

REPORT

THIRD PARTY AUDITING OF RAILWAY BRIDGES (PHASE I)

(Job No: CED/CON/TMMP/SAS/2019312)

**Br. No. 1274 UP (23 × 21.28m) + (4 × 45.72m) + (1 × 27.00m) Steel
Girder + PSC Girder + Steel Plate Girder at km. 881/600 – 882/480
between Ullal and Netravathi stations**

CLIENT



SOUTHERN RAILWAY PALGHAT DIVISION

CONSULTANT



**Department of Civil Engineering
NATIONAL INSTITUTE OF TECHNOLOGY CALICUT**

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1. PREAMBLE

Director, NIT Calicut received a request from Chief Bridge Engineer, Headquarters office, Chennai Works Branch, Southern Railway vide letter No. W.439/4/61/Vol.IV(W.245/I/1 Spl. Drive) dated 18 – 09 - 2018, exploring the possibility of whether NIT Calicut can undertake a one - time third party auditing of some selected railway bridges. This was consequent to the instruction issued by Railway Board to various railway divisions for undertaking such inspections of railway bridges through expert institutions. The letter was forwarded to the Civil Engineering Department (CED) and the Structural Consultancy Group of CED followed the matter up. The Railway Board was represented by Palakkad division of Indian Railway. Subsequently, on 05 – 10 - 2018 railway officials came to CED, NITC for discussions. Soft copies of the available drawings of some of the bridges were also shared by Railways to CED for initial assessments. Subsequently, CED expressed its willingness to take up the work and communicated the matter by letter, No. CED/CON/RLY/TPAUD/2018/1 dated 08-11- 2018. Through this letter it was conveyed to Railways that the work could be taken up in two phases, outlining the scope of work to be carried out in the different phases. This was accepted by Railways and they requested the details of consultancy charges for carrying out phase I of the work.

CED replied vide letter No. CED/CON/RLY/TPAUD/2019/1 dated 10 - 01- 2019, the requisite details regarding consultancy charges and scope of the work. Railways replied to this letter showing their interest to award the work to NITC and sought the signing of an MoU to proceed further. Mutual discussions followed and an MoU was signed in the FN of 19-09-2019.

The details of communications are given below:

- i. W.439/4/61/Vol.IV(W.245/I/1 Spl. Drive) dated 18-09-2018.
- ii. Meeting held on 28-09-2018 at NIT Calicut with railway officials.
- iii. J/W/71/G dated 23-10-2018 from Divisional Office, Palakkad division furnishing details of some of the bridges.
- iv. CED/CON/RLY/TPAUD/2019/1, dated 10th January 2019
- v. MoU between Southern Railway and NIT Calicut, dated 19-09-2019.
- vi. CED/CON/HOD/TMMP/SAS/2019312, dated 3rd October 2019
- vii. J/W.71/G, dated 03-12-2019
- viii. CED/CON/HOD/TMMP/SAS/2019312/1, dated 02-01-2020
- ix. CED/CON/HOD/TMMP/SAS/2019312/2, dated 02-01-2020
- x. CED/CON/HOD/TMMP/SAS/2019312/3 dated 23-01-2020

- xi. CED/CON/HOD/TMMP/SAS/2019312/4 dated 11-03-2020
- xii. CED/CON/HOD/TMMP/SAS/2019312/5 dated 12-03-2020
- xiii. CED/CON/HOD/TMMP/SAS/2019312/6 dated 13-03-2020
- xiv. CED/CON/HOD/TMMP/SAS/2019312/7 dated 11-05-2020
- xv. No. J/W.71/G dated 27-07-2020
- xvi. CED/CON/HOD/TMMP/SAS/2019312/8 dated 22-07-2020
- xvii. CED/CON/HOD/TMMP/SAS/2019312/9 dated 22-07-2020
- xviii. CED/CON/HOD/TMMP/SAS/2019312/10 dated 14-08-2020
- xix. No. J/W.71/G dated 26-09-2020
- xx. CED/CON/HOD/TMMP/SAS/2019312/13 dated 21-09-2020
- xxi. CED/CON/HOD/TMMP/SAS/2019312/15 dated 17-11-2020
- xxii. CED/CON/HOD/TMMP/SAS/2019312/17 dated 24-12-2020
- xxiii. CED/CON/HOD/TMMP/SAS/2019312/18 dated 19-01-2021
- xxiv. CED/CON/HOD/TMMP/SAS/2019312/19 dated 10-03-2021
- xxv. CED/CON/HOD/TMMP/SAS/2019312/20 dated 18-03-2021

2. SCOPE AND METHODOLOGY

Railway bridges, which are known for their permanent resistance to heavy movable loads, are exposed to large load ranges and severe environmental conditions that may lead to damage and failure of the system. Keeping in mind the large risk and economic ramifications, it is necessary to assess the health of these structures systematically by considering all loading types namely, gravity loading, wind loading and seismic loading and effect of corrosion and fatigue. In view of the above and based on the terms and conditions laid down in the MoU, the scopes of the present investigation are identified as below.

- i. Inspection of the superstructure of the bridge, bearings, piers and abutments and top surface of the bridge along with load transfer mechanism.
- ii. Visual assessment of corrosion in the bridge.
- iii. Collection of data pertaining to the state of the bearings and water drainage.
- iv. Collection of necessary data for structural modelling and analysis.
- v. Preliminary analysis of the bridges to check the adequacy of the bridge to carry 25 t axle load since the bridge was designed in the past for loads lesser than 25 t axle load.
- vi. Overall assessment of the bridge and its integrity.

Analysis for earthquake loads is not carried out as dimensions pertaining to the substructure and its condition is not completely available. This can be performed after the

underwater inspection and measurements in phase II of the audit.

In general, estimation of fatigue, corrosion, and their combined effects pose practical difficulties. However, an attempt will be made to collect all relevant data for predicting the residual life ^{[4][8]}, as part of phase II of the audit.

Considering the above aspects, the following methodology is finalised.

- i. Visual inspection of the bridge
- ii. Field measurements for modelling purpose
- iii. Condition assessment of various elements of the bridge and to summarize the results in a tabular form
- iv. Structural modelling and analysis of the bridge for stipulated loads.
- v. Assessment of the residual strength and safety based on the analysis results

3. INSPECTION OF BRIDGE

3.1 Visual Examination

The visual inspections were carried out on the bridge on 10-01-2020 (Tuesday) at 4:00 pm and 11-01-2020 (Wednesday) at 8:30 am. Officials from Kannur Railway section were also present during the inspection. The bridge consist of total 28 spans (23 spans of PSC Girder type each 21.28m long + 4 spans are of steel truss through type each 45.72 m + one span of steel plate girder type of span 27m). Super structure is supported on RCC piers and RCC abutments. Photographs of the bridge showing various type of spans and elements are shown in Figures 1 to 8.

On 15/01/2021 one more visit was conducted at this bridge site to assess the condition and for taking a few measurements pertaining to various members.



Figure 1. Side view of PSC Girder of Br.1274 UP between Ullal and Netravathi Station



Figure 2. Inside view of PSC Girder of Br.1274 UP between Ullal and Netravathi station



Figure 3. Inside view of the typical Steel truss girder of Br.1274 UP between Ullal and Netravathi Station



Figure 4. Top chord, bottom chord and diagonal members of steel truss



Figure 5. Steel roller type bearing



Figure 6. Elastomeric bearing of PSC box girder



Figure 7. Side view of steel plate girder of Br.1274 UP between Ullal and Netravathi Station



Figure 8. Typical RCC pier of the bridge

3.2 Field Measurements and condition assessment

3.2.1 Steel truss girder spans

Figures 9 and 10 show the schematic diagrams of details of the steel truss girder and typical member details prepared based on the available drawings. All the missing dimensions to complete the geometry of the bridge were measured from the site. During inspection it was noted that all the elements of bridge were in sound condition and there was no signs of distress or deterioration. Also it was seen that extent of corrosion is minimal in these spans.

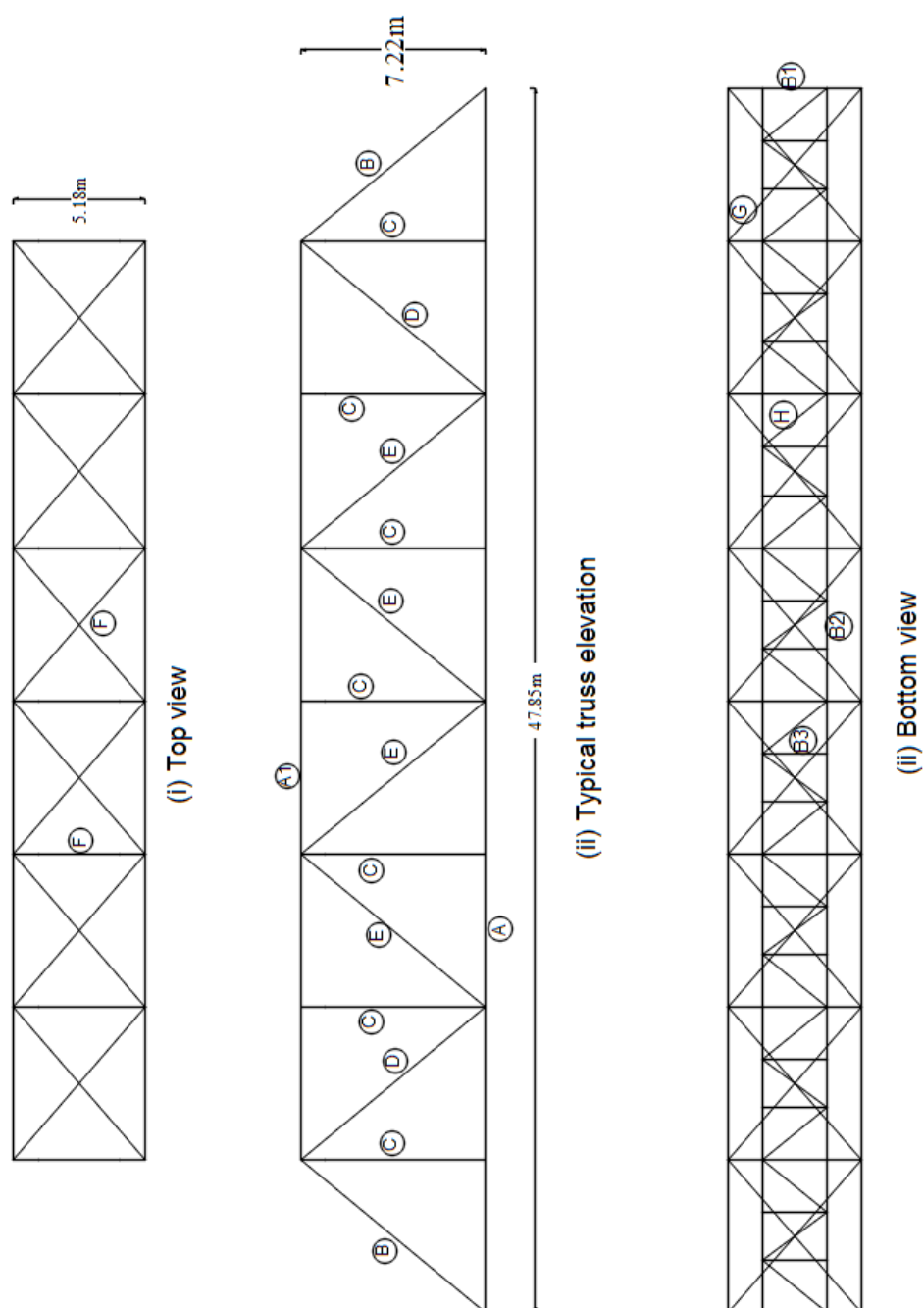


Figure 9. Geometrical details of a typical truss bridge (not to scale)

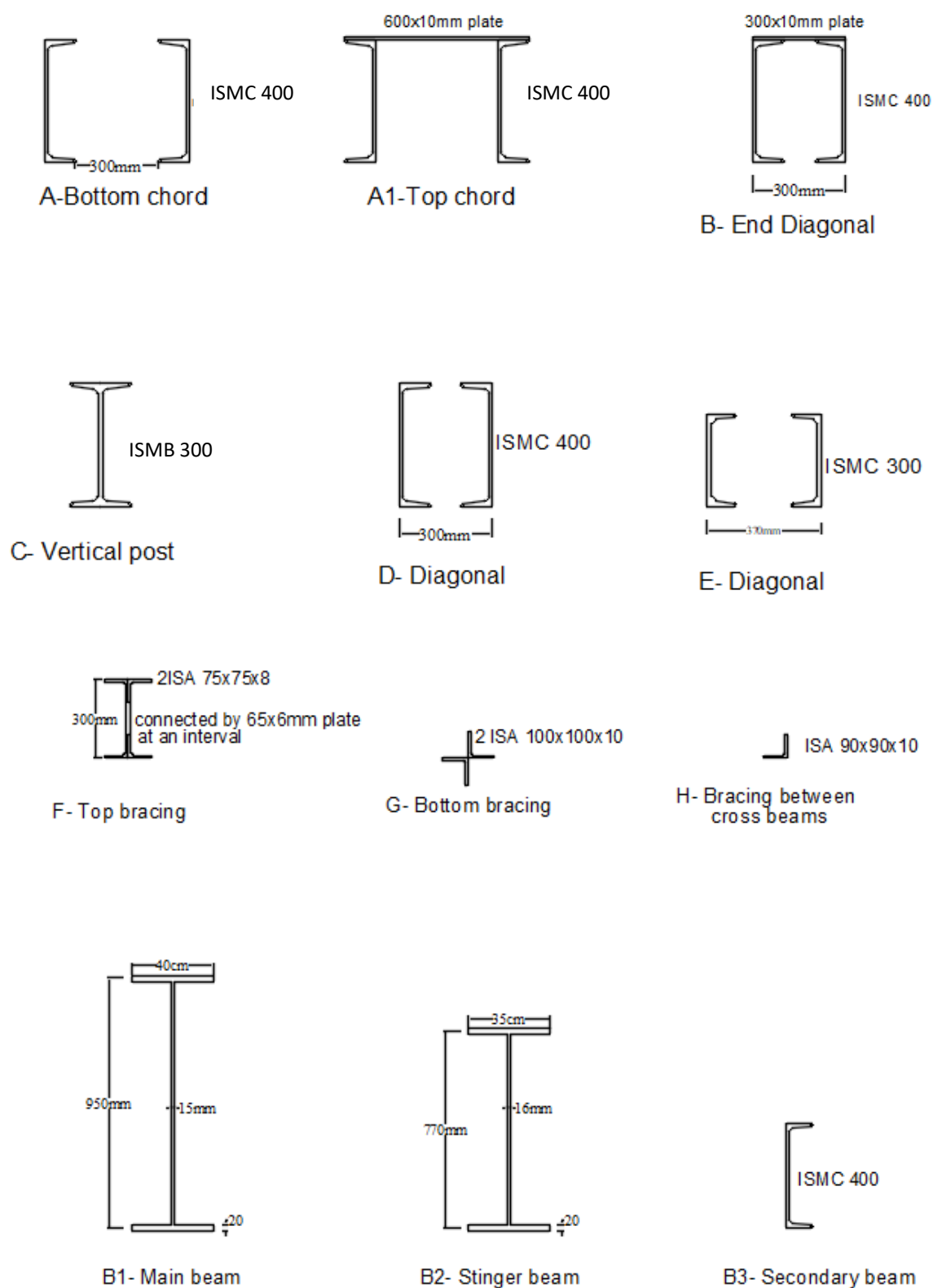


Figure 10. Section details of each elements

3.2.2 Plate Girder span

Schematic diagram of plate girder bridge is shown in Figure 11. The dimensions were taken from the field measurements. In this span also extent of corrosion was very minimal.

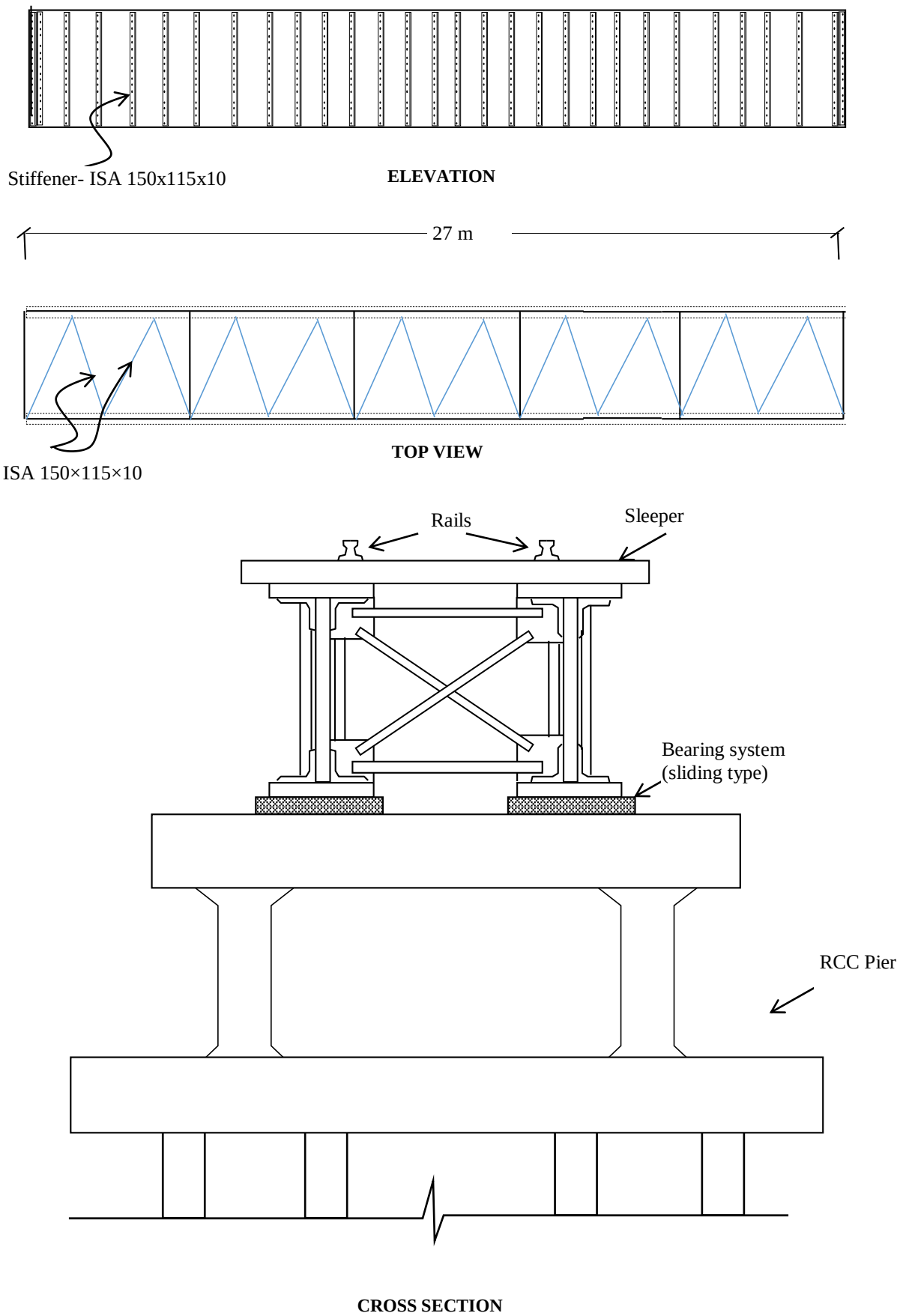


Figure 11. Geometrical details of the bridge (not to scale)

3.2.3 PSC Girder spans

All the 23 PSC spans of the bridge were inspected and were found to be in very good condition. As the bridge is a relatively new, no distresses could be noted in any of these spans. Figure 2 and Figure 6 shows the interior of a box girder and a typical bearing, respectively.

4. MODELING AND ANALYSIS OF BRIDGE SPANS

4.1 Description of loading

It is presumed that the design of the bridge is as per Bridge rules 1941. In the present work, one of the requirements is to check the adequacy of the bridge to carry 25-ton axle loading. In this report, the specification of loads is based on the first revision of Bridge rule 1964. Accordingly, equivalent uniformly distributed load (EUDL) specified for 25-ton axle load (Cl. 3.6) has been used for calculations. Also as per clause 2.4.1.1 of Bridge Rules, the augmentation in load due to dynamic effect is considered. The racking load and wind load are calculated as per clause 2.9.2.

With regard to PSC girder spans, the complete details for modeling the PSC span including cable profiles are not available to NITC. Hence the detailed modeling, structural analysis and design check for the PSC girder span could not be carried out. However, it is noted that the bridge is relatively new and the PSC girder spans of the bridge are found to be sound as per the visual inspection.

4.2 Modeling and analysis of a typical Steel truss girder span

Calculation of loads

DESIGN LOADS

Dead load

2.5m MCP 150@16.8kg/m sleepers are placed at a spacing of 40cm

Weight of sleeper per meter on each girder = 0.515 kN/m

Weight of rail per meter = 0.43 kN/m

Weight of corrugated sheeting per meter = 0.10 kN/m

Self weight of the structural elements were taken by default by the modelling software

Live Load (From IRBM Appendix-XXIII)

EUDL for bending moment for span 47.85m = 4379 kN

EUDL for Shear force for span 47.85m = 4823 kN

Impact factor CDA = 0.298

Total EUDL for bending moment on each girder = 2842 kN

Total EUDL for shear force on each girder = 3130 kN

Live Load (From IRBM Appendix-XXIII)

Basic wind pressure as per IS 875 part 3	=	0.85	m/s
Wind acting in the windward direction	=	3.27	kN/m
Wind acting in the leeward direction	=	0.81	kN/m
Wind acting on the train	=	3.15	kN/m
Racking force	=	5.88	kN/m

4.2.1 Result of Analysis

The modelling and analysis of the truss bridge were done in STAAD Pro. 2008 using the measured geometry and calculated loads. Model of the typical single span of 45.72 m is shown in Figure 12 and Figure 13.

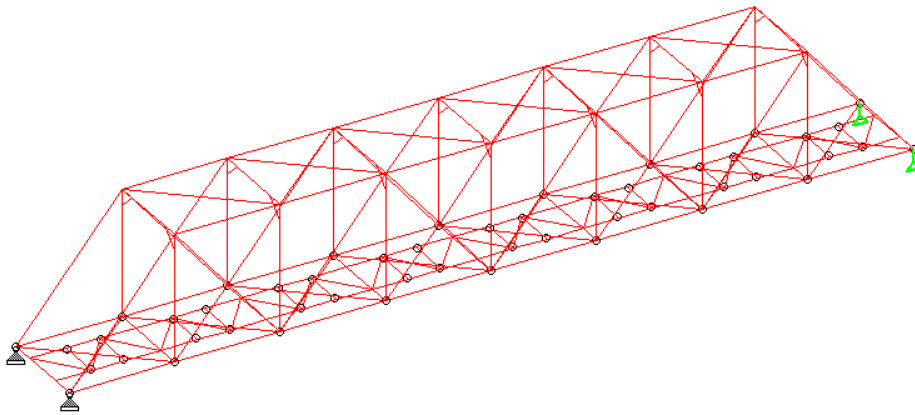


Figure 12. STAAD Pro. Model of the truss bridge

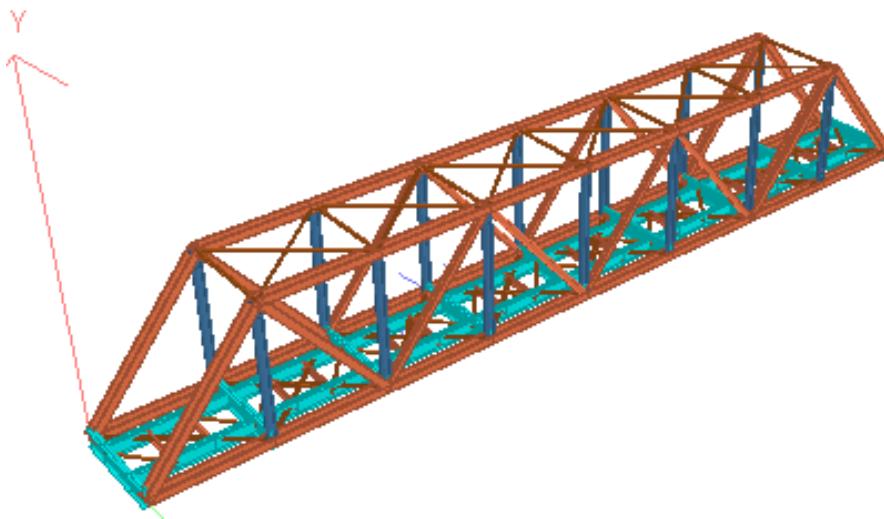


Figure 13. 3D rendered view of the bridge

Analysis was done and the design forces in each member is found out. The design forces in all elements of bridge is given in Table 1 and 2.

Table 1: Design Forces of the truss members

Member	Forces (kN)	Type
Top chord members (A)	3723	Compression
Bottom chord members(A1)	1148	Tension
End diagonals (B)	2650	Compression
Diagonals (D)	1849	Tension
Diagonals (E)	1171	Compression
	396	Tension
Vertical members (C)	556	Compression
Bottom bracings (G)	372	Compression
Top Bracings (F)	137	Compression

Table 2: Design Forces in beams

Beam	Moment (kNm)	Shear force (kN)
Main beam (B1)	722	538
Stinger beam (B2)	361	264
Bracing (H)	40kN (axial compression)	

4.2.2 Demand and Capacity

Here the integrity and safety of the bridge are assessed by the parameter 'Demand to Capacity Ratio' (DCR) of various elements. As the effect of corrosion is minimal in all the members, it is not considered for the calculation of capacity of the members.

4.2.3 Demand and Capacity of truss bridge

DESIGN CHECK OF TOP CHORD MEMBER (A1)

Check for compression

Design force in the member	=	3723	kN	compression
Area	=	18815	mm ²	
Moment of Inertia , I _{xx}	=	4.8E+08	mm ⁴	
Radius of gyration, r _{yy}	=	159.34	mm	

As per Table 2 IS 800:2007, the section is plastic.

$$\begin{aligned}\text{Effective length of member, } L_{\text{eff}} &= 5.265 \text{ m} \\ \text{Slenderness ratio, } \lambda &= L_{\text{eff}} / r \\ &= 33.04\end{aligned}$$

buckling class 'c' of table 9(c) of IS800:2007,

$$\text{Maximum permissible compressive stress, } f_{cd} = 207 \text{ N/mm}^2$$

$$\begin{aligned}\text{Load carrying capacity} &= f_{cd} \times A \\ &= 3894.705 \text{ kN} \quad \boxed{\text{Safe}}\end{aligned}$$

DESIGN CHECK OF BOTTOM CHORD MEMBER(A)

Check for Tension

$$\begin{aligned}\text{Design Force} &= 1148 \text{ kN} \\ \text{Area} &= 12586 \text{ mm}^2 \quad F_y = 250 \text{ N/mm}^2 \\ \text{Load carrying capacity} &= A_g f_y / \gamma_{mo} \\ &= 2860.45 \text{ kN} \quad \boxed{\text{Safe}}\end{aligned}$$

DESIGN CHECK OF END DIAGONAL (B)

Check for compression

$$\begin{aligned}\text{Design force in the member} &= 2650 \text{ kN} \quad \text{compression} \\ \text{Area} &= 15815 \text{ mm}^2 \\ \text{Moment of Inertia, } I_{yy} &= 2.4\text{E}+08 \text{ mm}^4 \\ \text{Radius of gyration, } r_{yy} &= 122 \text{ mm}\end{aligned}$$

As per table 2 IS 800:2007, the section is plastic.

$$\begin{aligned}\text{Effective length of member, } L_{\text{eff}} &= 8.352 \text{ m} \\ \text{Slenderness ratio, } \lambda &= L_{\text{eff}} / r \\ &= 68.46\end{aligned}$$

buckling class 'c' of table 9(c) of IS 800:2007,

$$\text{Maximum permissible compressive stress, } F_{cd} = 156 \text{ N/mm}^2$$

$$\begin{aligned}\text{Load carrying capacity} &= f_{cd} \times A \\ &= 2467.14 \text{ kN} \quad \boxed{\text{Low FOS}}\end{aligned}$$

DESIGN CHECK DIAGONAL (D)

Check for Tension

$$\begin{aligned}\text{Design Force} &= 1849 \text{ kN} \\ \text{Area} &= 12586 \text{ mm}^2 \quad F_y = 250 \text{ N/mm}^2 \\ \text{Load carrying capacity} &= A_g f_y / \gamma_{mo} \\ &= 2860.45 \text{ kN} \quad \boxed{\text{Safe}}\end{aligned}$$

DESIGN CHECK OF END DIAGONAL (E)

Check for compression

Design force in the member	=	1171	kN	compression
Area	=	9128	mm ²	
Moment of Inertia , I _{yy}	=	1.3E+08	mm ⁴	
Radius of gyration, r _{yy}	=	118	mm	

As per table 2 IS 800:2007, the section is plastic.

Effective length of member, L _{eff}	=	8.352	m
Slenderness ratio, λ	=	L _{eff} / r	
	=	70.78	

buckling class 'c' of table 9(c) of IS800:2007,

Maximum permissible compressive stress, F _{cd}	=	151	N/mm ²	
Load carrying capacity	=	f _{cd} x A		
	=	1378.328	kN	Safe

Check for Tension

Design Force	=	396	kN		
Area	=	9128	mm ²	F _y = 250	N/mm ²
Load carrying capacity	=	A _g f _y /γ _{mo}			
	=	2074.55	kN	Safe	

DESIGN CHECK OF VERTICAL MEMBER(C) TENSION MEMBER

Design Force	=	556	kN	(Tension)	
Area	=	5626	mm ²	F _y = 250	N/mm ²
Load carrying capacity	=	A _g f _y /γ _{mo}			
	=	1278	kN	Safe	

DESIGN CHECK OF BOTTOM BRACING (G) COMPRESSION MEMBER

section 2ISA 100X100X10, Properties of the section from steel tables:

Design force in the member	=	372	kN
Effective Length	=	3.56	m
Area	=	4518	mm ²
r _{yy}	=	30.3	mm
Slenderness ratio, λ	=	L _{eff} / r	
	=	117.49	

buckling class 'c' of table 9(c) of IS800:2007,

Maximum permissible compressive stress, F _{cd}	=	124	N/mm ²	
Load carrying capacity	=	f _{cd} x A		
	=	560.232	kN	Safe

DESIGN CHECK OF TOP BRACING (F)
COMPRESSION MEMBER

section 4 ISA 75X75X8, Properties of the section from steel tables:

Design force in the member	=	137	kN		
Effective Length	=	3.56	m		
Area	=	4552	mm ²		
Moment of Inertia , I _{yy}	=	4549656	mm ⁴		
r _{yy}	=	40.4	mm	k3=	20
Slenderness ratio, λ	=	L _{eff} / r			
	=	88.12			

buckling class 'c' of table 9(c) of IS800:2007,

Maximum permissible compressive stress, F _{cd}	=	91	N/mm ²		
Load carrying capacity	=	f _{cd} x A			
	=	414.232	kN		Safe

DESIGN CHECK OF MAIN BEAM B1

Span of Beam, l	=	5.18	m		
BM and SF					
Bending Moment ,Mu	=	722	kNm		
Shear Force,SF	=	538	kN		

$$Z_{pz} = 7440000 \text{ mm}^3$$

$$\text{Web area, } A_w = 13650 \text{ mm}$$

SECTION CLASSIFICATION

Neutral axis mid depth	=	d/t _w			
	=	60	<84ε		

Hence the section is plastic

Cross beams are provide connecting two beams , hence it is laterally supported beam

Moment Capacity, M _d	=	βZ _p f _y /γ _{mo}			
	=	1690.90	kNm		Safe
Shear Capacity, SF	=	d _t wf _y /γ _{mo} √3			
	=	1791	kN		Safe

DESIGN CHECK OF CROSS BEAM B2

Span of Beam, l	=	5.98	m		
BM and SF					
Bending Moment ,Mu	=	361	kNm		

$$\text{Shear Force, SF} = 264 \text{ kN}$$

$$Z_{pz} = 5250000 \text{ mm}^3$$

$$\text{Web area, } A_w = 11680 \text{ mm}$$

SECTION CLASSIFICATION

$$\begin{aligned} \text{Neutral axis mid depth} &= d/t_w \\ &= 45.62 < 84\epsilon \end{aligned}$$

Hence the section is plastic

Cross beams are provide connecting two beams , hence it is laterally supported beam

$$\begin{aligned} \text{Moment Capacity, } M_d &= \beta Z_p f_y / \gamma_{mo} \\ &= 1443.75 \text{ kNm} \quad \boxed{\text{Safe}} \\ \text{Shear Capacity, SF} &= d t_w f_y / \gamma_{mo} \sqrt{3} \\ &= 1532 \text{ kN} \quad \boxed{\text{Safe}} \end{aligned}$$

DESIGN CHECK OF BOTTOM LATERAL BETWEEN CROSS BEAMS (H) COMPRESSION MEMBER

section ISA 90x90x10, Properties of the section from steel tables:

$$\begin{aligned} \text{Design force in the member} &= 40 \text{ kN} \\ \text{Effective Length} &= 3.55 \text{ m} \\ \text{Area} &= 1703 \text{ mm}^2 \\ \text{Moment of Inertia} &= 516000 \text{ mm}^4 \\ I_{yy} &= 17.4 \text{ mm} \\ r_{yy} &= L_{eff} / r \\ \text{Slenderness ratio, } \lambda &= 204.02 \\ &\text{buckling class 'c' of table 9(c) of IS800:2007,} \\ \text{Maximum permissible compressive stress, } F_{cd} &= 34.5 \text{ N/mm}^2 \\ \text{Load carrying capacity} &= f_{cd} \times A \\ &= 58.75 \text{ kN} \quad \boxed{\text{Safe}} \end{aligned}$$

From the visual inspection, it was noted that the effect of corrosion was minimal in the connections when compared to the members. Therefore, connections are excluded in DCR calculations. Table 3 gives a summary of the DCRs for various elements.

Table 3: Demand capacity ratio for various elements

Sl No	Element/Section force	Demand (kN/kNm)	Capacity (kN/kNm)	DCR
1	Top chord members (A) - compression	3723	3894	0.96
2	Bottom chord members(A1) - tension	1148	2860	0.40
3	End diagonals (B) - compression	2650	2467	1.08
4	Diagonals (D) - tension	1849	2860	0.65
5	Diagonals (E) - Compression	1171	1378	0.85
6	- tension	394	2074	0.19
7	Vertical members(C) -	556	1278	0.44
8	Bottom bracings (G) - compression	372	560	0.66
9	Top Bracings (F) - compression	137	414	0.33
10	Main beam (B1) - moment	722	1690	0.43
	-Shear force	538	1791	0.3
11	Stinger beam (B2) - moment	361	1443	0.25
	-Shear force	264	1532	0.17
12	Bracing (H) - Compression	40	58	0.69

Important notes

- i. The demand includes load factor and capacity includes partial safety factors.
- ii. Here the analysis is performed for the superstructure alone presuming that the foundations are intact.

4.3. ANALYSIS OF THE PLATE GIRDER SPAN

The detailed calculation of loads and analysis of the plate girder bridge was done following relevant bridge codes ^{[1][2][3][6]}, which are furnished in the Appendix which gives the demand of the various elements of the plate girder bridge.

4.3.1 Demand and Capacity for the Plate girder span

Table 4 gives a summary of the DCRs for various element plate girder bridge

Table 4: Demand capacity ratio for various elements

Sl No	Element/Section force	Demand (kN/kNm)	Capacity (kN/kNm)	DCR
1	Girder/Moment	10184	14000	0.73
2	Girder/Shear	1776	2530	0.70
3	End panel/ Shear	680	1169	0.58
4	End panel/ Moment	200	230	0.87
5	Bearing stiffener/ bearing	1473	3548	0.42
6	Bearing stiffener/ buckling	1473	4144	0.36
7	Intermediate stiffener/ buckling	685	927	0.74
8	Top lateral bracing/ Axial force	59	156	0.38
9	Bottom lateral bracing/ Axial force	116	156	0.74
10	Cross frame- diagonal member /compression	41	181	0.23
11	Cross frame- diagonal member /Tension	41	522	0.08
12	Cross frame- horizontal member /Compression	19	306	0.06

Important note

- The demand includes load factor and capacity includes partial safety factors.*
- Here the analysis is performed for the superstructure alone presuming that the foundations are intact.*

5. COMMENTS AND INFERENCE

- Table 3 shows that all the elements of steel truss type spans have a DCR value less than unity except the end diagonal from which it can be inferred that the condition of super structure of these spans is **by and large satisfactory**.
- The DCR value of 1.08 in the end diagonal implies that the **margin of safety available for that member is lower than that required by the code**.
- Table 4 shows that all the elements of plate girder have a DCR value less than unity. This implies that the condition of the superstructure of these spans is satisfactory

On the basis of the above points it is felt that conclusions on the stability and safety of the bridge can be made only after conducting phase II of the audit, which shall include assessing the details pertaining to the substructure and its condition by underwater inspection and measurements. The modeling and design check for the PSC girder in the superstructure also shall be carried out along with that. Till then it is recommended to continue the safety precautions being followed currently including the restricted speed of trains through the bridge, if any.

Comment on the maintenance and repair practices

It is observed that repair works are being carried out by Railways to rectify the issues especially due to corrosion. It is also informed that member replacements are occasionally done when the need arises. However, looking at the present condition of the bridge with regard to corrosion it is felt that there is a delay in implementing the maintenance works. It is suggested to conduct more regular and systematic maintenance. Moreover, the need for a record-keeping of the member replacements performed at various instances is felt. This can be achieved by maintaining the as-built drawing and recording the interventions as and when it is performed. This will help in assessing the overall integrity of the bridge at any point of time during its life span.

6. ASSESSMENT PROPOSED IN PHASE II

It is proposed to do the underwater inspection and condition assessment of the substructure in phase II. This will also provide necessary data for performing analysis for earthquake loads. Assessment of corrosion-fatigue strength of components and their life expectancy is required as there can be strength degradation due to corrosion and repeated loadings. Tests required for the above assessments as well as forecasting of residual life of the bridge are planned to be carried out in phase II. Dynamic testing and tests for assessing the stiffness degradation, if any, of the super structure also are envisaged in the Phase II of the audit.

Members of the Investigation team

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APPENDIX

DESIGN OF DECK PLATE STEEL GIRDER BRIDGE (25 T Loading 2008 Broad gauge)

Span of girder = 27 m

1. Dead load Calculation

Dead load Calculation

1) Total dead Load of the bridge = 602.8 kN
 weight of sleeper per m = 2.06 kN/m
 weight of rails per m = 0.86 kN/m
 weight of track = 2.92 kN/m
 2) Total Weight of track = 78.84 kN
 Total Dead Load = 681.63 kN

Depth of Web = 2212 mm
 Thickness of web = 12 mm
 flange thickness = 48 mm
 Width of flange = 480 mm

Wt of Girder = 22.32555 kN
 Sleeper MCP 150@16.8kg/m
 length = 2.5 m
 spacing = 0.4 m
 wt = 16.8 kg/m
 Wt of Rail = 0.43 kN/m

Live Load (From IRBM APPENDIX- XX111)

EUDL for BM = 2656.21 kN
 EUDL for SF = 2914.06 kN
 CDA = 0.392

Total Load for BM = 4379.1 kN

Maximum BMu in each Girder = 11084.5 kNm

Total Load for SF in each Girder = 4738.0 kN

Maximum SF in each Girder = 1776.8 kNm

2. Section Classification and Capacity

bi = 480 mm be = 232 mm

b/t_f = 10 < 29.3ξ Flanges are plastic b/t_f = 4.8 < 8.4ξ Flanges are plastic
 d/t_w = 184.3333 > 126ξ web Semi compact

β = 1

Z_p = 7.70E+07

Design moment carrying of girder

M_d = β_bZ_pf_y/γ_{m0} = 14000 kNm

Safe

3. End Panel Design (Tension Field Method)

Spacing of Transverse vertical stiffeners = 750 mm f_{yw} = 250 MPa

ζ_b = 41.74 MPa

f_v = 186.0 MPa

For calculating maximum BM at the section 750 mm from left support, Live load should be considered on the right of the section

Loaded Length = 26.25 m

EUDL for BM = 2592 kN

CDA = 0.398

Total Live Load = 3623.6 kN

Reaction @ R_A = 2102.3 kN

M_u in each plate girder at 2.8m from left end = 1177.2 kNm

Axial force in the Flange due to BM, N_f = 520.9 kN

V_d = 2530.4 kN

>V safe

Check for shear capacity of end panel

$$\text{shear capacity of longitudinal section of the end panel} = A_v f_y / \sqrt{3} = 1169 \text{ kN}$$

$$V_{cr} = 1108.0 \text{ kN}$$

$$H_q = 4037.7 \text{ kN}$$

Since $V < V_{tf}$ H_q may be reduced by the ratio

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}} = 0.34$$

$$\text{hence } H_q = 1360.6 \text{ kN}$$

$$R_{tf} = 680.3 \text{ kN}$$

< V_p Safe

Check for moment capacity of end panel

$$M_{tf} = H_q d / 10 = 301.0 \text{ kNm}$$

Since the single stiffener is provide at the support panel need to resist only 2/3 M_{tf}

$$2/3 M_{tf} = 200.6 \text{ kNm}$$

$$M_q = z f_y / x \gamma_{mo} = 230.1 \text{ kNm}$$

Safe

5. Design of Bearing Stiffener at the end

Bearing stiffener consist of double angle ISA 150x115x10 at either side of the web

$$\text{Force on end post due to moment, } 2/3M_{tf} = 106.2 \text{ kN}$$

$$\text{Desing force at the end post} = 1882.9 \text{ kN}$$

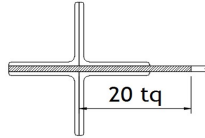
$$\text{Area required} = 6627.8 \text{ mm}^2$$

$$\text{Bearing Area of stiffener provided} = 20264 \text{ mm}^2$$

Safe

$$b = 100 \text{ mm}$$

$$n = 50 \text{ mm}$$



$$\text{Local capacity of web, } F_w = (b_1 + n_2)t_w f_y / \gamma_{mo} = 409.1 \text{ kN}$$

$$\text{Bearing Force on stiffners} = 1473.8 \text{ kN}$$

$$\text{Bearing Capacity of stiffners} = 4144.9 \text{ kN}$$

Safe

$$\text{Moment of inertia of stiffners} = 5.07E+07 \text{ mm}^4$$

$$\text{Bearing area} = 2.03E+04$$

$$\text{Radius of Gyration, } r = 50.03 \text{ mm}$$

$$KL/r = 31$$

$$f_{cd} = 214 \text{ Mpa}$$

$$\text{Design compressive capacity} = 3548.042 \text{ kN}$$

Safe

6. Design of Intermediate Stiffener

Intermediate stiffener consist of angle ISA 150x115x10 at either side of the web

$$\text{width of outstand of stiffner} = 142 \text{ mm} \quad c/d = 2.106667$$

$$\text{Required moment of inertia} = 2866752 \text{ mm}^4$$

Determination of shear force at 750 mm from the left support, Live load is applied to the right of the stiffener

$$\text{Loaded length} = 26.25 \text{ m}$$

$$\text{EUDL for SF} = 2847 \text{ kN}$$

$$\text{CDA} = 0.398$$

$$\text{Total Live Load} = 3980.1 \text{ kN}$$

$$\text{Reaction @ } R_A = 2275.6 \text{ kN}$$

$$\text{Shear force in each plate girder at 0.68 m from the support} = 1128 \text{ kN}$$

$$\text{Factored shear force} = 1692 \text{ kN}$$

Buckling Check

$$V_{cr} = 1108.0 \text{ kN}$$

$$\text{Force on the stiffeners, } F_q = 685.2 \text{ kN}$$

$$\text{Moment of Inertia of stiffeners, } I_s = 25357934 \text{ mm}^4$$

$$\text{Area of cross section of stiffeners, } A_s = 5132 \text{ mm}^2$$

$$\text{radius of gyration, } r = 70.29325 \text{ mm}$$

$$KL/r = 22.02772$$

$$f_{cd} = 221 \text{ MPa}$$

$$\text{Design compression Capacity} = 927.9589091 \text{ kN}$$

Safe

7. Design of Bottom and Top lateral bracing

Since the span is less than 20 m,

Basic wind speed (as per IS 875 part 3) =	0.83	kN/m ²	
Wind acting in the wind ward direction =	2.07	kN/m	
Wind acting in the leeward direction =	0.52	kN/m	<0.5d
Wind acting on the train =	2.91	kN/m	<1.5d
racking force =	5.88	kN/m	
Total lateral force=	11.37	kN/m	

Lateral load bottom intermediate panel pt = 7.1 kN

$$\theta = 50$$

End reaction from the leeward Girder =	35.50	kN
Length of lateral =	3.53	m

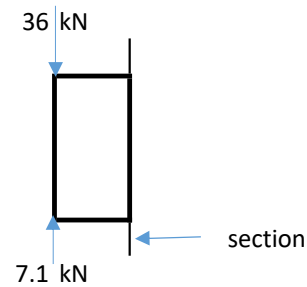
Considering a horizontal section cutting the end lateral as shown Fig.2. For Calculating the forces in the lateral, the part of Left section is considered

Let S be the force in the lateral

$$S \cos 50.0 = 25.30 \text{ kN}$$

$$S = 39.36 \text{ kN}$$

Factored force in lateral = 59.03967 kN



ISA 150 x 115 x 10 Provide as bottom Lateral Bracing

Area of cross section =	2552	mm ²	b1 =	150	mm
r _{xx} =	47.4	mm	b2 =	115	mm
r _{yy} =	33.9	mm	t =	10	mm
r _{uu} =	52.9	mm			
r _{vv} =	24.4	mm			

Since the end of lateral are rivitted

$$K1 = 0.7$$

$$K2 = 0.6$$

$$K3 = 5$$

$$f_{cd} = 67.9 \text{ N/mm}^2$$

Design Load Carrying capacity = 156.05 kN

Safe

Lateral load top intermediate panel pt = 10.2 kN

$$\theta = 50$$

End reaction from the leeward Girder =	50.10	kN
Length of lateral =	3.20	m

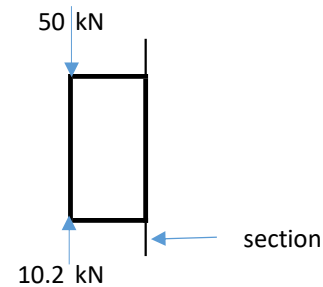
Considering a horizontal section cutting the end lateral as shown Fig.2. For Calculating the forces in the lateral, the part of Left section is considered

Let S be the force in the lateral

$$S \cos 50.0 = 50.10 \text{ kN}$$

$$S = 77.94 \text{ kN}$$

$$\text{Factored force in lateral} = 116.9125 \text{ kN}$$



ISA 150 x 115 x 10 Provide as top Lateral Bracing

$$\text{Area of cross section} = 2552 \text{ mm}^2$$

$$r_{xx} = 47.4 \text{ mm}$$

$$r_{yy} = 33.9 \text{ mm}$$

$$r_{uu} = 52.9 \text{ mm}$$

$$r_{vv} = 24.4 \text{ mm}$$

$$b1 = 150 \text{ mm}$$

$$b2 = 115 \text{ mm}$$

$$t = 10 \text{ mm}$$

Since the end of lateral are rivitted

$$K1 = 0.7$$

$$K2 = 0.6$$

$$K3 = 5$$

$$f_{cd} = 67.9 \text{ N/mm}^2$$

$$\text{Design Load Carrying capacity} = 156.05 \text{ kN}$$

Safe

7. End cross frame

$$\text{Spacing between the girder, } S = 0.52 \text{ m}$$

$$\text{Height of web} = 2.212 \text{ m}$$

$$\text{Angle between horizontal and diagonal members} = 63$$

$$\text{Lateral force acting on the end of cross frame} = 25.30 \text{ kN}$$

Due to thos lateral force one diagonal member is subjected to compression and other diagonal member is subjected to tension, If S is the force in diagonal member,

$$2S \cos (63) = 25.30$$

$$s = 27.86397 \text{ kN}$$

a) Design of diagonal member

$$\text{Factored force in the diagonal member} = 41.80 \text{ kN}$$

$$\text{Length of diagonal member} = 3.02 \text{ m}$$

ISA 150 x115x 10 Provide as Diagonal member

$$\text{Area of cross section} = 2552 \text{ mm}^2$$

$$r_{xx} = 47.4 \text{ mm}$$

$$r_{yy} = 33.9 \text{ mm}$$

$$r_{uu} = 52.9 \text{ mm}$$

$$r_{vv} = 24.4 \text{ mm}$$

$$b1 = 150 \text{ mm}$$

$$b2 = 115 \text{ mm}$$

$$t = 10 \text{ mm}$$

Since the end of lateral are rivitted

$$K1 = 0.7$$

$$K2 = 0.6$$

$$K3 = 5$$

$$f_{cd} = 78.9 \text{ N/mm}^2$$

$$\text{Design Load carrying capacity} = 181.242 \text{ kN}$$

Safe

Check for tension

$$\text{Design strength of the angle due to yielding, } T_{dg} = A_g f_y / \gamma_{m0} = 522 \text{ kN}$$

$$\text{Design strength of the angle due to rupture, } T_{du} = 612.6 \text{ kN}$$

$$\text{Ultimate Design strength} = 522 \text{ kN}$$

Safe

a) Design of horizontal member

both horizontal member are subjected to compression

$$\text{Force in horizontal member} = S \cos (63^\circ) = 12.7 \text{ kN}$$

$$\text{Factored force in the horizontal member} = 18.975 \text{ kN}$$

$$\text{Length of member} = 0.52 \text{ m}$$

ISA150x115x10 Provide as Diagonal member

$$\text{Area of cross section} = 2552 \text{ mm}^2$$

$$b_1 = 150 \text{ mm}$$

$$r_{xx} = 47.4 \text{ mm}$$

$$b_2 = 115 \text{ mm}$$

$$r_{yy} = 33.9 \text{ mm}$$

$$t = 10 \text{ mm}$$

$$r_{uu} = 52.9 \text{ mm}$$

$$r_{vv} = 24.4 \text{ mm}$$

Since the end of lateral are rivitted

$$K_1 = 0.7$$

$$K_2 = 0.6$$

$$K_3 = 5$$

$$f_{cd} = 133.6 \text{ N/mm}^2$$

$$\text{Design Load carrying capacity} = 306.846 \text{ kN}$$

Safe