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SEISMIC ANALYSIS STEEL TRUSS BRIDGE USING SPLICES CONNECTION

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Abstract

The aim of the research project is to resist the seismic force/ vibration force in railway steel truss bridges using splice connection. Using the Warren type of railway truss bridges Analysis and designed by Indian standard railway code (IRC) and IS 800 -2007. The connection of the railway truss bridge is bolted with splice connection. Same cross sectional area has been carried for both theoretical and experimental investigation. Observed from the results, the splice connection has high load carrying capacity, low deflection and high level seismic resistance.

Key words: warren truss, railway traffic, splices connection, load carrying capacity, deflection.

I. INTRODUCTION

1.1 GENERAL

The bridges are made in different types of material such as timber, steel, concrete and aluminium etc.. The wooden bridges are used for small spans, light loads and for passing bridges. The masonry bridges are also used for small spans. The masonry bridges are generally arch bridges. The reinforced cement concrete bridges are used for different spans and different site conditions. There are many advantages of structural steel over other materials as regards its strength and ductility. Steel bridges are more efficient to defend against earthquake forces and explosion loading than other bridges nowadays. developed types of steel and applying special paints, the corrosion may be reduced. **K.K. Sangle et al., (2012) reported that,** in steel structures, **the lateral displacement** at the roof level controls the seismic performance and vibration by bracing system. and also measured the displacement at roof level , the displacement value has reduced about 40% to 60%. Finally observed from the results, the diagonal bracing system has given good results and is more economical compared to the other bracing system. **Jeffrey W. et al., (2012)** reported that, the different types of guest plates are used in the railway transportation system for railway truss bridges by bolted and riveted connection. Finally observed from the results the guest plate's connection gave the best results compared to the other connection. The guest plate connection has given higher shear strength, bearing, low deflection and high load carrying capacity. Different codes have been used for the design of the steel structures such as railway truss bridges, steel structures and steel connection. The Indian railway system has followed the Association of State Highway and Transportation officials (AASHTO) provisions for design of railway truss bridges. **Nandar Elwin (2014)** reported that the retrofitting and seismic performance has been done in the railway truss bridges based on the American Association of State Highway and Transportation officials (AASHTO) provisions. The aim of the research, to increase the life of structures, reduced the corrosion level, durability, wearing and high level seismic resistance based on the AASHTO-LRFD 2007.

Tsutomu Usami (2015) reported that, the experiential Pseudo-dynamic test is performed in the existing railway steel truss bridges with bracing system. There the different types of bracing system used for the for the experimental results. Finally observed from the results the BRB ed" H -section diagonal bracing members have given the best seismic resistance and also time history analysis also measured in the truss bridge, the BRBs with H-section core members have given the best results compare than the other bracing system. Now days the cold form section has mostly used in the construction field for the reduced the dead weight of the structures and prevent the base shear of the structures. **Shah Foram Ashokbhai et al., (2017)** he reported that, the weight has reduced in the of industrial shed about 10435 Kg by cold formed sections compare with hot rolled sections. The cold formed steel structures; the weight has reduced about 32.03% compared with hot rolled sections. Finally observed from the results, the cold form section has given best economical construction when compare than the other structures. **A Jayaraman et al., (2018**) reported that comparison and behaviour of cold formed steel channel section and built up section are used same cross sectional area. Channel section is high bending strength, more loads carrying capacity, deflection is minimum and distortional buckling minimum & local buckling compare with channel built up section by same cross sectional area. The main aim of the research work to reduce the seismic force and base shear in railway truss bridge using the splice connection through bolted.

II OBJECTIVE OF THE RESEARCH

➢ Analysis and design of Steel Truss Bridge using Splice Connection.

➢ To Study about the guidelines for the design of Steel Truss Bridge according to the IS Code. Calculations and assumptions are taken from the review of past practices and also by the bridge rules, Ministry of Indian railways.

➢ To know about the design philosophy for the safe and economical design of Steel Truss Bridge.

➢ To perform a experimental Investigation of Steel Truss railway bridges.

➢ To Compare the Theoretical and Experimental of Steel Truss railway bridges Results.

III EXPERIMENTAL INVESTIGATION

3. 1 Materials

3.1.1 Steel section: A steel section has been used for experimental work ISMC 100 and ISMC 75 and also finds the young modulus and ultimate load carrying capacity of the steel section.

3.1.2 Splice plate: A solid steel plate is used to make the connections between the structural steel members.

3.1.3 Splice joint: A splice joint is a method of joining two members end to end in woodworking. The splice joint is used when the material being joined is not available in the length required. It is an alternative to other joints such as the butt joint and the scarf joint.

3.1.4 IS codes for the railway truss bridge design

Design of the steel wrought iron bridge, road traffic and design of pedestrian using Indian Railway Standard Code of Practice (IRRC)

Design of bridge substructures and superstructures using Indian Railway Standards.

Specification for Fabrication and Erection of Steel Girder Bridges and Locomotives using Turn-Tables and Bridge Rules.

IV. EXPERIMENTAL PROCEDURE

Consider the steel channel sections ISMC 100 and ISMC 75. A 1m length channel of ISMC 100 and ISMC 75 is taken respectively. For the design of splice connections the design load to be carried by the section and the length of the section is reduced. The corresponding sections are reduced to 0.5m length and their bolted connections are made with splice plates. The testing is carried out in the computerized Universal Testing Machine of capacity 400 k N. The result is taken as the load carrying capacity and deflection of the steel sections and compared between the sections with and without splice connections.

V RESULT AND DISCUSSION

5.1 Theoretical investigation of analysis and design of Warren Truss Bridge

The typical plan, section and elevation warren truss bridge is shown below.1 The specifications are,

Span of the truss = 49 m
Inclination of the diagonals = 60°

Height of the truss bridge 6.125 m Number of panels = 7

Panel length = 7 m

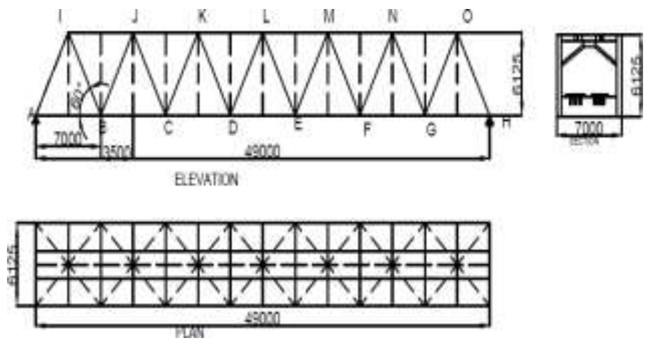


Fig.1 Plan, section and elevation of warren truss bridge

5.1.1 Influence Line Diagram For The Top Chord Members

Member force for the top chord members

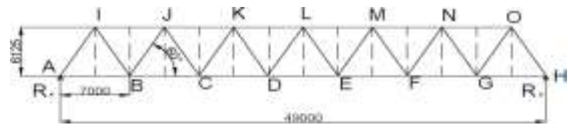


Fig: 1.1 Warren truss

To find the member force ij:

Applying unit load at B, finding the support reactions,
RA×49-1×42 = 0

RA = 0.857 kN, RH = 0.142 kN

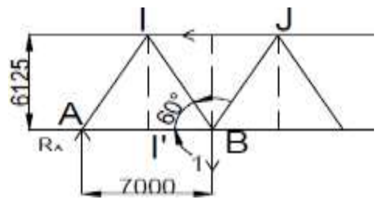


Fig: 1.1.1 Unit load at B

To find FIJ about II'

R_A×3.5+FIJ×6.125=0

F_{IJ} = - 0.479 KN

To Find The Member Force Jk:

Applying unit load at C, finding the support reactions,
RA×49-1×35 = 0,

RA = 0.719 kN, RH = 0.285 kN

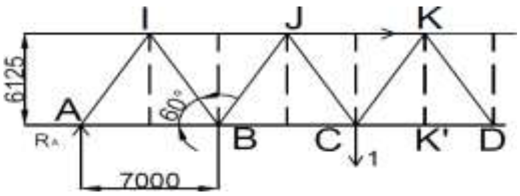


Fig: 1.1.2 Unit load at C

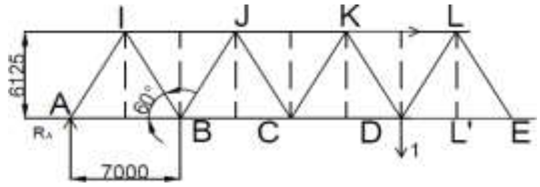
To find FJK about KK'

RA×17.5-1×3.5+FJK×6.125 = 0

8.995+FJK×6.125 = 0 FJK = -1.46 kN

To find the member force KL

Applying unit load at D, finding support reactions, RA×49-1×28 = 0



RA = 0.571 kN, RH = 0.42 kN

Fig: 1.1.3 Unit load at D

To find Fkl about Ll'

RA×24.5-1×3.5+FKL×6.125 = 0 FKL = -1.713 kN

To Find the Member Force LM

Applying unit load at D, finding support reactions, RA× 49 − 1×28 = 0

RA = 0.571 kN, RH = 0.428 kN

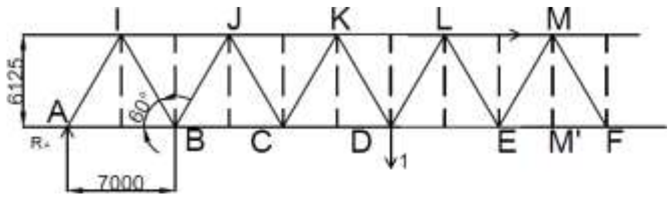


Fig: 1.1.4 Unit load at D

To find FLM about MM'

RA×31.5-1×10.5+FLM×6.125 = 0 = -1.22 kN

To find the member force MN

Applying unit load at E, finding the support reactions RA×49-1×21 = 0

RA = 0.428 kN, RH = 0.571 kN

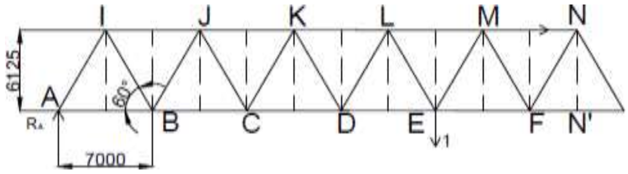


Fig: 1.1.5 Unit load at E

To find FMN about NN'

RA×38.5-1×10.5+FMN×6.125 = 0 FMN = -0.976 kN

To find the member force NO

Applying unit load at F, finding support reactions RA×49-1×14 = 0

RA = 0.285 kN, RH = 0.714 kN

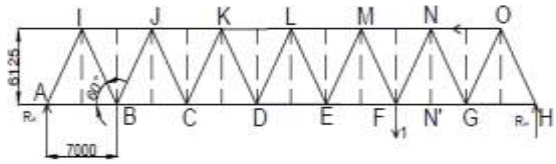


Fig: 1.1.6 Unit load at F

To find FNO about OO'

RH×10.5+FNO×6.125 = 0 FNO = 1.22kN

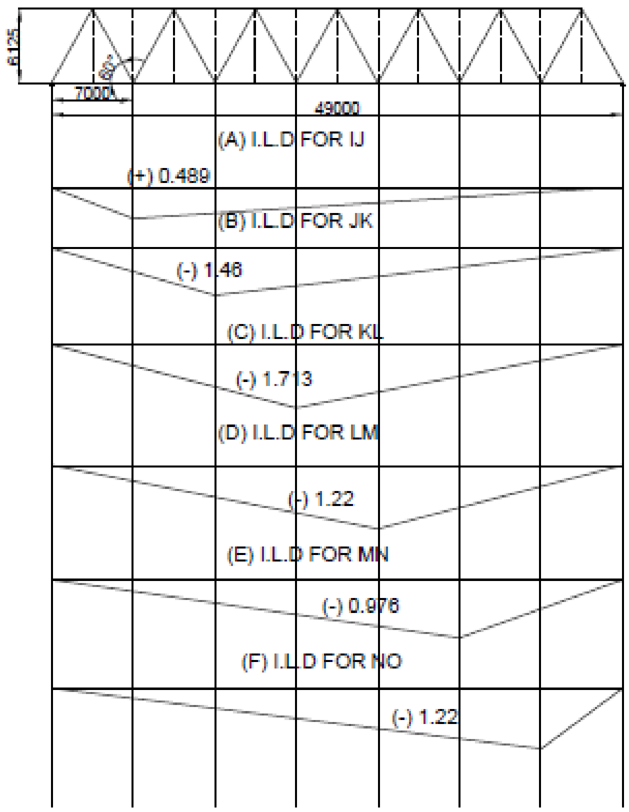


Fig.1.1.7 Influence line diagram for the top chord members

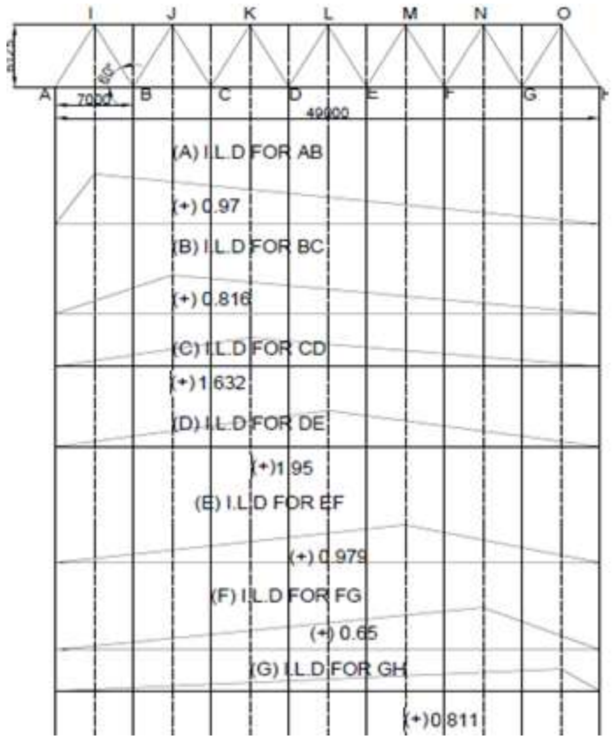


Fig.1.1.8 Influence line diagram for the bottom chord members

5.2 Calculation of Forces in the members

5. 2. 1Forces In The Top Chord Members Due To Dead Load: (Compression)

Calculated dead load = 11.668 kN/m

Force in the member IJ = (1/2×49×0.479×11.668) = 136.929 kN

Force in the member JK = (1/2×49×1.46×11.668) = 417.36 kN

Force in the member KL = (1/2×49×1.733 ×11.668) = 495.40 kN

Force in the member LM = (1/2×49×1.22×11.668) = 348.756 kN

Force in the member MN = (1/2×49×0.976×11.668) = 279.00 kN

Force in the member NO = (1/2×49×1.22×11.668) = 348.75 KN

5 .2. 2 Forces in the bottom chord members due to dead load:

Force in the member AB = (1/2×49×0.97×11.668) = 277 kN

Force in the member BC = (1/2×49×0.816×11.668) =233.26 kN

Force in the member CD = (1/2×49×1.632×11.668) = 466.53 kN

Force in the member DE = (1/2×49×1.950×11.668) = 557.43kN

Force in the member EF = (1/2×49×0.979×11.668) = 279.86 kN

Force in the member FG = (1/2×49×0.65×11.668) = 185.81kN

Force in the member GH = (1/2×49×0.811×11.668) = 231.83 kN

5.2.3 Forces in the inclined members due to dead load:

Force in the member AI = (1/2×3.5×1.94-1/2×45.5×1.15) = -22.76 kN

Force in the member BI = (-1/2×3.6×1.94+1/2×45.4×1.15) = 22.615 kN

Force in te member BJ = (1/2×1.632×10.5-1/2×1.15×38.5) = -13.569 kN

Force in the mmber CJ = (-1/2×1.632×10.7+1/2×1.15×38.3) = 13.292 kN

Force in the memer CK = (1/2×3.26×17.5-1/2×1.15×31.5) = 10.41 kN

Force in the member DK = (-1/2×18.7×3.26+1/2×31.3×1.15) = -12.48 kN

Force in the member DL = (1/2×24.9×3.9-1/2×24.1×1.15) = 34.6 kN
Force in the member EL = (1/2×27×3.9+1/2×22×1.15) = -40 kN

Force in the member EM = (1/2×31.5×1.94-1/2×21×1.15) = 18.485 Kn

Force in the member FM = (-1/2×33×1.94+1/2×16×1.15) = -22.81 kN

Force in the member FN = (1/2×38.5×1.3-1/2×10.5×1.15) = 18.98 kN

Force in the member GN = (-1/2×40.5×1.3+1/2v8.5×1.15) = -21.43 kN

Force in the member GO = (1/2×45.5×1.622-1/2×3.5×1.15) = 34.88 kN

Force in the member HO = (-1/2×47.5×1.622+1/2×1.5×1.15) = -37.65 kN

5.3 Forces in the Members Due to Live load and Impact Load:

5.3.1 Top chord members: member JK:

Loaded length = 49 m, Impact factor = 0.317

From bridge rules, for metre gauge, 49 m loaded length Live load + impact load per girder 1556.317 kN

Forces in the member due to live load and impact load = -1/2×1.46×49× (1556.317/49)

= -1136.11 kN (C)

Member KL:

Loaded length = 49 m, Impact factor = 0.317

Forces in the member due to live load and impact load = -1/2×1.713×49× (1556.317/49)

= -1332.98 kN (C)

TABLE 1 Calculated design forces in the members are as follows:

MEMBER	FORCES IN THE MEMBERS				DESIGN FORCES	
	D.L		(L.L + I.L)		D.L+L.L+I.L	
	COMP (kN)	TEN (kN)	COMP (kN)	TEN (kN)	COMP (kN)	TEN (kN)
IJ	136.92	-	570.39	-	707.31	-
JK	417.36	-	1136.11	-	1553.47	-
KL	495.40	-	1332.98	-	1828.38	-
LM	348.76	-	949.4	-	1298.16	-
MN	279.00	-	759.5	-	1038.5	-
NO	348.75	-	949.4	-	1298.15	-
AB	-	277.0	-	754.81	-	1031.81
BC	-	233.26	-	634.97	-	868.23
CD	-	466.53	-	1269.95	-	1737.28
DE	-	557.43	-	1517.40	-	2094.83
EF	-	279.86	-	761.81	-	1041.67
FG	-	185.81	-	505.80	-	691.61
GH	-	231.83	-	631.08	-	862.91
AI	22.76	-	1191.97	507.02	1214.73	484.26
BI	-	22.615	513.23	1191.97	490.62	1214.59
BJ	13.569	-	1077.5	730.56	1091.07	716.99
CJ	-	13.292	738.19	1074.50	724.9	1087.8
CK	-	10.41	944.13	1899.5	933.72	1909.9
DK	12.49	-	1959.60	919.84	1972.1	907.35
DL	-	34.6	792.99	2749.03	758.4	2783.63
EL	40	-	2900.23	746.6	2940.23	706.6
EM	-	18.485	669.7	1592.70	651.22	1611.2
FM	22.81	-	1642.8	642.4	1665.6	619.6
FN	-	18.98	514.8	1218.03	495.82	1237.01
GN	21.43	-	1257.3	312.26	1278.73	290.83
GO	-	34.88	522.9	1681.20	488.02	1716.08
HO	37.65	-	1724.02	191.6	1761.67	153.95

5.4 Experimental investigation of railway steel truss bridges

The load carrying capacity and deflections are presented in the Table 2.

Table. 2: Experimental details of a specimen with and without splice connections

SECTION DETAIL	SPLICE DETAIL	LENGTH			
		1M		0.5 M	
		LOAD(k N)	DEF (mm)	LOAD (k N)	DEF (mm)
ISMC 100	WITHO UT	110	6.8	102	5.4
ISMC100	WITH	140	4.9	122	4
ISMC75	WITHO UT	80	3.7	94	2.8
ISMC75	WITH	100	3	112	2.5

5.4.1- Load carrying capacity in k N for 1m length without splices

The test is carried out by taking the load carrying capacity of the steel channels ISMC 100 and ISMC 75 of 1m length. The obtained results shows that the load carrying capacity of the section ISMC 100 is greater than the ISMC 75 section by 27 %.(Fig 2)

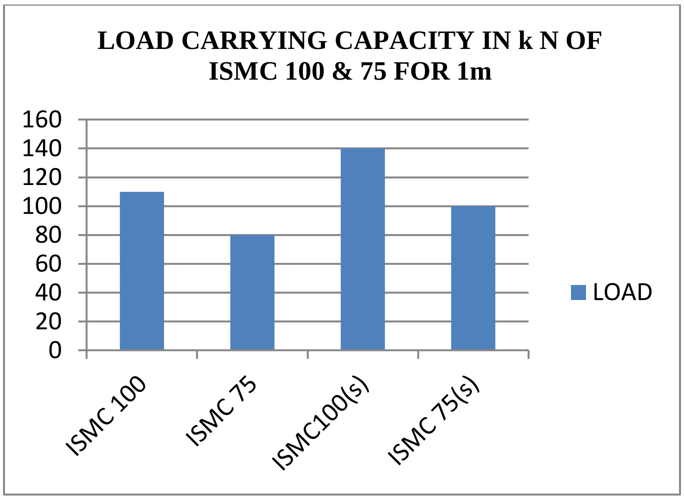


Figure 2: Load carrying capacity in k N of the ISMC 100 and ISMC 75 sections for 1m length

5.4.2. Load carrying capacity in k N for 0.5m length without splices

The test is carried out by taking the load carrying capacity of the steel channels ISMC 100 and ISMC 75 of 0.5m length. The obtained results shows that the load carrying capacity of the section ISMC 100 is greater than the ISMC 75 section by 8 %.(Fig 3)

5.4.3. Load carrying capacity in k N for 1m length with splices

The test is carried out by taking the load carrying capacity of the steel channels ISMC 100 and ISMC 75 with splice connection. The obtained results shows that the load carrying capacity of the section ISMC 100 is greater than the ISMC 75 section by 30 %.(Fig 2)

5.4.4. Load carrying capacity in k N for 0.5m length without splices

The test is carried out by taking the load carrying capacity of the steel channels ISMC 100 and ISMC 75 with splice connection. The obtained results shows that the load carrying capacity of the section ISMC 100 is greater than the ISMC 75 section by 11 %.(Fig 3)

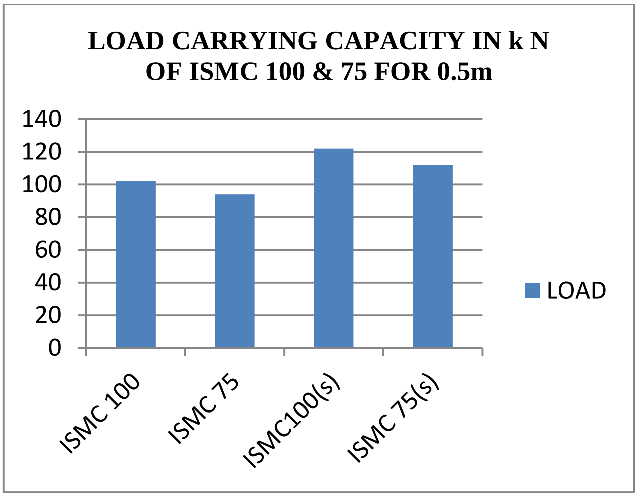


Figure 3: Load carrying capacity in k N of the ISMC 100 and ISMC 75 sections for 0.5m

length

5.4.5. Deflection in mm for 1m length without splices

The test is carried out by taking the deflection of the steel channels ISMC 100 and ISMC 75 of 1m length. The obtained result shows that the deflection of the section ISMC 75 is lesser than the ISMC 100 section by 45 %.(Fig 4)

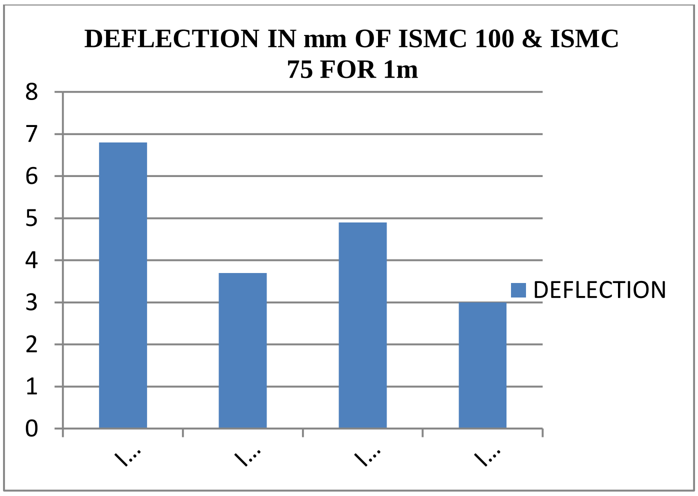


Fig 4- Deflection in mm of ISMC 100 & ISMC 75 for 1m.

5.4.6. Deflection in mm for 0.5m length without splices

The test is carried out by taking the deflection of the steel channels ISMC 100 and ISMC 75 of 0.5m length. The obtained result shows that the deflection of the section ISMC 75 is lesser than the ISMC 100 section by 48 %.(Fig 5)

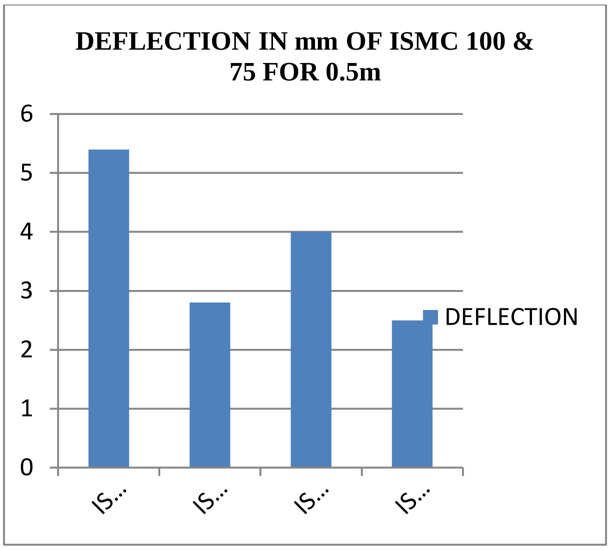


Fig 5- Deflection in mm of ISMC 100 & ISMC 75 for 0.5m.

5.4.7. Deflection in mm for 1m length with splices

The test is carried out by taking the deflection of the steel channels ISMC 100 and ISMC 75 with splice connection. The obtained result shows that the deflection of the section ISMC 75 is lesser than the ISMC 100 section by 25 %(Fig 4)

4.8. Deflection in mm for 0.5m length with splices

The test is carried out by taking the deflection of the steel channels ISMC 100 and ISMC 75 with splice connection. The obtained results shows that the deflection of the section ISMC 75 is lesser than the ISMC 100 section by 38 %.(Fig 5)

V. CONCLUSIONS

- The members designed with splices show greater reduction in the structural weight.
- Experimental evaluation shows increase in the load carrying capacity and decrease in the deflection while using members with splices by 24%
- Splice connection best seismic performance compared to the other type of connection.

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