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Comparative Study of Economical Design Aspect of Steel-Concrete Composite Bridge with MS, HPS AND Hybrid Steel

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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. Hybrid steel girder, comprising mild steel for top flange and web, and HPS for bottom flange, has outstanding potential for steel-concrete composite bridge.*

A study is performed to compare the cost differences between bridge designs using conventional mild steel Fe 410, high tensile steel Fe 590 and a combination of the two grades of steel. 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.

It has been found that hybrid steel girders are most economical, for which there is a saving of about 34.7% in steel weight and 29.1% in steel cost in comparison to the mild steel girder bridge for the un-supported span case, while in supported span case it is 50% and 46%, respectively. 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 both HPS as well as hybrid steel girder in comparison to the mild steel girder case

Keywords— *High performance steel, Bridge, Deflection, Cost, Hybrid*

I. 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 [14]. The service limit state should be checked for deflection, handling, shipping and construction procedures and sequences.

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, two lanes Class A load and Bogie load are considered for calculating the live load effects on the bridges. Super imposed dead load (SIDL) is also considered as per IRC code. 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 the comparison between MS, HPS and hybrid steel, 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. A study is performed to compare the cost differences between composite bridge designs using conventional mild steel Fe 410, high tensile steel Fe 590 and a combination of the two grades of steel. 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.

Steel-concrete composite bridge is 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 (and 4 Nos for Study 3).
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 x 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 and Hybrid
xvii. σst (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.

II. Analysis of Bridge

This study is performed to compare the bridges designed using MS, HPS and hybrid (MS for top flange and web, and HPS for bottom flange) steel. Two cases of span supported and un-supported during construction are considered for comparison. 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.

Let the dimensions of girder section are as given below:

Depth of web	=	dw
Thickness of web	=	tw
Width of top flange	=	wt
Thickness of top flange	=	tt
Width of bottom flange	=	wb
Thickness of bottom flange	=	tb

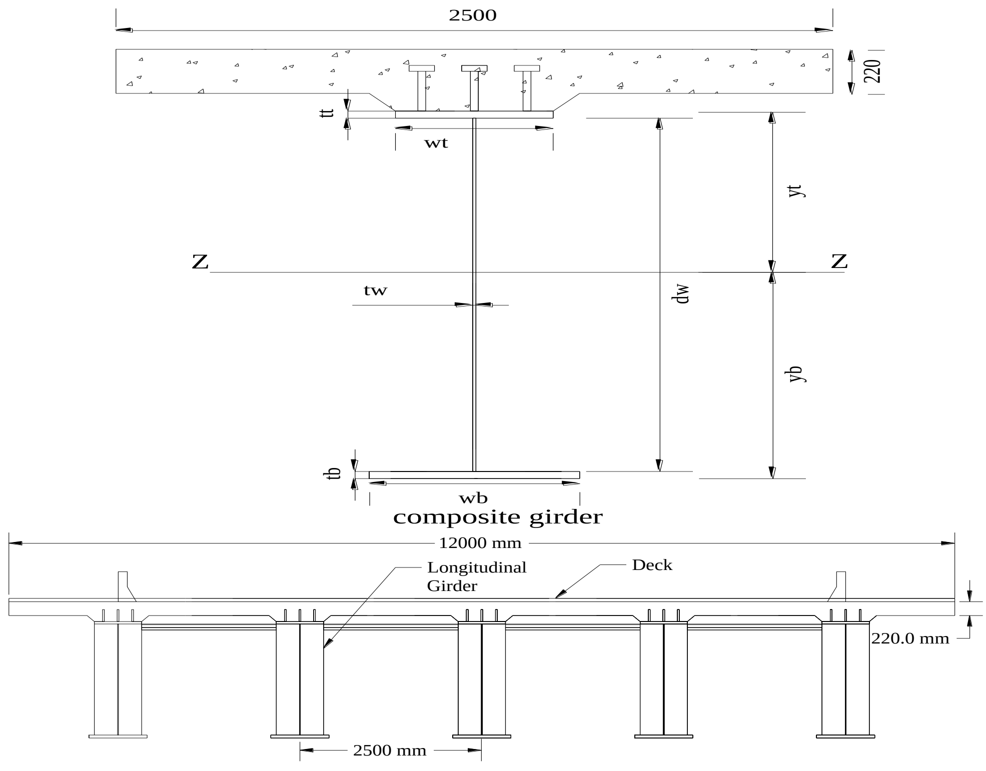

Fig. 2.1 Details of Composite Girder Bridge

Various combinations of cross-section were generated to optimize the resulting bridge profiles keeping the maximum flexural stresses within the permissible limits. Resulting bridges were studied to investigate the influence of steel grade on weight, performance, and deflection issues.

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

III. Result and Discussion

Table-3.1.A and 3.1.B give the geometrical properties of composite sections for the un-supported and supported span bridge cases, respectively.

Table-3.1.A. Details of SCC 40 m span Bridge for un-supported Span

Sectional Properties		Type of Steel		
		MS	HPS	Hybrid
Girder Web (m)	Depth (dw)	3.0	2.5	2.5
	Thickness (tw)	0.012	0.012	0.012
Girder Top Flange (m)	Width (wt)	0.4	0.3	0.4
	Thickness (tt)	0.05	0.03	0.04
Girder Bottom Flange (m)	Width (wb)	0.7	0.5	0.45
	Thickness(tb)	0.05	0.03	0.03
Girder Cross Section (m^2)	Girder	**0.0910**	**0.0540**	**0.0595**
	Composite	0.1864	0.1494	0.1549
Moment of inertia about major axis, Iz (m^4)	Girder	0.1492	0.0530	0.0629
	Composite	0.3348	0.1432	0.1381
Distance of neutral axis from girder bottom, yb (m)	Girder	1.2986	1.1394	1.3345
	Composite	2.3187	2.1690	2.2136
Distance of neutral axis from girder top, yt (m)	Girder	1.8014	1.4206	1.2355
	Composite	0.7813	0.3910	0.3564
Section modulus from girder bottom (m^3)	Girder	0.1149	0.0465	0.0471
	Composite	0.1444	0.0660	0.0624
Section modulus from girder top (m^3)	Girder	0.0828	0.0373	0.0509
	Composite	0.4285	0.3663	0.3875

Table-3.1.B. Details of SCC 40 m span Bridge for supported span

Sectional Properties		Type of Steel		
		MS	HPS	Hybrid
Girder Web (m)	Depth (dw)	2.5	2.0	2.0
	Thickness (tw)	0.012	0.012	0.012
Girder Top Flange (m)	Width (wt)	0.4	0.2000	0.2
	Thickness (tt)	0.05	0.015	0.015
Girder Bottom Flange (m)	Width (wb)	0.8	0.4	0.4
	Thickness(tb)	0.05	0.045	0.045
Girder Cross Section (m^2)	Girder	**0.090**	**0.045**	**0.045**
	Composite	0.1854	0.1404	0.1404
Moment of inertia about major axis, Iz (m^4)	Girder	0.1060	0.0246	0.0246
	Composite	0.2525	0.0985	0.0985
Distance of neutral axis from girder bottom, yb (m)	Girder	1.0167	0.7032	0.7032
	Composite	1.9301	1.7555	1.7555
Distance of neutral axis from girder top, yt (m)	Girder	1.5833	1.3568	1.3568
	Composite	0.6699	0.3045	0.3045
Section modulus from girder bottom (m^3)	Girder	0.1042	0.0350	0.0350
	Composite	0.1308	0.0561	0.0561
Section modulus from girder top (m^3)	Girder	0.0669	0.0181	0.0181
	Composite	0.3769	0.3236	0.3236

From Table-3.1.A and 3.1.B it is observed that the girder cross-sectional area required in HPS and hybrid steel cases is nearly 50% of MS case, for both the un-supported and supported bridge span cases. Further, it is seen that the required cross-sectional area in case of supported condition is 20% lower that of un-supported case for all type of steel grades. Thus, it is concluded that the minimum cross-section is obtained by using hybrid steel in supported span condition.

The maximum flexural stresses under dead load, SIDL and live load are given in Table-3.2.A and 3.2.B for the un-supported and supported bridge cases, respectively. The section used to calculate the maximum flexural stresses is taken as girder only before start of the composite action, i.e. till the maximum stresses in the top fibre of girder reach 60N/mm^2, after which the composite action is considered for calculating the maximum flexural stresses.

Table-3.2.A. Maximum Flexural Stresses (N/mm^2) in girder for un-supported span

Load Type	Section	Location	Maximum Flexural Stresses (N/mm^2) in girder		
			MS	HPS	Hybrid
Dead Load	Girder only	Bottom	39.6	85.3	86.0
		Top	55.0	106.3	79.6
SIDL	Girder only	Bottom	39.6	48.9 (50%)$^\$$	62.7 (65%)$^\$$
		Top	54.9	61.0 (50%)$^\$$	58.1 (65%)$^\$$
	Composite	Bottom	0	34.4	25.5
		Top	0	6.2	4.1
Live Load	Girder only	Bottom	5.6 (10%)$^\$$	0	0
		Top	7.8 (10%)$^\$$	0	0
	Composite	Bottom	40.5	98.4	104.1
		Top	13.6	17.7	16.7
Total		Bottom	125.4	**267.1**$^\#$	**278.5**$^\#$
		Top	131.5	191.3	**154.6**$^\#$

$^\#$ Permissible flexural stress limit-Mild steel (Fe250) = 155 N/mm^2, HPS (Fe590) = 279.0 N/mm^2
$^\$$Bracketed figures give the percent of loading after which the composite action starts.

From Table-3.2.A, it is observed that for the case of main girders, not supported during launching and casting of the deck slab, the maximum flexural stresses at the top fiber of the steel girder in case of MS, due to SIDL is 54.9 N/mm^2. Since in case of MS the flexural stress is less than the required stress to overcome deck slab shrinkage strain (60.0 N/mm^2) composite action will not start even after launching of deck slab and SIDL. Hence due to shrinkage of deck slab, 10.0% of the live load will be supported by the girder alone, before the start of composite action.

Table-3.2.B. Maximum Flexural Stresses (N/mm^2) in girder for supported span

Load	Section	Location	Maximum Flexural Stresses (N/mm^2) in girder		
			MS	HPS	Hybrid
Dead Load	Girder only	Bottom	34.8 (80%)$^\$$	32.7 (30%)$^\$$	32.7 (30%)$^\$$
		Top	54.3 (80%)$^\$$	63.2 (30%)$^\$$	63.2 (30%)$^\$$
	Composite	Bottom	6.9	47.6	47.6
		Top	2.4	8.2	8.2
SIDL	Composite	Bottom	34.7	81.0	81.0
		Top	12.0	14.0	14.0
Live Load	Composite	Bottom	49.6	115.7	115.7
		Top	17.2	20.0	20.0
Total		Bottom	126.2	**277.2**$^\#$	**277.2**$^\#$
		Top	86.0	105.6	105.6

$^\#$ Permissible flexural stress limit-Mild steel (Fe250) = 155 N/mm^2, HPS (Fe590) = 279.0 N/mm^2
$^\$$Bracketed figures give the percent of loading after which the composite action start.

In the case of HPS, maximum stress in the top fibre of girder due to SIDL will overcome deck equivalent shrinkage stress (60.0 N/mm^2) after 50.0% of SIDL loading. Hence, composite action will start after 50.0% load due to of SIDL. While in the case of hybrid, stress in the top fibre of girder due to SIDL will overcome deck equivalent shrinkage stress (60.0 N/mm^2) after 65.0% of SIDL loading. Hence composite action will start well before the live load. Thus, in the HPS and hybrid steel cases, full live load will be supported by composite section.

From Table-3.2.B, it is observed that for the case of main girder supported during launching and casting of the deck, the maximum flexural stresses at the top fiber of the steel girder in case MS, due to dead load will overcome deck equivalent shrinkage stress (60.0 N/mm^2) after the 80.0% of loading. While in HPS and hybrid cases, stresses in the top fibre of girder due to dead load will overcome deck equivalent shrinkage stress (60.0 N/mm^2) after 30% of dead load casting.

Hence in case of supported condition for all the cases the composite action will start well before the SIDL and live load. Thus, in case of supported bridge cases SIDL and live load will be supported by composite section.
The maximum deflection under dead load, SIDL and live load are given in Table-3.3.A and 3.3.B for the un-supported and supported bridge cases, respectively.

Table-3.3.A. Maximum Deflection for un-supported span

	Load	Section	MS	HPS	Hybrid
Maximum deflection (mm)	Dead Load	Girder Only	25.4	62.2	53.6
	SIDL	Girder Only	25.5	35.9	39.3
		Composite	0	13.2	9.6
	Live Load	Girder Only	3.2	0	0
		Composite	12.2	34.3	35.5
	Total		**66.4**	145.8*	138.2*

* Permissible deflection limit (L/600) =66.7 mm for MS and deflection limit criteria for HPS are considered optional or neglected (AASHTO standard specifications).

From Table-3.3.A, it is observed that for the case of main girder not supported during launching and casting of the deck slab, the deflection in case of MS is 66.4 mm, which is within the permissible deflection limit of 66.7 mm (L/600).

In case of HPS and hybrid steel cases, the deflections are 145.8 mm and 138.2 mm, respectively. These deflections are above the permissible deflection limit of 66.7 mm (L/600), and can be overlooked as given in AASHTO Standard Specifications. Thus, it is concluded that in HPS and hybrid steel cases deflections are 230% times more than that of MS case.

Table-3.3.B. Maximum Deflection (mm) for supported span

	Load	Section	MS	HPS	Hybrid
Maximum deflection (mm)	Dead Load	Girder Only	28.5	38.7	38.7
		Composite	2.9	22.5	22.5
	SIDL	Composite	15.0	38.6	38.6
	Live Load	Composite	19.4	49.8	49.8
	Total		**66.0**	149.7*	149.7*

* Permissible deflection limit (L/600) =66.7 mm for MS and deflection limit criteria for HPS are considered optional or neglected (AASHTO standard specifications).

From Table-3.3.B, it is observed that for the case of main girder supported during launching and casting of the deck slab, the deflection in case MS is 66.0 mm which is within the permissible deflection limit of 66.7 mm (L/600). In HPS and hybrid steel cases, the deflection is 149.7 mm which is 230% times the permissible deflection limit.
The comparison of weight and cost for both the un-supported (US) and supported (S) cases for different type of steel grades are given in Table-3.4.

Table-3.4. Weight and Cost comparison in % of MS

Weight and Cost comparison in % of MS		MS	HPS	Hybrid
Un-supported span	Weight	100	59.3	65.3
	Cost	100	71.2	70.9
Supported Span	Weight	100	50.0	50.0
	Cost	100	60.0	54.0

From Table 3.2, it is concluded that the HPS weight is reduced to 59.3% and 50.0% as compared to the MS girders, for un-supported and supported span cases, respectively. Cost was reduced to 71.2% and 60.0% compared to the MS girders, for un-supported and supported span cases, respectively.
In the case of hybrid steel, the weight is reduced to 65.3% and 50.0% compared to the MS girders, for un-supported and supported span cases, respectively. And reduction in the cost of steel is 70.9% and 54.0%, for un-supported and supported span cases, respectively.

IV. CONCLUSIONS

This study has described the development of HPS and hybrid steel composite bridge, and presented the comparison between mild steel, HPS and hybrid steel girder. HPS steel is found to be most beneficial and economical in bridge design when used in hybrid combination with MS.

The following main conclusions are drawn from the study.

1. The minimum cross-section of steel girder is obtained by using hybrid steel in supported span condition.
2. In the case of supported span condition for all the grades of steel, the composite action starts under dead load itself.
3. In case of MS girder for un-supported span condition, the deflection under total load is 66.4 mm, which is within the permissible deflection limit of 66.7 mm (L/600).
4. In case of MS girder for un-supported span condition, due to shrinkage of deck slab, 10.0% of the live load is supported by the girder alone, before start of composite action between the deck slab and steel girder.
5. For un-supported span condition, in HPS and hybrid steel cases, the deflections are 145.8 mm and 138.2 mm, respectively. These deflections are 230% of deflection in MS case.
6. In case of MS girder for supported span condition, the deflection is 66.0 mm, which is within the permissible deflection limit. In both HPS and hybrid steel cases, for supported span condition, the deflection is 149.7 mm which is 230% of permissible deflection limit.
7. For un-supported and supported span cases, HPS weight reduces to 59.3% and 50.0%, respectively, in comparison to the MS girders.
8. For un-supported and supported span cases, cost of HPS reduces to 71.2% and 60.0%, respectively, in comparison to the MS girders.
9. For un-supported and supported span cases, in the case of hybrid steel girder, the weight reduces to 65.3% and 50.0%, respectively, in comparison to the MS girders.
10. For un-supported and supported span cases, in the case of hybrid steel girder, the cost reduces to 70.9% and 54.0%, respectively, in comparison to the MS girders.

With all the advantages of HPS and hybrid steel, their main disadvantage is that the deflection is more than two times of 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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