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# Economical Design of Steel-Concrete Composite Bridge with MS and HPS

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## Abstract

A new class of high strength steel with excellent toughness, ductility, and good weldability is emerging world-wide, named as High Performance Steel (HPS). HPS can be designed as having an optimized balance of these properties to give maximum performance in bridge structures while remaining cost-effective.

A study is performed to compare the cost differences between bridge designs using conventional mild steel Fe 410 and high tensile steel Fe 590. Two cases of span supported and un-supported during construction are considered for comparison. Maximum flexural stresses, maximum deflection, weight and cost are compared for 40m span steel-concrete composite bridge for both the unsupported and supported conditions of the bridge span during construction.

HPS steel is found to be most beneficial and economical in bridge design as compare to MS. However, the maximum deflection is found to increase more than two times the permissible deflection of L/600 for total dead and live load, for HPS girder in comparison to the mild steel girder case.

## 1. Introduction

The application of high strength steel [1] makes it possible to design not only lightweight structures, but also simple structures with simple weld details. As the spans of bridges are getting longer and longer, there is strong demand for steel with regard to the increased strength. However, careful attention must be paid for the fabrication of structural members using high strength steel due to their inherent poor weldability. The fatigue performance [2] of structural welded members of high strength steel indicates the inverse material dependence. The biggest problem in high strength steel is to achieve a balance between tensile strength and fatigue performance without loosing good weldability. Another important problem is to overcome corrosion which is a drawback of steel bridges.

Steel processing has undergone significant development in the past ten years. In addition to the traditional hot rolling, controlled rolling, normalizing, and quenching and tempering, various combinations of rolling practices and cooling rates have opened new opportunities to develop high strength with very attractive properties. The word “High Performance Steel (HPS)” has been used as the steel having higher ductility, better fracture toughness, better weldability, better cold formability, and better corrosion resistance besides higher strength [3].

When HPS, first became available for use [4], it was attractive steel to bridge engineers because of its superior weldability, fracture toughness and weathering characteristics. Since its first introduction to the market, HPS has been implemented in bridge design and construction in several states. However, though HPS offers the above positive attributes, it does have higher material costs [5]. Therefore, it is important to develop an understanding of how this material may most economically be incorporated in the design of composite I-girder bridges. A few studies have been performed to explore this issue and the benefits realized by weight savings and reduced fabrication costs, which may offset the increased material costs.

For deflection control, the structural designer [6] should select maximum deflection limits that are appropriate to the structure and its intended use. The calculated deflection (or camber) must not exceed these limits. Codes [7] of practice give general guidance for both the selection of the maximum deflection limits and the calculation of deflection. Again, the existing code [8] procedures do not provide real guidance on how to adequately model the time-dependent effects of creep and shrinkage in deflection calculations [9-12].

HPS design follows the same design criteria and good practice as provided in Section-6 of Steel Structures of the AASHTO LRFD Bridge Design Specifications [13]. Use of HPS generally results in smaller members and lighter structures. The designers should pay attention to deformations, global buckling of members, and local buckling of components.

For HPS, the live load deflection criteria are considered optional as stated in Section 2, Article 2.5.2.6.2 of the AASHTO LRFD [9]. The reason for this is that past experience with bridges designed under the previous editions of the AASHTO standard specifications has not shown any need to compute and control live load deflections based on the heavier live load required by AASHTO LRFD. However, if the designers choose to invoke the optional live load deflection criteria specified in Article 2.5.2.6.2, the live load deflection should be computed as provided in Section 3, Article 3.6.1.3.2 of the AASHTO LRFD. It may be expected that HPS designs would exceed the live load deflection limit of L/800. The designers have the discretion to exceed this limit or to adjust the sections by optimizing the web depth and/or increasing the bottom flange thickness in the positive moment region to keep the deflection within limit.

Study is performed to compare the bridges designed using MS and HPS. In India the HPS is still not in use and IS codes has no specification for HPS. So criteria of HPS used for comparison is assumed as given in HPS Designer Guide. As per Indian Standard Codes, mild steel (MS) Fe 410 (yield stress = 250 MPa) and high tensile steel (HPS) Fe 590 (yield stress = 450 MPa) are used to compare the steel grades. For the cost comparison cost of HPS is approximately taken as 1.2 times the cost of MS.

IRC: 6-2000 code is considered for Class 70R wheeled and tracked loads, and two lanes Class A load are considered for calculating the live load effects on the bridges. Super imposed dead load (SIDL) is also considered as per IRC. Maximum bending moment and deflection are calculated using the composite bridge model in STAAD.Pro V8i software. Different cross-sections of steel girders are analyzed to obtain the weight and cost effective section keeping the maximum flexural stresses within the permissible limits.

Total shrinkage strain in steel-concrete composite deck slab concrete may be taken as 0.0003. For composite action to start, this strain must be first overcome, for which additional flexural stress of 60.0 $N/mm^2$ is required at the top fiber of the steel girders. Thus, the load will be taken by the girder alone till the composite action starts, and only after the start of composite action, the load will be supported by the composite section.

The primary objective of this paper is to comparison between MS, and HPS, and to investigate the economy of HPS in bridge design using various span lengths, girder spacing and steel grade combinations. This study also emphasis on the effect of live load deflection criteria of using HPS.

The study examined the use of HPS by designing the homogenous girders using MS and HPS with the following parameters:

- Span lengths: 40 m, 50 m, 60 m and 70 m
- Span-to-depth ratio (*L/D*) = 15, 20, 25 and 30

This study investigated the influence of steel grade on weight, performance, and deflection issues.

Steel-concrete composite bridge are designed as per IS 1343:1999 and IS 2062: 1999 codes for comparison using the following parameters.

i. Effective span = 40.0m.
ii. No. of Main Girders = 5 Nos.
iii. No. of Cross Girders = 4 Nos.
iv. Width of deck slab = 12000 mm
v. Width of footpath = 1750 mm
vi. Carriage width = 7500 mm
vii. Size of kerbs = 500 x 400 mm
viii. Railings = 250 mm
ix. Yield strength of steel (fy) = 250 Mpa
x. Young's modulus of steel = $2 \times 10^5$ Mpa
xi. Grade of concrete = M40
xii. Impact factor = as per IRC 6
xiii. Thickness of deck slab = 220 mm
xiv. Depth of haunch = 80 mm
xv. Width of railings = 250 mm
xvi. Grade of reinforcing steel = Fe 410, Fe 590
xvii. $\sigma$st (as per IRC 21) = 200 Mpa
xviii. Cover provided = 40 mm

Two types of spans of the bridge have been considered for design

i) Un-supported span:

In the unsupported span it has been assumed that site conditions are such that it is not possible to support the bridge during construction. Therefore, the steel girder will deflect when it is launched, then it will further deflect under the load of shuttering and bridge deck slab concrete. After hardening of the deck slab concrete, the composite action of the steel girder and RCC deck slab will start. Therefore, under live load conditions the composite section will be available to take up the load.

ii) Supported span:

In the supported span case, it is assumed that it is possible to erect temporary support to the bridge span. Therefore, there will not be any deflection of the steel girder or the deck slab until the supports are removed after hardening of the deck slab concrete. Thus, the composite sections will resist all the loads after removal of the support.

## 2. Analysis of Bridge

The primary objective of this section was to conduct a set of parametric design studies incorporating two grades of bridge steels: MS and HPS. Various combinations of cross-section and span configurations were generated to optimize the resulting bridge profiles. Resulting bridges were studied to investigate the influence of steel grade on weight, performance, and deflection issues.

Maximum flexural stresses and deflection, weight and cost are compared for 40.0m span steel-concrete composite bridge (Fig. 2.1) for both the unsupported and supported span conditions of the bridge during construction.

Results of calculation of sectional properties of girder section only and composite section bridges are are given in Table 3.1.A and 3.1.B.

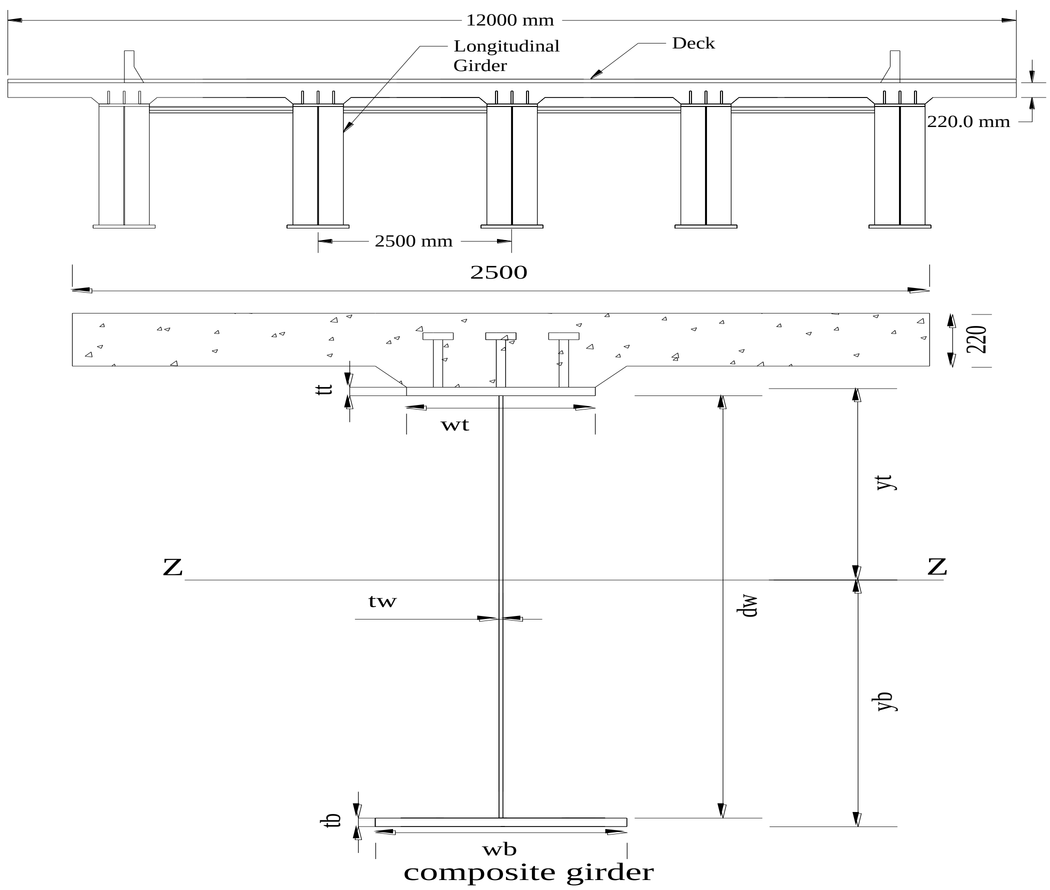


**Fig. 2.1 Details of Composite Girder Bridge**

By analyzing a two-lane composite bridge with five welded steel plate girders spaced at 2.5 m, a matrix of parametric combinations was developed with the following variable design parameters:

- Span lengths: $L$ = 40 m, 50 m, 60 m and 70 m
- Targeted span-to-depth ratio: $L/D$ = 15, 20, 25 and 30
- Steel compared: MS and HPS

In the span to depth ratio the depth, $D$, is taken as the entire superstructure depth, i.e. the sum of the structural thickness of the deck, haunch, web depth, and thickness of both flanges.

To begin a design, a preliminary superstructure depth based on the targeted $L/D$ was calculated.

Once the preliminary superstructure depth, $D$, was calculated, the structural depth of the deck, the haunch, and the thickness of both flanges was subtracted to achieve the web depth. From this web depth, an initial flange width was selected such that $D/bf$ fell in the range of 3.00 to 4.5 where $D$ is the web depth and $bf$ is the flange width. It should be noted that target $D/bf$ ratios resulted from previous research by [3]. After a preliminary girder was chosen, the appropriate non-composite and composite dead loads were calculated. In some cases for high $L/D$, the $D/bf$ fell below 2.0. Bridge designs were developed neglecting the live load deflection limits of $L/800$.

## 3. Result and Discussion

Summaries of resulting designs are presented in Tables 3.1. It should be noted that the bold values in Table 3.1 shows the deflection values that failed the permissible live load deflection limit (L/800). The weight given in the tables is the weight of a single girder. These designs were performed to observe qualitative trends between the variables described above. Changes in the design assumptions will naturally change the resulting design values.

**Table 3.1: Summary of Study**

| Steel | Span Length (L), m | Web Depth (d), m | L/D Ratio | Permissible Limit L/800, mm | Deflection (δ), mm | L/ δ | Weight (tons) |
|---|---|---|---|---|---|---|---|
| MS | 40 | 2.267 | 15.0 | 50 | 27.3 | 1463 | 13.3 |
| MS | 40 | 1.600 | 20.0 | 50 | 39.7 | 1008 | 13.3 |
| MS | 40 | 1.167 | 25.0 | 50 | **55.0** | 727 | 14.7 |
| MS* | 40 | 0.900 | 30.0 | 50 | **63.7** | 628 | 20.0 |
| HPS | 40 | 2.267 | 15.0 | 50 | 27.3 | 1463 | 13.3 |
| HPS | 40 | 1.600 | 20.0 | 50 | 40.0 | 1000 | 13.3 |
| HPS | 40 | 1.167 | 25.0 | 50 | **54.3** | 736 | 14.7 |
| HPS* | 40 | 0.900 | 30.0 | 50 | **73.0** | 548 | 17.3 |
| MS | 50 | 3.028 | 15.0 | 62.5 | 23.6 | 2118 | 33.3 |
| MS | 50 | 2.194 | 20.0 | 62.5 | 46.7 | 1071 | 22.2 |
| MS | 50 | 1.667 | 25.0 | 62.5 | **62.8** | 796 | 23.3 |
| MS | 50 | 1.333 | 30.0 | 62.5 | **76.1** | 657 | 27.8 |
| HPS | 50 | 3.028 | 15.0 | 62.5 | 22.2 | 2250 | 36.7 |
| HPS | 50 | 2.194 | 20.0 | 62.5 | 50.3 | 994 | 22.2 |
| HPS | 50 | 1.667 | 25.0 | 62.5 | **69.2** | 723 | 22.2 |
| HPS | 50 | 1.333 | 30.0 | 62.5 | **88.1** | 568 | 23.3 |
| MS | 60 | 3.725 | 15.0 | 75 | 23.0 | 2609 | 72.0 |
| MS | 60 | 2.725 | 20.0 | 75 | 52.3 | 1148 | 45.0 |
| MS | 60 | 2.075 | 25.0 | 75 | 70.6 | 848 | 41.0 |
| MS | 60 | 1.675 | 30.0 | 75 | **83.8** | 716 | 48.0 |
| HPS | 60 | 3.725 | 15.0 | 75 | 22.0 | 2727 | 79.0 |
| HPS | 60 | 2.725 | 20.0 | 75 | 57.5 | 1043 | 46.0 |
| HPS | 60 | 2.075 | 25.0 | 75 | **82.3** | 730 | 36.0 |
| HPS | 60 | 1.675 | 30.0 | 75 | **105.3** | 570 | 37.0 |
| MS | 70 | 4.410 | 15.0 | 87.5 | 24.7 | 2830 | 122.3 |
| MS | 70 | 3.220 | 20.0 | 87.5 | 49.5 | 1415 | 84.0 |
| MS | 70 | 2.543 | 25.0 | 87.5 | 78.4 | 893 | 68.1 |
| MS | 70 | 2.007 | 30.0 | 87.5 | 83.1 | 843 | 86.8 |
| HPS | 70 | 4.410 | 15.0 | 87.5 | 23.3 | 3000 | 136.3 |
| HPS | 70 | 3.220 | 20.0 | 87.5 | 61.4 | 1141 | 80.3 |
| HPS | 70 | 2.543 | 25.0 | 87.5 | **90.1** | 777 | 63.5 |
| HPS | 70 | 2.007 | 30.0 | 87.5 | **121.1** | 578 | 67.9 |

**Notes:**

1) L = span length, D = total superstructure depth (bot. flange + web + haunch + slab)
2) Weight shown is for one steel girder in the system

3) *denotes D/bf < 2.0
4) Bold values showed the deflection values that failed the permissible live load deflection limit).

### 3.1 Influence of L / D Ratio on girder weight

Requirement of girder weight versus the span to depth ratio for a given span lengths are showed in figures 3.1 to 3.4. Two curves are plotted on each figure, one for the MS designs and one for the HPS designs. In many cases, the weights are similar for both the MS and HPS designs.

This is attributable to following points:

• These studies cover a wide range of girder depths, with web depths variations for a given series of girder varying from as low as 0.9 m to as high as 4.4 m.

• At high *L/D* ratios of 25 and 30, the HPS girders tend to weight slightly less than the comparable MS girders.

• Except for ratio L/D of 15, the average weight savings for all span ranges considered, for HPS versus MS is 8.2%.

• Except for 40 m span, at low L/D ratio of 15, the weight of HPS girders is more than comparable MS girders for all span ranges.

• At *L/D* ratios of 20 and 25, MS and HPS girders tended to weigh less than at *L/D* ratios of 15 and 30. It showed that optimum weight of girders is obtained at certain mid value of L/D ratio, which should not be too high or too low.

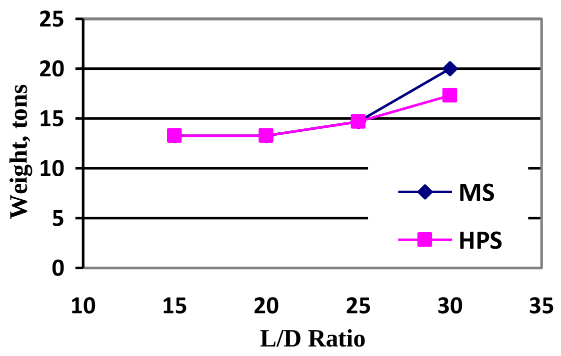


**Fig. 3.1 Requirement of steel with L/D ratio for 40.0 m span girder**

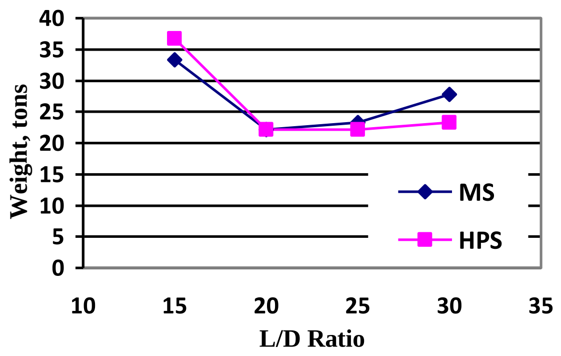


**Fig. 3.2 Requirement of steel with L/D ratio for 50.0 m span girder**

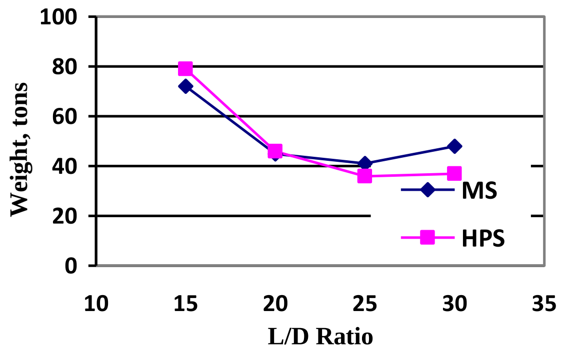


**Fig. 3.3 Requirement of steel with L/D ratio for 60.0 m span girder**

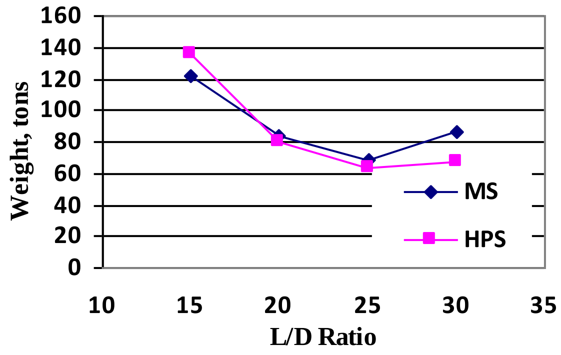


**Fig. 3.4 Requirement of steel with L/D ratio for 70.0 m span girder**

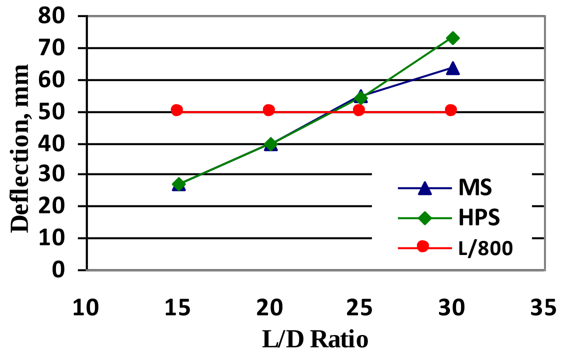


**Fig. 3.5 Maximum deflection at mid span with L/D ratio for 40.0 m span girder**

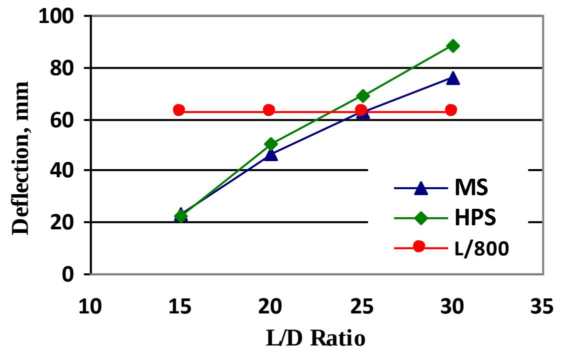


**Fig. 3.6 Maximum deflection at mid span with L/D ratio for 50.0 m span girder**

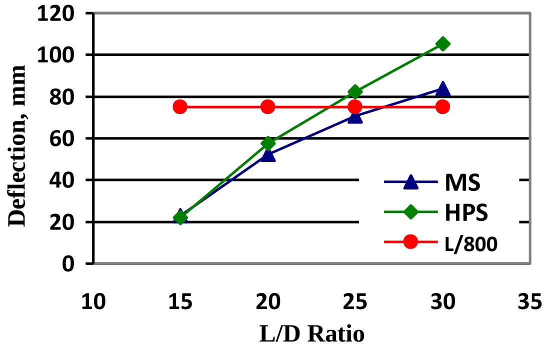


**Fig. 3.7 Maximum deflection at mid span with L/D ratio for 60.0 m span girder**

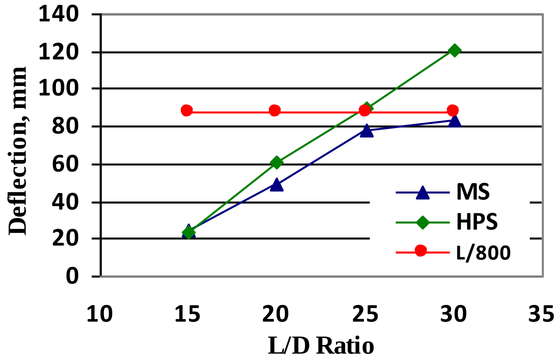


**Fig. 3.8 Maximum deflection at mid span with L/D ratio for 70.0 m span girder**

### 3.2 Influence of L/D on Live-Load Deflection Limits

Figures 3.5 through 3.8 show the maximum deflection versus the span to depth ratio for a given span length. Again, these plots give two curves, one for the MS designs and one for the HPS designs.

• At *L/D* ratios of 15 and 20 no girder failed the deflection criteria. In fact, it is observed from Figs. 3.5 through 3.8 that the deflections in these girders are significantly lower than those associated with *L*/800.

• At *L/D* = 25 and 30 all HPS girders failed the deflection criteria.

• Except for *L/D* of 15, the deflection in HPS girders is more than that of MS girders for all range of span considered.

## 4. Conclusion

This study has presented the comparison between mild steel and HPS girder. HPS steel is found to be most beneficial and economical in bridge design as compare to MS. Several trends can be observed from the comparison of the data. Main conclusions drawn from the study are:

(1) In all cases, the HPS girder bridge resulted in the lightest than the MS girder bridge design.

(2) For all the cases, the deflection in HPS girders design is more than that of MS girders design.

With all the advantages of HPS, its main disadvantage is that the deflection is more than the permissible deflection limit. This has further adverse effects of increased flexural stresses in the deck slab, and its deterioration under increased fatigue loading.

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