Design Comparison of Steel Railway Bridge-International Codes of Practice

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Abstract

Structures made from steel are emerging worldwide nowadays due to its high strength, high weld-ability, high weight and high weight to strength ratio. That's the reason its usage are stretching into the bridges section. Constructors make use of several codes of practice to maintain economy, quality and safety the best. Many countries of Asia use another country's design codes along with their own design to aim best quality, safety and economy. It is very essential to compare the codal provisions of Indian standards and European standards in the design of railway bridges made of steel. This will enable us to know that whether India or Europe standards are economical, better quality and safer than compared to another. Construction procedures of both countries in design of web, moment check, shear capacity, end stiffener, intermediate stiffeners are shown. By varying the Steel grade, Aspect Ratio (A.R), d/t_w (web slenderness ratio) total steel (in tonnes) used in girder is taken. Also the variation of steel grades, A.R, d/t_w along with steel weight and deflection are shown for deriving better results

Keywords- *Steel railway bridge, design, comparison, web slenderness ratio, deflection, aspect ratio, weight of steel*

I. INTRODUCTION

Steels possess high ductility, strength, weld ability and strength to weight ratio. Due to more efficient and economy it has become a necessary choice for bridges having long spans such as truss bridges, plate girder bridges and box type girder bridges. Without affecting the strength of sections, amount of steel has to be reduced as cost of steel is rising in high rate. In order to achieve better economic sections various optimization techniques are used.

Composition of Structural Steel

(1) Iron

(2) A very small amount of carbon and manganese

(3) Impurities like sulphur and phosphorous, that cannot be fully removed from the ore

(4) Copper, silicon, nickel, chromium, molybdenum, vanadium, columbium and zirconium are few alloys added in very small quantities to improve the properties of the finished product

As carbon content in steel increases, strength also increases, but reduction in ductility and weld ability is observed. Sulphur and phosphorus have undesirable effects and hence their maximum amount is controlled.

Structural steel may be grouped into the following three categories:

(1) Carbon steels

(2) High-strength steels

(3) Heat-treated carbon steels

(4) Weathering steel

Table I
Types of Structural Steel Along with their Properties

Sl.no	Structural steel	Properties
1	Carbon steels	Components-Mn, Si, Cu High welding ability Yield stress-270N/mm^2
2	High-strength steels	By adding Si, Cu it possess high strength Yield strength- 300-390N/mm^2
3	Heat-treated carbon steels	
4	Weathering steel	Possess corrosion resistance and can be left unpainted

A. Actions and Nature of Rail Traffic Loads

Both Indian (IRS) and European codes (EN 1991-2) consider following loads due to normal railway operations for their designs

- Dynamic effects
- Vertical loading for earthworks
- Vertical loading
- Centrifugal forces
- Nosing forces
- Traction & braking forces
- Combined response of a structure and track to variable actions
- Aerodynamic effects from passing trains
- Actions due to overhead line equipment & other railway infrastructure and equipment

Few other loads or stresses by other sources are:

- Centrifugal forces on a horizontally curved bridge
- Accidental load due to skidding or collision with parapet
- Creep and Shrinkage of concrete
- Snow load on bridge deck, cables, etc.
- Friction at, or shearing resistance of, bearings
- Earth pressure on retaining structures
- Stream flow pressure, floating ice, buoyancy

- Earthquake or ground movement due to other causes
- Settlement of supports
- Impact from shipping.

The three basic parameters which determines the form of construction of a railway bridge:

- Available construction depth
- Span and geometric configuration
- Limitation imposed by substructure

B. Functional Requirements

There are two key functional requirements for a bridge carrying railway:

- Provision of satisfactory support to the railway traffic and infrastructure throughout the life of a bridge.
- Provision of adequate clearances between the structure and the traffic on and beneath the bridge

First requirement can be expressed in terms of requirements for-

- Strength & Fatigue endurance
- Limiting the bridge deformation
- Robustness
- Durability

The second requirement is expressed in terms of various “clearance gauges”, defined by railway & highway authorities

To verify that the requirements continue to be met throughout the life of the structure (i.e., to ensure on-going serviceability), there is also a need to make provision for access to inspect and maintain the elements of the structure in a safe and convenient way, as in [9].

C. Construction Requirements

The design of railway bridges has always required the design engineer to give detailed consideration to the possible methods of construction that might be available at a particular site. This is considered a fundamental requirement in order to produce a design solution that can be translated into reality within the very short periods usually available for such activities. This is particularly so in case of underline bridges because they are required to be capable of supporting the imposed railway loads by the time the structure is ready for reinstatement of the track.

With the introduction of the Construction, Design and Management Regulations in 1995, the need for the designer to consider carefully the effect of his proposals, from a safety point of view, was formalised.

When the requirements of the CDM regulations are considered together with the client’s operational requirements, it becomes obvious that knowledge of methods of construction, and the stability of each, play an important part in selecting the appropriate design solution for particular site.

1) Minimizing Disruption to the operational Railway:

Minimizing disruption to the railway is one of the key criteria to be understood by the designer in developing a successful solution. Due to the nature of railway, access on or near the railway is restricted and generally personnel or operations that can lead to objects coming within a horizontal distance of 3m of the nearest operational rail can only be undertaken during closures of the railway, called possessions. Even work outside the 3m limit (such as preparatory works) may be subject to speed restrictions being placed on the line and this clearly is another disruption to railway operations.

The level of disruption that would be acceptable on the railway line(s) that would be affected by the works has to be done prior to the commencing the design of a new project.

Two main categories of disruption-

- Restrictions to speed
- Possessions

Speed Restriction

A temporary speed restriction (TSR) has to imposed if any works affect the increase risk of derailment and affecting the stability of the track. Imposition of a TSR is required for the installation of track and ballast on to a newly bridge constructed. Amount of time available for the track-work contractor to lay and bed down the track affects the duration and severity of the TSR. At a given particular time, only a certain level of disruption is permitted on railway track.

Possessions

Closure of a section of the railway to normal rail traffic is defined as a possession. The availability of possessions is classified as either ‘Rules of Route’, which are those available for the day to day maintenance of the railway or ‘Outside Rules of the Route’ (also known as ‘abnormal’), which are special possessions usually of longer duration and booked for specific activities.

Rules of Route Possessions, available for regular maintenance of the railway, usually vary between 4 and 29 hours depending on the lines affected. These are generally booked three months in advance. Sometimes it is possible to reduce this booking period by undertaking works on the back of a possession provided for other work, as long as it does not adversely affect the works the possession was originally booked for.

Outside Rules of the Route (abnormal) possessions are the ones most commonly used for bridgeworks because they generally offer a longer duration for construction activity. These possessions are usually booked up to two years in advance and are commonly up to two days duration or, in exceptional circumstances (such as over periods with public holidays), longer. Occasionally the abnormal possession duration for the main bridge works will have been agreed in advance of the design commencing, thus representing a constraint on the solution to be designed.

Because of the importance of handing back the possessions on time for train operations to recommence, designer needs to produce a viable scheme that can be constructed within the available possession period, during any reasonably foreseeable inclement weather and with consideration given to the robustness of every detail. Specifying that all components are trial erected, and ensuring there are adequate tolerances in the design for fit up, will significantly help to achieve it.

D. General Site Constraints

For most structures, it is necessary to understand the impact of site constraints before developing the design solution, as these factors can dictate the method of erection and the form of structure.

1) Site Access

This is the single most important consideration because access by road to the bridge site is not always available, particularly if bridge spans over an obstruction other than a road (e.g. a canal, river, or flood plain). In such cases careful thought needs to be given to researching the types and quality of access that might be arranged to enable the particular design solution being considered.

The quality of site access will determine the type and size of bridge elements (and the type and size of plant) that can be brought to site. Particular care needs to be taken to make sure that the load carrying capacity of any bridges supporting the access road are adequate for the weight of plant and bridge elements being considered.

2) Available Working Place

Many rail sites are in heavily built up areas and often buildings have been erected adjacent to the railway after the line was constructed. These may preclude various methods of erection and significantly constrain others.

3) Services

Services include all statutory undertakers plant, from sewers and fibre optic telecommunications cables buried in the road through to overload power lines. As part of initial option development, the location of utilities services need to be confirmed and their impact on the proposed scheme identified. This will need to cover the viability of any diversionary works together with an order of costs.

E. Bridge Erection Methods

Depending on whether it is a completely new structure or the reconstruction of an existing superstructure, erection of a new steel railway bridge involves following activities

- Crane lifting
- Rolling / Sliding
- Transporting

1) Crane Lifting:

By use of one or more cranes it is the most commonly used method, either piecemeal or in a single lift. Actual amount of time for erection depends on overall size & type of structure, particularly on type of connections to be made & on type of crane to be used.

The cranes are usually road-mobile but can also be either rail-mounted or on a floating vessel. Developments in both road mobile and rail-mounted cranes have significantly increased the size of elements that can be installed.

Before the final selection of carnage as the erection method, consideration should also be given to the following:

- Access to site for crane and bridge elements
- Overhead power lines
- Overhead electrification equipment
- Underground services
- Exposure to wind or flooding
- Available possession time
- Availability and locality of back up plant, spares and fitters

2) Sliding/ Rolling:

Sliding and Rolling are 2 different processes. Sliding a structure on low friction surfaces. Phosphor/bronze or PTFE sledge on stainless steel used in combinations making large number of systems together. From minimum 5% to a maximum of 12% co-efficient at breakout is likely to vary. But this reduces to 2% and 8% during sliding.

Rolling is made on ball bearings or on proprietary rollers. The co-efficient of friction for a 75mm diameter steel balls for ball bearings has 10% at breakout and 2.5% at once rolling .Whereas in case of proprietary rollers, 2.5% at both cases

Possible to use sliding/rolling to remove existing bridge so that it can be safely demolished without affecting critical items of work.

3) Transporting:

This is a relatively recent technique for moving railway bridges. It uses Self Propelled Lifting Vehicles (SPLVs), to lift heavy assembled bridge from temporary works at near-by sites & transport it to its final position. This method is more expensive than other methods. This is particularly suited to bridges over highways or presence of OHE (Over Head Electrification).

F. Recent Developments

There has been a higher requirements of steel in off-shore structures because of low temperature, high tides and greater water depths. By adding alloys such as Va, Ni, Cu, Mu to high strength Ca-Mn steel, strength up to 500N/mm^2 are reached. To attain high ductility, high weld ability and good notch toughness properties quenching-tempering to be done having strength of 700N/mm^2

G. Objectives

Following are the objectives kept in mind while designing the bridge -

- To determine remaining fatigue life of the existing bridge
- Total deflection bridge undergoes by varying the web slenderness ratio, steel grades and aspect ratio
- Total weight of steel
- To determine ultimate strength

II. DESIGN PROCEDURES

A. General

Entire bridge and its components designing- basic form of bridge selection, aligning its components, elements sizing,etc.,the three basic endurances required for the adequacy of structural performance are- deformation, strength and fatigue. All these factors have high effect on railway bridges. Hence, prior to the beginning of the bridge design all three has to be taken into the account.

Primarily, the site details, topography details, geographical details, soil profile, relevant codes, and much more are to be collected. Only after having all these details principal elements sizes, construction form can then be selected.

Load effects such as forces, moments and displacements are determined after checking of initial selection and global analysis.

B. Basis for Design

Design loads are products of load factors and nominal loads, as per BS 5400

i.e., $D.L = \gamma_{Fl\,.}\ N.L$

where, DL= Design loads

γ_{fl}=load factors

NL= Nominal loads

The obtained DL are used to calculate displacements, BM, SF and these inturn used to determine resultant design load effects.

Expression for structural adequacy is,

$$\mathbf{R_d \geq L_d}$$

R_d = design resistance

L_d= design load effects

C. Types of Loads

Following are the loads to be taken in to account-

- Dead Load
- Superimposed dead loads
- Railway Live Loading
- Accidental loading
- Loads due to wind effects
- Loads due to temperature effects

1) Dead Load:

Structural elements load are considered as dead weight. It is the product of material densities and size of material selected.

2) Superimposed Dead Load:

Parapet wall, walkways, balconies, water proofing layer, ballast, etc., are superimposed dead load. Even the superimposed dead loads are calculated in same way as that of dead load. Usually partial factors applied to SDL are larger than those applied to the DL since there may be less control over extent of items making up the SDL

Allowance should be made for increased ballast depth where the deck is sloped for drainage and so the depth is usually taken to the top of sleepers.

3) Track Weights (TW):

To avoid 'double-counting', track weight are often coated as 'extra over' values (i.e. the weight of the sleepers, rails, etc.) minus (the weight of the displaced ballast).

4) Railway Live Loading (RL):

Primary live load and secondary live load are two types of railway live loading. Weights of vertical loads are primary live loads. Horizontal loads are considered as the secondary loads. Type RL loading deals with passenger rapid transit railway system, SW/0 deals with alternative loading, and type RU Loading deals with nominal loading in bridges carrying mainline traffic.

5) RU and SW/O Load Models:

When applied to a simple supported beam would produce load effects approximately equal to or slightly in excess of those that would be derived if the static weight of trains are applied, simplified model Type SW/0 loading is a special loading that only needs to be applied to continuous bridges, where the RU model does not give the worst loading effects at the intermediate supports. The load model should be curtailed if it produces a more onerous effect.

6) Secondary Live Loads (SLL):

Change in direction, speed of train that causes primary loading causes secondary live loads. Three categories- longitudinal, centrifugal and nosing effects

7) Centrifugal Loading:

The centrifugal load due to a mass travelling around a curve at speed is easily calculated.

$$Fc = [P\ (vt + 10)2 \text{ x } f] / 127r$$

Where, P = static load

Vt = train speed (kmph)

r= radius of curvature in m
f= reduction factor that recognizes that train travelling in excess of 120kmph will be lighter (lower mass) and thus the centrifugal force is less.

*8) **Nosing:***

Lateral oscillation of the train on the track also gives rise to lateral forces. the dynamic wheel / rail interaction force from measurements of forces on rails should be applied on both straight and curved track.

*9) **Longitudinal Loads:***

Forces due to traction and breaking both acts along the rails. With continuous welded tracks, some of the force is transmitted beyond the bridge. Traction and braking differ in that there are usually only a small number of driving axles, but wheels are braked all along the train. For long loaded lengths, the braking loads are therefore significantly higher than the traction loads.

*10) **Wheel Loads and Axle Loads Distribution:***

The railway loading may be applied along the lines of the rails which are usually at 1.5m centres, for determining bending moments in transversely spanning elements. RL loading should be shared 56%:44% and RU loading has to be shared equally between two rails.

*11) **Loads Due to Accidents:***

Impact of a vehicle collision or from derailment of train causes Accidental loading.

III.INDIAN AND EUROPEAN STANDARDS DESIGN PROCEDURES COMPARISON

Following table gives a comparison made between the design procedures using Indian standards and European standards for calculating dimensions, design checks, design of end stiffeners, and design of intermediate stiffeners, as in [8].

Table II
Dimensions Comparison of International standards

Indian Standard (IS 800:2007)	**European Standard (BS EN 1993)**
1. Web depth D/L (depth/span)= 0.10 to 0.0667 $D=(M_k/f_y)^{1/3}$ $K= d/t_w$, t_w=web thickness	**Web depth** Depth / span =0.0667 Minimum thickness= 10 to 20
2. Design of Flange Flange area, $A_f \geq M*1.1/(f_y*d)$ M=maximum moment For semi plastic section Flange breadth ,$b_f \leq 13.6\ d_f$	**Flange design** $A_f = M_{max}/d*f_y$ f_y=Yield strength of material $b_f=0.3*d_f$ d_f= depth of flange
3. Moment capacity M-maximum moment $M_{design} = Z_e*f_y/\gamma_{mo}$ Z_e= Section modulus γ_{mo}- Material factor	**Moment capacity** $M_c = f_y*A_f*h_s$ f_y=Yield strength A_f=Area of flange h_f=Centre to centre distance between flanges
4. Shear buckling Stiffener spacing/depth ≥1 K_s =5.15+4/(stiffener/depth)² Shear stress for Elastically critical, τ_{cr} = KΠ²E/(12*(1-μ²)*(d/t_w)²) Non dimensionless slenderness, $\lambda_w=\sqrt{}(f_{yw}/\sqrt{3}*\tau_{cr})$ λ_w > 1.2 shear stress τ_b $=f_{yw}/(\sqrt{3}*1.83^2)$ Shear force $V_n=V_{cr}=A_v*\tau_b > V$	Check for serviceability d/t < t Check for flange buckling in to the web Thickness, t ≥ (d/294)*√(P_{yf}/250) q_e(a/d >1) - [1.0=0.75/(a/d)²][1000/(d/t)]² λ_w -√[0.6*(f_{yw}/γ_m)/q_e] > (a/d) $q_{cr} = q_e\ f_v = F_V A/d*t$
5. Web local capacity Clause8.7.4 Local capacity, fw =(b1+n2)*tw*fyw/γ mo If fw<Fv end stiffeners should be provided	υt=1.5qcr/√(1+(a/d)²,yb=√(Pyw²-3qcr²+υ²)–υt qb=qcr+(yb/2[a/d + √(1+(a/d)²)] end panel is safe if qb >fv Resisting shear force, V_{res}=H/2 Av=t*a Pv=0.6*Pyw*Av End panel is safe if Rtf<Pv , M_{res}= H*d/10 I=(1/12)*t*a³ M= I*Py/y If M_{res} > M end panel is safe
6. End stiffener Effective length on one side – 14tf Area of buckling resistance A Moment of resistance Ix r =√(I/A) , slenderness ratio λ =KL/r fcd – from IS 800	$F_m=M_{res}/a$, Compression in total, $F_c=F_b+F_m$ Area of stiffener – $(0.8*F_c/P_{ys})$ $P_c=(\sigma_c*A_e/\lambda_m)>F_c$ P_{crip} =(b1+n2)*t *p_{yw} , FA=F_c- P_{crip} PA=P_{ys}*A PA> FA

Table 9c Bearing resistance – $f_{cd} * A > F_v$ Bearing strength of stiffener fpsd – $A_q*f_y*q/0.8\ \lambda_{mo}$ should be greater than shear load	
7. Intermediate stiffener $I_s = (1.5\ d^3t^3/c^2)$ $V_{buckling} = V - V_{cr}/\Upsilon_{m0}$ Resistance for buckling $= f_{cd} * A$	**Intermediate stiffener** If $a \geq d\sqrt{2}\, I_s \geq 0.75dt^3$ If $a < d\sqrt{2}\, I_s \geq 0.75dt^3/a^3$ $F_q = V - V_s$ $R_{res} = \sigma_c * A/1.15$ $R_{res} \leq 13.7ts\varepsilon$

IV. OPEN WEB GIRDER BRIDGE

Ministry of state for railways K.H. Muniyappa who laid the foundation stone for the two bridges on May 28, 2008 had given his assurance that work could be complete by March 2011. It's part of the on-going track building work between the two cities, executed to a cost of Rs.497crores. The project has been undertaken on a cost – sharing basis with the state government agreeing to contribute 2/3rd Rs.231crore and the railways chipping in with the balance amount of Rs.166crore.

New bridge will be 13m wide

Cauvery North Bridge will be 325m long whereas Cauvery south bridge 275m long

Cost of both bridges – around 60crores

Figure 1: Both Old Bridge and the Proposed New Bridge in Sri Rangapatna

A. *General*

- Safe bearing Capacity of soil is 180T/m² at the depth of average 3m from EGL
- Standard of loading considered for the design of proposed structure is 25Tons-2008
- Steel ladders have been provided on the abutment and piers to get down to the top of the bed block
- Foundation pressure developed for the abutment max, 36.70 T/Sq.m
- Minimum grade of concrete M25
- The load testing of the open web girder has been carried out and the deflection found within permissible limits.
- Fabrications are done at Jaibalpur
- Camber has been provided in order to avoid negative deflection

B. *Specifications*

- S.W.R railway standard specifications for all materials and works of 2008
- I.R.S concrete bridge code 1997(REVD) including latest correction
- I.R.S bridge substructure and foundation code 2004 (REVD)

Table III : Grades of Concrete and Aggregate sizes Used for Different Components

NO	Description of component	Grade	Aggregate*
1	RCC bed block , Pedestals, Dirt wall	M35	20
2	RCC Abutments and Foundation	M30	20
3	Coping	M30	20

*mm maximum size graded hard stone aggregate of approved quality

1) *Grade for Reinforcement Steel:*

HYSD /TMT/ BARS, Fe 500 to IS:1786

2) *Bearings:*

- Rocker Roller Bearings for open web girder
- Elastomeric Bearings for plate girder

3) *Weep holes:*

Weep holes and back filling are provided as per addendum and correction slip no. 3 dated 30-5-1989 of substructure code

4) *Bed Block:*

- Reinforced cement concrete M30 grade using 20mm max size
- Graded hard stone aggregate of approved quality

5) *Coping:*

- Cement concrete M30 grade using 20mm max size
- Graded hard stone aggregate of approved quality

C. Track details

Table IV Track Details Used in Project

Description	
Loading	25T
Alignment	Straight
Grade	Level
Pro. Rail level	682.521
Pro. Formation level	681.783
Span	37.7m

D. Hydraulic Particulars

Table V : Required and Provided Dimensions for Hydraulic Particulars

Description	Required	Provided
Catchment area	-	-
Waterway	1020 Sq.m	1967.746 Sq.m
Vertical clearance	1500 mm	3434 mm
Free board	1000 mm	3846 mm
Scour depth		

Table VI :Safe Bearing Capacity and Foundation Pressures Developed for Abutments and Piers

Foundation Pressure Developed		Safe Bearing Capacity
For abutments	For piers	
36.70 T/sq.m	57.706 T/Sq.m	130 T/Sq.m

Table VII : Depth of Construction Chosen for North Cauvery Project

Sl.no	Description	Depth
1	Rail 52kg	172 mm
2	Thickness of GRP	10 mm
3	Thickness of canted bearing plate	18 mm
4	Height of steel channel sleeper	150 mm
5	Thickness of rubber pad	25 mm
6	Thickness of stringer	720 mm
7	Bottom of stringer to bottom of cross girder	85 mm
	Depth of construction	2500 mm

Table VIII : Depth of Construction Chosen for South Cauvery Project

Sl.no	Description	Depth
1	Rail 52kg	172 mm
2	Thickness of GRP	10 mm
3	Thickness of canted bearing plate	18 mm
4	Depth from steel channel sleeper top to top of cross girder	80 mm
5	Cross girder depth	900 mm
	Depth of construction	2500 mm

Figure 2: Two Lane Steel Railway Bridge

V. RESULTS AND DISCUSSIONS

A. Steel grade effects on weight and deflection

1) Design results for Indian Standards:

Table IX : Effect of Grade of Steel as per Indian Standards

Steel	Span (L), m	Web depth(D), m	Permissible Limit L/600, m	Deflection (δ), m	Weight (tons)
E450	60	2.5	0.1	0.09931	78.18
E410	60	2.5	0.1	0.09494	81.25
E250	60	2.5	0.1	0.08865	86.24
E450	50	2.5	0.08333	0.07659	40.01
E410	50	2.5	0.08333	0.07141	42.17
E250	50	2.5	0.08333	0.06719	44.21
E450	40	2.5	0.06666	0.05312	19.93
E410	40	2.5	0.06666	0.04952	20.77
E250	40	2.5	0.06666	0.03322	26.87

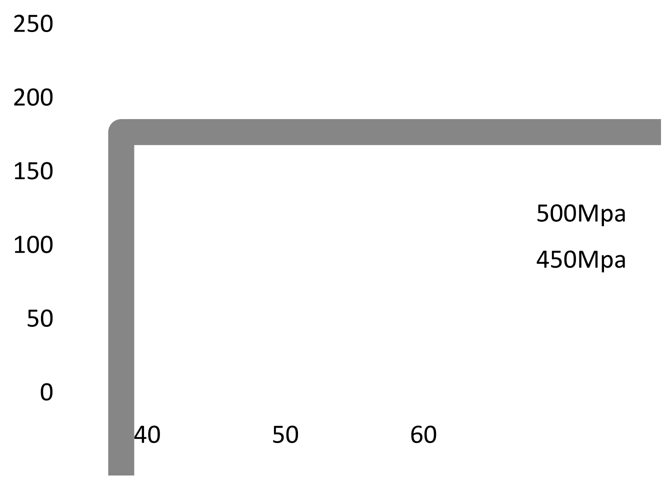


Figure 3: Plot of Deflection Along Y-Axis Against Span Length Along X-Axis for Different Steel Grades

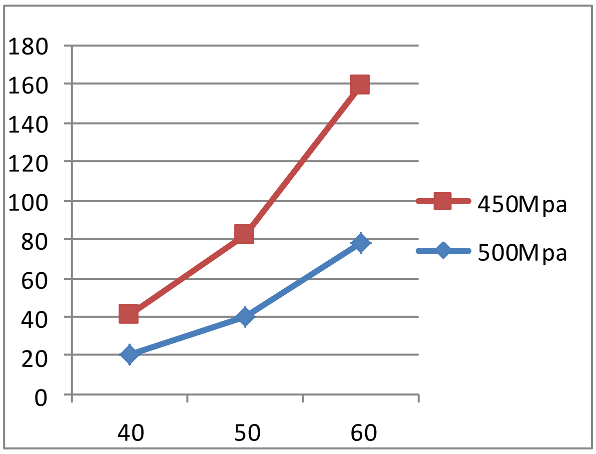


Figure 4: Plot Of Weight Along Y-Axis Against Span Length Along X-Axis for Different Steel Grades

2) *Design results for European Standards:*

Table X : Effect of Grade of Steel as per Euro Standards

Grade of Steel	Bridge Span (L), m	Depth of web (D), m	L/600 (Permissible limit), m	Δ (deflection), m	W (Weight) ,Tons
460	60	2.5	0.1	0.0979	54.53
420	60	2.5	0.1	0.0918	55.35
235	60	2.5	0.1	0.0772	59.44
460	50	2.5	0.0833	0.0797	38.84
420	50	2.5	0.0833	0.0749	40.62
235	50	2.5	0.0833	0.0664	42.73
460	40	2.5	0.0666	0.0489	17.21
420	40	2.5	0.0666	0.0459	17.81
235	40	2.5	0.0666	0.0322	19.33

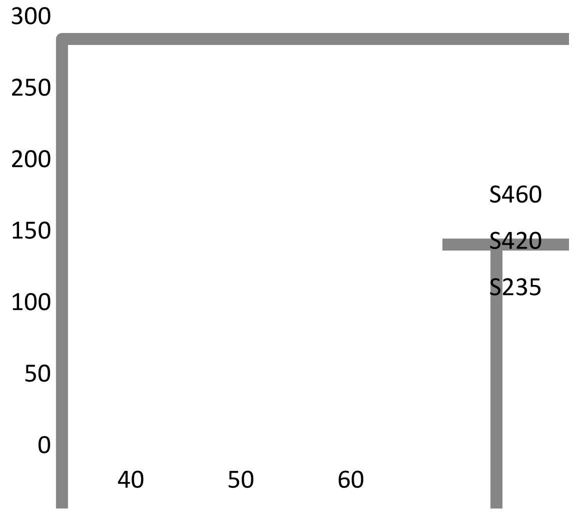


Figure 5: Plot of Deflection Along Y-Axis Against Span Length Along X-Axis for Different Steel Grades

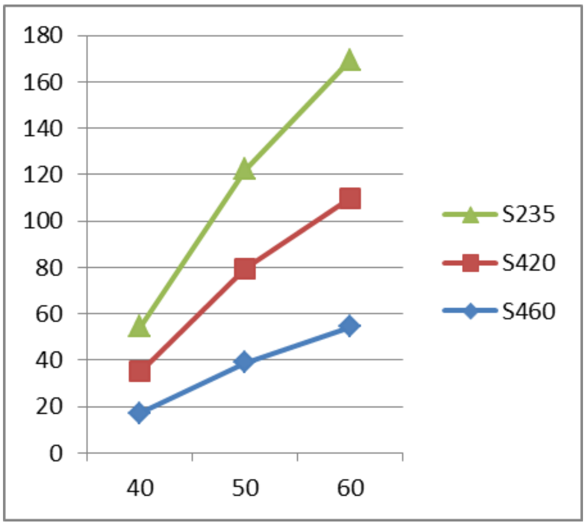


Figure 6: Plot of Weight Along Y-Axis Against Span Length Along X-Axis for Different Steel Grades

B. *Panel Aspect Ratio v/s Weight and Deflection*

By varying the aspect ratio (c/d) of web panel from 0.8 to 1.6, the railway bridge plate girders were modelled. By keeping the depth of web as constant, the aspect ratio was determined by varying the width(c) of the web panel. Also the effect of aspect ratio on deflection and weight of the panel was compared by varying the grade of steel. The comparison of the effect of aspect ratio on deflection and weight for various grades of steel is shown in table below.

1) *Design Results for Indian Standards:*

Table XI : Effect of Variation of Aspect Ratio on Total Deflection and Weight

Grade of Steel	Bridge Span(L), m	Depth of web(D), m	Panel aspect ratio	L/600 (Permissible limit), m	Δ (Deflection), m	W (weight),tons

E250	60	2.5	1.6	0.1	**0.1151**	64.89
E250	60	2.5	1.4	0.1	**0.1075**	65.12
E250	60	2.5	1.2	0.1	0.09931	65.48
E250	60	2.5	1.0	0.1	0.09494	65.82
E250	60	2.5	0.8	0.1	0.08865	66.24
E250	50	2.5	1.6	0.0833	0.08958	41.98
E250	50	2.5	1.4	0.0833	0.08443	42.32
E250	50	2.5	1.2	0.0833	0.07859	42.81
E250	50	2.5	1.0	0.0833	0.07221	43.67
E250	50	2.5	0.8	0.0833	0.06719	44.21
E250	40	2.5	1.6	0.0666	0.05568	25.07
E250	40	2.5	1.4	0.0666	0.04893	25.35
E250	40	2.5	1.2	0.0666	0.04267	25.96
E250	40	2.5	1.0	0.0666	0.03793	26.49
E250	40	2.5	0.8	0.0666	0.03321	26.87

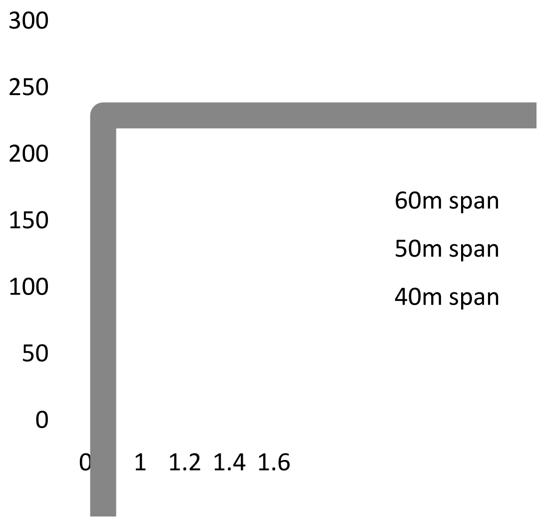


Figure 7: Plot of Deflection Along Y-Axis Against Panel Aspect Ratio Along X-Axis for 250Mpa

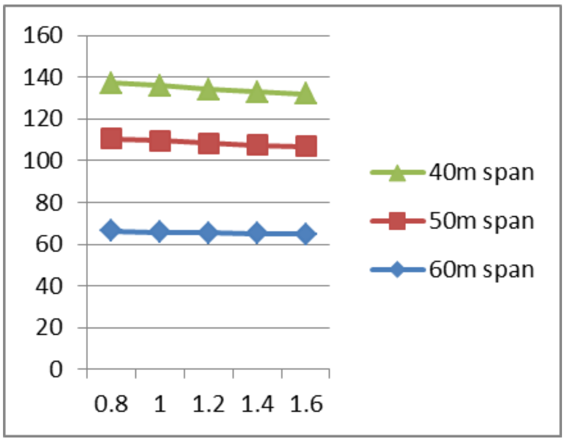


Figure 8: Plot of Weight Along Y-Axis Against Panel Aspect Ratio Along X-Axis for 250Mpa

		m		limit), m		
S235	60	2.5	1.6	0.1	**0.1071**	57.89
S235	60	2.5	1.4	0.1	**0.09945**	58.12
S235	60	2.5	1.2	0.1	0.09131	58.48
S235	60	2.5	1.0	0.1	0.08394	58.82
S235	60	2.5	0.8	0.1	0.0.0772	59.24
S235	50	2.5	1.6	0.0833	0.08858	39.98
S235	50	2.5	1.4	0.0833	0.08359	40.32
S235	50	2.5	1.2	0.0833	0.07783	40.81
S235	50	2.5	1.0	0.0833	0.07151	41.67
S235	50	2.5	0.8	0.0833	0.06691	42.73
S235	40	2.5	1.6	0.0666	0.05439	18.07
S235	40	2.5	1.4	0.0666	0.04758	18.35
S235	40	2.5	1.2	0.0666	0.04143	18.96
S235	40	2.5	1.0	0.0666	0.03687	19.49
S235	40	2.5	0.8	0.0666	0.03227	19.33

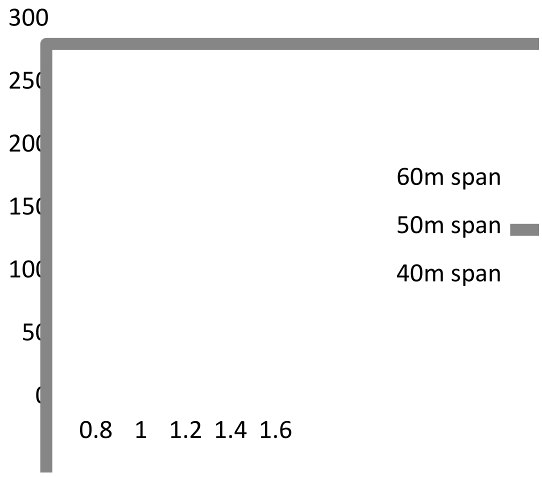


Figure 9: Plot of Deflection Along Y-Axis Against Panel Aspect Ratio Along X-Axis for S235

2) Design results for European standards:

Table XII : Effect of Variation of Aspect Ratio on Total Deflection and Weight

Grade of Steel	Bridge Span(L), m	Depth of web(D),	Panel aspect ratio	L/600 (Permissible	Δ (deflection), m	W (weight),tons

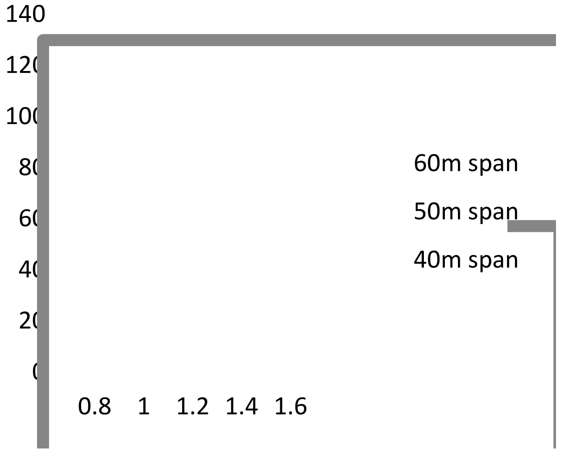


Figure 10: Plot of Weight Along Y-Axis Against Panel Aspect Ratio Along X-Axis for S235

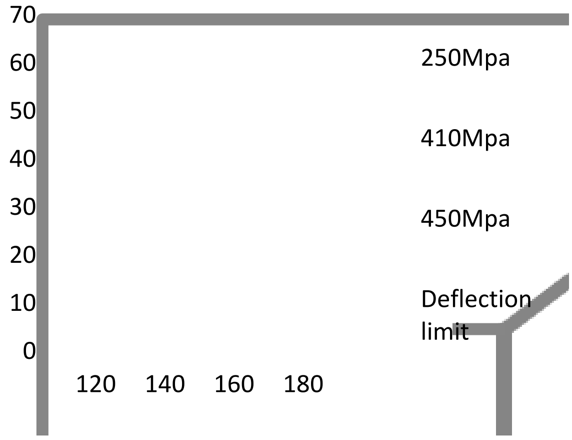


Figure 11: Plot of Deflection Along Y-Axis Against Web Slenderness Ratio Along X-Axis

C. Web Slenderness Ratio v/s Deflection and Weight

Variation of thickness of web from 14mm to 20mm is achieved by varying the web slenderness ratio (d/t_w), by keeping depth as constant. The varying slenderness ratio is monitored to get variation in total deflection and total weight. Also for different grades of steel of web, calculations are done. Results tabulated below

1) Design Results of Indian Standards:

Table XIII : Effect of Slenderness Ratio on Total Deflection and Weight

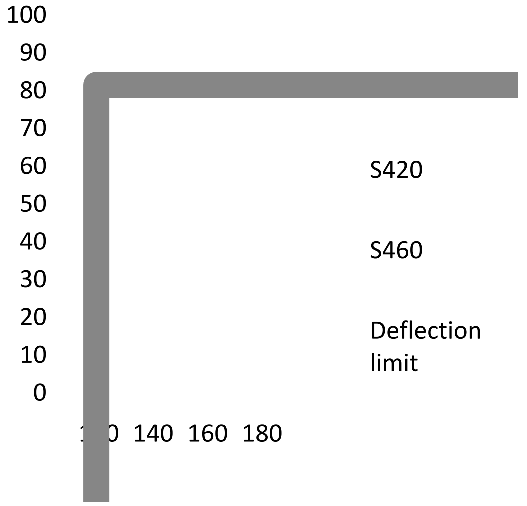
Grade of Steel	Bridge Span(L),m	Depth of web(D),m	Slenderness ratio, (d/t_w)	L/600 (Permissible limit),m	Deflection(δ), mm	Weight (tons)
450	50	2.5	178	0.0833	50.78	18.07
450	50	2.5	156	0.0833	48.64	18.07
450	50	2.5	138	0.0833	47.32	18.07
450	50	2.5	125	0.0833	45.18	18.07
410	50	2.5	178	0.0833	47.16	20.74
410	50	2.5	156	0.0833	45.23	20.74
410	50	2.5	138	0.0833	44.2	20.74
410	50	2.5	125	0.0833	42.54	20.74
250	50	2.5	178	0.0833	33.33	25.88
250	50	2.5	256	0.0833	31.63	25.88
250	50	2.5	138	0.0833	30.85	25.88
250	50	2.5	125	0.0833	29.87	25.88

2) Design results of European standards:
3)

Table XIV : Effect of Slenderness Ratio on Total Deflection and Weight

Grade of Steel	Bridge Span(L),m	Depth of web(D),m	Slenderness ratio, (d/t_w)	L/600 (Permissible limit),m	Deflection(δ), mm	Weight (tons)
460	50	2.5	178	0.0833	**99.74**	47.14
460	50	2.5	156	0.0833	**98.62**	49.68
460	50	2.5	138	0.0833	**97.2**	51.99
460	50	2.5	125	0.0833	94.28	56.89
420	50	2.5	178	0.0833	94.06	49.84
420	50	2.5	156	0.0833	93.15	51.38
420	50	2.5	138	0.0833	91.63	56.24
420	50	2.5	125	0.0833	87.86	58.06
235	50	2.5	178	0.0833	90.67	50.01
235	50	2.5	156	0.0833	85.45	53.18
235	50	2.5	138	0.0833	87.00	58.45
235	50	2.5	125	0.0833	83.10	61.2

Figure 12: Plot of Deflection Along Y-Axis Against Web Slenderness Ratio Along X-Axis



VI. CONCLUSIONS

By comparing the code provisions as per Indian and European standards in the design of steel railway bridge, following conclusions were derived from design results:

1. According to the Indian standards design, the railway bridge of constant span and depth shows that as the grade of steel increases the total deflection of the girder increases but the total weight decreases.
2. European standard design depicted that, deflection increases and weight decreases as grade of steel increases.
3. The maximum deflections obtained for a 60m span bridge with varying aspect ratio from 0.8 to 1.6 as per Indian standards is more while compared to the European standards. Similar behaviour is observed for 40m and 50m span bridge.
   Increasing the web slenderness ratio from 120-180, the deflection also increases.
4. From the results it is clear that the deflection is inversely proportional to the thickness of the web
5. By observing the design results of Indian and European standards, it is evident that as web thickness increases deflection reduces.
6. As per Indian and European design standard results, stiffener spacing have much impact on the deflection of a plate girder bridge

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