

REPORT
THIRD PARTY AUDITING OF RAILWAY BRIDGES (PHASE I)

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

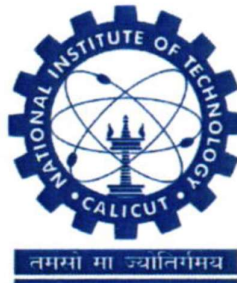
**Br. No. 1087 DN (3×18.29 m) Steel Girder at km.
736/100 – 200 between Thalassery & Ettakot Stations**

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

2. SCOPE AND METHODOLOGY

Railway bridges, which are known for their permanent resistance to heavy movable loads, are exposed to large stress 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 main 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 along with field measurements were carried out on the bridge on 20 – 12 – 2019 (Friday) at 11.00 am. Officials from Kannur Railway section were also present during the inspection. Photographs of the bridge showing various elements are shown in Fig. 1 and Fig. 2. The bridge is a deck type riveted steel plate girder with three spans of 18.29 m, supported on Random Rubble (RR) masonry piers with masonry well foundation. Cement Concrete (CC) bed - blocks are seen on top of the RR masonry pier. One of the piers is seen to be strengthened by confining it with steel rails all around at four locations as shown in Fig.3. It was noted that the abutments were also of RR masonry. The bearings on the piers and abutments are of centralized steel sliding type. It was reported that the speed of the trains is restricted to 30 km/hr as the piers were showing signs of distress (in mortar joints). Vegetation growth is seen on pier as shown in Fig.3.



Figure 1. View of Br.1087 Plate girder bridge



Figure 2. View of the bridge from inside



Figure 3. Pier strengthened using rail steel confinement

3.2 Field measurements and condition assessment

The geometry of the bridge is obtained by taking measurements in the field. Fig. 4 shows the details of geometry and typical member details. Based on a careful examination of the bridge, the results of observations are tabulated in the following Table 1. The Table shows various elements of the bridge, defect type, follow up actions required and remarks. Labels are assigned to various types of defects as given in Table 2.

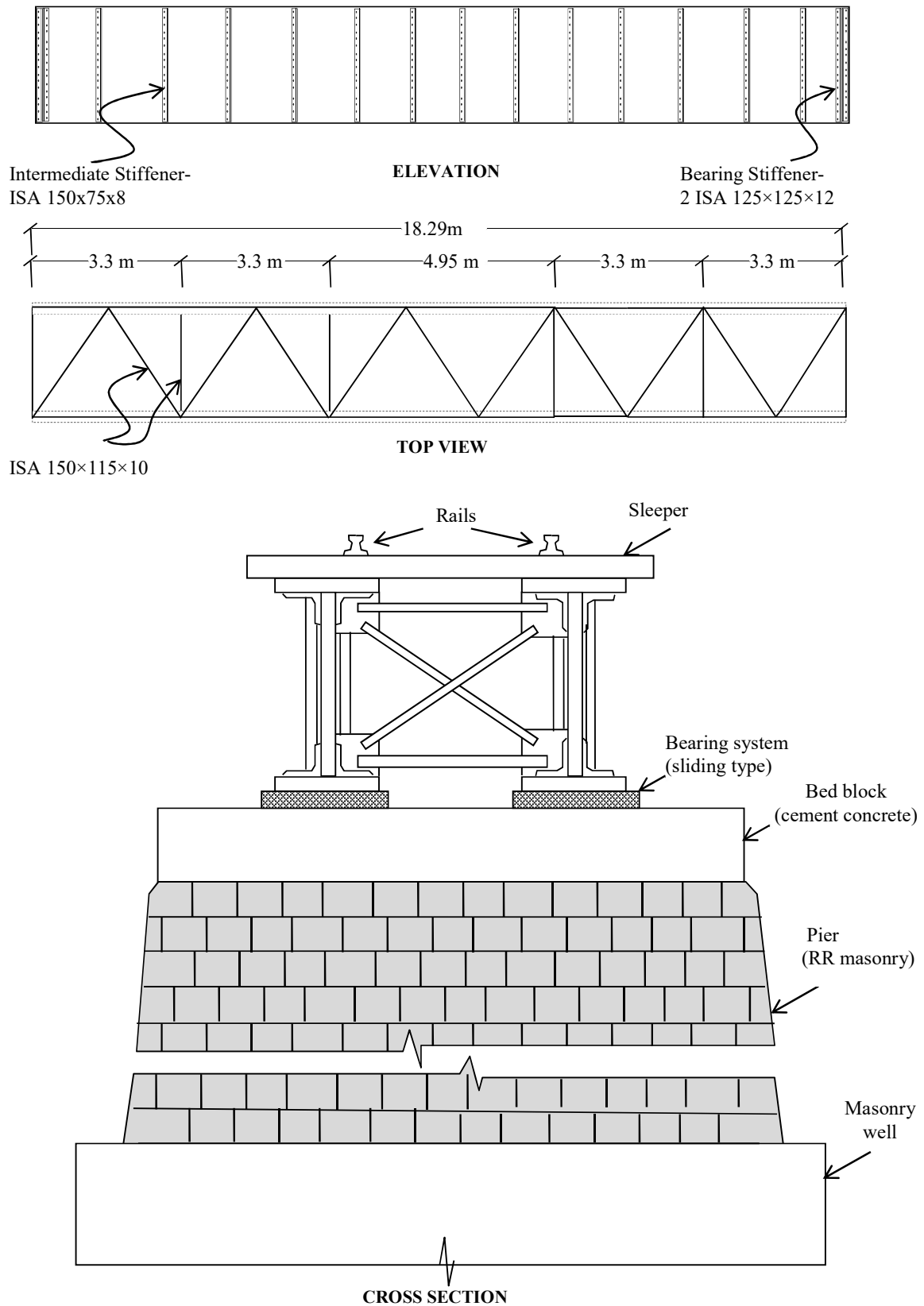


Figure 4. Geometrical details of the bridge (not to scale)

Table 1: Condition assessment of bridge

Element	Defect Type (Ref. Table 2 for details)	Follow-up actions required			Remarks
		DI	Re	De	
Abutments	00	-	-	-	
Piers	13,17		✓	-	
Bed-blocks	00	-	-	-	
Girder bearings	00	-	-	-	
Flange of Girder	02, 03, 08	-	✓	-	
Web of girder	03	-	✓	-	
Bearing stiffeners	03	-	✓	-	
Intermediate stiffeners	03	-	✓	-	
Vertical bracings	03, 08	-	✓	-	
Horizontal bracings	03,08	-	✓	-	

* Legend: DI – Detailed Investigation; Re – Repair; De – Demolition

Table 2: Different defect types and their designation

Defect Type	Label	Defect Type	Label
Dampness	01	Lack of redundancy	10
Loss of material due to corrosion	02	Foundation settlement	11
Staining due to corrosion	03	Percolation of water through RR masonry	12
Buckling of plate	04	Vegetation growth	13
Tearing of steel	05	Cracks in stone masonry	14
Excessive deflection	06	Floor settlement	15
Excessive deformation	07	Signs of overloading	16
Connection failure	08	Distress in masonry (in mortar joints)	17
Cracks in concrete	09	No visible defect	00

3.2.1 Extent of corrosion

The corrosion process of steel is affected by many factors, such as the kind of steel and surface protection; environmental impact; and the presence of pollutants, cracks, and stresses. The loss of material may result in a reduction in the cross-sectional area and thereby increase the stress level for a given load or increase the stress range for cyclic loads [5]. Besides, it can lead to a

reduction in the buckling resistance of elements or the initiation of fractures. In general, the loss of cross-sectional area results in a decrease in geometric parameters such as the moment of inertia and radius of gyration.

In the present bridge, severe corrosion has been noticed. Especially at the joints, it is seen that the gusset plate, flanges of girder and end cross bracings were corroded severely (Fig. 5 and Fig.9). Also, rivet heads at several locations were damaged. For many of the rivets, head is observed to be lost (Fig.6 and Fig.7), which is a matter of concern. The top flange of the girder has corroded considerably at several locations especially below the sleepers as shown in Fig.8. This is due to the wastewater from the toilets of the train. Flanges are noted to have become very thin at many locations due to corrosion. Material loss due to corrosion has been noticed in the horizontal bracings also. Corrosion in the web of the girder and intermediate stiffeners were minimal.

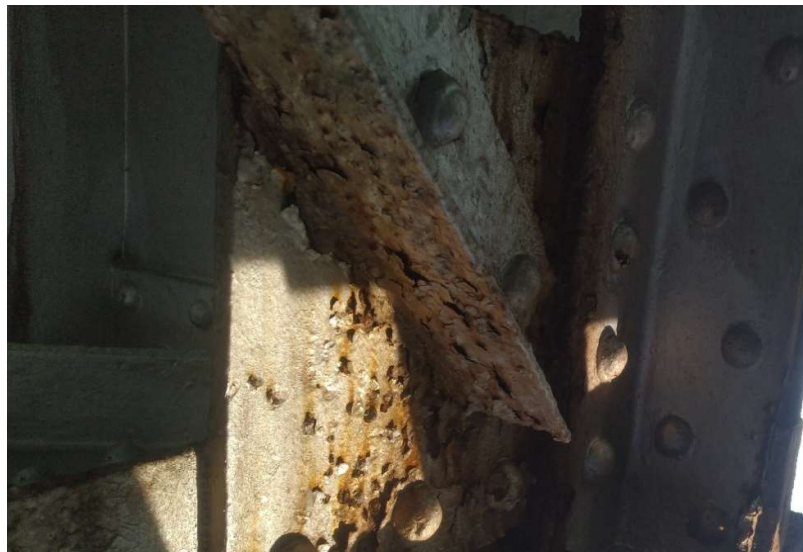


Figure 5. Corrosion and material loss in the gusset plate, cross-bracing elements



Figure 6. Corrosion of bottom flange over base plate



Figure 7. Severe corrosion in rivets



Figure 8. Corrosion in top flange under rail sleepers

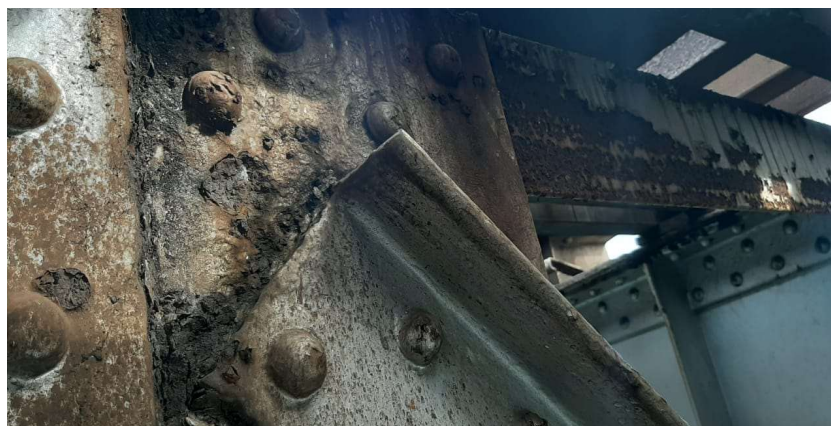


Figure 9. Corrosion in end cross bracing

4. MODELING AND ANALYSIS

4.1 Description of loading

It is understood 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. Also racking load and wind load is calculated as per clause 2.9.2.

The detailed calculation of loads and analysis 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 bridge.

4.2 Demand and Capacity

Here the integrity and safety of the bridge are assessed by the parameter ‘Demand to Capacity Ratio’ (DCR) of various elements. The capacity of various elements of the superstructure is evaluated considering the effective section incorporating the material loss due to corrosion, judged conservatively during the condition assessment. Though the joints were affected due to the corrosion, while calculating the DCRs of various elements, joints were assumed intact. 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	Girder/Moment	5420	5360	1.01
2	Girder/Shear	1296	1862	0.70
3	End panel/ Shear	530	883	0.60
4	End panel/ Moment	128	158	0.81
5	Bearing stiffener/ bearing	1023	3429	0.30
6	Bearing stiffener/ buckling	1364	2982	0.48
7	Intermediate stiffener/ buckling	580	1203	0.59
8	Top lateral bracing/ Axial force	132	211	0.63
9	Cross frame- diagonal member/compression	69	104	0.66
10	Cross frame- diagonal member/Tension	69	389	0.18
11	Cross frame- horizontal member/Compression	49	248	0.2

Important notes

- i. *The effect of material loss due to corrosion is conservatively accounted for as a reduction of 10% in the evaluation of the capacity of members except for moment capacity in which a reduction of 20% considered in the flange due to excessive corrosion. The estimation of material loss due to corrosion at the joints was not as practical as in the case of members. Hence this is not considered in the calculation of DCR.*
- ii. *The demand includes load factor and capacity includes partial safety factors*

5. COMMENTS AND INFERENCE

Table 3 shows that all the elements of bridge superstructure have a DCR value except the **flange of the plate girder, which has a DCR value of 1.01.**

In view of the above, it is inferred that the condition of the superstructure is not satisfactory. The situation would have been more critical if the actual status (concerning corrosion) of the joints were considered.

It has to be noted that the strength reduction due to fatigue is not considered here. If the maintenance is not done regularly and systematically, there is a possibility of further distress and strength reduction due to corrosion and fatigue. **Hence it is recommended to replace the corroded rivets, damaged members and connections at the earliest with new elements as per the existing design. As the flange plate and flange angle are having a DCR value of 1.01, it is suggested to replace the same with new elements as per the existing design.** Conclusions on the stability and safety of the bridge can be made only after conducting phase II of the audit.

It is informed that speed restrictions are imposed on this bridge. At present, adequate data is not available to conduct studies and to make recommendations on restricted speed. Hence this aspect is kept out of the scope of the present study.

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 in the life span of the bridge.

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.

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 = 18.29 m

1. Dead load Calculation

Dead load Calculation

1) Total dead Load of the bridge = 276.6 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 = 53.41 kN
 Total Dead Load = 330.02 kN

Depth of Web = 1814 mm
 Thickness of web = 10 mm
 Thickness of top plate = 18 mm
 Width of flange = 410 mm
 flange angle = 150x150x18
 Wt of Girder = 15.12349 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 = 1913 kN
 EUDL for SF = 2113 kN
 CDA = 0.48

Total Load for BM = 3161.3 kN

Maximum BMu in each Girder = 5420.6 kNm

Total Load for SF in each Girder = 3457.3 kN

Maximum SF in each Girder = 1296.5 kNm

2. Section Classification and Capacity

bi = 310 mm

be = 50 mm

$b/t_f = 17.222222$ < 29.3 ξ Flanges are plastic
 $d/t_w = 181.4$ > 126 ξ web Semi compact
 $\beta = 1$
 $Z_p = 2.95E+07$

$b/t_f = 2.8$

< 8.4 ξ Flanges are plastic

Design moment carrying of girder

$M_d = \beta_b Z_p f_y / \gamma_{m0} = 5360$ kNm

Revise Section

3. End Panel Design (Tension Field Method)

Spacing of Transverse vertical stiffeners = 680 mm $f_{yw} = 250$ MPa

$\zeta_b = 38.61$ MPa

$f_v = 193.8$ MPa

For calculating maximum BM at the section 680 mm from left support, Live load should be considered on the right of the section

Loaded Length = 17.61 m

EUDL for BM = 1862 kN

CDA = 0.4885

Total Live Load = 2771.6 kN

Reaction @ $R_A = 1499.3$ kN

M_u in each plate girder at 2.8m from left end = 761.5 kNm

Axial force in the Flange due to BM, $N_f = 415.7$ kN

$V_d = 1862.1$ kN

> V safe

Check for shear capacity of end panel

$$\text{shear capacity of longitudinal section of the end panel} = A_v f_y / \sqrt{3} = 883 \text{ kN}$$

$$V_{cr} = 700.4 \text{ kN}$$

$$H_q = 2801.1 \text{ kN}$$

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

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}} = 0.38$$

$$\text{hence } H_q = 1059.7 \text{ kN}$$

$$R_{tf} = 529.9 \text{ kN}$$

< V_p Safe

Check for moment capacity of end panel

$$M_{tf} = H_q d / 10 = 192.2 \text{ kNm}$$

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

$$2/3 M_{tf} = 128.2 \text{ kNm}$$

$$M_q = z f_y / x \gamma_{mo} = 157.6 \text{ kNm}$$

Safe

5. Design of Bearing Stiffener at the end

Bearing stiffener consist of double angle ISA 125x125x12 at either side of the web

$$\text{Force on end post due to moment, } 2/3M_{tf} = 67.8 \text{ kN}$$

$$\text{Desing force at the end post} = 1364.3 \text{ kN}$$

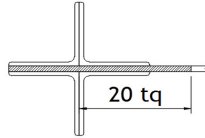
$$\text{Area required} = 4802.3 \text{ mm}^2$$

$$\text{Bearing Area of stiffener provided} = 16767 \text{ mm}^2$$

Safe

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

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



$$\text{Local capacity of web, } F_w = (b_1 + n_2)t_w f_y / \gamma_{mo} = 340.9 \text{ kN}$$

$$\text{Bearing Force on stiffners} = 1023.4 \text{ kN}$$

$$\text{Bearing Capacity of stiffners} = 3429.6 \text{ kN}$$

Safe

$$\text{Moment of inertia of stiffners} = 4.74\text{E}+07 \text{ mm}^4$$

$$\text{Bearing area} = 1.91\text{E}+04$$

$$\text{Radius of Gyration, } r = 49.79 \text{ mm}$$

$$KL/r = 25.5$$

$$f_{cd} = 217.4 \text{ Mpa}$$

$$\text{Design compressive capacity} = 2982.392 \text{ kN}$$

Safe

6. Design of Intermediate Stiffener

Intermediate stiffener consist of angle ISA 150x75x8 at either side of the web

$$\text{width of outstand of stiffner} = 142 \text{ mm} \quad c/d = 1.727619$$

$$\text{Required moment of inertia} = 1360500 \text{ mm}^4$$

Determination of shear force at 680 mm from the left support, Live load is applied to the right of the stiffener

$$\text{Loaded length} = 17.61 \text{ m}$$

$$\text{EUDL for SF} = 2053 \text{ kN}$$

$$\text{CDA} = 0.491$$

$$\text{Total Live Load} = 3061.0 \text{ kN}$$

$$\text{Reaction @ } R_A = 1638.6 \text{ kN}$$

$$\text{Shear force in each plate girder at 0.68 m from the support} = 813 \text{ kN}$$

$$\text{Factored shear force} = 1220 \text{ kN}$$

Buckling Check

$$V_{cr} = 700.4 \text{ kN}$$

$$\text{Force on the stiffeners, } F_q = 583.0 \text{ kN}$$

$$\text{Moment of Inertia of stiffeners, } I_s = 17678365 \text{ mm}^4$$

$$\text{Area of cross section of stiffeners, } A_s = 7503 \text{ mm}^2$$

$$\text{radius of gyration, } r = 48.54043 \text{ mm}$$

$$KL/r = 26.15964$$

$$f_{cd} = 196 \text{ MPa}$$

$$\text{Design compression Capacity} = 1203.208364 \text{ kN}$$

Safe

7. Design of Top lateral bracing

Since the span is less than 20 m,

Entire span is divided into	5.5	Panels	
Length of each panel =	3.3	m	
Spacing between the girder, S =	1.85	m	>0.5d <1.5d

Total lateral force = 8.82 kN/m

(Since the span is less than 20 m)

Lateral load at each intermediate panel pt = 29.1 kN

Lateral load at each panel pt = 14.6 kN

$\tan \theta = 0.89$

$\theta = 41.7$

End reaction from the leeward Girder = 80.04 kN

Length of lateral = 2.48 m

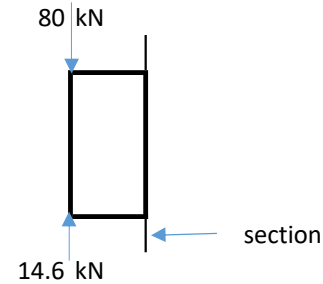
Considering a horizontal section cutting the end lateral as shown Fig.2. For Calculating the forces in the lateral, the part of Left section is considered

Let S be the force in the lateral

$S \cos 41.7 = 65.49$ kN

$S = 87.75$ kN

Factored force in lateral = 131.6272 kN



ISA 150 x 115 x 10 Provide as Lateral Bracing

Area of cross section = 2552 mm²

$r_{xx} = 47.4$ mm

$r_{yy} = 33.9$ mm

$r_{uu} = 52.9$ mm

$r_{vv} = 24.4$ mm

$b_1 = 150$ mm

$b_2 = 115$ mm

$t = 10$ mm

Since the end of lateral are rivitted

$K_1 = 0.7$

$K_2 = 0.6$

$K_3 = 5$

$f_{cd} = 92.0$ N/mm²

Design Load Carrying capacity = 211.28 kN

Safe

7. End cross frame

Spacing between the girder, $S = 1.85$ m
Height of web = 1.814 m

Angle between horizontal and diagonal members = 44.43711

Lateral force acting on the end of cross frame = 65.49 kN

Due to this 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,

$$2S \cos (44.43710585) = 65.49$$
$$S = 45.85902 \text{ kN}$$

a) Design of diagonal member

Factored force in the diagonal member = 68.79 kN
Length of diagonal member = 2.590964 m

ISA 125 x75x 10 Provide as Diagonal member

Area of cross section =	1902	mm ²	b1 =	125	mm
$r_{xx} =$	40	mm	b2 =	75	mm
$r_{yy} =$	20.9	mm	t =	10	mm
$r_{uu} =$	42.1	mm			
$r_{vv} =$	16.1	mm			

Since the end of lateral are rivetted

$K1 = 0.7$
 $K2 = 0.6$
 $K3 = 5$
 $f_{cd} = 61.3$ N/mm²

Design Load carrying capacity = 104.971 kN

Safe

Check for tension

Design strength of the angle due to yielding, $T_{dg} = A_g f_y / \gamma_{m0} = 389$ kN

Design strength of the angle due to rupture, $T_{du} = 504.3$ kN

Ultimate Design strength = 389 kN

Safe

a) Design of horizontal member

both horizontal member are subjected to compression

Force in horizontal member = $S \cos (44.43711) = 32.7$ kN

Factored force in the horizontal member = 49.11638 kN

Length of member = 1.85 m

ISA150x115x10 Provide as Diagonal member

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

$$K1 = 0.7$$

$$K2 = 0.6$$

$$K3 = 5$$

$$f_{cd} = 108.0 \text{ N/mm}^2$$

$$\text{Design Load carrying capacity} = 248.057 \text{ kN}$$

Safe