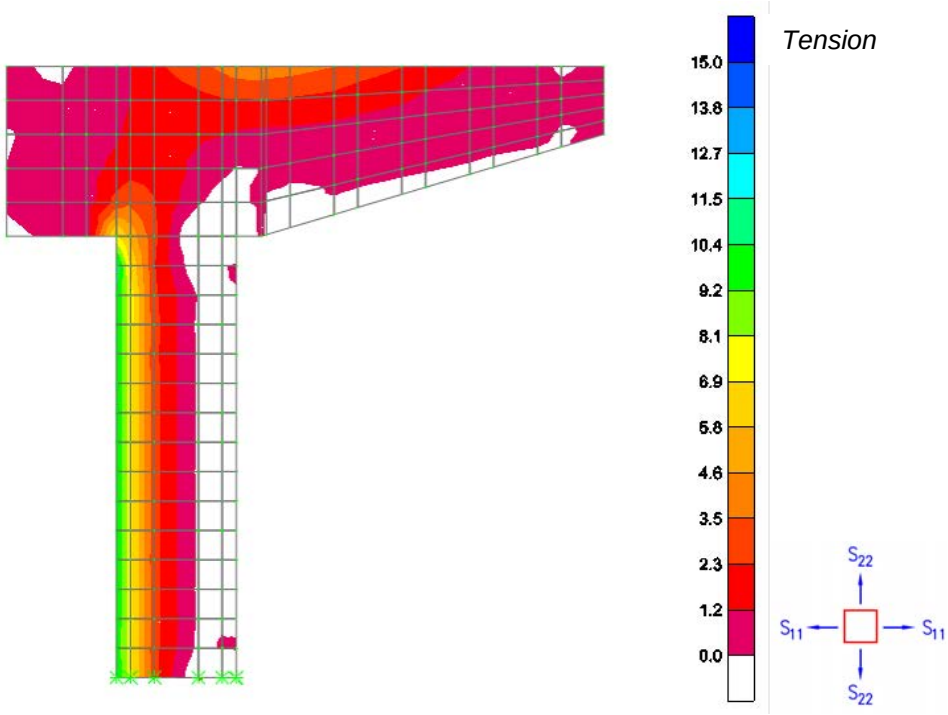


Figure 41. *P-11A SMAX tension stress diagram*

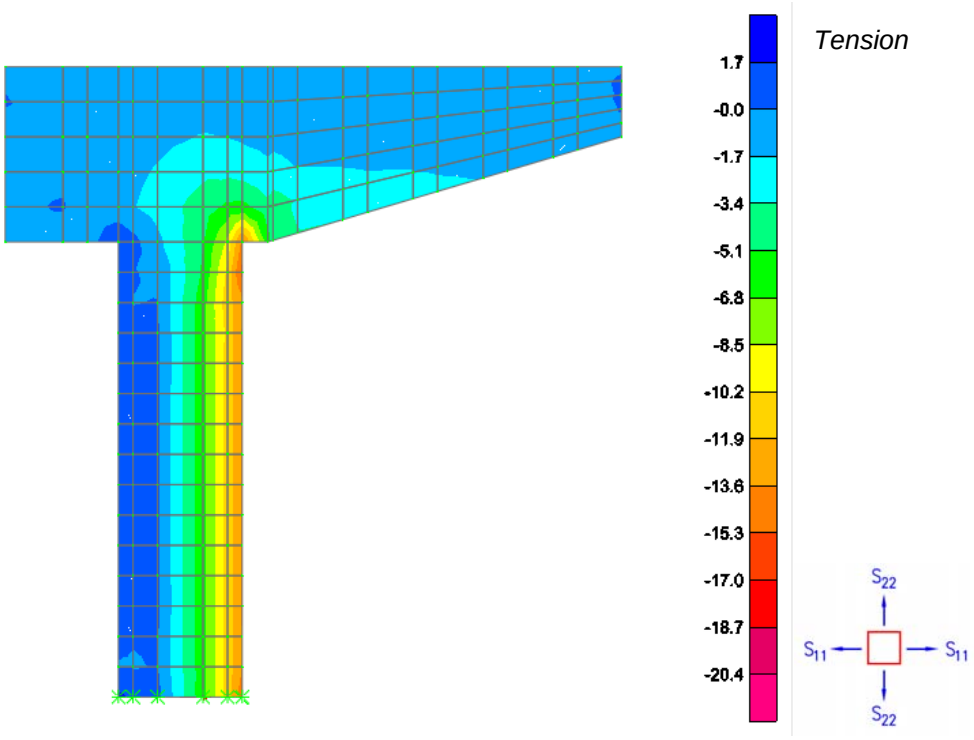


Figure 42. P-11A SMIN tension stress diagram

Based on Figure 39, it is shown that the S22 tension stress extends approximately 2.0m from the top of the pier into the crosshead. Therefore, it would be suggested that the pier main reinforcement should be extended up for a minimum of 0.8 depth of the crosshead followed by tension anchorage length to cater for tension of the reinforcement bars.

7.5. Summary of Design Review for Pier P-11A

(a) The pier column ultimate capacity (ULS) and crack width (SLS) check is summarized as below.

Table 35. P-11A – Summary of pier ULS moment capacity

Pier Type	Ultimate Moment Capacity (kN.m)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-C (P-11A)	O.K	O.K	O.K

Table 36. P-11A – Summary of pier SLS crack width

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-C (P-11A)	***0.416***	***0.370***	***0.398***

Based on the checking, the existing pier column design satisfied the ULS capacity. However, the crack widths computed are more than 0.250mm which exceeded the SLS criteria.

(b) The crosshead ultimate moment capacity (ULS) check is summarized as below.

Table 37. P-11A – Summary of crosshead ULS moment capacity

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	*24,439*	26,069	25,805	1.06
BD 37/88 (2 Notional Lanes)	*24,439*	26,069	24,442	1.00
JKR MTAL (3 Notional Lanes)	*24,439*	26,069	25,173	1.03

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The checking shows that the existing design of crosshead did not meet the ULS moment capacity for BD 37/88 (3 Notional Lanes) and JKR MTAL (3 Notional Lanes) loading criteria when analyzed without the side reinforcement. When side reinforcement is taken into consideration, the crosshead moment capacity is adequate for all three (3) loading cases.

(c) The crosshead ultimate shear capacity (ULS) check is summarized as below.

Table 38. P-11A – Summary of crosshead ULS shear capacity

BD 37/88 (3 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.72	8.04	0.96
ULS2C1	7.28	8.04	0.90
ULS3C1	6.96	8.04	0.87
ULS4C1	7.71	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

BD 37/88 (2 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	6.90	8.04	0.86
ULS2C1	6.64	8.04	0.83
ULS3C1	6.94	8.04	0.86
ULS4C1	7.70	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

JKR MTAL @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.57	8.04	0.94

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

The checking shows that the existing shear capacity design of the crosshead is adequate at ULS.

(d) The crosshead crack width (SLS) check is summarized as below.

Table 39. P-11A – Summary of crosshead SLS crack width

Crosshead Type	3 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	***0.270***	***0.255***

Crosshead Type	2 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	0.196	0.181

Crosshead Type	3 Notional Lanes (JKR MTAL)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	***0.252***	0.238

The checking shows that the existing design of the crosshead did not meet the SLS crack width criteria of 0.250mm for BD 37/88 (3 Notional Lanes) loading criteria. When BD 37/88 (3 Notional Lanes) loading criteria is checked against the existing design, the calculated crack width satisfies the allowable limit of 0.250mm. However, the crack width calculated under JKR MTAL loading criteria only satisfies the 0.250mm limit criteria when side reinforcement is taken into consideration.

(e) The following table shows the comparison of ULS design between conventional beam theory and STM.

Table 40. P-11A – Conventional beam theory vs. STM

Element Force	Conventional Beam Theory	STM
Bending Moment, $As_{req'd}$	2 x 17T32	2 x 15T32
Shear Force, $Asv_{req'd}$	3T16-150	4T16-150

Based on the comparison, it is found that the reinforcement required the resist the bending moment by using STM method is less than the conventional beam theory method. In the other hand, STM design method requires more shear links compared to conventional beam theory design.

(f) Based on Figure (42), the pier column main vertical reinforcement of T32-150 was terminated near top of crosshead without 90° anchorage bent.

From Figure (31) of the STM analysis, the vertical ultimate tension of 10,685kN on the tension tie member (Node 103 to 203) extends from the bottom compression strut to the top tension tie. Although, the reinforcement provided in the pier column tension zone of 32T32 and 3T16-150 of crosshead links is sufficient to resist the tension tie force, but they don't have sufficient anchorage length into the nodal zone to satisfy STM design philosophy.

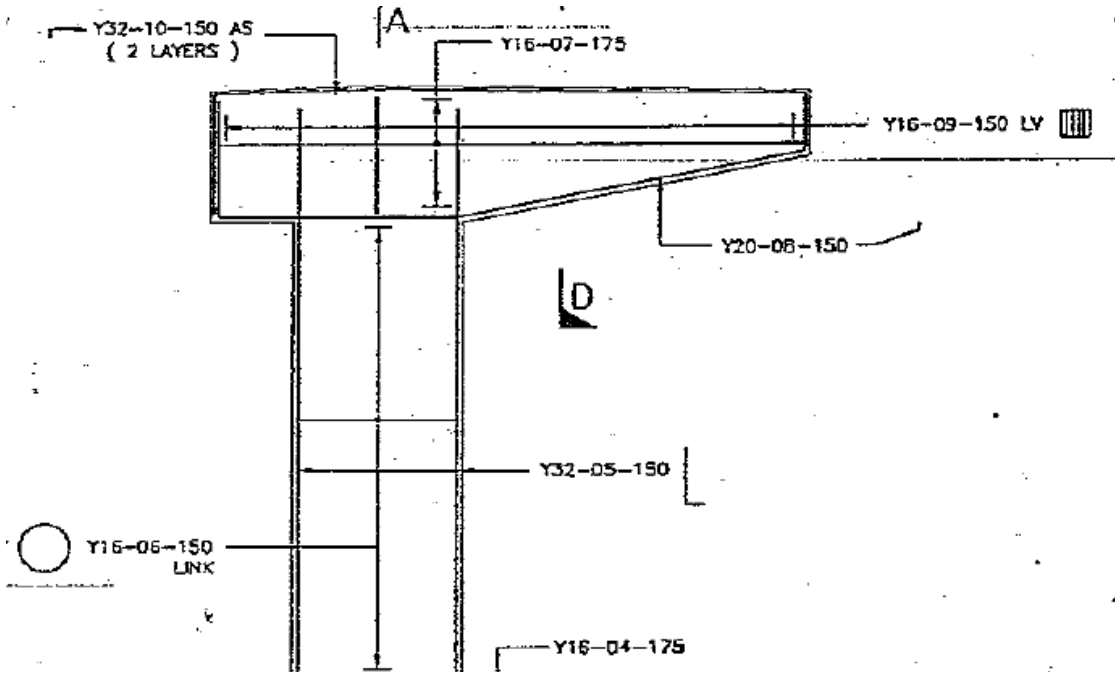


Figure 43. P-11A as-built detailing

Moreover, from Figure (38) of the FEM analysis, the crosshead S22 tension stress is found to extend into crosshead about 2.0m from top of pier column (i.e. soffit of crosshead). Therefore, it is essential that the main column reinforcement should extend further up and followed by a bend for another minimum tension anchorage length beyond the tension zone.

(g) It is found that the conventional method of analysis does not capture the tensile stress in the crosshead as compared to STM or FEM analysis. This is due to the conventional method assumes the crosshead and pier column as frame elements connected at the centroid of the respected elements. Therefore, it is recommended to perform STM and FEM analysis to investigate and capture the behaviour of deep crosshead.

8. DESIGN REVIEW FOR PIER P-25 (TYPE P1-A)

From the 3D analysis, the load effects under each load case can be obtained for P-25.

8.1. Pier Column Check for Pier P-25

The member forces for pier column are presented below for various load combinations. The design checks for pier column members under ULS and SLS are performed.

8.1.1 Analysis Results for Pier P-25 Column

The maximum design forces at pier column base are tabulated.

8.1.1.1 BD 37/88 (3 Notional Lanes)

Table 41. P-25 pier force – BD 37/88 (3 Notional Lanes)

No.	Load Case	N (kN)	M (kN.m)	Combination 1		
				γ_{fL}		γ_{f3}
				SLS	ULS	ULS
1	SW	11789	0	1.00	1.15	1.10
2	Deck Slab	2092	0	1.00	1.15	1.10
3	SDL (Parapet)	1235	0	1.00	1.20	1.10
4	Premix	616	0	1.20	1.75	1.10
5	HA+KEL (1 CARRIAGEWAY)	2558	-11025	1.20	1.50	1.10
6	HA+KEL (2 CARRIAGEWAY)	5115	0	1.20	1.50	1.10
7	HA+HB30 (1 CARRIAGEWAY)	1889	-7833	1.10	1.30	1.10
8	HA+HB30 (2 CARRIAGEWAY)	3973	2380	1.10	1.30	1.10
9	HB45	1588	-9718	1.10	1.30	1.10
10	SV20	3145	-12579	1.10	1.30	1.10

**SW includes 12 nos. precast U (LHS), 14nos. precast M10, 2 nos. precast UM10 (RHS), diaphragms, crosshead and column*

Table 42. P-25 pier force load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)	N_g (kN)	M_g (kN.m)	M_g (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 1 CARRIAGEWAY)	18925	-13230	15856	0	-13230
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 2 CARRIAGEWAY)	21994	0	15856	0	0
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 1 CARRIAGEWAY)	17934	-8617	15856	0	-8617
SLS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 2 CARRIAGEWAY)	20226	2618	15856	0	2618
SLS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	17602	-10689	15856	0	-10689
SLS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	19315	-13837	15856	0	-13837

ULS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 1 CARRIAGEWAY)	24596	-18191
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 2 CARRIAGEWAY)	28817	0
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 1 CARRIAGEWAY)	23078	-11202
ULS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 2 CARRIAGEWAY)	26057	3404
ULS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	22647	-13896
ULS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	24874	-17988

8.1.2 Sectional Capacity Check (ULS) for P-25 Column

The pier section capacity is calculated based on the following as-built information:-

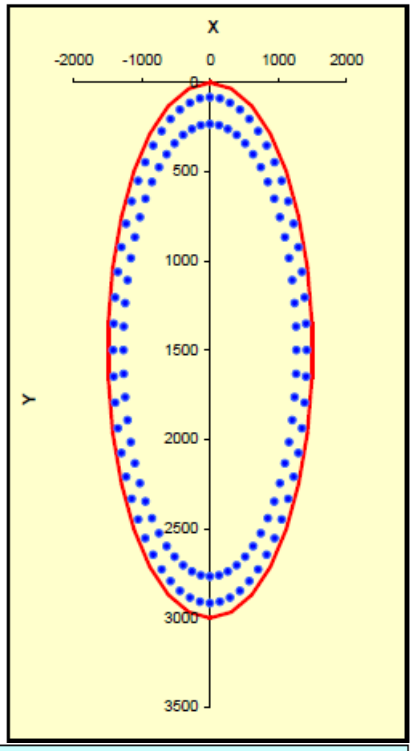
- *Ø3000mm, f_{cu}=40MPa, 120-T32*

8.1.2.1 BD 37/88 (3 Notional Lanes)

Ultimate Section Capacity BS5400

Design Information: PIER TYPE P25 (P1 -A) (3 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1500.000	mm	: Centroid of section
ΣAg=	7,017,020	mm²	: Area of concrete section
ΣAs=	109,378	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	1.56	%	
Ap/Ag=	0.00	%	
Pu,max=	145,983	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

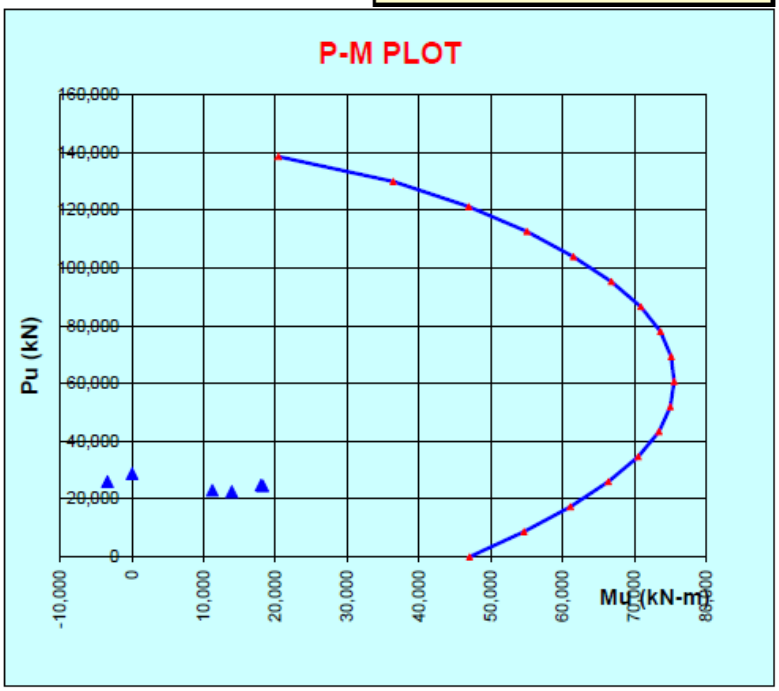
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	6,979,891	mm²	: Area of concrete in compression
Fc=	-111,678	kN	: Force on concrete
Fs=	-27,005	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.232E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	2930.67	mm	
Pu=	138,683	kN	: Ultimate Axial Force (+) compression
Mu=	20,386	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	47,035
2	0.06	8,668	54,611
3	0.12	17,335	61,058
4	0.18	26,003	66,354
5	0.24	34,671	70,509
6	0.30	43,339	73,430
7	0.36	52,006	74,988
8	0.42	60,674	75,521
9	0.48	69,342	75,102
10	0.53	78,009	73,623
11	0.59	86,677	70,872
12	0.65	95,345	66,744
13	0.71	104,013	61,458
14	0.77	112,680	55,019
15	0.83	121,348	46,900
16	0.89	130,016	36,392
17	0.95	138,683	20,386



The applied forces lies within the P-M interaction envelopes; hence the existing design of Pier P-25 is adequate at ULS.

8.1.3 Crack Width Check (SLS) for P-25 Column

The pier crack width is calculated based on the following parameter;

Ø3000mm, f_{cu}=40MPa, 120-T32

8.1.3.1 BD 37/88 (3 Notional Lanes)

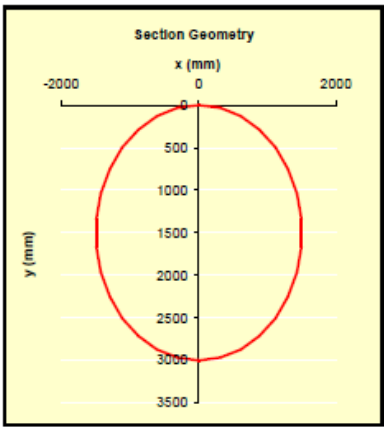
TITTLE : Pier P25 Type P1-A SLS1C1 (3 Notional Lanes)

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m²	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm²	: Steel Yield Strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



Neutral Axis (Elastic Analysis)

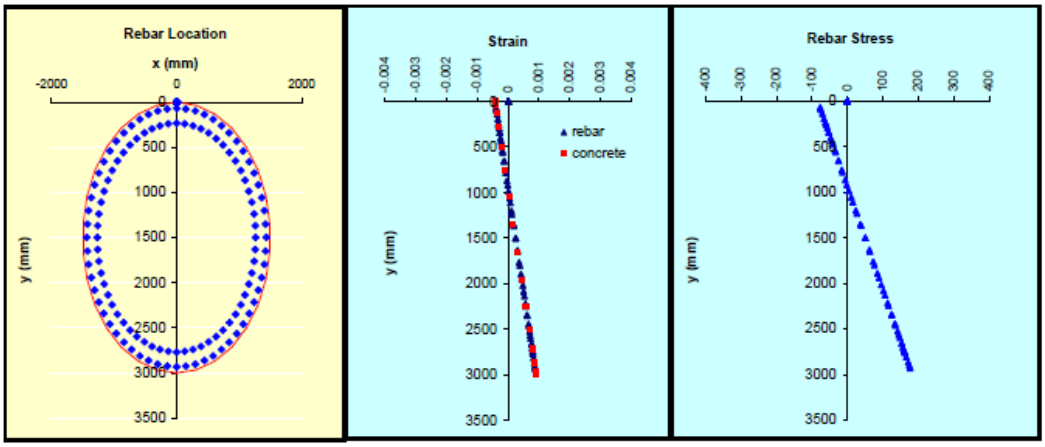
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

ΣAc=	1,850,932	mm²	: Area of concrete
ΣAs=	1,245,287	mm²	: Transformed area of rebar
ΣA=	3,096,219	mm²	: Gross area
Err(x)=	0.0		: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	931.50	mm	

RE_ITERATE

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	-15856	kN	: Permanent Axial Force; (-) compression
M_g =	0	kN-m	: Permanent moment
M_q =	13230	kN-m	: Live load moment
M_s =	13230	kN-m	: Applied SLS moment
h=	3000	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	50	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	5.87E-04		: Average strain at point considered
ε_o =	-3.30E-04		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	0.00E+00		: Strain due to tension stiffening effect
(1-Mq/Mg) =	#DIV/0!		



Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	ε_{sm}	ε_m	W_{max} (mm)
0	0	0	66	32	50	-0.00041	-3.30E-04	0	-7.44E-04	uncracked
0	0	0	0	0	0	-0.00041	-3.30E-04	0	-7.44E-04	uncracked
0	0	0	0	0	0	-0.00041	-3.30E-04	0	-7.44E-04	uncracked
0	0	0	0	0	0	-0.00041	-3.30E-04	0	-7.44E-04	uncracked
0	0	0	0	0	0	-0.00041	-3.30E-04	0	-7.44E-04	uncracked
0	3000	0	2934	32	50	0.000918	-3.30E-04	0.00E+00	5.87E-04	0.087

The computed crack width for Pier P-25 (Type P1-A) is summarized as follows:-

Table 43. Summary of P-25 SLS crack width check

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-25)	0.087	-	-

Hence, the existing design of Pier P-25 is less than the allowable crack width of 0.25mm.

8.2. Crosshead Check for Pier P-25

The member forces of crosshead are presented below for various load combinations. The design checks for crosshead members under ULS and SLS are performed based on the following as-built drawing.

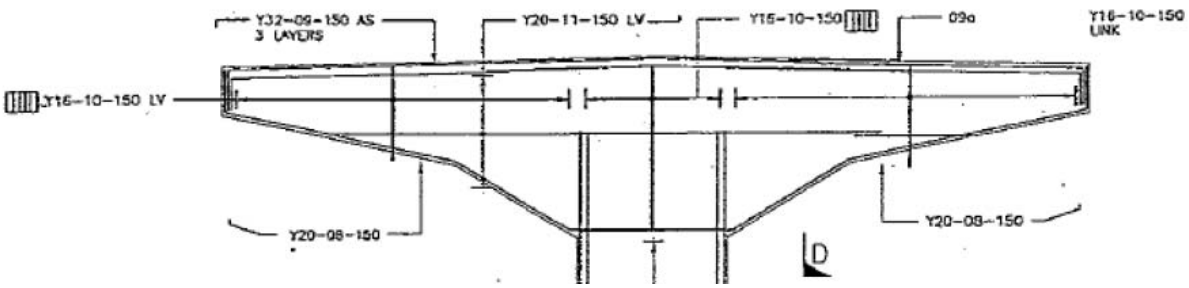


Figure 44. P-25 As-built crosshead reinforcement

8.2.1 Analysis Results for Pier P-25 Crosshead

The maximum design forces for crosshead are tabulated.

8.2.1.1 BD 37/88 (3 Notional Lanes)

Table 44. P-25 crosshead moment – BD 37/88 (3 Notional Lanes)

No.	Load Case	Mmax (kN.m)	Combination 1		
			γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	18509	1.00	1.15	1.10
2	Deck Slab	4325	1.00	1.15	1.10
3	SDL (Parapet)	3014	1.00	1.20	1.10
4	Premix	1273	1.20	1.75	1.10
5	HA+KEL	11041	1.20	1.50	1.10
6	HA+HB30	10191	1.10	1.30	1.10
7	HB45	9913	1.10	1.30	1.10
8	SV20	12675	1.10	1.30	1.10

**SW includes 12 nos. precast U (LHS), 14nos. precast M10, 2 nos. precast UM10 (RHS), diaphragms and crosshead*

Table 45. P-25 crosshead moment load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	M (kN.m)	M_g (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	40624	27375	13249
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	38585	27375	11210
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	38279	27375	10904
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	41317	27375	13942

ULS Design to Load Combination 1

Case #	Load Combination	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	53530
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	49886
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	49488
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	53438

Table 46. P-25 crosshead shear @ 2.0m depth – BD 37/88 (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	-2191	1.00	1.15	1.10
2	Deck Slab	-535	1.00	1.15	1.10
3	SDL (Parapet)	-362	1.00	1.20	1.10
4	Premix	-158	1.20	1.75	1.10
5	HA+KEL	-1431	1.20	1.50	1.10
6	HA+HB30	-1028	1.10	1.30	1.10
7	HB45	-1412	1.10	1.30	1.10
8	SV20	-1641	1.10	1.30	1.10

*StaadPro member 6208

**SW includes 12 nos. precast U (LHS), 14nos. precast M10, 2 nos. precast UM10 (RHS), diaphragms and crosshead*

Table 47. P-25 crosshead shear @ 2.0m depth load combination – BD 37/88 (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	-6590
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	-5700
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	-6249
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	-6576

Table 48. P-25 crosshead shear @ 3.5m depth – BD 37/88 (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	-3666	1.00	1.15	1.10
2	Deck Slab	-809	1.00	1.15	1.10
3	SDL	-447	1.00	1.20	1.10
4	Premix	-238	1.20	1.75	1.10
5	HA+KEL	-1977	1.20	1.50	1.10
6	HA+HB30	-1343	1.10	1.30	1.10
7	HB45	-1502	1.10	1.30	1.10
8	SV20	-2625	1.10	1.30	1.10

*StaadPro member 6213

**SW includes 12 nos. precast U (LHS), 14nos. precast M10, 2 nos. precast UM10 (RHS), diaphragms and crosshead*

Table 49. P-25 crosshead shear @ 3.5m load combination – BD 37/88 (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	-9973
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	-8631
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	-8859
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	-10465

8.2.2 Section Capacity Check (ULS) for Pier P-25 Crosshead

The crosshead is checked for its moment and shear capacity under Ultimate Limit State (ULS)

The crosshead section capacity is calculated based on the following as built information:-

Crosshead P-25 (*Type P1-A*)

- *Width = 3000mm, Depth = 3500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (3 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

8.2.2.1 Ultimate Moment Capacity Check for P-25 Crosshead

The computed crosshead ultimate moment capacity for Pier P-25 (Type P1-A) is computed and compared with the ULS applied moments.

Table 50. Summary of P-25 crosshead ULS moment capacity check

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	59,830	65,605	53,530	0.89

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The applied moment lies within the P-M interaction envelope. Hence the existing crosshead design for P-25 (Type P1-A) is adequate at ULS.

The detailed computations of the sectional moment capacity are presented below.

8.2.2.1.1 BD 37/88 (3 Notional Lanes)

**Without Side Reinforcement*

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P25 TYPE P1 -A (WITHOUT SIDEBARS) 3 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1750.000	mm	: Centroid of section
ΣAg=	10,500,000	mm²	: Area of concrete section
ΣAs=	54,538	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.52	%	
Ap/Ag=	0.00	%	
Pu,max=	184,809	kN	: 0.4fcuAg+0.67fyAs

Neutral Axis

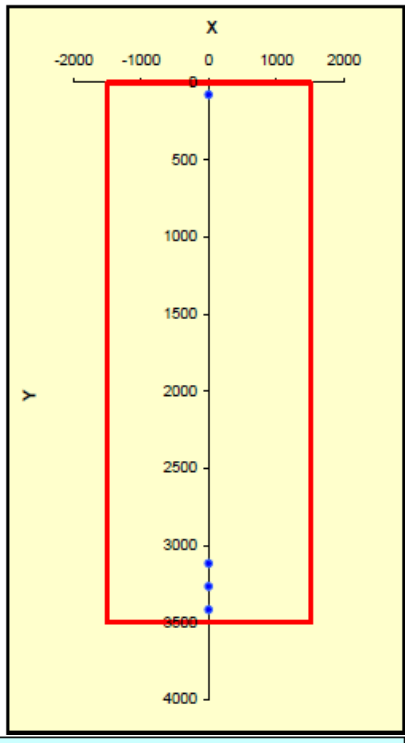
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	1,075,472	mm²	: Area of concrete in compression
Fc=	-17,208	kN	: Force on concrete
Fs=	17,208	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)

Yn= 358.49 mm

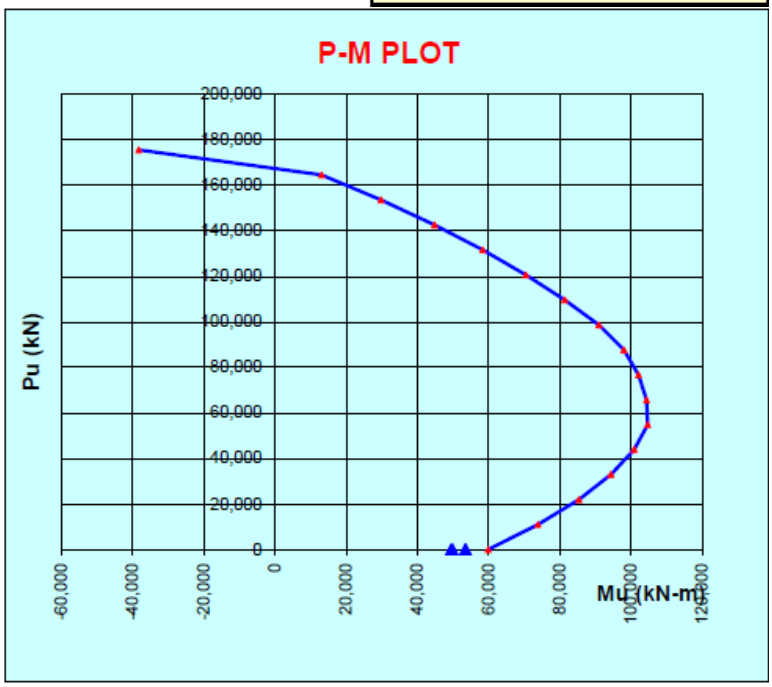
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	59,830	kN-m	: Ultimate Bending Capacity

Go



P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	59,830
2	0.06	10,973	73,845
3	0.12	21,946	85,351
4	0.18	32,919	94,349
5	0.24	43,892	100,838
6	0.30	54,865	104,628
7	0.36	65,838	104,379
8	0.42	76,811	102,067
9	0.48	87,784	97,987
10	0.53	98,757	90,929
11	0.59	109,730	81,341
12	0.65	120,703	70,459
13	0.71	131,676	58,330
14	0.77	142,649	44,807
15	0.83	153,622	29,758
16	0.89	164,595	13,065
17	0.95	175,568	-38,278

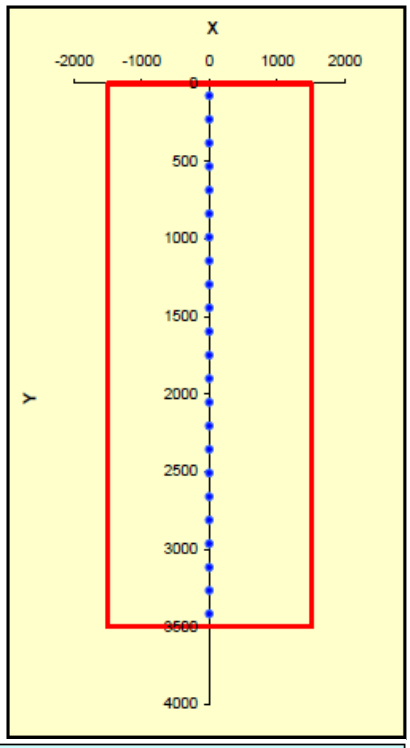


** With Side Reinforcement T20-150 (Both Sides)*

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P25 TYPE P1 -A (WITH SIDEBARS) 3 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1750.000	mm	: Centroid of section
ΣAg=	10,500,000	mm²	: Area of concrete section
ΣAs=	66,476	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.63	%	
Ap/Ag=	0.00	%	
Pu,max=	188,488	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

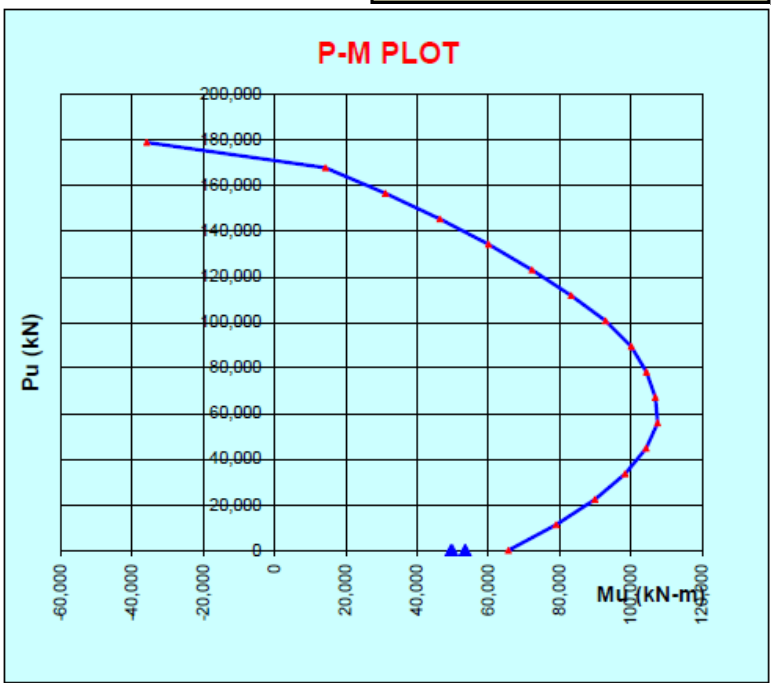
Neutral Axis, Yn, where **Σ(Fc+Fp+Fs+Pu)=0**

ΣAc=	1,312,843	mm²	: Area of concrete in compression
Fc=	-21,005	kN	: Force on concrete
Fs=	21,005	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.025E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	437.61	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	65,605	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	65,605
2	0.06	11,191	78,986
3	0.12	22,383	89,895
4	0.18	33,574	98,332
5	0.24	44,766	104,296
6	0.30	55,957	107,493
7	0.36	67,149	106,843
8	0.42	78,340	104,352
9	0.48	89,532	100,097
10	0.53	100,723	92,887
11	0.59	111,915	83,216
12	0.65	123,106	72,240
13	0.71	134,298	60,010
14	0.77	145,489	46,369
15	0.83	156,681	31,157
16	0.89	167,872	14,273
17	0.95	179,064	-35,889



8.2.2.2 Ultimate Shear Capacity Check for P-25 Crosshead

The crosshead ultimate shear link required for Pier P-25 (Type P1-A) is computed and compared to the shear link provided.

Table 51. Summary of P-25 ULS shear force capacity check @ 2.0m depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	**8.46**	8.04	**1.05**
ULS2C1	7.19	8.04	0.89
ULS3C1	7.98	8.04	0.99
ULS4C1	**8.44**	8.04	**1.05**

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Table 52. Summary of P-25 ULS shear force capacity check @ 3.5m depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.60	8.04	0.94
ULS2C1	6.57	8.04	0.82
ULS3C1	6.74	8.04	0.84
ULS4C1	7.97	8.04	0.99

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Based on the checking, it is found that the shear link provided at the 2.0m depth crosshead section is marginally insufficient for load case ULS1C1 (HA+KEL) and ULS4C1 (SV20).

However, the shear link provided at the 3.5m depth crosshead section is sufficient to resist the ultimate shear force for all the load cases.

The detailed computations of the sectional shear capacities are presented as below.

8.2.2.1.1 BD 37/88 (3 Notional Lanes)

*ULS1C1 @ 2.0m Depth

Element ID = **P-25 Crosshead (ULS1C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	6,590	kN			
v	=	1.25	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	8.46				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		

Not Sufficient!

*ULS2C1 @ 2.0m Depth

Element ID = **P-25 Crosshead (ULS2C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	5,700	kN			
v	=	1.08	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	7.19				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		

Sufficient!

*ULS3C1 @ 2.0m Depth

Element ID = **P-25 Crosshead (ULS3C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	6,249	kN			
v	=	1.19	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	7.98				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

*ULS4C1 @ 2.0m Depth

Element ID = **P-25 Crosshead (ULS4C1 - SV20) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	6,576	kN			
v	=	1.25	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	8.44				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Not Sufficient!				

*ULS1C1 @ 3.5m Depth

Element ID = **P-25 Crosshead (ULS1C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	9,973	kN				
v	=	1.02	N/mm^2	Remarks :	**O.K**		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.60					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS2C1 @ 3.5m Depth

Element ID = **P-25 Crosshead (ULS2C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	8,631	kN				
v	=	0.88	N/mm^2	Remarks :	**O.K**		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.57					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS3C1 @ 3.5m Depth

Element ID = **P-25 Crosshead (ULS3C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	8,859	kN				
v	=	0.91	N/mm^2	Remarks :	**O.K**		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.74					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS4C1 @ 3.5m Depth

Element ID = **P-25 Crosshead (ULS4C1 - SV20) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	10,465	kN				
v	=	1.07	N/mm^2	Remarks :	**O.K**		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.97					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

8.2.3 Crack Width Check (SLS) for P-25 Crosshead

The crosshead crack width is calculated based on the following parameter;

Crosshead P-25 (*Type P1-A*)

- *Width = 3000mm, Depth = 3500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (3 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

The computed crosshead crack width for Pier P-25 (Type P1-A) is summarized as follows:-

Table 53. Summary of P-25 SLS crack width check

Loading Criteria	Crack Width (mm)	
	Without Sidebar	With Sidebar
BD 37/88 (3 Notional Lanes)	0.138	0.115

The computed crack width is 0.138mm without taking into account side reinforcement and 0.115mm with side reinforcement. Hence, the crack width is less than the allowable limit of 0.250mm

8.2.3.1 BD 37/88 (3 Notional Lanes)

** Without Side Reinforcement*

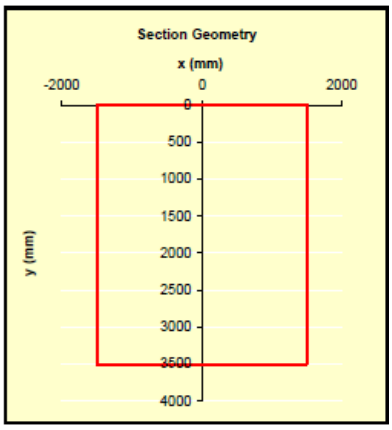
TITTLE : Crosshead P25 Type P1-A SLS1C1 (WITHOUT SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



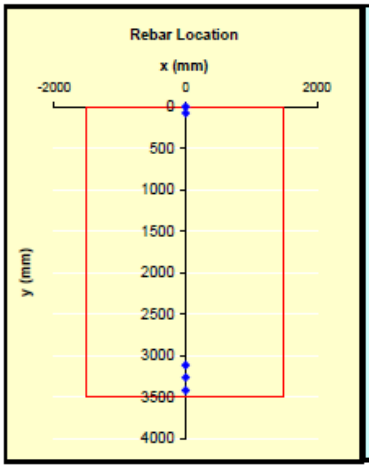
Neutral Axis (Elastic Analysis)

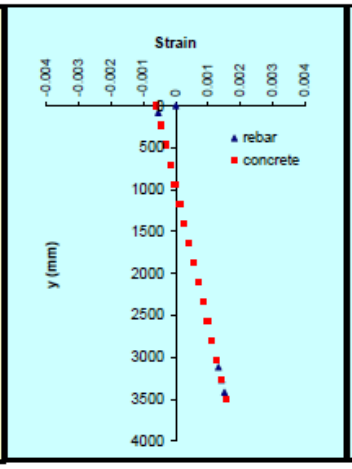
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

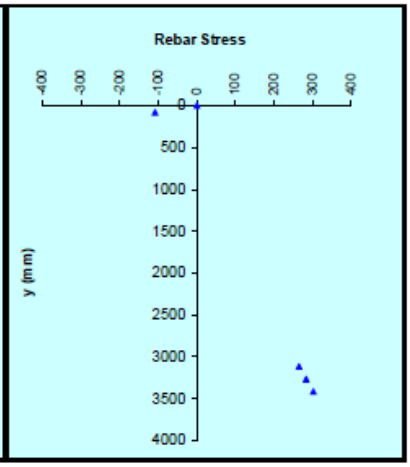
ΣAc=	2,865,904	mm^2	: Area of concrete
ΣAs=	703,717	mm^2	: Transformed area of rebar
ΣA=	3,569,621	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	955.30	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	27379	kN-m	: Permanent moment
M_q =	13249	kN-m	: Live load moment
M_s =	40628	kN-m	: Applied SLS moment
h=	3500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	7.13E-04		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-8.48E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	5.16E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.86E-04	0.00E+00	0.00E+00	-5.86E-04	uncracked
0	0	0	0	0	0	-5.86E-04	0.00E+00	0.00E+00	-5.86E-04	uncracked
0	0	0	0	0	0	-5.86E-04	0.00E+00	0.00E+00	-5.86E-04	uncracked
0	0	0	0	0	0	-5.86E-04	0.00E+00	0.00E+00	-5.86E-04	uncracked
0	0	0	0	0	0	-5.86E-04	0.00E+00	0.00E+00	-5.86E-04	uncracked
0	3500	0	3418	32	66	1.56E-03	0.00E+00	-8.48E-04	7.13E-04	0.138

* With Side Reinforcement T20-150 (Both Sides)

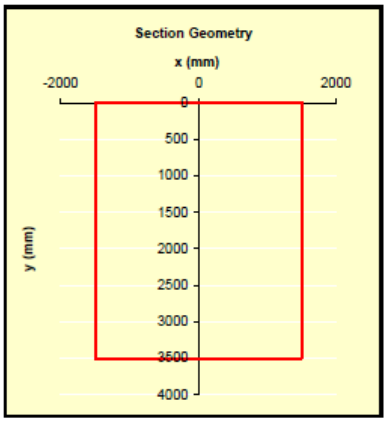
TITTLE : Crosshead P25 Type P1-A SLS1C1 (WITH SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



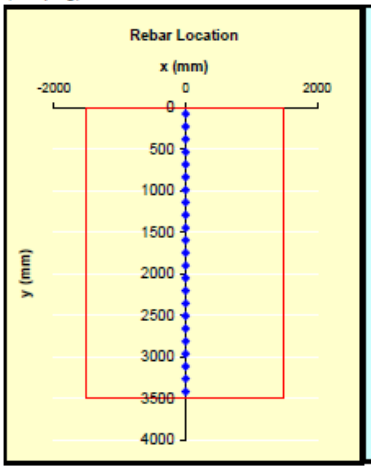
Neutral Axis (Elastic Analysis)

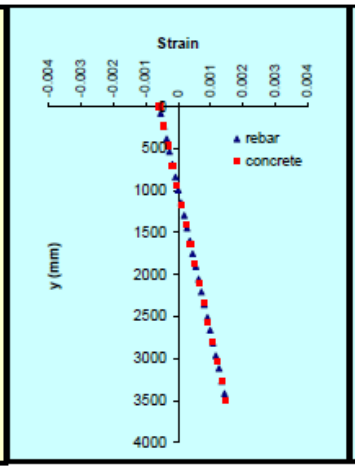
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

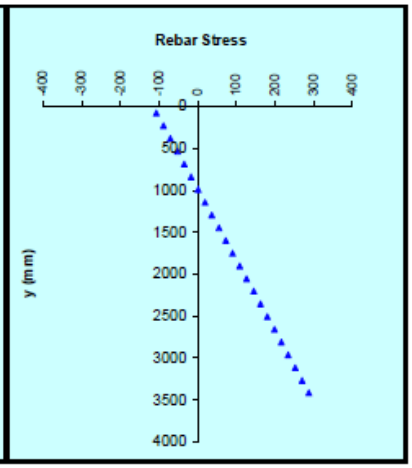
ΣAc=	2,944,950	mm^2	: Area of concrete
ΣAs=	857,756	mm^2	: Transformed area of rebar
ΣA=	3,802,706	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	981.65	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	27375	kN-m	: Permanent moment
M_q =	13249	kN-m	: Live load moment
M_s =	40624	kN-m	: Applied SLS moment
h=	3500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	5.96E-04		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-8.90E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	5.16E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{suff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.79E-04	0.00E+00	0.00E+00	-5.79E-04	uncracked
0	0	0	0	0	0	-5.79E-04	0.00E+00	0.00E+00	-5.79E-04	uncracked
0	0	0	0	0	0	-5.79E-04	0.00E+00	0.00E+00	-5.79E-04	uncracked
0	0	0	0	0	0	-5.79E-04	0.00E+00	0.00E+00	-5.79E-04	uncracked
0	0	0	0	0	0	-5.79E-04	0.00E+00	0.00E+00	-5.79E-04	uncracked
0	3500	0	3418	32	66	1.49E-03	0.00E+00	-8.90E-04	5.96E-04	0.115

8.3. Summary of Design Review for P-25

(a) The pier ultimate capacity (ULS) and crack width (SLS) check is summarized as below.

Table 54. P-25 - Summary of pier ULS moment capacity

Pier Type	Ultimate Moment Capacity (kN.m)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-25)	O.K	-	-

Table 55. P-25 - Summary of pier SLS crack width

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-25)	0.087	-	-

Based on the checking, the existing pier column design satisfied the ULS and SLS criteria.

(b) The crosshead ultimate moment capacity (ULS) check is summarized as below.

Table 56. P-25 – Summary of crosshead ULS moment capacity

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	59,830	65,605	53,530	0.89

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The checking shows that the existing design of the crosshead satisfies the ULS moment capacity.

(c) The crosshead ultimate shear capacity (ULS) check is summarized as below.

Table 57. P-25 – Summary of crosshead ULS shear capacity @ 2.0m depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	8.46	8.04	1.05
ULS2C1	7.19	8.04	0.89
ULS3C1	7.98	8.04	0.99
ULS4C1	8.44	8.04	1.05

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Table 58. P-25 – Summary of crosshead ULS shear capacity @ 3.5m depth

Load Case	Asv/sv $_{req'd}$	Asv/sv $_{prov}$	Capacity Ratio
ULS1C1	7.60	8.04	0.94
ULS2C1	6.57	8.04	0.82
ULS3C1	6.74	8.04	0.84
ULS4C1	7.97	8.04	0.99

*Capacity ratio is based on Asv/sv $_{req'd}$ / Asv/sv $_{prov}$

The checking shows that the existing shear capacity design at 2.0m crosshead depth section did not fulfill the ULS requirement for load case ULS1C1 (HA+KEL) and ULS4C1 (SV20). In the other hand, the existing shear capacity design at 3.5m crosshead depth section satisfies the ULS criteria.

(d) The crosshead crack width (SLS) check is summarized as below.

Table 59. P-25 – Summary of crosshead SLS crack width

Crosshead Type	3 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-A (P-25)	0.138	0.115

The checking shows that the existing design of the crosshead satisfies the SLS crack width criteria of 0.250mm.

9. DESIGN REVIEW FOR PIER P-33 (TYPE P1-A)

From the 3D analysis, the load effects under each load case can be obtained for P-33.

9.1. Pier Column Check for Pier P-33

The member forces for pier column are presented below for various load combinations. The design checks for pier column members under ULS and SLS are performed.

9.1.1 Analysis Results for Pier P-33 Column

The maximum design forces at pier column base are tabulated.

9.1.1.1 BD 37/88 (3 Notional Lanes)

Table 60. P-33 pier force – BD 37/88 (3 Notional Lanes)

No.	Load Case	N (kN)	M (kN.m)	Combination 1		γ_{f3}
				γ_{fL}		
				SLS	ULS	ULS
1	SW	10350	-4	1.00	1.15	1.10
2	Deck Slab	1860	-2	1.00	1.15	1.10
3	SDL (Parapet)	1098	-3	1.00	1.20	1.10
4	Premix	547	-1	1.20	1.75	1.10
5	HA+KEL (1 CARRIAGEWAY)	2574	-11595	1.20	1.50	1.10
6	HA+KEL (2 CARRIAGEWAY)	5148	0	1.20	1.50	1.10
7	HA+HB30 (1 CARRIAGEWAY)	1881	-8130	1.10	1.30	1.10
8	HA+HB30 (2 CARRIAGEWAY)	4182	2250	1.10	1.30	1.10
9	HB45	1551	-10063	1.10	1.30	1.10
10	SV20	3005	-12640	1.10	1.30	1.10

**SW includes 14 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms, crosshead and column*

Table 61. P-33 pier force load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)	N_q (kN)	M_q (kN.m)	M_q (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 1 CARRIAGEWAY)	17053	-13923	13965	-9	-13914
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 2 CARRIAGEWAY)	20142	-9	13965	-9	0
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 1 CARRIAGEWAY)	16034	-8952	13965	-9	-8943
SLS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 2 CARRIAGEWAY)	18565	2466	13965	-9	2475
SLS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	15671	-11079	13965	-9	-11070
SLS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	17270	-13913	13965	-9	-13904

ULS Design to Load Combination 1

Case #	Load Combination	N (kN)	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 1 CARRIAGEWAY)	22195	-19143
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL 2 CARRIAGEWAY)	26442	-12
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 1 CARRIAGEWAY)	20639	-11637
ULS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30 2 CARRIAGEWAY)	23929	3206
ULS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	20167	-14402
ULS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	22246	-18087

9.1.2 Section Capacity Check (ULS) for Pier P-33 Column

The pier section capacity is calculated based on the following parameter;

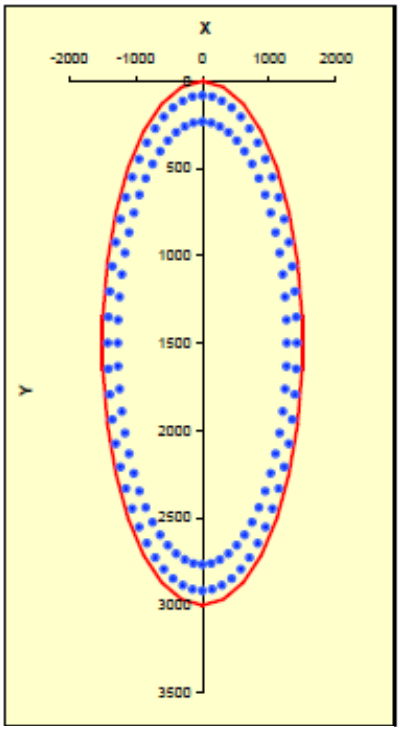
- *Ø3000mm, f_{cu}=40MPa, 120-T32*

9.1.2.1 BD 37/88 (3 Notional Lanes)

Ultimate Section Capacity BS5400

Design Information: PIER TYPE P33 (P1 -A) (3 Notional Lanes)

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1500.000	mm	: Centroid of section
ΣAg=	7,017,020	mm²	: Area of concrete section
ΣAs=	109,378	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	1.56	%	
Ap/Ag=	0.00	%	
Pu,max=	145,983	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

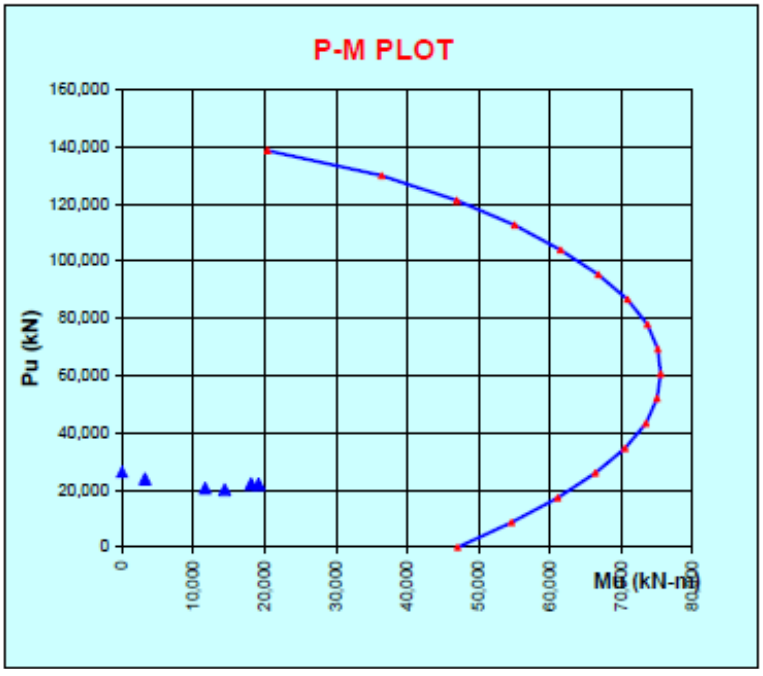
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	6,979,891	mm²	: Area of concrete in compression
Fc=	-111,678	kN	: Force on concrete
Fs=	-27,005	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.232E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	2930.67	mm	
Pu=	138,683	kN	: Ultimate Axial Force (+) compression
Mu=	20,386	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	47,035
2	0.06	8,668	54,611
3	0.12	17,335	61,058
4	0.18	26,003	66,354
5	0.24	34,671	70,509
6	0.30	43,339	73,430
7	0.36	52,006	74,988
8	0.42	60,674	75,521
9	0.48	69,342	75,102
10	0.53	78,009	73,623
11	0.59	86,677	70,872
12	0.65	95,345	66,744
13	0.71	104,013	61,458
14	0.77	112,680	55,019
15	0.83	121,348	46,900
16	0.89	130,016	36,392
17	0.95	138,683	20,386



The applied forces lies within the P-M interaction envelopes; hence the existing design of Pier P-11A is adequate at ULS.

9.1.3 Crack Width Check (SLS) for Pier P-33 Column

The pier crack width is calculated based on the following parameter;

Ø3000mm, f_{cu}=40MPa, 120-T32

9.1.3.1 BD 37/88 (3 Notional Lanes)

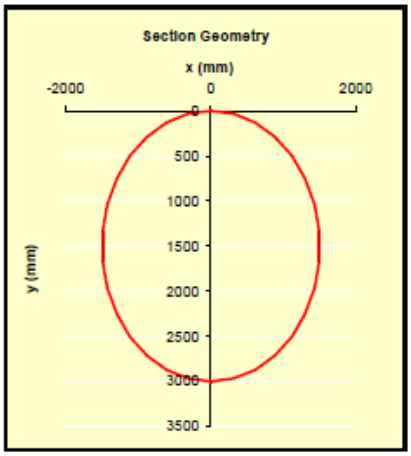
TITTLE : Pier P33 Type P1-A SLS1C1 (3 Notional Lanes)

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m²	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm²	: Steel Yield Strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



Neutral Axis (Elastic Analysis)

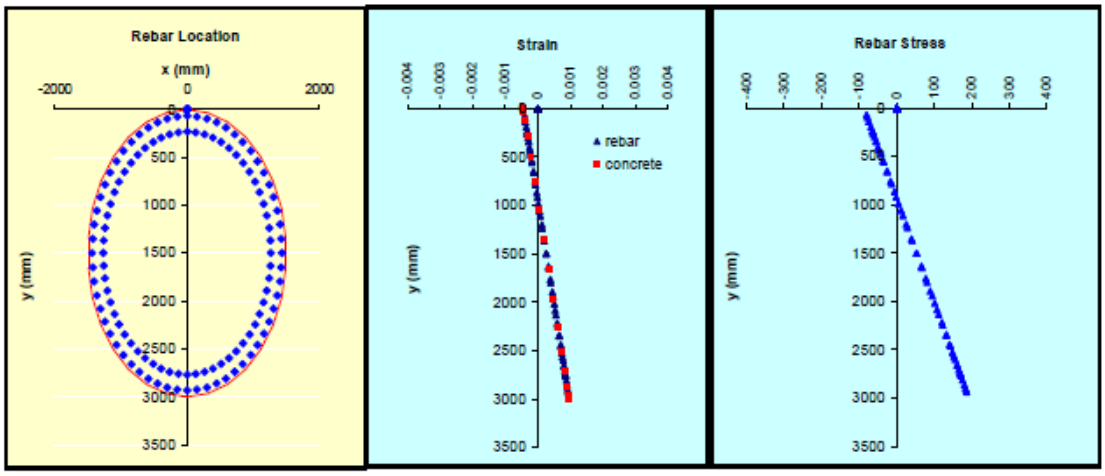
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

ΣAc=	1,850,932	mm²	: Area of concrete
ΣAs=	1,245,287	mm²	: Transformed area of rebar
ΣA=	3,096,219	mm²	: Gross area
Err(x)=	0.0		: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	931.50	mm	

RE_ITERATE

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	-13965	kN	: Permanent Axial Force; (-) compression
M_g =	9	kN-m	: Permanent moment
M_q =	13914	kN-m	: Live load moment
M_s =	13923	kN-m	: Applied SLS moment
h=	3000	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	50	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	6.75E-04		: Average strain at point considered
ε_o =	-2.91E-04		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	0.00E+00		: Strain due to tension stiffening effect
(1-Mq/Mg) =	-1.55E+03		



Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	*0*	*0*	*66*	*32*	*50*	*-0.00043*	*-2.91E-04*	*0*	*-7.26E-04*	*uncracked*
0	0	0	0	0	0	-0.00043	-2.91E-04	0	-7.26E-04	uncracked
0	0	0	0	0	0	-0.00043	-2.91E-04	0	-7.26E-04	uncracked
0	0	0	0	0	0	-0.00043	-2.91E-04	0	-7.26E-04	uncracked
0	0	0	0	0	0	-0.00043	-2.91E-04	0	-7.26E-04	uncracked
0	*3000*	*0*	*2934*	*32*	*50*	*0.000966*	*-2.91E-04*	*0.00E+00*	*6.75E-04*	*0.100*

The computed crack width for Pier P-33 (Type P1-A) is summarized as follows;

Table 62. Summary of P-33 crack width check

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-33)	0.100	-	-

Hence, the existing design of Pier P-33 is less than the allowable crack width of 0.25mm.

9.2. Crosshead Check for Pier P-33

The member forces of crosshead are presented below for various load combinations. The design checks for crosshead members under ULS and SLS are performed based on the following as-built drawing.

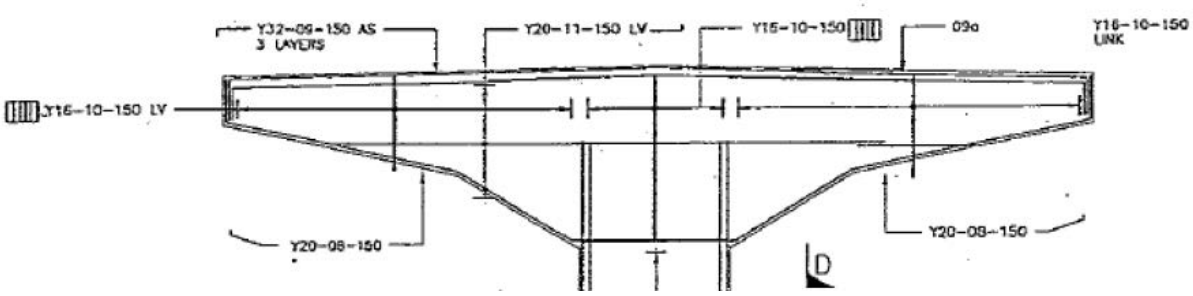


Figure 45. P-33 As-built crosshead reinforcement

9.2.1 Analysis Results for Pier P-33 Crosshead

The maximum design forces for crosshead are tabulated for various cases.

9.2.1.1 BD 37/88 (3 Notional Lanes)

Table 63. P-33 crosshead moment – BD 37/88 (3 Notional Lanes)

No.	Load Case	Mmax (kN.m)	Combination 1		γ_{f3}
			γ_{fL}		
			SLS	ULS	ULS
1	SW	15939	1.00	1.15	1.10
2	Deck Slab	3834	1.00	1.15	1.10
3	SDL (Parapet)	2636	1.00	1.20	1.10
4	Premix	1128	1.20	1.75	1.10
5	HA+KEL	11353	1.20	1.50	1.10
6	HA+HB30	10336	1.10	1.30	1.10
7	HB45	9875	1.10	1.30	1.10
8	SV20	12393	1.10	1.30	1.10

**SW includes 14 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 64. P-33 crosshead moment load combination – BD 37/88 (3 Notional Lanes)

SLS Design to Load Combination 1

Case #	Load Combination	M (kN.m)	M_q (kN.m)	M_g (kN.m)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	37386	23763	13623
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	35132	23763	11369
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	34626	23763	10863
SLS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	37395	23763	13632

ULS Design to Load Combination 1

Case #	Load Combination	M (kN.m)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	49396
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	45444
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	44786
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	48386

Table 65. P-33 crosshead shear @ 2.0m depth – BD 37/88 (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	-1959	1.00	1.15	1.10
2	Deck Slab	-475	1.00	1.15	1.10
3	SDL (Parapet)	-321	1.00	1.20	1.10
4	Premix	-140	1.20	1.75	1.10
5	HA+KEL	-1457	1.20	1.50	1.10
6	HA+HB30	-1047	1.10	1.30	1.10
7	HB45	-1413	1.10	1.30	1.10
8	SV20	-1608	1.10	1.30	1.10

*StaadPro member 6205

**SW includes 14 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 66. P-33 crosshead shear @ 2.0m depth load combination – BD 37/88 (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	-6176
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	-5269
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	-5793
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	-6072

Table 67. P-33 crosshead shear @ 3.5m depth – BD 37/88 (3 Notional Lanes)

No.	Load Case	Vmax (kN)	Combination 1 γ_{fL}		γ_{f3}
			SLS	ULS	ULS
1	SW	-3472	1.00	1.15	1.10
2	Deck Slab	-810	1.00	1.15	1.10
3	SDL (Parapet)	-445	1.00	1.20	1.10
4	Premix	-238	1.20	1.75	1.10
5	HA+KEL	-2294	1.20	1.50	1.10
6	HA+HB30	-1605	1.10	1.30	1.10
7	HB45	-1569	1.10	1.30	1.10
8	SV20	-2689	1.10	1.30	1.10

*StaadPro member 6208

**SW includes 14 nos. precast M10, 2 nos. precast UM10 (LHS & RHS), diaphragms and crosshead*

Table 68. P-33 crosshead shear @ 3.5m depth load combination – BD 37/88 (3 Notional Lanes)

ULS Design to Load Combination 1

Case #	Load Combination	V (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)	-10248
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)	-8757
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HB45)	-8706
ULS4C1	(SW+Deck Slab+SDL+Premix) + (SV20)	-10308

9.2.2 Section Capacity Check (ULS) for P-33 Crosshead

The crosshead section capacity is calculated based on the following parameter;

Crosshead P-33 (*Type P1-A*)

- *Width = 3000mm, Depth = 3500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (3 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

9.2.2.1 Ultimate Moment Capacity Check for P-33 Crosshead

The computed crosshead ultimate moment capacities for P-33 (Type P1-A) is computed and compared with the ULS applied moments.

Table 69. Summary of P-33 crosshead ULS moment capacity check

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	59,830	65,605	49,396	0.83

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The applied ULS moment is within the P-M interaction envelope. Hence, the existing crosshead moment capacity design for P-33 (Type P1-A) is adequate at ULS.

The detailed computations of the sectional moment capacities are presented below.

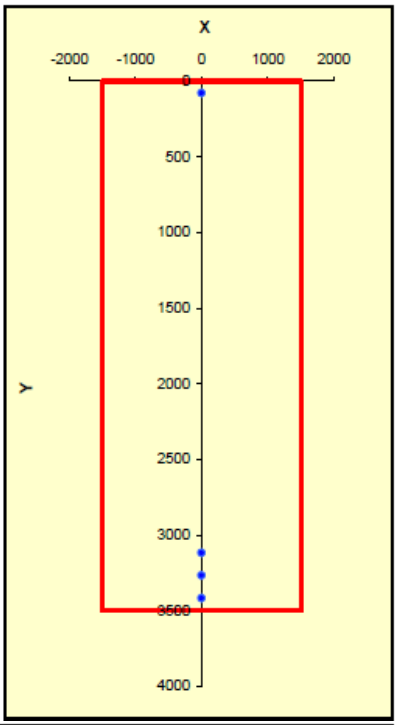
9.2.2.1.1 BD 37/88 (3 Notional Lanes)

**Without Side Reinforcement*

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P33 TYPE P1 -A (WITHOUT SIDEBARS) 3 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1750.000	mm	: Centroid of section
ΣAg=	10,500,000	mm²	: Area of concrete section
ΣAs=	54,538	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.52	%	
Ap/Ag=	0.00	%	
Pu,max=	184,809	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	1,075,472	mm²	: Area of concrete in compression
Fc=	-17,208	kN	: Force on concrete
Fs=	17,208	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.000E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)

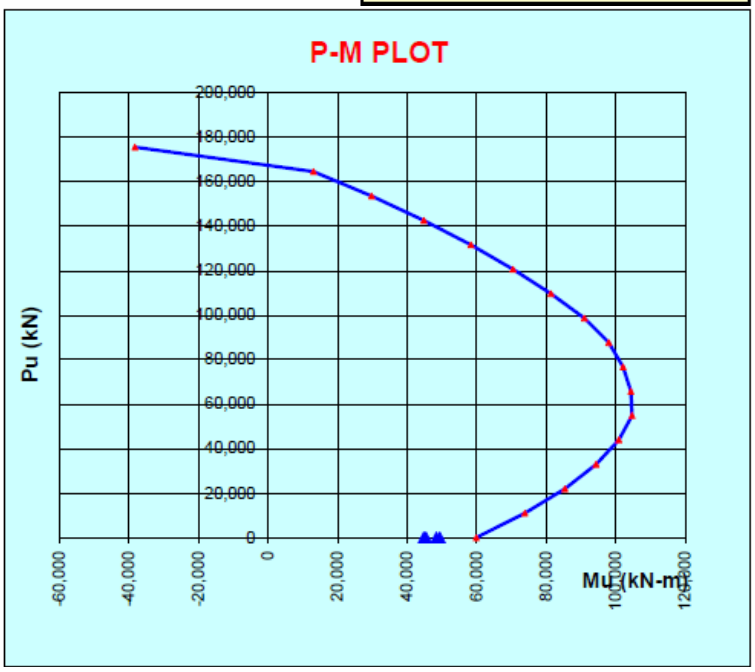
Yn= 358.49 mm

Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	59,830	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	59,830
2	0.06	10,973	73,845
3	0.12	21,946	85,351
4	0.18	32,919	94,349
5	0.24	43,892	100,838
6	0.30	54,865	104,628
7	0.36	65,838	104,379
8	0.42	76,811	102,067
9	0.48	87,784	97,987
10	0.53	98,757	90,929
11	0.59	109,730	81,341
12	0.65	120,703	70,459
13	0.71	131,676	58,330
14	0.77	142,649	44,807
15	0.83	153,622	29,758
16	0.89	164,595	13,065
17	0.95	175,568	-38,278

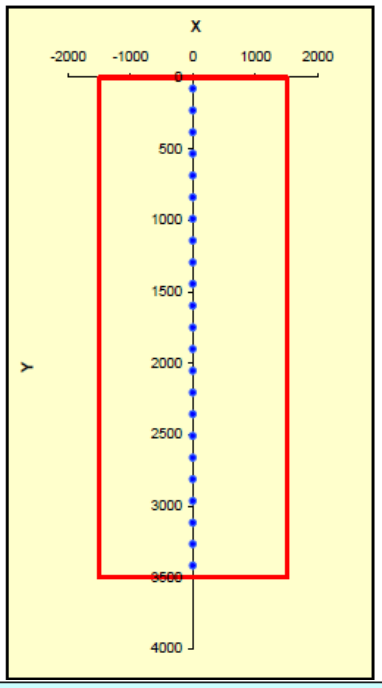


* With Side Reinforcement T20-150 (Both Sides)

Ultimate Section Capacity BS5400

Design Information: CROSSHEAD P33 TYPE P1 -A (WITH SIDEBARS) 3 Notional Lanes

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Modulus of elasticity of concrete; short term
f_y =	460	N/mm²	: Yeild strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
Ycg=	1750.000	mm	: Centroid of section
ΣAg=	10,500,000	mm²	: Area of concrete section
ΣAs=	66,476	mm²	: Area of Rebars
ΣAp=	-	mm²	: Area of prestressing strand
As/Ag=	0.63	%	
Ap/Ag=	0.00	%	
Pu,max=	188,488	kN	: 0.4fcuAg+0.67fyAs



Neutral Axis

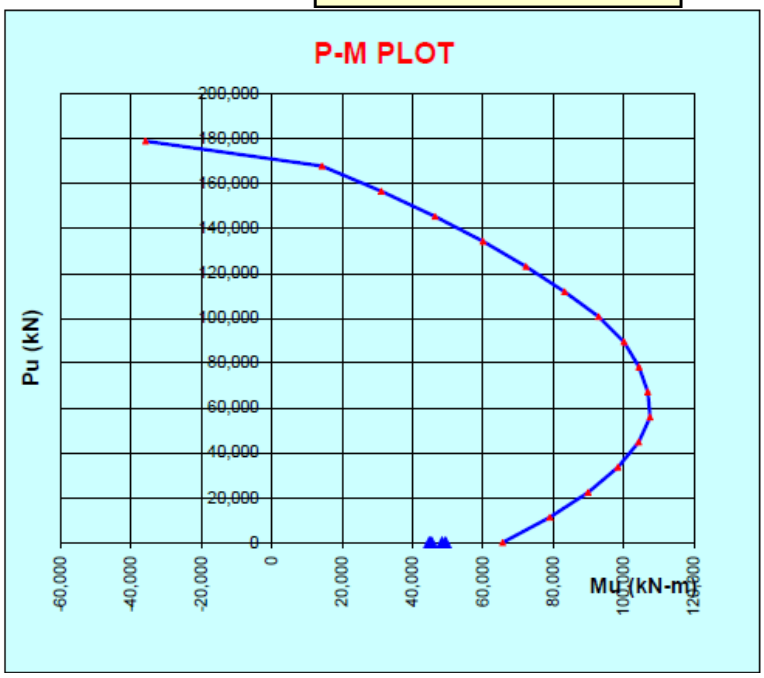
Neutral Axis, Yn, where Σ(Fc+Fp+Fs+Pu)=0

ΣAc=	1,312,843	mm²	: Area of concrete in compression
Fc=	-21,005	kN	: Force on concrete
Fs=	21,005	kN	: Force on rebars
Fp=	0	kN	: Force on prestressing strands
Err(x)=	1.025E-03		: Err(Yn)= Σ(Fc+Fs+Fp+Pu)
Yn=	437.61	mm	
Pu=	-	kN	: Ultimate Axial Force (+) compression
Mu=	65,605	kN-m	: Ultimate Bending Capacity

Go

P-M Graph

	$P_u/P_{u,max}$	P_u	M_u
1	0.00	0	65,605
2	0.06	11,191	78,986
3	0.12	22,383	89,895
4	0.18	33,574	98,332
5	0.24	44,766	104,296
6	0.30	55,957	107,493
7	0.36	67,149	106,843
8	0.42	78,340	104,352
9	0.48	89,532	100,097
10	0.53	100,723	92,887
11	0.59	111,915	83,216
12	0.65	123,106	72,240
13	0.71	134,298	60,010
14	0.77	145,489	46,369
15	0.83	156,681	31,157
16	0.89	167,872	14,273
17	0.95	179,064	-35,889



9.2.2.2 Ultimate Shear Capacity Check for P-33 Crosshead

The crosshead ultimate shear link required for Pier P-33 (Type P1-A) is computed and compared to the shear link provided.

Table 70. Summary of P-33 ULS shear capacity check @ 2.0m depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.87	8.04	0.98
ULS2C1	6.58	8.04	0.82
ULS3C1	7.87	8.04	0.98
ULS4C1	7.73	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Table 71. Summary of P-33 ULS shear capacity check @ 3.5m depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.81	8.04	0.97
ULS2C1	6.66	8.04	0.83
ULS3C1	6.62	8.04	0.82
ULS4C1	7.85	8.04	0.98

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

Based on the checking, the shear link provided is more than required. Hence, the existing crosshead shear design for P-33 (Type P1-A) is adequate at ULS.

The detailed computations of the sectional shear capacities are presented as below.

9.2.2.2.1 BD 37/88 (3 Notional Lanes)

*ULS1C1 @ 2.0m Depth

Element ID = **P-33 Crosshead (ULS1C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	6,176	kN			
v	=	1.18	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	7.87				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

*ULS2C1 @ 2.0m Depth

Element ID = **P-33 Crosshead (ULS2C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2			
f_y	=	460	N/mm^2			
b	=	3,000	mm			
d	=	1,752	mm			
V_{ult}	=	5,269	kN			
v	=	1.00	N/mm^2	Remarks :	O.K	
Depth Factor, ξ_s =		0.731				
A_s	=	48,240	mm^2	(3 layers of 20T32)		
v_c	=	0.72	N/mm^2	$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$				
$A_{sv}/s_{v,req'd}$	=	6.58				
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)		
		Sufficient!				

*ULS3C1 @ 2.0m Depth

Element ID = **P-33 Crosshead (ULS3C1 - BD 37/88 3 Notional Lanes) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	1,752	mm				
V_{ult}	=	5,793	kN				
v	=	1.10	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.731					
A_s	=	48,240	mm^2	(3 layers of 20T32)			
v_c	=	0.72	N/mm^2		$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.33					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS4C1 @ 2.0m Depth

Element ID = **P-33 Crosshead (ULS4C1 - SV20) @ 2.0m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	1,752	mm				
V_{ult}	=	6,072	kN				
v	=	1.16	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.731					
A_s	=	48,240	mm^2	(3 layers of 20T32)			
v_c	=	0.72	N/mm^2		$\xi_s v_c$	=	0.52
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.73					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS1C1 @ 3.5m Depth

Element ID = **P-33 Crosshead (ULS1C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	10,248	kN				
v	=	1.05	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.81					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS2C1 @ 3.5m Depth

Element ID = **P-33 Crosshead (ULS2C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	8,757	kN				
v	=	0.90	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.66					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS3C1 @ 3.5m Depth

Element ID = **P-33 Crosshead (ULS3C1 - BD 37/88 3 Notional Lanes) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	8,706	kN				
v	=	0.89	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	6.62					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

*ULS4C1 @ 3.5m Depth

Element ID = **P-33 Crosshead (ULS4C1 - SV20) @ 3.5m Depth**

f_{cu}	=	40	N/mm^2				
f_y	=	460	N/mm^2				
b	=	3,000	mm				
d	=	3,252	mm				
V_{ult}	=	10,308	kN				
v	=	1.06	N/mm^2	Remarks :	O.K		
Depth Factor, ξ_s =		0.700					
A_s	=	48,240	mm^2	(3 layers 20T32)			
v_c	=	0.58	N/mm^2		$\xi_s v_c$	=	0.41
v	>	$\xi_s v_c$					
$A_{sv}/s_{v,req'd}$	=	7.85					
$A_{sv}/s_{v,prov}$	=	8.04		(3T16-150)			
		Sufficient!					

9.2.3 Crack Width Check (SLS) for Pier P-33 Crosshead

The crosshead crack width is calculated based on the following parameter;

Crosshead P-33 (*Type P1-A*)

- *Width = 3000mm, Depth = 3500mm, f_{cu}=40MPa*
- *Top Reinforcement = T32-150 (3 layers)*
- *Bottom Reinforcement = T20 – 150 (1 layer)*

The computed crosshead crack width for Pier P-33 (Type P1-A) is summarized as follows;

Table 72. Summary of P-33 crosshead SLS crack width check

Loading Criteria	Crack Width (mm)	
	Without Sidebar	With Sidebar
BD 37/88 (3 Notional Lanes)	0.130	0.110

The computed crack width is 0.130mm without taking into account side reinforcement and 0.110mm with side reinforcement. Hence, the crack width is less than the allowable limit of 0.250mm

The detailed computation of the crack widths are presented below.

9.2.3.1 BD 37/88 (3 Notional Lanes)

** Without Side Reinforcement*

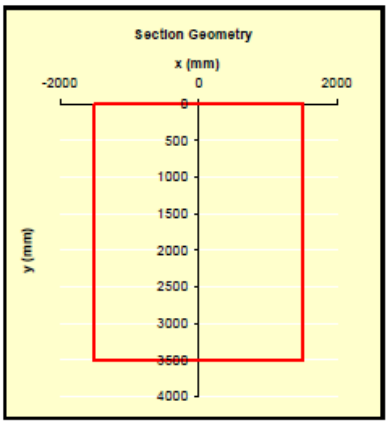
TITTLE : Crosshead P33 Type P1-A SLS1C1 (WITHOUT SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm²	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m²	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m²	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm²	: Steel Yield Strength
Es =	2.00E+08	kN/m²	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/Ecl



Neutral Axis (Elastic Analysis)

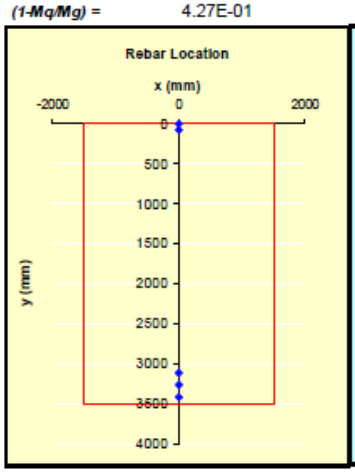
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

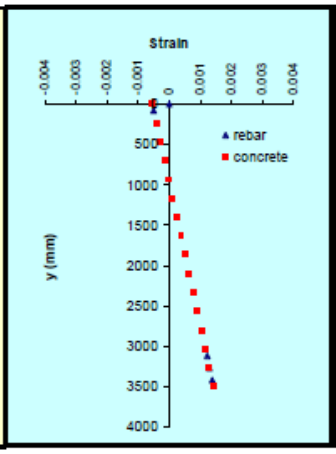
ΣAc=	2,865,904	mm²	: Area of concrete
ΣAs=	703,717	mm²	: Transformed area of rebar
ΣA=	3,569,621	mm²	: Gross area
Err(x)=	0.0		: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	955.30	mm	

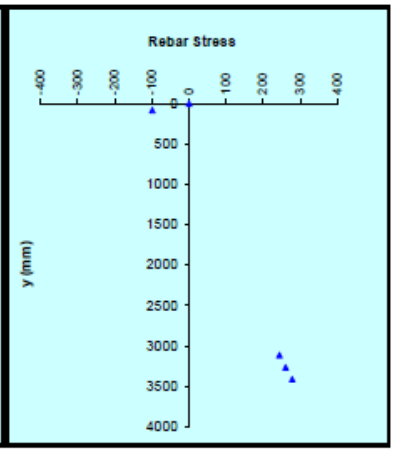
RE_ITERATE

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	23763	kN-m	: Permanent moment
M_q =	13623	kN-m	: Live load moment
M_s =	37386	kN-m	: Applied SLS moment
h=	3500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	6.75E-04		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-7.62E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.27E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.39E-04	0.00E+00	0.00E+00	-5.39E-04	uncracked
0	0	0	0	0	0	-5.39E-04	0.00E+00	0.00E+00	-5.39E-04	uncracked
0	0	0	0	0	0	-5.39E-04	0.00E+00	0.00E+00	-5.39E-04	uncracked
0	0	0	0	0	0	-5.39E-04	0.00E+00	0.00E+00	-5.39E-04	uncracked
0	0	0	0	0	0	-5.39E-04	0.00E+00	0.00E+00	-5.39E-04	uncracked
0	3500	0	3418	32	66	1.44E-03	0.00E+00	-7.62E-04	6.75E-04	0.130

* With Side Reinforcement T20-150 (Both Sides)

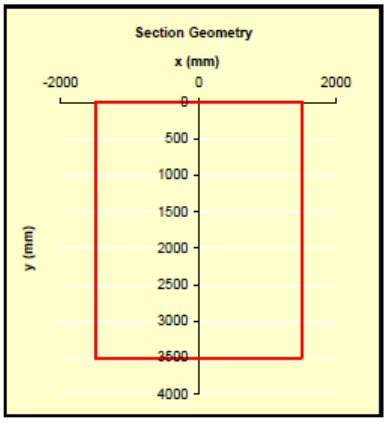
TITTLE : Crosshead P33 Type P1-A SLS1C1 (WITH SIDEBAR) 3 Notional Lanes

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ =	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α =	12.90		: Long term ratio Es/EcL



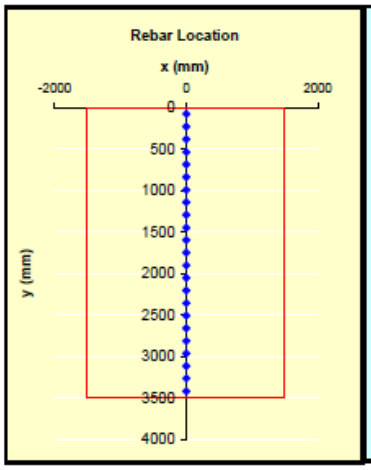
Neutral Axis (Elastic Analysis)

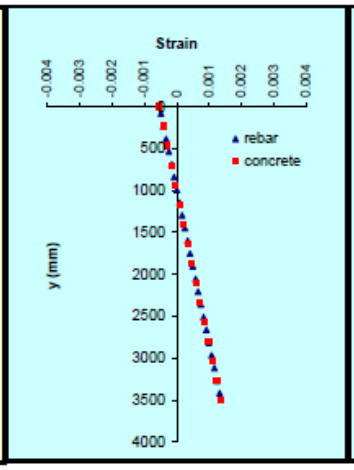
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

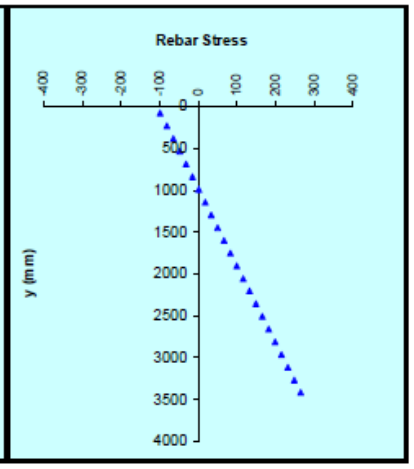
ΣAc=	2,944,950	mm^2	: Area of concrete
ΣAs=	857,756	mm^2	: Transformed area of rebar
ΣA=	3,802,706	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	981.65	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	23763	kN-m	: Permanent moment
M_q =	13623	kN-m	: Live load moment
M_s =	37386	kN-m	: Applied SLS moment
h=	3500	mm	: Overall depth of section
C_{nom} =	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	66	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	5.68E-04		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-8.00E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	4.27E-01		







Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	79	20	69	-5.33E-04	0.00E+00	0.00E+00	-5.33E-04	uncracked
0	0	0	0	0	0	-5.33E-04	0.00E+00	0.00E+00	-5.33E-04	uncracked
0	0	0	0	0	0	-5.33E-04	0.00E+00	0.00E+00	-5.33E-04	uncracked
0	0	0	0	0	0	-5.33E-04	0.00E+00	0.00E+00	-5.33E-04	uncracked
0	0	0	0	0	0	-5.33E-04	0.00E+00	0.00E+00	-5.33E-04	uncracked
0	3500	0	3418	32	66	1.37E-03	0.00E+00	-8.00E-04	5.68E-04	0.110

9.3. Strut and Tie Analysis (STM) for Pier P-33 Crosshead

The STM model was based on BD 37/88 3 notional lanes loading criteria for Ultimate Limit State Load Combination 1.

Summary of maximum bearing force based on BD 37/88 (3 Notional Lanes)

No.	Load Case	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)	Combination 1 γ_{fL} SLS	Combination 1 γ_{fL} ULS	γ_{f3} ULS
1	SW	324	351	365	372	367	347	303	398	1.00	1.15	1.10
2	Deck Slab	144	105	118	118	116	110	99	120	1.00	1.15	1.10
3	SDL (Parapet)	286	***-42***	38	45	42	38	37	104	1.00	1.20	1.10
4	Premix	42	31	35	35	34	32	29	35	1.20	1.75	1.10
5	HA+KEL (1)	554	278	265	446	248	231	273	151	1.20	1.50	1.10
6	HA+KEL (2)	419	336	274	458	273	269	315	229	1.20	1.50	1.10
7	HA+HB30 (1)	492	179	273	182	117	154	208	114	1.10	1.30	1.10
8	HA+HB30 (2)	352	240	283	194	142	193	252	192	1.10	1.30	1.10
9	HB45	787	149	326	181	75	39	12	36	1.10	1.30	1.10
10	SV20	260	309	543	531	543	319	184	175	1.10	1.30	1.10

No.	Load Case	N_9 (kN)	N_{10} (kN)	N_{11} (kN)	N_{12} (kN)	N_{13} (kN)	N_{14} (kN)	N_{15} (kN)	N_{16} (kN)	Combination 1 γ_{fL} SLS	Combination 1 γ_{fL} ULS	γ_{f3} ULS
1	SW	398	303	347	367	372	364	352	324	1.00	1.15	1.10
2	Deck Slab	120	99	110	116	119	117	106	144	1.00	1.15	1.10
3	SDL (Parapet)	104	37	38	42	45	37	***-41***	286	1.00	1.20	1.10
4	Premix	35	29	32	34	35	34	31	42	1.20	1.75	1.10
5	HA+KEL (1)	77	42	39	25	12	10	58	***-135***	1.20	1.50	1.10
6	HA+KEL (2)	230	316	257	294	449	256	363	409	1.20	1.50	1.10
7	HA+HB30 (1)	58	34	35	26	18	15	39	***-62***	1.10	1.30	1.10
8	HA+HB30 (2)	207	252	237	275	371	262	278	451	1.10	1.30	1.10
9	HB45	***-5***	***-13***	***-5***	***-5***	***-7***	***-9***	***-16***	6	1.10	1.30	1.10
10	SV20	92	45	40	24	10	7	50	***-127***	1.10	1.30	1.10

SLS Design to Load Combination 1

Case #	Load Combination	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(1)	1470	785	880	1112	863	810	802	847
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(2)	1308	854	891	1127	894	857	852	940
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(1)	1347	647	862	777	694	703	702	790
SLS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(2)	1193	714	873	791	722	746	751	876
SLS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	1671	615	921	777	648	577	487	704
SLS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	1091	790	1159	1161	1163	885	676	858

SLS Design to Load Combination 1 (cont'd)

Case #	Load Combination	N_9 (kN)	N_{10} (kN)	N_{11} (kN)	N_{12} (kN)	N_{13} (kN)	N_{14} (kN)	N_{15} (kN)	N_{16} (kN)
SLS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(1)	758	525	580	596	592	571	524	643
SLS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(2)	940	854	841	919	1116	866	889	1296
SLS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(1)	729	512	571	595	597	576	497	736
SLS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(2)	893	751	794	869	985	847	760	1301
SLS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	660	459	528	560	570	550	436	812
SLS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	766	524	577	592	588	566	509	665

ULS Design to Load Combination 1

Case #	Load Combination	N_1 (kN)	N_2 (kN)	N_3 (kN)	N_4 (kN)	N_5 (kN)	N_6 (kN)	N_7 (kN)	N_8 (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(1)	1966	1040	1164	1483	1141	1071	1064	1111
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(2)	1743	1135	1179	1503	1183	1135	1133	1239
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(1)	1756	836	1118	1007	899	911	911	1024
ULS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(2)	1556	923	1132	1025	935	967	974	1136
ULS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	2178	793	1193	1006	839	747	630	912
ULS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	1424	1022	1503	1506	1509	1147	877	1112

ULS Design to Load Combination 1 (cont'd)

Case #	Load Combination	N_9 (kN)	N_{10} (kN)	N_{11} (kN)	N_{12} (kN)	N_{13} (kN)	N_{14} (kN)	N_{15} (kN)	N_{16} (kN)
ULS1C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(1)	989	683	755	773	767	739	680	828
ULS2C1	(SW+Deck Slab+SDL+Premix) + (HA+KEL)(2)	1240	1135	1113	1218	1487	1146	1183	1726
ULS3C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(1)	944	663	739	770	773	745	640	962
ULS4C1	(SW+Deck Slab+SDL+Premix) + (HA+HB30)(2)	1158	973	1028	1126	1277	1098	982	1696
ULS5C1	(SW+Deck Slab+SDL+Premix) + (HB45)	854	594	683	725	738	711	562	1060
ULS6C1	(SW+Deck Slab+SDL+Premix) + (SV20)	993	678	747	766	761	733	657	870

Note : N_1 & N_{16} is located at the tip of the cantilever (furthest from the pier)
HA+KEL(1) denotes 1 carriageway loaded, HA+KEL(2) denotes 2 carriageway loaded simultaneously

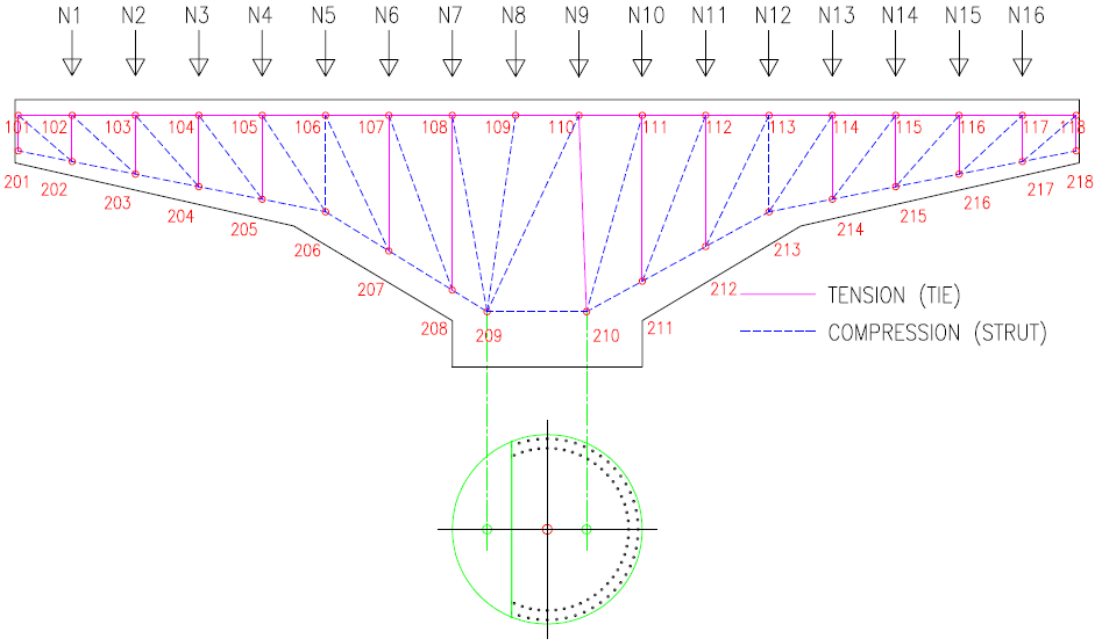


Figure 46*. P-33 STM Analysis Model*

CENTROID OF TENSION REINFORCEMENT

Area: 988169.2881
Perimeter: 11259.8615
Bounding box: X: −1450.0000 -- 562.5000
Y: −1450.0000 -- 1450.0000
Centroid: X: −620.3035
Y: 0.0000
Moments of inertia: X: 1.0884E+12
Y: 7.4359E+11
Product of inertia: XY: 0.0000
Radii of gyration: X: 1049.4821
Y: 867.4630
Principal moments and X−Y directions about centroid:
I: 1.0884E+12 along [1.0000 0.0000]
J: 3.6337E+11 along [0.0000 1.0000]

CENTROID OF COMPRESSION ZONE

Area: 1887221.8085
Perimeter: 6340.2731
Bounding box: X: 562.5000 -- 1500.0000
Y: −1390.5372 -- 1390.5372
Centroid: X: 949.8032
Y: 0.0000
Moments of inertia: X: 8.0949E+11
Y: 1.8178E+12
Product of inertia: XY: 0.0000
Radii of gyration: X: 654.9305
Y: 981.4266
Principal moments and X−Y directions about centroid:
I: 8.0949E+11 along [1.0000 0.0000]
J: 1.1526E+11 along [0.0000 1.0000]

Figure 47*. P-33 tension and compression zone based on ULS1C1*

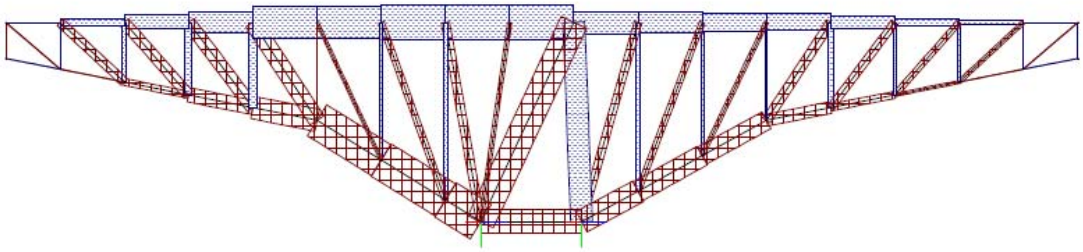


Figure 48. *P-33 STM axial force dDiagram (Blue = Tension, Red = Compression)*

The support for the STM model is modeled based on the centroid of the tension reinforcement zone and concrete compression zone as shown in Figure 47.

The tie tension forces obtained from the analysis are checked as follows;

Top Tension Tie *(109-110)* **Check**

fy =	460	Mpa	
As,prov =	48,240	mm^2	(3 x 20T32 top reinforcement)

Tu =	11,541	kN
As,req =	28,839	mm^2

Remarks : As,prov > As,req O.K!

Vertical Tension Tie *(110-210)* **Check**

fy =	460	Mpa	
As,prov =	48,240	mm^2	(2x30T32 column main reinforcement - half column zone)
	16,578	mm^2	(3T16-150 links in 2062mm tension zone)
	64,818	mm^2	

Tu =	7,157	kN
As,req =	17,884	mm^2

Remarks : As,prov > As,req O.K!

Vertical Tension Tie *(105-205)* **Check**

fy =	460	Mpa	
As,prov =	0	mm^2	
	8,040	mm^2	(3T16-150 links in 1000mm tension zone)
	8,040	mm^2	
Tu =	2,945	kN	
As,req =	7,359	mm^2	
Remarks :	As,prov > As,req	O.K!	

The check shows that the reinforcement provided is sufficient to resist the tension tie forces.

The bottom compression strut forces obtained from the analysis are checked as follows;

Bottom Compression Strut *(208-209)* **Check**

fcu =	40	Mpa	
b =	3,000	mm	(Crosshead width)
d =	970	mm	(Crosshead compression zone)
Su =	13,244	kN	
σ =	4.55	Mpa	
Remarks :	< 0.4fcu	O.K	

Based on the check, the concrete stress calculated is 4.55 N/mm^2, which is less than $0.4f_{cu}$; 16.0 N/mm^2. Thus, the bottom strut concrete compression stress is within the strength limit.

The diagonal compression strut is checked as follows;

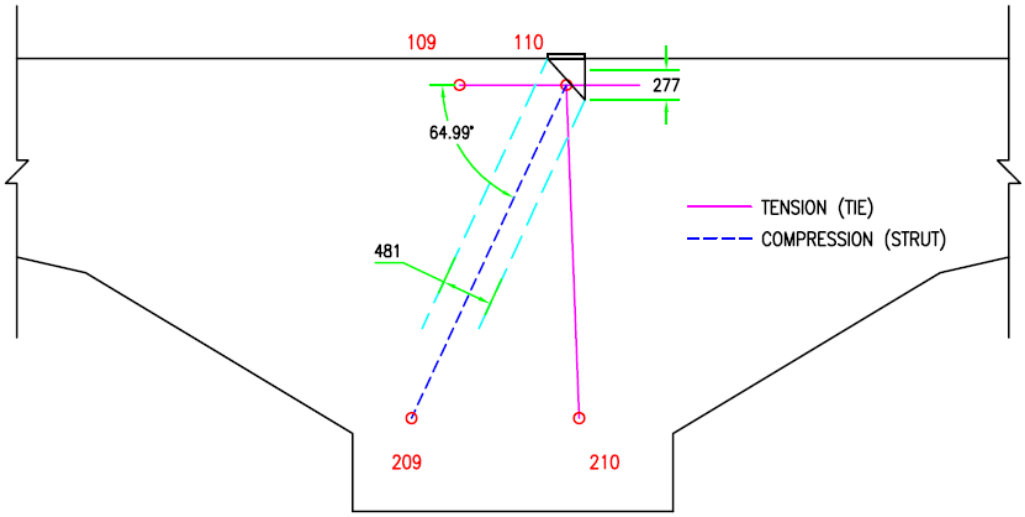


Figure 49. P-33 diagonal strut check

Diagonal Strut *(110-209)* Calculation based on ACI

Effective Compressive Strength for Node *110*

β_n =	0.6	(CTT)	
f'_c =	40	Mpa	*(cube strength)*
	4,640	psi	
$f_{c3(110)}$ =	$0.85\beta_n f'_c$		*(eq. A-8)*
=	2.37	ksi	
=	16.32	Mpa	

Calculate Width of Tie *109-110*

ϕ =	0.85		*(cl. C.9.3.2.6)*
b_w =	3,000	mm	
	118.2	in.	
F =	11,541	kN	
	2,594	k	

$w_{(109-110)} = F / \phi(b_w)f_{cu}$

=	10.9	in.
=	277.2	mm

Effective Compressive Strength for Strut $_{110\text{-}209}$

β_s =	1.00		*(cl. A.3.2.1)*
f'_c =	40	Mpa	*(cube strength)*
	4,640	psi	*(cylinder strength)*
$f_{ce(110\text{-}209)}$ =	$0.85\beta_s f'_c$		*(eq. A-3)*
=	3.94	ksi	
=	27.20	Mpa	

Check Strut $_{110\text{-}209}$ Capacity

$w_{s(110\text{-}209)}$ =	480.6	mm			*(measured from drawing)*	
	18.9	in.				
$\phi F_{ns(110\text{-}209)}$ =	$\phi f_{ce} w_{s(110\text{-}209)} b_w$				*(eq. A-2)*	
=	7,498	k				
=	33,349	kN	>	Su =	9,226	kN
	O.K!					

Based on the checking, the diagonal compression strut width is measured to be 480.6mm. The maximum ultimate compression strut force from the analysis is 9,226 kN, which is lower than the calculated capacity of 33,349 kN. Therefore, the diagonal compression strut capacity satisfies the ultimate limit force from the analysis.

9.4. Finite Element Analysis (FEM) for P-33

The FEM model was based on BD 37/88 3 notional lanes loading criteria for Serviceability Limit State Load Combination 1.

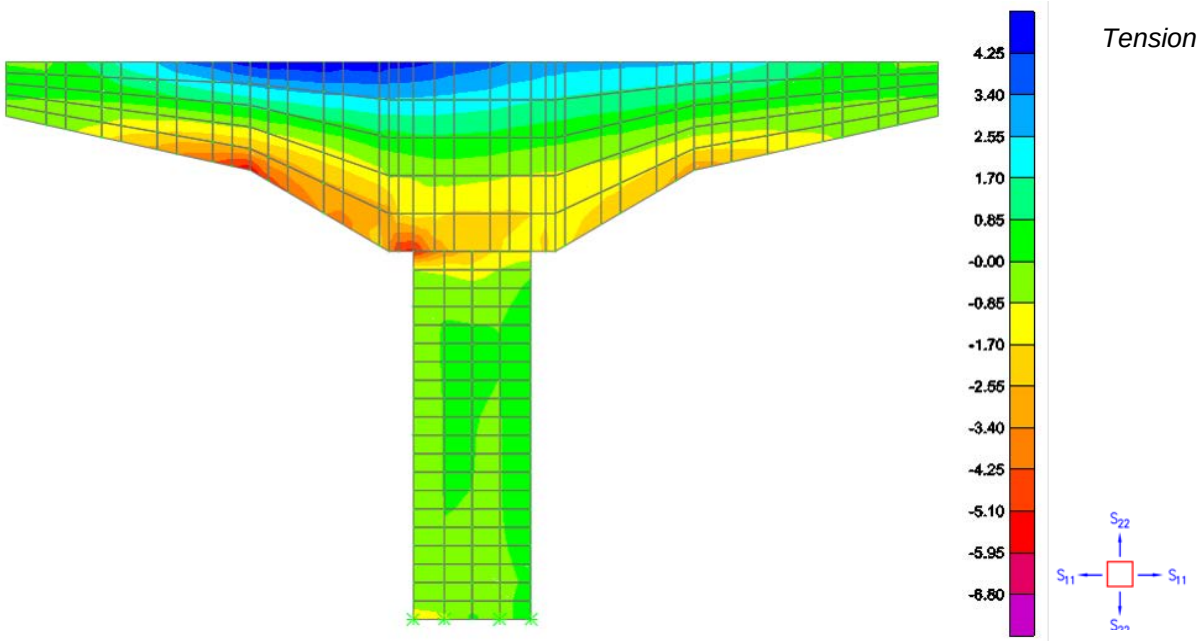


Figure 50. *P-33 S11 stress diagram*

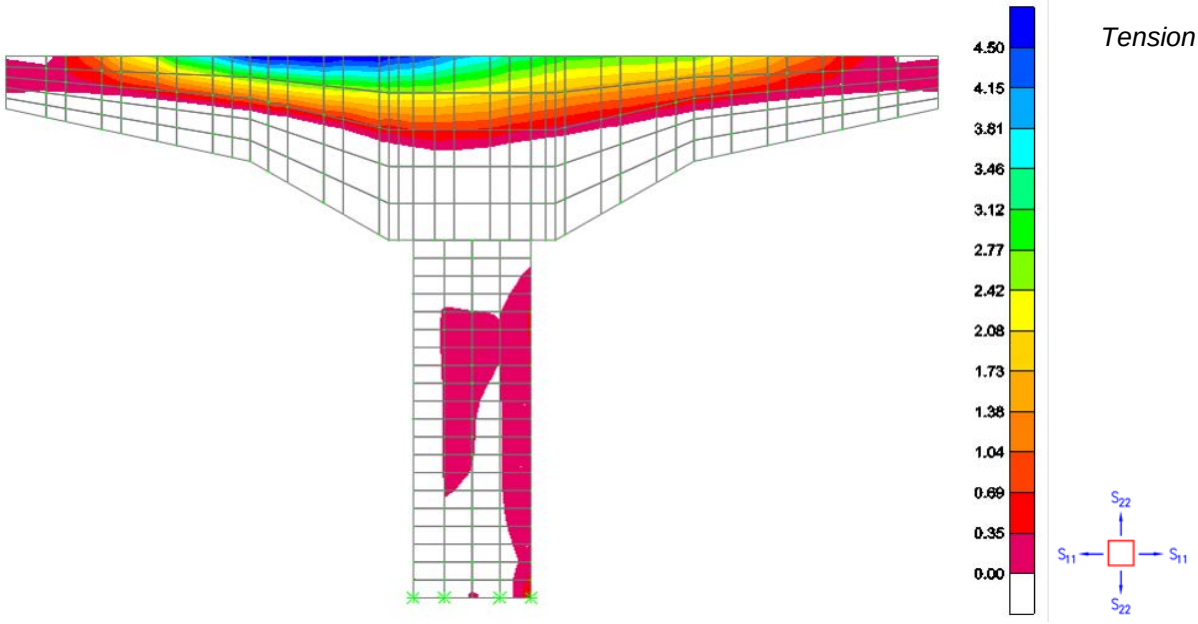


Figure 51. P-33 S11 tension stress

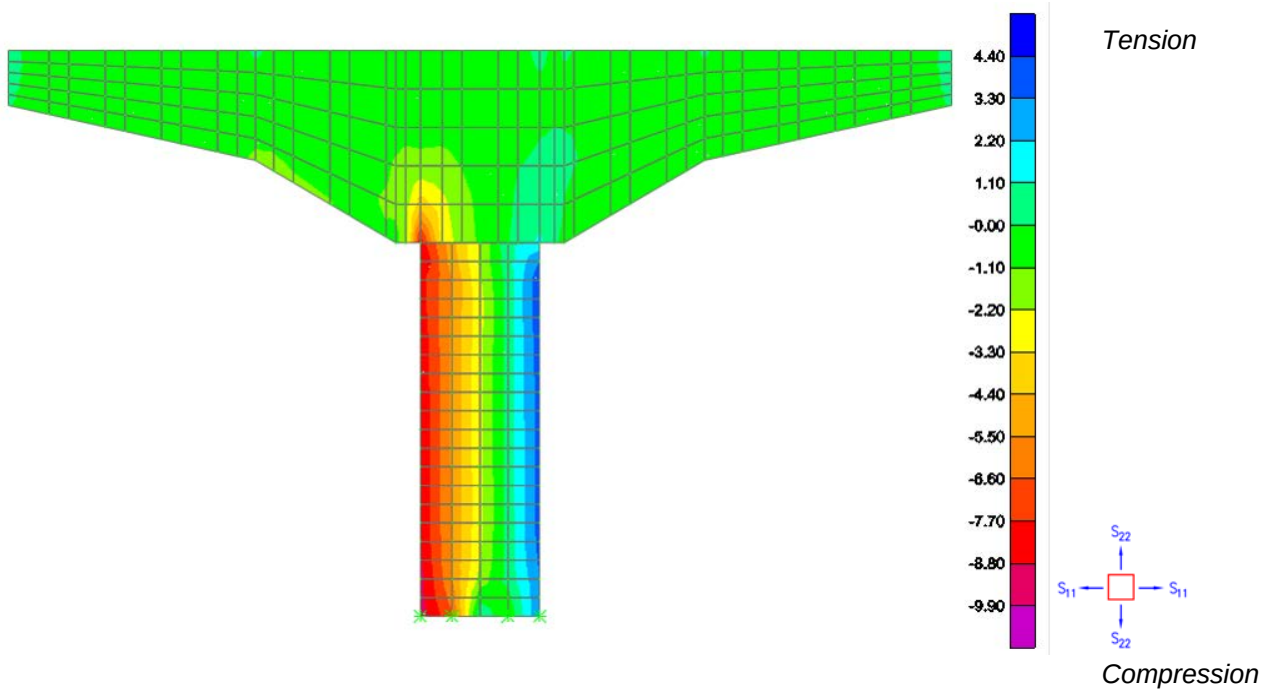


Figure 52. P-33 S22 stress diagram

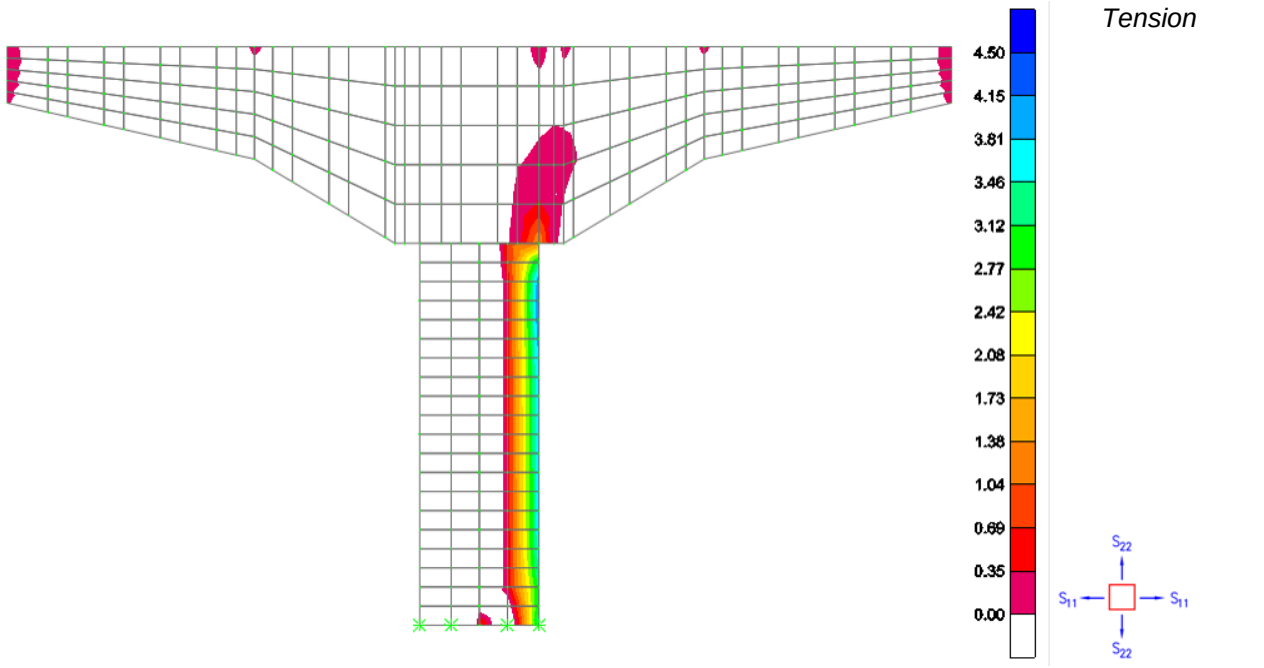


Figure 53. P-33 S22 tension stress diagram

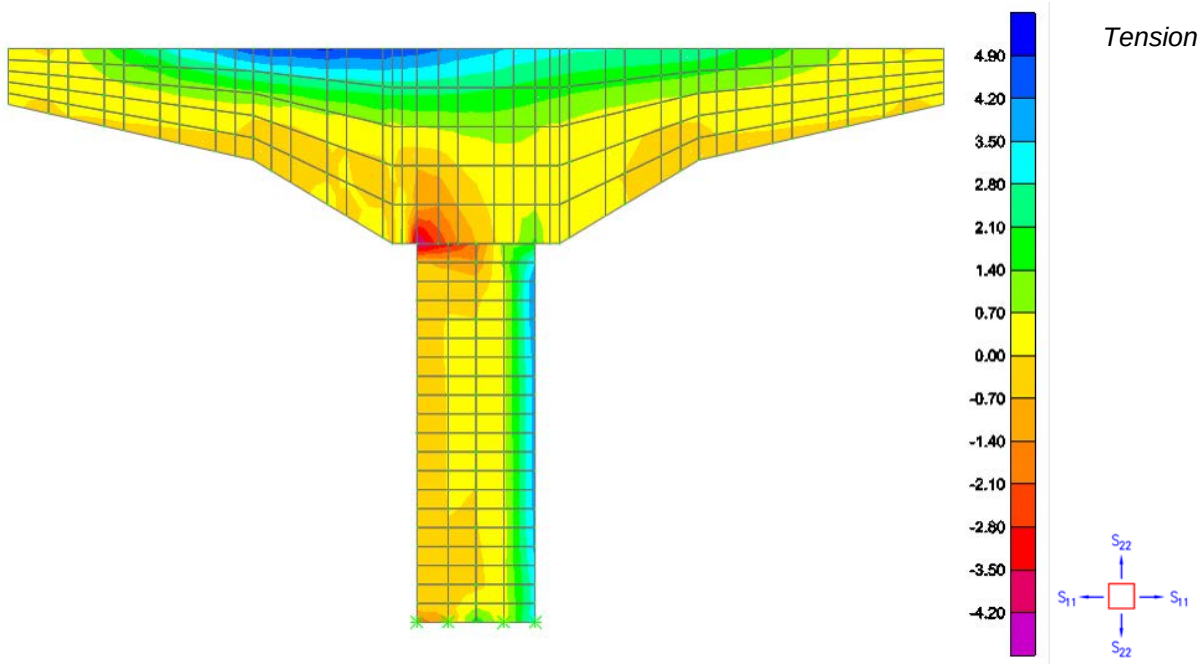


Figure 54. P-33 SMAX stress diagram

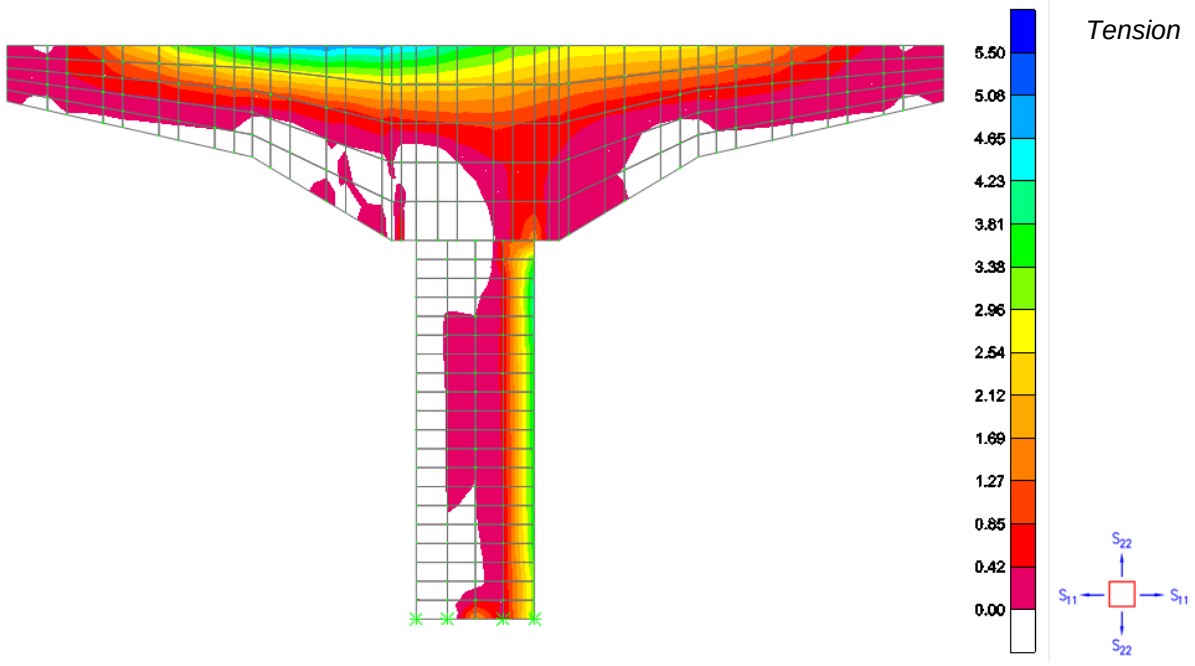


Figure 55. P-33 SMAX tension stress diagram

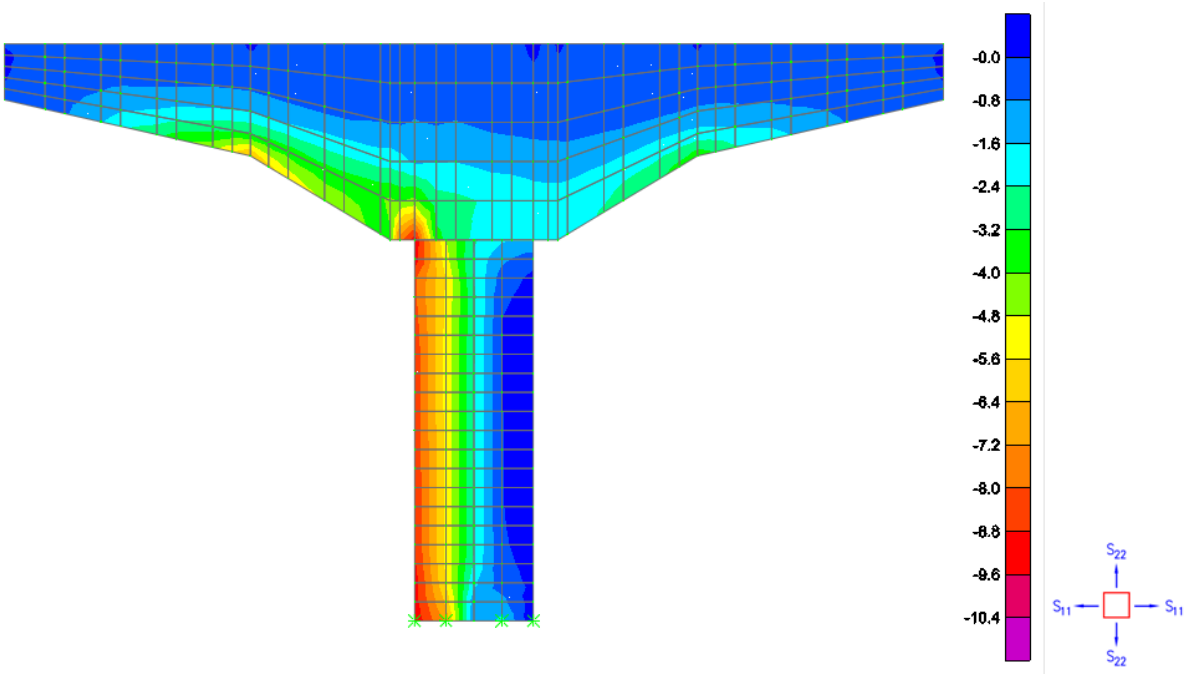


Figure 56. P-33 SMIN stress diagram

Based on Figure 53, it is shown that the S22 tension stress extends approximately 2.1m from the top of the pier into the crosshead. Therefore, it would be suggested that the pier main reinforcement should be extended up for a minimum of 0.6 depth of the crosshead followed by the tension anchorage length to cater for the tension stresses.

9.5. Summary of Design Review for P-33

(a) The pier ultimate capacity (ULS) and crack width (SLS) check is summarized as below.

Table 73. P-33 – Summary of pier ULS moment capacity

Pier Type	Ultimate Moment Capacity (kN.m)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-33)	O.K	-	-

Table 74. P-33 Summary of pier SLS crack width

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-A (P-33)	0.100	-	-

Based on the checking, the existing pier column design satisfied the ULS and SLS criteria.

(b) The crosshead ultimate moment capacity (ULS) check is summarized as below.

Table 75. P-33 Summary of crosshead ULS moment capacity

Loading Criteria	Ult. Moment Capacity (kN.m)		Maximum ULS Moment (kN.m)	Capacity Ratio
	Without Sidebar	With Sidebar		
BD 37/88 (3 Notional Lanes)	59,830	65,605	49,396	0.83

*Capacity ratio is based on Maximum ULS Moment / Ult. Moment Capacity (without sidebar)

The checking shows that the existing design of crosshead satisfies the ULS criteria.

(c) The crosshead ultimate shear capacity (ULS) check is summarized as below.

Table 76. P-33 – Summary of crosshead ULS shear capacity

BD 37/88 (3 Notional Lanes) @ 2.0m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.87	8.04	0.98
ULS2C1	6.58	8.04	0.82
ULS3C1	7.87	8.04	0.98
ULS4C1	7.73	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

BD 37/88 (3 Notional Lanes) @ 3.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.81	8.04	0.97
ULS2C1	6.66	8.04	0.83
ULS3C1	6.62	8.04	0.82
ULS4C1	7.85	8.04	0.98

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

The checking shows that the existing shear capacity design of the crosshead is adequate at ULS.

(d) The crosshead crack width (SLS) check is summarized as below.

Table 77. P-33 – Summary of crosshead SLS crack width

Crosshead Type	3 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-A (P-33)	0.130	0.110

The checking shows that the existing design of the crosshead satisfies the SLS criteria of 0.25mm crack width.

(e) The following table shows the comparison of ULS design between conventional beam theory and STM.

Table 78. P-33 Conventional beam theory vs. STM

Element Force	Conventional Beam Theory	STM
Bending Moment, $As_{req'd}$	3 x 20T32	2 x 20T32
Shear Force, $Asv_{req'd}$	3T16-150	3T16-150

Based on the comparison, it is found that the reinforcement required the resist the bending moment by using STM method is less than the conventional beam theory method. The required shear link to resist the shear force is found to be similar between STM method and conventional beam theory method.

(f) Based on Figure (54), the pier column main vertical reinforcement of 2xT32-150 are terminated slightly over the mid depth of the crosshead without 90° anchorage bent.

From Figure (43) & (45) of the STM analysis, the vertical ultimate tension of 7,157kN on the tension tie member (Node 110 to 210) extends from the bottom compression strut to the top tension tie. Although, the reinforcement provided in the pier column tension zone of 2x30T32 and 3T16-150 of crosshead links is sufficient to resist the tension tie force, but they don't have sufficient anchorage length into the nodal zone to satisfy STM design philosophy.

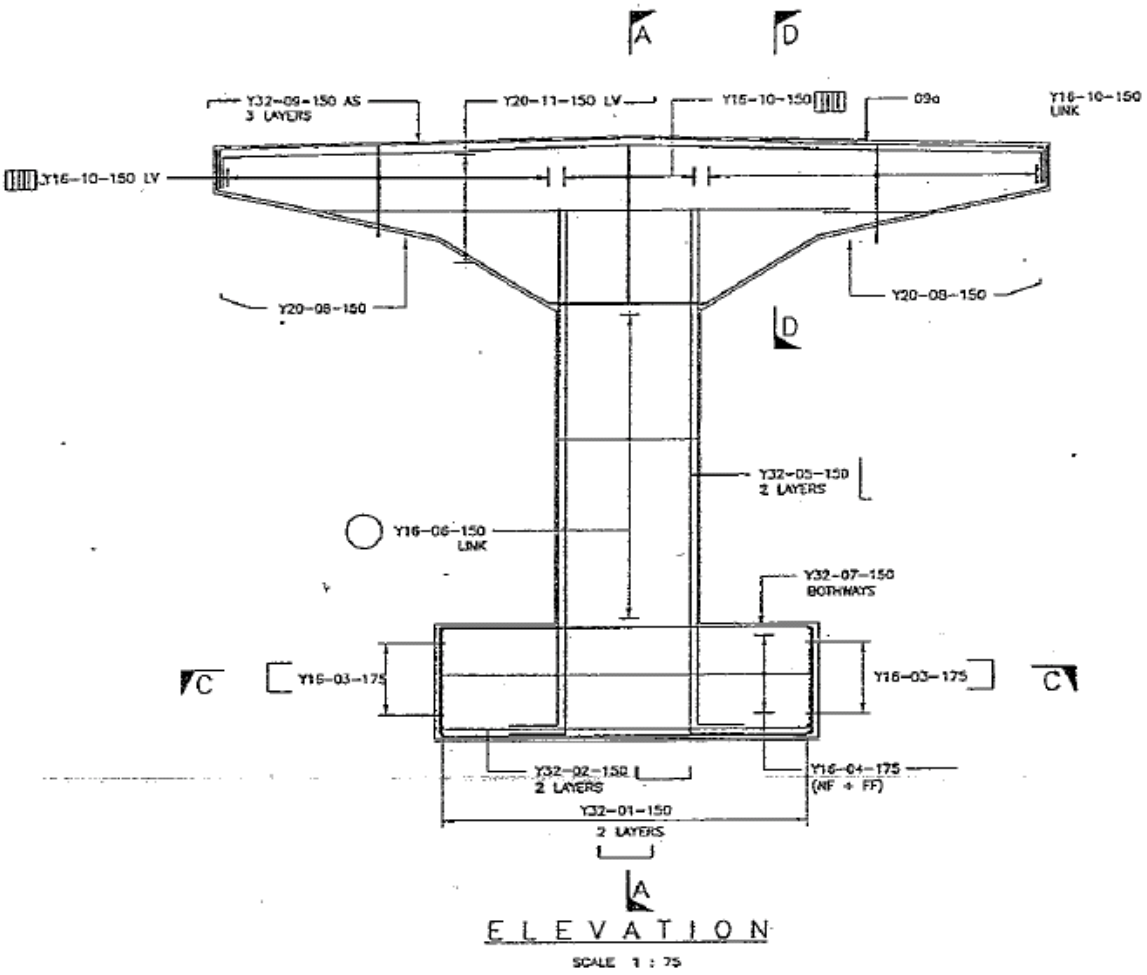


Figure 57. P-33 as-built detailing

Moreover, from Figure (50) of the FEM analysis, the crosshead S22 tension stress is found to extend into crosshead about 2.1m from top of pier column (i.e. soffit of crosshead). Therefore, it is essential that the main column reinforcement should extend up to the top of crosshead and followed by a bend for another minimum tension anchorage length beyond the tension zone.

(g) It is found that the conventional method of analysis does not capture the tensile stress in the crosshead as compared to STM or FEM analysis. This is due to the conventional method assumes the crosshead and pier column as frame elements connected at the centroid of the respected elements. Therefore, it is recommended to perform STM and FEM analysis to investigate and capture the behaviour of deep crosshead.

10. CONCLUSION

A design review has been carried out for the affected Pier P-11A (Type P1-C), P-25 (Type P1-A) and P-33 (Type P1-A). The affected pier and crosshead designs are checked based on the available as built drawings. Three (3) independent analytical models were established based on BD 37/88, JKR SV20 and JKR MTAL traffic live load criteria to determine the maximum induced forces acting on the structures.

The following table summarized the different traffic live load analysis for this study.

Table 79. Summary of traffic live load analysis

Pier Type	BD 37/88		SV20	JKR MTAL (3 Notional Lanes)
	3 Notional Lanes	2 Notional Lanes		
P1-C (P-11A)	√	√	√	√
P1-A (P-25)	√		√	
P1-A (P-33)	√		√	

10.1. Pier Column Check

The pier ultimate capacity (ULS) and crack width (SLS) check is summarized as follows:-

Table 80. Summary of pier ULS moment capacity

Pier Type	Ultimate Moment Capacity (kN.m)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-C (P-11A)	O.K	O.K	O.K
P1-A (P-25)	O.K	-	-
P1-A (P-33)	O.K	-	-

Table 81. Summary of pier SLS moment capacity

Pier Type	Crack Width (mm)		
	3 Notional Lanes (BD37/88)	2 Notional Lanes (BD37/88)	3 Notional Lanes (JKR MTAL)
P1-C (P-11A)	***0.416***	***0.370***	***0.398***
P1-A (P-25)	0.087	-	-
P1-A (P-33)	0.100	-	-

The maximum ultimate load acting on the pier columns are within the ultimate capacity of the structure. Therefore, the existing design of the pier columns satisfies the ULS criteria.

The pier column crack width calculated for P-25 and P-33 is less than 0.25mm, which satisfies the SLS criteria. However, the crack widths for P-11A under 3 different load conditions are found to be more than 0.25mm. Hence, the existing design of P-11A doesn't fulfill the SLS criteria.

Hence, remedial work shall be implemented for all inverted "L" shape pier columns in order to enhance the durability of the structure, especially against the corrosion of the main pier column reinforcement.

10.2. Crosshead Check

The crosshead ultimate moment capacity (ULS) check is summarized as follows;

Table 82. Summary of crosshead ULS moment capacity

Crosshead Type	Ult Moment Capacity (kN.m)		3 Notional Lanes (BD 37/88)
	Without Sidebar	With Sidebar	Maximum ULS Moment (kN.m)
P1-C (P-11A)	*24,439*	26,069	25,805
P1-A (P-25)	59,830	65,605	53,530
P1-A (P-33)	59,830	65,605	49,396

Crosshead Type	Ult Moment Capacity (kN.m)		2 Notional Lanes (BD 37/88)
	Without Sidebar	With Sidebar	Maximum ULS Moment (kN.m)
P1-C (P-11A)	*24,439*	26,069	24,442
P1-A (P-25)	-	-	-
P1-A (P-33)	-	-	-

Crosshead Type	Ult Moment Capacity (kN.m)		JKR MTAL Criteria
	Without Sidebar	With Sidebar	Maximum ULS Moment (kN.m)
P1-C (P-11A)	*24,439*	26,069	25,173
P1-A (P-25)	-	-	-
P1-A (P-33)	-	-	-

The crosshead ultimate shear capacity (ULS) check is summarized as follows;

Table 83. Summary of crosshead ULS shear capacity

P-11A Crosshead ULS Shear Check

BD 37/88 (3 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.72	8.04	0.96
ULS2C1	7.28	8.04	0.90
ULS3C1	6.96	8.04	0.87
ULS4C1	7.71	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

BD 37/88 (2 Notional Lanes) @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	6.90	8.04	0.86
ULS2C1	6.64	8.04	0.83
ULS3C1	6.94	8.04	0.86
ULS4C1	7.70	8.04	0.96

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

JKR MTAL @ 2.5m Depth

Load Case	$Asv/sv_{req'd}$	Asv/sv_{prov}	Capacity Ratio
ULS1C1	7.57	8.04	0.94

*Capacity ratio is based on $Asv/sv_{req'd}$ / Asv/sv_{prov}

P-25 Crosshead ULS Shear Check

BD 37/88 (3 Notional Lanes) @ 2.0m Depth

Load Case	Asv/sv $_{req'd}$	Asv/sv $_{prov}$	Capacity Ratio
ULS1C1	**8.46**	8.04	**1.05**
ULS2C1	7.19	8.04	0.89
ULS3C1	7.98	8.04	0.99
ULS4C1	**8.44**	8.04	**1.05**

*Capacity ratio is based on Asv/sv $_{req'd}$ / Asv/sv $_{prov}$

BD 37/88 (3 Notional Lanes) @ 3.5m Depth

Load Case	Asv/sv $_{req'd}$	Asv/sv $_{prov}$	Capacity Ratio
ULS1C1	7.60	8.04	0.94
ULS2C1	6.57	8.04	0.82
ULS3C1	6.74	8.04	0.84
ULS4C1	7.97	8.04	0.99

*Capacity ratio is based on Asv/sv $_{req'd}$ / Asv/sv $_{prov}$

P-33 Crosshead ULS Shear Check

BD 37/88 (3 Notional Lanes) @ 2.0m Depth

Load Case	Asv/sv $_{req'd}$	Asv/sv $_{prov}$	Capacity Ratio
ULS1C1	7.87	8.04	0.98
ULS2C1	6.58	8.04	0.82
ULS3C1	7.87	8.04	0.98
ULS4C1	7.73	8.04	0.96

*Capacity ratio is based on Asv/sv $_{req'd}$ / Asv/sv $_{prov}$

BD 37/88 (3 Notional Lanes) @ 3.5m Depth

Load Case	Asv/sv $_{req'd}$	Asv/sv $_{prov}$	Capacity Ratio
ULS1C1	7.81	8.04	0.97
ULS2C1	6.66	8.04	0.83
ULS3C1	6.62	8.04	0.82
ULS4C1	7.85	8.04	0.98

*Capacity ratio is based on Asv/sv $_{req'd}$ / Asv/sv $_{prov}$

The crosshead crack width (SLS) check is summarized as follows;

Table 84. Summary of crosshead SLS crack width check

Crosshead Type	3 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	***0.270***	***0.255***
P1-A (P-25)	0.138	0.115
P1-A (P-33)	0.130	0.110

Crosshead Type	2 Notional Lanes (BD 37/88)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	0.196	0.181
P1-A (P-25)	-	-
P1-A (P-33)	-	-

Crosshead Type	3 Notional Lanes (JKR MTAL)	
	Crack Width (mm)	
	Without Sidebar	With Sidebar
P1-C (P-11A)	***0.252***	0.238
P1-A (P-25)	-	-
P1-A (P-33)	-	-

The maximum ultimate moment acting on P-25 and P-33 crossheads is within the ultimate capacity of the structure. Therefore, the existing design of the piers satisfies the ULS moment capacity criteria. In the other hand, the maximum ultimate moment acting on P-11A crosshead is only satisfactory when side reinforcement is taken into account in the ultimate capacity calculation of the crosshead.

The maximum ultimate shear acting on P-11A and P-33 crossheads is within the ultimate capacity of the structure. Therefore, the existing design of the piers satisfies the ULS moment capacity criteria. In the other hand, the maximum ultimate shear acting on P-25 crosshead exceeded the shear capacity of the crosshead for load case ULS1C1 (HA+KEL) and ULS4C1 (SV20)

P-11A and P-33 STM indicates that sufficient top reinforcement has been provided to resist the crosshead ULS tensile force. The resultant compressive stress at the bottom compression strut is found to be less than the strength limit and the diagonal strut force is calculated to be less than the capacity. The vertical tensile force of P-11A is found to exceed the shear capacity calculated from the existing shear link provided. However, the vertical tensile force of P-33 is within the shear capacity calculated from the existing shear link provided.

Crack widths calculated for P-25 and P-33 crosshead are within the allowable limit of 0.25mm, which satisfies the SLS criteria. The crack width for P-11A only satisfies the allowable limit under BD 37/88 two (2) notional lane criteria. When checked against BD 37/88 three (3) notional lane criteria, the crack width is found to exceed the allowable 0.25mm limit. Under load combination with JKR MTAL, the crack width only satisfies the allowable limit when side reinforcement is considered in the crack width calculation.

11. APPENDIX A – CRACK WIDTH VERIFICATION

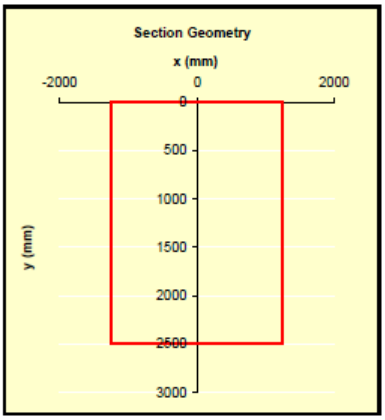
PROJECT : Port Klang Bridge Design Audit
TITTLE : Crack Width Calculation Verification

CRACK WIDTH DESIGN TO BS5400-4:1990 (AXIAL & FLEXURAL)

*cl.4.2.2 Crack width check applies only for Load Combination 1

Design Parameters

f_{cu} =	40	N/mm^2	: Characteristic cube strength at 28 days
E_c =	3.10E+07	kN/m^2	: Short term modulus of elasticity of concrete
Φ=	2.00		: Creep coefficient
E_{cl} =	1.55E+07	kN/m^2	: Long term modulus of elasticity of concrete (allowed for creep effect)
f_y =	460	N/mm^2	: Steel Yield Strength
Es =	2.00E+08	kN/m^2	: Modulus of elasticity of rebar
α=	12.90		: Long term ratio Es/EcL



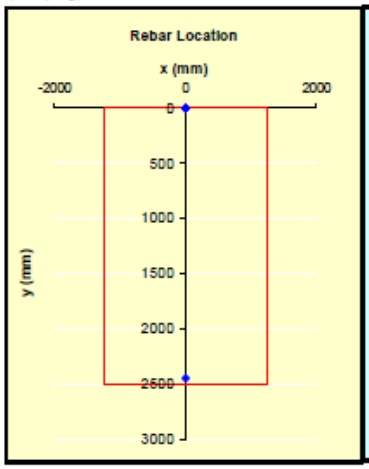
Neutral Axis (Elastic Analysis)

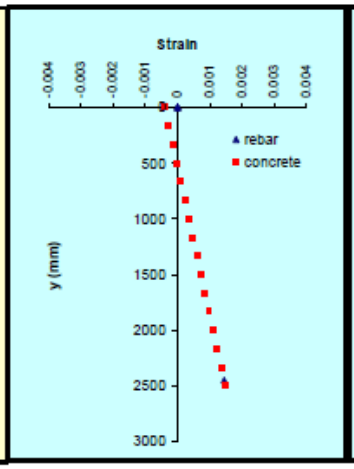
*Neutral Axis, Yn = Σ(A*Yi)/ΣA*

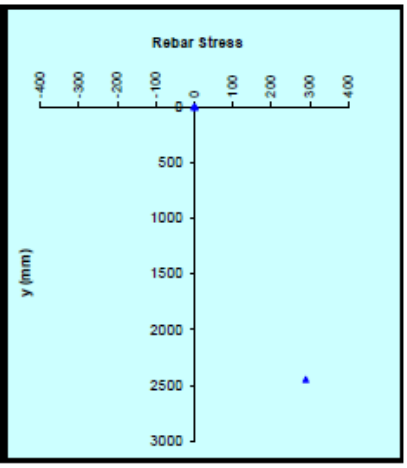
ΣAc=	1,303,899	mm^2	: Area of concrete
ΣAs=	176,416	mm^2	: Transformed area of rebar
ΣA=	1,480,315	mm^2	: Gross area
Err(x)=	0.0	RE_ITERATE	: Err(Yn)= Σ(A*Yi)-Yn*ΣA
Yn=	521.56	mm	

Crack Width Calculation (BS 5400, cl. 5.8.8.2)

P_g =	0	kN	: Permanent Axial Force; (-) compression
M_g =	5000	kN-m	: Permanent moment
M_q =	4000	kN-m	: Live load moment
M_s =	9000	kN-m	: Applied SLS moment
h=	2500	mm	: Overall depth of section
C_{nom}=	35	mm	: Nominal concrete clear cover as per BS5400, Part 4 -table (13)
a_{cr} =	35	mm	: Distance from the point considered (x,y) to the surface of the nearest rebar
ε_m =	1.24E-03		: Average strain at point considered
ε_o =	0.00E+00		: Initial strain due to axial load
$\varepsilon_{stiff.}$ =	-2.40E-04		: Strain due to tension stiffening effect
(1-Mq/Mg) =	2.00E-01		



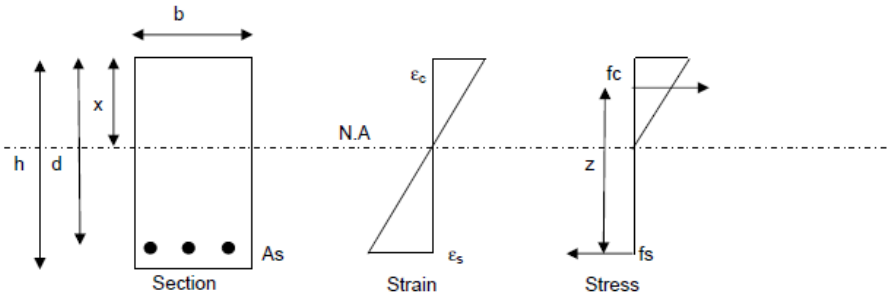




Location		To Nearest Rebar								
x (mm)	y = a' (mm)	xr (mm)	yr (mm)	Ø (mm)	a_{cr} (mm)	ε_1	ε_o	$\varepsilon_{stiff.}$	ε_m	W_{max} (mm)
0	0	0	0	0	0	-3.91E-04	0.00E+00	0.00E+00	-3.91E-04	uncracked
0	0	0	0	0	0	-3.91E-04	0.00E+00	0.00E+00	-3.91E-04	uncracked
0	0	0	0	0	0	-3.91E-04	0.00E+00	0.00E+00	-3.91E-04	uncracked
0	0	0	0	0	0	-3.91E-04	0.00E+00	0.00E+00	-3.91E-04	uncracked
0	0	0	0	0	0	-3.91E-04	0.00E+00	0.00E+00	-3.91E-04	uncracked
0	2500	0	2449	32	35	1.48E-03	0.00E+00	-2.40E-04	1.24E-03	0.131

Crack Width Verification (BS 5400)

Serviceability Crack Width Calculation



f_{cu} =	40	N/mm^2	Concrete Characteristic Strength	
f_y =	460	N/mm^2	Steel Specified Characteristic Strength	
E_s =	200	kN/mm^2	Steel Elastic Modulus	
b_t =	2500	mm	Breadth of Section	
h =	2500	mm	Depth of Section	
c_{nom} =	35	mm	Concrete Cover	
ϕ =	32	mm	Bar Diameter	
M_g =	5000.0	kN.m	Permenant Moment	
M_q =	4000.0	kN.m	Live Load Moment	
M_s =	9000.0	kN.m	Applied Service Moment	
d =	2449	mm	Effective Depth of Tension Reinforcement	
A_s =	13672	mm^2	Tension Reinforcement Steel Area	
α_e =	12.90		Modular ratio	
x =	521.56	mm	Neutral axis	
a_{cr} =	35	mm	Dist. to the nearest longitudinal bar	
z =	2275	mm	Lever arm	
f_s =	289	N/mm^2	Steel Stress,	$f_s = M_s/(A_s*z)$
f_c =	6.07	N/mm^2	Concrete Stress,	$f_c = (f_s*A_s)/(0.5*b*x)$
ε_1 =	1.485E-03			$\varepsilon_1 = (f_s/E_s)*((h-x)/(d-x))$
$3.8b_th(a'-dc)$=	4.699E+10			
ε_s =	1.447E-03			$\varepsilon_s = f_s/E_s$
$\varepsilon_s As(h-dc)$=	3.913E+04			
$1-(M_q/M_g)$=	0.200			

$\varepsilon_{stiff.} = [(3.8b_th(a'-d_c)/(\varepsilon_s As(h-d_c)]*[(1-(M_q/M_g)10^{-9}]$

$= 2.402E-04$

$\varepsilon_m = \varepsilon_1 - [(3.8b_th(a'-d_c)/(\varepsilon_s As(h-d_c)]*[(1-(M_q/M_g)10^{-9}]$ eq. 25

(but not greater than ε_1)

$= 1.245E-03$

$w_{max} = 3a_{cr}\varepsilon_m / [1+2(a_{cr}-c_{nom})/(h-d_c)]$ eq. 24

$= 0.131$ mm

12. APPENDIX B – PROPOSED REMEDIAL WORK

12.1. Pier Type P1-C (Inverted "L" Pier)

12.1.1 Pier Column

Remedial work using flexible coatings are proposed for inverted pier (Type P1-C) i.e. P-10A, 11A, 12A, 13B, 14B and 15B (Total 6 nos.) to fulfil the Serviceability Limit State (SLS) requirement.

Base on the as-built information, there are 17 nos. of inverted "L" for the existing elevated bridge. Namely,

7A, 8A, 9A, 10A, 11A, 12A, 13A, 14A (8 nos.)

8B, 9B, 10B, 11B, 12B, 13B, 14B, 15B, 16B (9 nos.)

In order to prevent deterioration of structure due to ingress of water and rebar corrosion due to the excessive design crack widths, some flexible coatings shall be used. All cracks to be sealed and apply two coats of polymer-modified cementitious waterproofing coating (SIKA Top Seal 109 MY) which has good crack bridging capacity up to 1.0mm. This flexible coating shall have the ability to accommodate the future cracks induced. However, it has limited life span.

12.1.2 Crosshead

Carbon fibre plate is proposed to strengthen the region where the main pier column reinforcement anchorage length into the crosshead is insufficient. A combination of (3 x 3 x 3 layers) + (3 x 3 x 3 layers) + (3 x 3 x 3 layers) Sika CarboDur S1012 is to be applied on each side of the crosshead. The detailed calculation for the proposed strengthening work is shown as below.

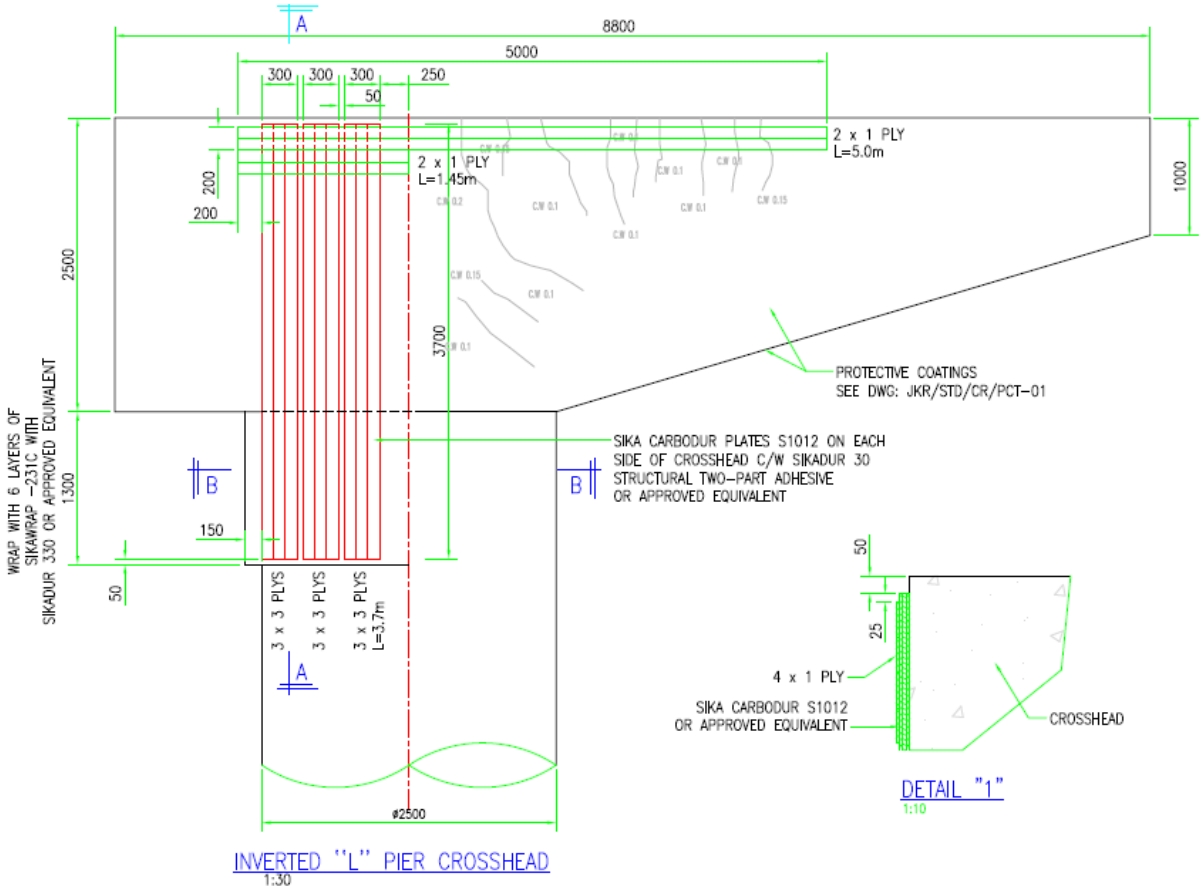


Figure 58. FRP Strengthening for Crosshead Type P1-C (Anchorage)

Crosshead Anchorage using Sika CarboDur Plate S1012

Width	=	100	mm
Thickness	=	1.2	mm
ϕ Tensile	=	0.60	
Min. Tensile Strength	=	2,800	MPa
T_U	=	10,685	kN
$A_{s,req}$	=	6,360	mm^2
	=	53	strips
$A_{s,prov}$	=	54	strips

The optimal bond length for the plate is calculated based on "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures" published by the National Research Council advisory committee on technical recommendations for constructions, 2004.

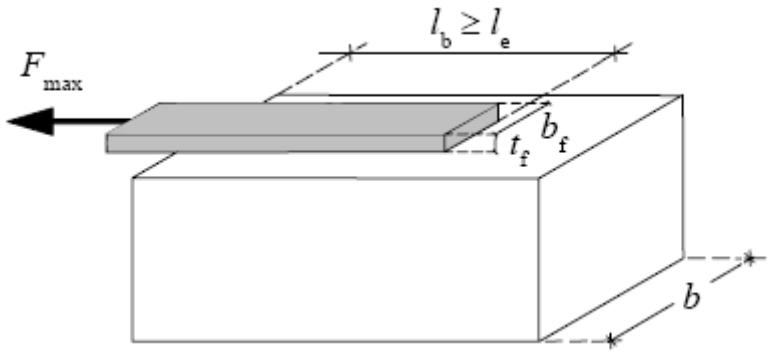


Figure 59. Optimal bond length, l_e

E_f	=	165,000	MPa	(Young modulus of elasticity)
t_f	=	3* 1.2	mm	
f_{ctm}	=	2.0	MPa	(average tensile strength of concrete)

$$l_e = [(E_f.t_f)/(2.f_{ctm})]^{0.5}$$
$$= 385 \text{ mm}$$

Based on Figure 39, it is shown that tensile stress S22 extends up to approximately 0.8 depth of the crosshead. This resulted in approximately 2000mm depth of tensile zone and 500mm depth of compression zone. Therefore, the calculated 385mm bond length is sufficiently bonded in the 500mm compression zone. The average tensile strength of concrete is to be verified by Tensile Pull Off Test on site.

6 layers of carbon fibre wrap using SikaWrap – 231C are introduced at the top of pier where the bottom of carbon fibre plate is anchored to resist the tensile force resulting from the carbon fibre plate.

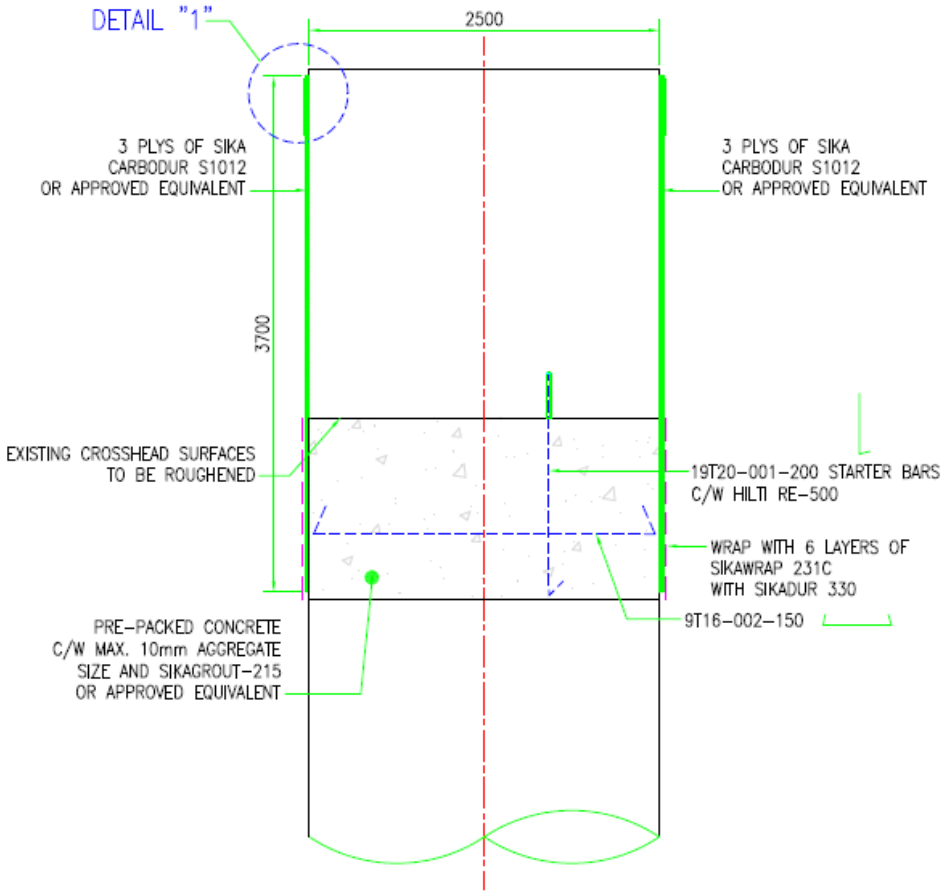


Figure 60. Carbon Fibre Wrap Type P1-C

T_u=	10,685	kN	
$T_u/2$=	5,343	kN	
Spread angle =	45	degrees	
H=	5,343	kN	
Tensile Strength=	4,900	N/mm2	
Thk=	0.127	mm	
T_u=	420	kN/m	(Recommended by SIKA)
Width of Wrapping=	1.25	m	
No. of Layer Required=	5.09		**(Use 6 layers)**

12.2. Pier Type P1-A ("T" Pier)

12.2.1 Pier Column

No strengthening work is required for "T" shaped Pier; P-25 and P-33 (Type P1-A). However, it is recommended that a layer of protective coating to be applied on the pier columns. As cracks were observed at surface, all cracks to be sealed and surfaces shall apply two coats of polymer-modified cementitious waterproofing coating (SIKA Top Seal 109 MY) which has good crack bridging capacity up to 1.0mm.

12.2.2 Crosshead

Carbon fibre plate is proposed to strengthen the region where the main pier column reinforcement anchorage length into the crosshead is insufficient. A combination of (3 x 3 x 2 layers) + (3 x 3 x 2 layers) + (3 x 3 x 2 layer) Sika CarboDur S1012 is to be applied on each side of the crosshead. The detailed calculation for the proposed strengthening work is shown as below.

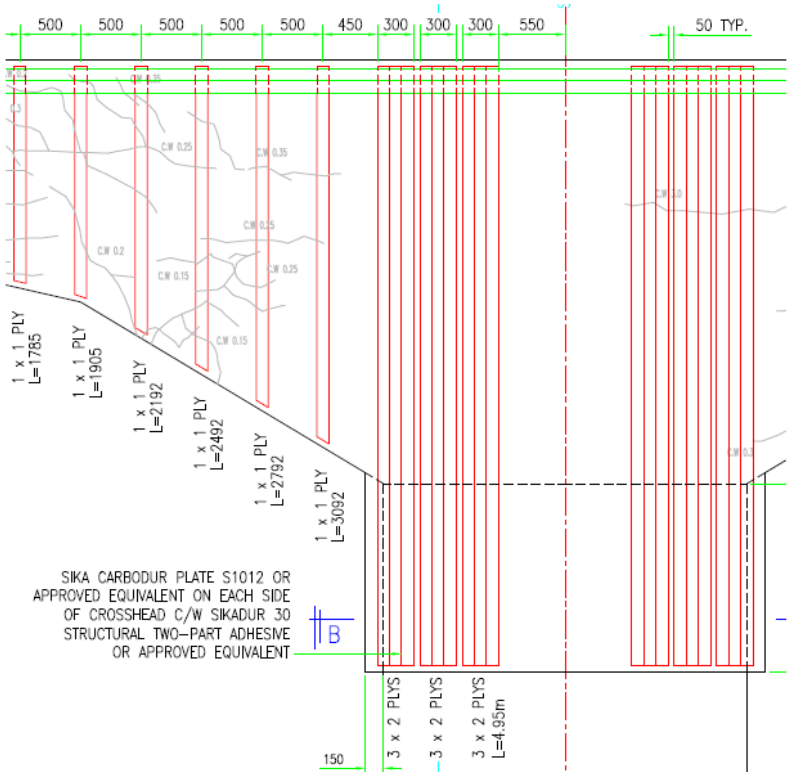


Figure 61. FRP Strengthening for Crosshead Type P1-A (Anchorage)

Crosshead Anchorage using Sika CarboDur Plate S1012

Width	=	100	mm
Thickness	=	1.2	mm
ϕ Tensile	=	0.60	
Min. Tensile Strength	=	2,800	MPa
T_U	=	7,157	kN
$A_{s,req}$	=	4,260	mm^2
	=	36	strips
$A_{s,prov}$	=	36	strips

The optimal bond length for the plate is calculated based on "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures" published by the National Research Council advisory committee on technical recommendations for constructions, 2004.

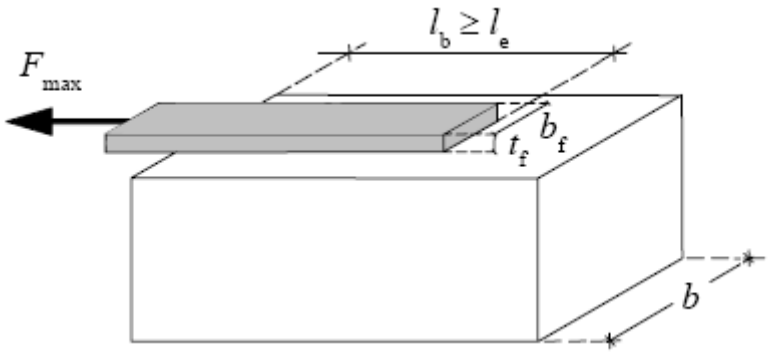


Figure 62. Optimal bond length, l_e

E_f	=	165,000	MPa (Young modulus of elasticity)
t_f	=	2* 1.2	mm
f_{ctm}	=	2.0	MPa (average tensile strength of concrete)

$l_e = [(E_f.t_f)/(2.f_{ctm})]^{0.5}$

$= 315$ mm

Based on Figure 53, it is shown that tensile stress S22 extends up to approximately 0.6 depth of the crosshead. This resulted in approximately 2100mm depth of tensile zone and 1400mm depth of compression zone. Therefore, the calculated 315mm bond length is sufficiently bonded in the 1400mm compression zone. The average tensile strength of concrete is to be verified by Tensile Pull Off Test on site.

4 layers of carbon fibre wrap using SikaWrap – 231C are introduced at the top of pier where the bottom of carbon fibre plate is anchored to resist the tensile force resulting from the carbon fibre plate.

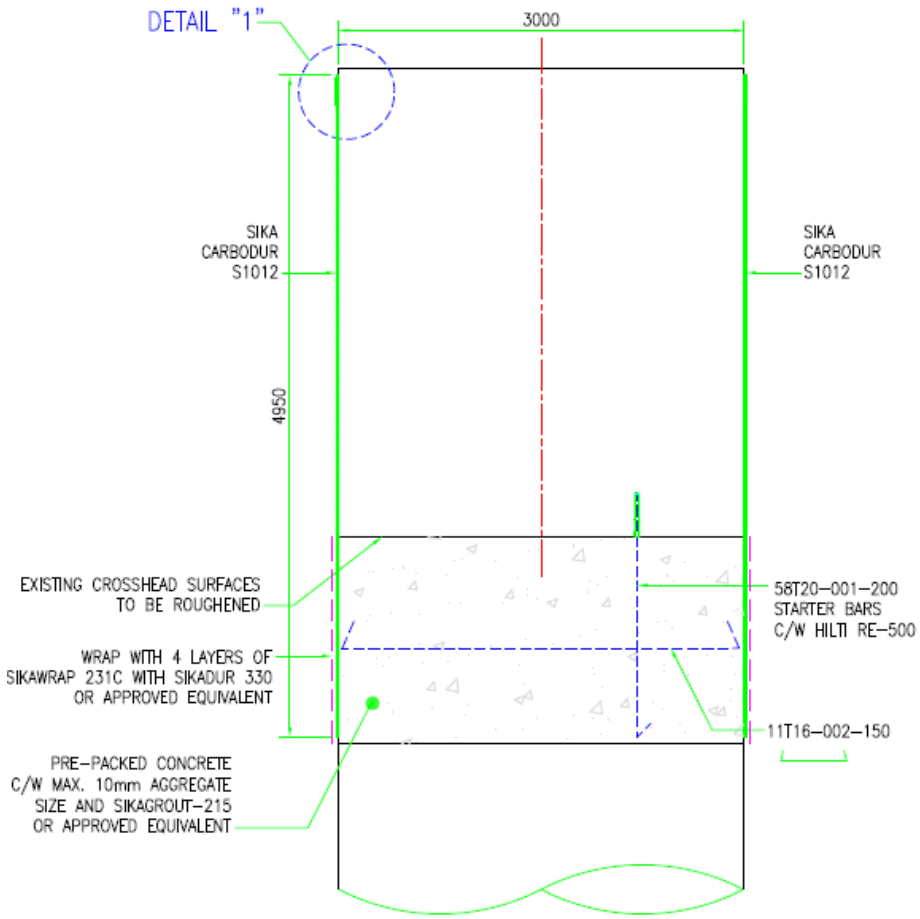


Figure 63. Carbon Fibre Wrap Type P1-A

T_u=	7,157	kN	
T_u/2=	3578.5	kN	
Spread angle =	45	degrees	
H=	3578.5	kN	
Tensile Strength=	4,900	N/mm2	
Thk=	0.127	mm	
T_u=	420	kN/m	(Recommended by SIKA)
Width of Wrapping=	1.5	m	
No. of Layer Required=	2.84		**(Use 4 layers)**

For the shear strengthening at crosshead depth of 2.0m, nine (9) strips of Sika CarboDur S1012 at 500mm spacing is proposed at each face of the crosshead. The detailed calculation for the proposed strengthening work is shown as below.

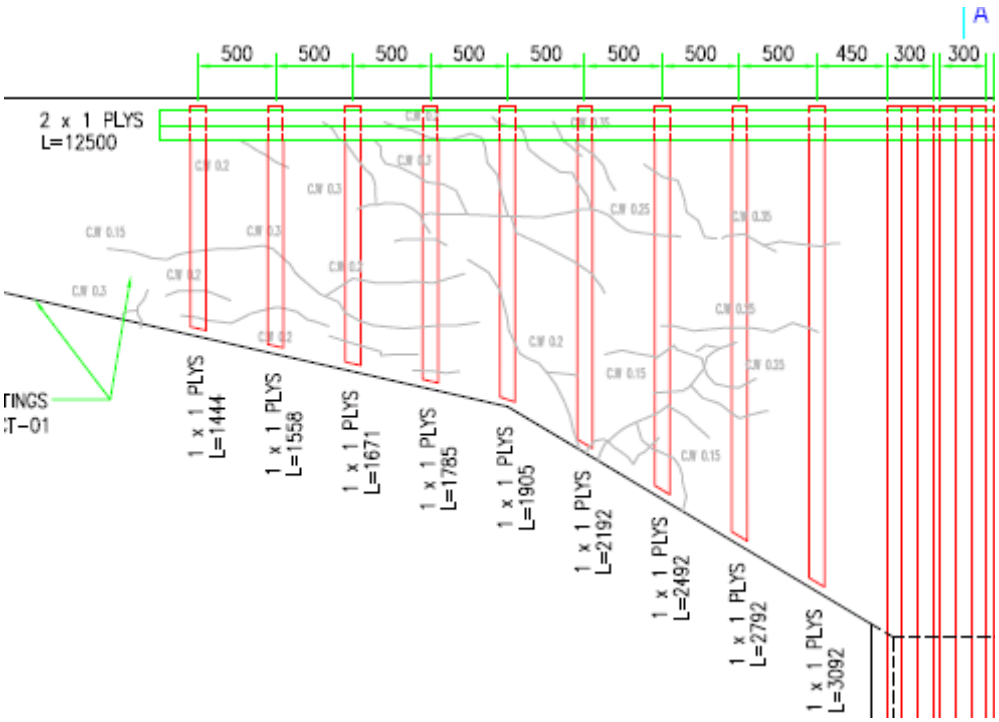


Figure 64. FRP Strengthening for Crosshead Type P1-A (Shear)

Crosshead Shear Strengthening using Sika CarboDur Plate S1012

f_{cu}	=	40	MPa	(28 days concrete cube compressive strength)
f'_c	=	32	MPa	(28 days concrete cylinder compressive strength)
f_y	=	460	MPa	(yield strength of shear steel)
b_w	=	3000	mm	(width of beam)
h	=	2000	mm	(depth of beam)
d	=	1854	mm	(distance to flexural steel)
h_c	=	1044.7	mm	(distance from tension face to neutral axis)
A_s	=	48240	mm^2	(area of flexural steel)
A_v	=	1206	mm^2	(area of shear steel)
V_U	=	6590	kN	(ultimate shear force)
V_U	=	6275	kN	(ultimate shear capacity)

Strengthening of the section (Using modified USD approach)

Required $\phi_L V_L = V_U - \phi_c V_c - \phi_s V_s$

$= 315.0$ kN

Product = Sika CarboDur S1012

θ	=	45	degrees to shear crack	
E_t	=	165000	MPa	
F_{tu}	=	2800	MPa	
t_L	=	1.2	mm	
ε_L	=	0.004	mm/mm	
s_L	=	500	mm	
b_L	=	100	mm	(width of laminate strip)
L_b	=	100	mm	(effective bond length of each FRP anchorage areas; lesser of the length of FRP reinforcing that extends into the compression zone of beam and 100mm)

Required nominal shear strength

$\phi_L V_L$ = 315.0 kN

Check Nominal Shear Strength based on Concrete Bond Strength

ϕ_{Lc} = 0.5

V_b = $0.91(f'_c)^{0.5}$
= 5.15 MPa

A_b = $h_c \cot\theta L_b$ (for wrap)
104470 mm^2

A_b = $(b_L/s_L) h_c \cot\theta L_b$ (for laminate strips)
20894 mm^2

$\phi_{Lc} V_L$ = $\phi_{Lc} 2 V_b A_b$ (2.3.3)
= 537.8 kN > 315.0 kN ***O.K***

Check Nominal Shear Strength based on Tensile Strength

ϕ_L = 0.45

$\phi_L V_L$ = $\phi_L 2 f_{Lu} t_L h_c \cot\theta$ (for wrap) (2.3.4)
= 3159.2 kN > 315.0 kN ***O.K***

ϕ_L = 0.6

$\phi_L V_L$ = $\phi_L 2 f_{Lu} t_L (b_L/s_L) h_c \cot\theta$ (for laminate strips) (2.3.5)
= 842.4 kN > 315.0 kN ***O.K***

Check Nominal Shear Strength based on Shear Strength of Sidakur Epoxy

ϕ_{La} = 0.40
f_{fa} = 24.80 MPa (ultimate shear strength of epoxy resin adhesive)

$\phi_{La} V_L$ = $\phi_{La} 2 A_b f_{fa}$ (2.3.6)
= 2072.7 kN > 315.0 kN ***O.K***

PRODUCT DATA SHEET AND RELEVANT INFORMATION