Influence of high strength steel on behavior of steel concrete composite girder models

Prof. Dr. A. Q. Melhem[1]
[1]*Department of structural engineering, University of Aleppo, Aleppo, Syria*

ABSTRACT: This paper is dealing with the effect of high strength steel (HSS) versus ordinary strength steel (OSS) on the performance of composite girder models theoretically and experimentally. It shows how the HSS can be used to its greater benefit in hybrid composite structures. The prospective gains of using HSS in hybrid composite structures discussed along with some disadvantages also presented. Two set of composite girder models studied implementing fabricated hybrid HSS and homogeneous OSS I-section connected via stud shear connectors to concrete slab. The models are identical; having the same cross section, span length, slab dimensions and concrete compressive strength. They are different in yield and ultimate stress of steel section and composite action by means of shear connection. The shear connection varied to accomplish both full and partial composite action.

1 GENERAL VIEW

At present, steel is one of the most important structural materials. Properties of particular importance in structural usage are high strength, compared to any other available material, and ductility. With rising of economical issue and coming up of constructional renaissance, the advent of HSS has helped to stop the progress of those predicaments. The steel up-to-the-minute mills produce quite a lot of grades of steel of interest to the structural designer. In contrast with many of the higher strength steels that have been available in the past, the modern higher strength steels show more favorable price-to-strength ratios than structural steel carbon steel. However, prospective disadvantages associated with the use of HSS include reduced ductility and poor weld characteristics.
The most prospective gain of using HSS in composite girder models, as been shown in this paper, is reducing the structural depth and weight. Not many researchers have coped with application of HSS in composite structures. Preceding studies on HSS were, focused mostly on bare steel elements (Haaijer 1961, Frost, Schilling 1964 and Suzuki et al. 1994). However, some researches (Sloane 1998) have worked on application of HSS into composite girder models. His work been reworked out with more models in this work, such as hybrid HSS steel beam and OSS composite and OSS steel beam. In this paper, simple steel-concrete composite girder models are studied using I-section connected via stud shear connectors to concrete slab. The experiment out comes are compared with numerical models to verify the accuracy of designing equations for both HSS and OSS, full and partial composite action.

2 THE SIGNIFICANCE OF HSS AND THE RESULTS FROM ITS USE

The optimum height, area and moment of inertia of I-shaped steel beam outlined by these equations

$$c_{opt} = \left[\frac{2}{3}\alpha\, S\right]^{1/3},\ A_{min} = \left[\frac{18\, S^{2}}{\alpha}\right]^{1/3},\ I = \frac{\alpha\, A_{min}^{2}}{12} \quad (1)$$

Where:
c = Distance between two flange centers, A = Cross section area, S = Section modulus, I = Moment of inertia, α = Ratio: (c / t_w) , t_w = Web thickness

If two members of the same length and the same ratio α, made of two kind of steels, having different yield points (may be HSS versus OSS) and designed to carry the same load, the relation between two areas and weights using equations (1) is

$$\frac{A_2}{A_1} = \left(\frac{S_2}{S_1}\right)^{2/3} = \left(\frac{F_1}{F_2}\right)^{2/3},\ \frac{w_2}{w_1} = \left(\frac{S_2}{S_1}\right)^{2/3} = \left(\frac{F_1}{F_2}\right)^{2/3} \quad (2)$$

Where:
F_1 and F_2 = Yield stress, E_1 and E_2 = Modulus of elasticity of the two steels
I_1 and I_2 = Modulus of inertia of the two beams (or girders)

If the two members have the same α values, such as a value imposed by the manufacturing process for rolled beams, then the relative cost from Equation (2)

$$\frac{Cost_2}{Cost_1} = \frac{p_2}{p_1}\left(\frac{F_1}{F_2}\right)^{2/3} \quad (3)$$

Where:
p_1 and p_2 = Material prices per unit weight

The relative deflection by means of equations (1)

$$\frac{\Delta_2}{\Delta_1} = \frac{E_1}{E_2}\frac{I_1}{I_2} = \frac{E_1}{E_2}\left(\frac{F_1}{F_2}\right)^{4/3} \quad (4)$$

However, if two members have the maximum α value that rules out elastic web buckling, a condition of interest in designing fabricated plate girders, the relation is

$$\frac{A_2}{A_1} = \left(\frac{E_1}{E_2}\right)^{1/6}\left(\frac{F_1}{F_2}\right)^{1/2},\ \frac{w_2}{w_1} = \left(\frac{E_1}{E_2}\right)^{1/6}\left(\frac{F_1}{F_2}\right)^{1/2} \quad (5)$$

The relative cost from Equation (5):

$$\frac{Cost_2}{Cost_1} = \frac{p_2}{p_1}\left(\frac{E_1}{E_2}\right)^{1/6}\left(\frac{F_1}{F_2}\right)^{1/2} \quad (6)$$

Figure 1 shows two curves of relative weights and relative material costs for several structural steels in Table 1, in favor of plate girders, based on Equation 6. The curves prevail that weights are getting fewer while the prices getting extra. Though the prices in Equation 6 have been in use from previous studies (Brockenbrough et al. 1994). Other relative elastic equations have been derived, shown in Table 2, based on earlier previously work (Haaijer 1962).

Table 1. Some of relative elastic structural steel properties

Structural Steel	A36	A572 Grade 42	A572 Grade 50	A588 Grade A	A852	A514 Grade B
Yield stress F_y (MPa)	240	289.6	344.7	344.7	482.6	689.5

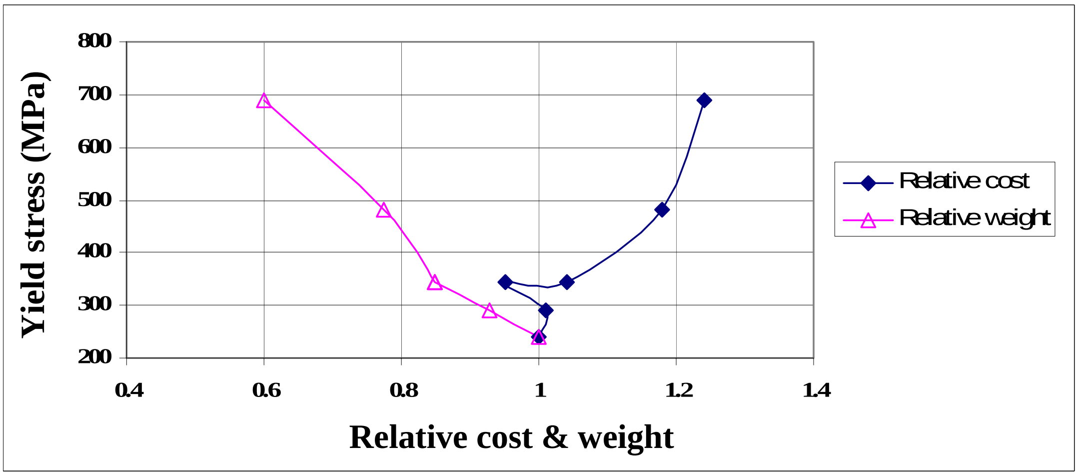
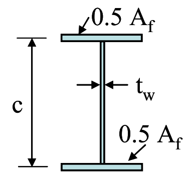


Figure 1. Relative material weight & cost

Table 2. Some of relative elastic structural steel properties

Beams & girders	I = Moment of Inertia S = Section modulus, A = Cross section area α = Web depth-to-thickness ratio (c / t_w) E = Modulus of elasticity , F = Yield stress of steel	
	Relative relations between two beams (girders) of equal length L and load but different web depth- to- thickness ratios	
	Relative web depth- to- thickness ratios	$\frac{\alpha_2}{\alpha_1}=\left(\frac{E_2}{E_1}\frac{F_1}{F_2}\right)^{1/2}$
	Relative area	$\frac{A_2}{A_1}=\left(\frac{E_1}{E_2}\right)^{1/6}\left(\frac{F_1}{F_2}\right)^{1/2}$
	Relative distances between centers of flanges	$\left(\frac{c_2}{c_1}\right)_{opt}=\left(\frac{E_2}{E_1}\right)^{1/6}\left(\frac{F_1}{F_2}\right)^{1/2}$
	Relative deflection of two beams	$\frac{\Delta_2}{\Delta_1}=\left(\frac{E_1}{E_2}\right)^{7/6}\left(\frac{F_2}{F_1}\right)^{3/2}$
	Long girders	
	Minimum area of cross section	$A_{\min}=\frac{9}{32}\frac{w^2L^4}{F^2\alpha}$
	Relative area	$\frac{A_2}{A_1}=\left(\frac{w_2}{w_1}\right)^2\left(\frac{E_1}{E_2}\right)^{1/2}\left(\frac{F_1}{F_2}\right)^{3/2}$, $\frac{w_2}{w_1}=\frac{c_2}{c_1}\frac{F_2}{F_1}$
	Two different hybrid girders	
	Relative cost	$\frac{Cost_2}{Cost_1}=\frac{A_2}{A_1}\left(1+\gamma_2-\gamma_2\rho_2\right)$

	Where: $\rho = \dfrac{2(1+\beta)R}{(2+2\beta R+R)}$, $R = \dfrac{1+\gamma}{1-\gamma+2\beta}$ $\beta = (F_f/F_w) - 1$, Where: F_f = yield stress of flanges, F_w = yield stress of web $\gamma = (p_f / p_w) - 1$, Where: p_f = Price of flange material, p_w = Price of web material

It should be noted that using of HSS in all steel members of one structure is uneconomical. For example, it is uneconomical to use HSS for axially compressed members, whereas using HSS in tension members is economical.

3 SHEAR CONECTION

Under the ultimate strength approach, the full shear connection is determined by assuming the concrete crushes with a compressive force of $0.85f'_c\, b_e\, t_c$. If the ultimate tensile force below the bottom of the slab is less than the compressive force, use $\Sigma A_s F_y$. Therefore total required number of shear connectors (Figure 2.a) for full shear connection are distributed uniformly over the region of the beam between maximum and zero bending moments is (AISC):

$$N_{st} = V_h / q_{ult} \tag{7}$$

$$V_h = 0.85\, f'_c\, b_e t_c < \Sigma A_s F_y = (A_{bf}\, t_{bf}\, F_{ybf} + A_w\, t_w\, F_{yw} + A_{tf}\, t_{tf}\, F_{ytf})$$

$$q_{ult} = 0.4\, d_{st}^{\,2} \sqrt{(f'_c E_c)} \le A_{st} F_u \;,\; H_{st} / d_{st} \ge 4$$

Where:

V_h = Horizontal shear to be resisted between the points of maximum positive moment and points of zero moment (AISC)
b_e = Effective width of slab, t_c = Thickness of slab
f'_c = Compressive strength of concrete, E_c = Modulus of elasticity of concrete
F_y = Yield stress of the steel
A_s = Area of steel section (b, t are subscripts for top and bottom flange)
q_{ult} = Ultimate shear capacity of shear connector
F_u = Specified tensile strength of connector
H_{st} , d_{st} = Height and diameter of stud shear connector

For partial shear connection, the ultimate strength is determined by assuming the shear connectors fail prior to the slab concrete crushing (Figure 2.b), where P_{shear} is the connector strength within the shear span z. The total required number of shear connectors is:

$$N_{st} = V'_h / q_{ult} \;,\; V'_h = N_{st}\; q_{st} \;\ge\; \frac{1}{4} V_h$$

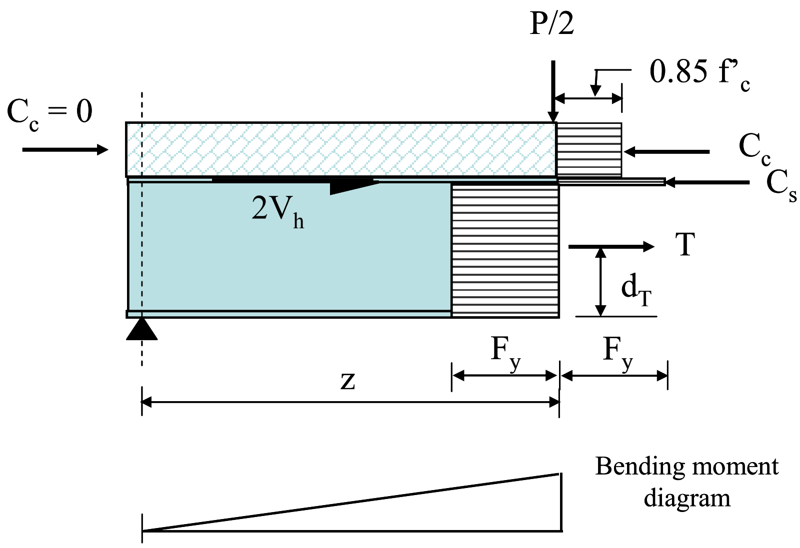
In this study: $V'_h = 0.5\, V_h$. The effective moment of inertia according to AISC:

$$I_{eff.} = I_S + \sqrt{\frac{V_h'}{V_h}}(I_{tr} - I_s)$$

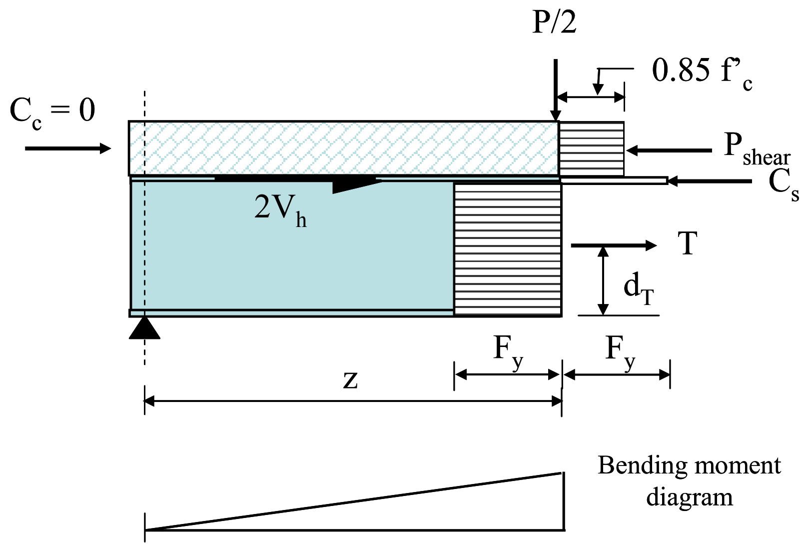
Where:

I_s = Moment of inertia of steel section, I_{tr} = Moment of inertia of composite section

In determining the ultimate moment capacity, the concrete is assumed to take only compressive strength (uniform stress of $0.85f'_c$ acting over a depth a).
The neutral axis in these models is located in the slab, very close to the bottom surface of the slab.



(a) Full shear connection



(b) Partial shear connection

Figure 2. Rigid plastic analysis

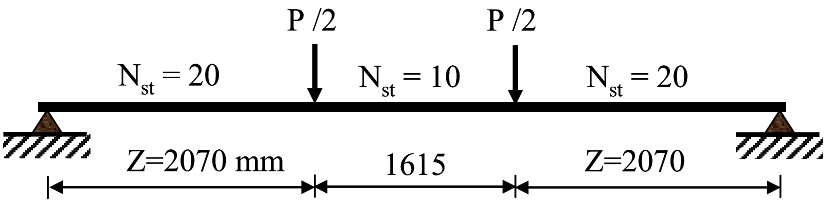
The compact section requirements according to AISC are satisfied for the steel beam section. Accordingly, the plastic moment capacity of the composite section is

$$M_{plastic} = 0.85\, f'_c\, b_e\, a\, \left(0.5\, d + t_c - 0.5\, \frac{\sum A_s F_y}{0.85 f'_c b_e}\right) \qquad (8)$$

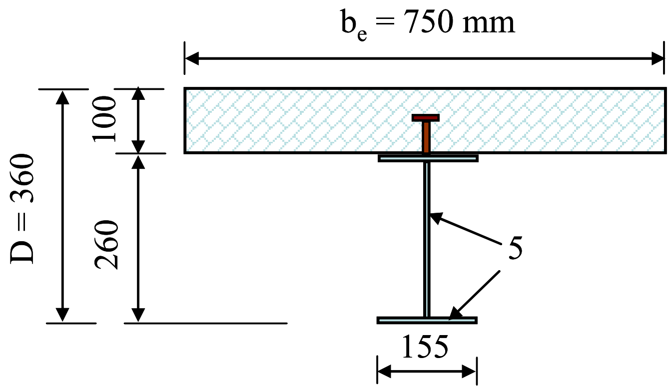
Where: d = Depth of steel section

4 STYDYING MODELS

The studying models consist of simple composite beam subjected to two concentrated loads as been shown in Figure 3. Tables 3 reviews the studying model features. The hybrid HSS beams were put together from quenched and tempered high strength structural steel. The yield stress of HSS models is 750 MPa and 710 MPa for tension flange and compression flange correspondingly, with limit stress of 810 and 800 MPa. The yield stress of OSS models is 345 MPa for tension flange and compression flange equally, with limit stress of 485 MPa. Tables 4 sums up the studying models aspects and properties. Table 5 goes over the main points of studying results.



(a) Beam model



(b) Cross section

Figure 3. Studying models

Table 3. Studying models

First set	Steel beam (HSSB)	HSS	Steel beam (HSSB)	
	B1	HSS	Composite	Full composite action
	B2	HSS	Composite	Partial composite action
Second set	B3	OSS	Composite	Full composite action
	B4	OSS	Composite	Partial composite action
	Steel beam (OSSB)	OSS	Steel beam (OSSB)	

Table 4. Studying model properties

Model Properties	HSSB	B1	B2	B3	B4	OSSB
	HSS			OSS		
f'_c	No slab	32 MPa				No slab
F_y	Top flange & web 710 Bottom flange 750			345		
F_{ult}	Top flange & web 800 Bottom flange 810			485		
Area, cm^2	28	128.27	128.27	128.27	128.27	28
Moment of inertia, cm^4	3171.08	11098.28	8776.46	11098.28	8776.46	3171.08

Table 5. Studying results

Model	Elastic load, KN		Deflection, mm		Plastic load, KN		
	Theory	Test	Theory	Test	Theory	Test	R. J. Sloane
HSSB	110.44	115.00	61.74	66.00	192.92	200.00	-

B1	129.35	127.50	20.66	25.00	352.13	355.56	376.81
B2	125.00	150.00	26.13	28.500	338.00	300.00	351.69
B3	90.19	-	14.41	-	192.56	-	-
B4	114.04	-	18.22	-	166.00	-	-
OSSB	53.66	-	30.00	-	91.91	-	-

Figure 4 shows load - deflection curves for six models. It demonstrates that both hybrid HSS composite and OSS composite models have similar structural behavior during much of elastic range. However, they diverge before the end of elastic range. Figure 5 illustrates two curvature - slip curves for B1 and B2 models. The slip of HSS full composite model is much less than the slip of HSS partial composite model. It should be noted that the governing failure mode of B1 model (full shear connection) was concrete crushing in the surrounding area of left concentrated load, while the leading failure mode of B2 model (partial shear connection) was concrete crushing in locality of right concentrated load in company with local buckling of top flange of steel beam.

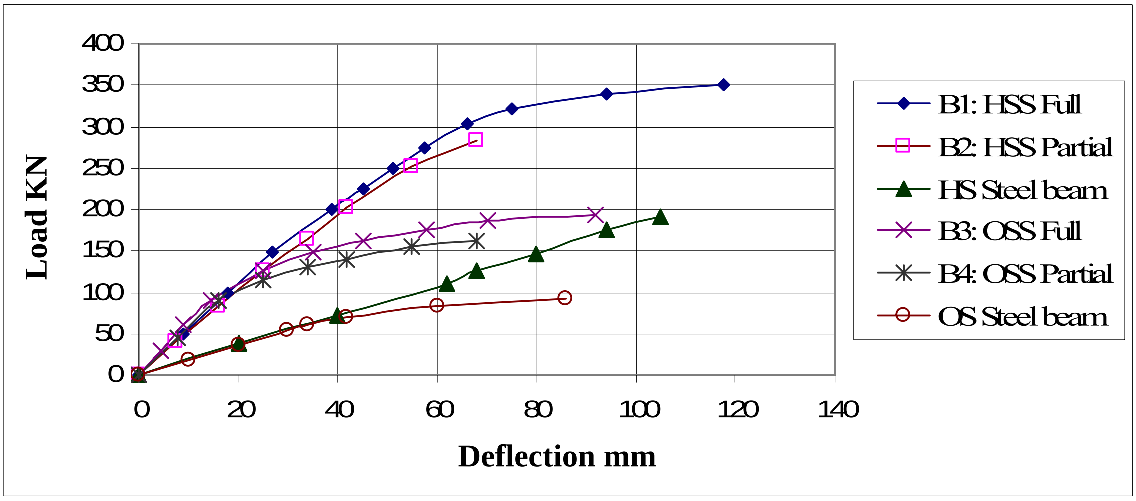


Figure 4. Load deflection curves for models

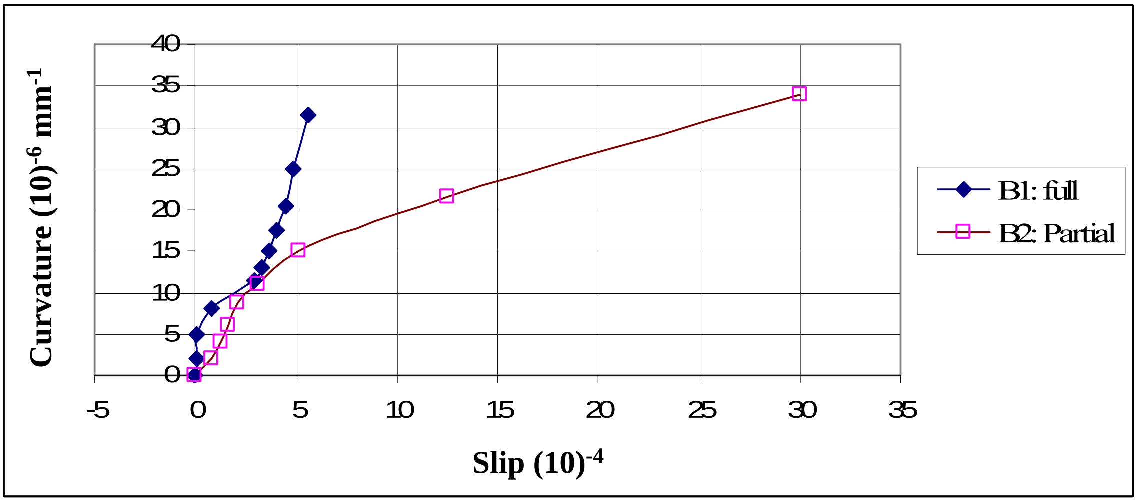


Figure 5. Curvature slip strain curves for models

5 DESIGNING CURVES AND EQUATONS

The deflection equation of simple beam subjected to two concentrated loads, may be rewritten Employing; $\Delta_{max} = L/\chi$ (where: χ = 360, 300,240 and 200)

$$\Delta = \frac{F_b}{12 E_s d}\left[3L^2 - 4z^2\right], \quad \frac{L}{d} = \frac{1}{3}\left[\frac{24\,E_s}{\chi\,F_b} + 4\Psi\right] \qquad (9)$$

Where: $\psi = z^2/(L/d) \approx 2, 3, 4$

Figure 6 represents designing curves of Equation (9) for composite beam model (case: χ = 360 and ψ = 2.1). The horizontal line stands for span length to steel beam height ratio, while vertical line is a symbol of allowable steel stress.

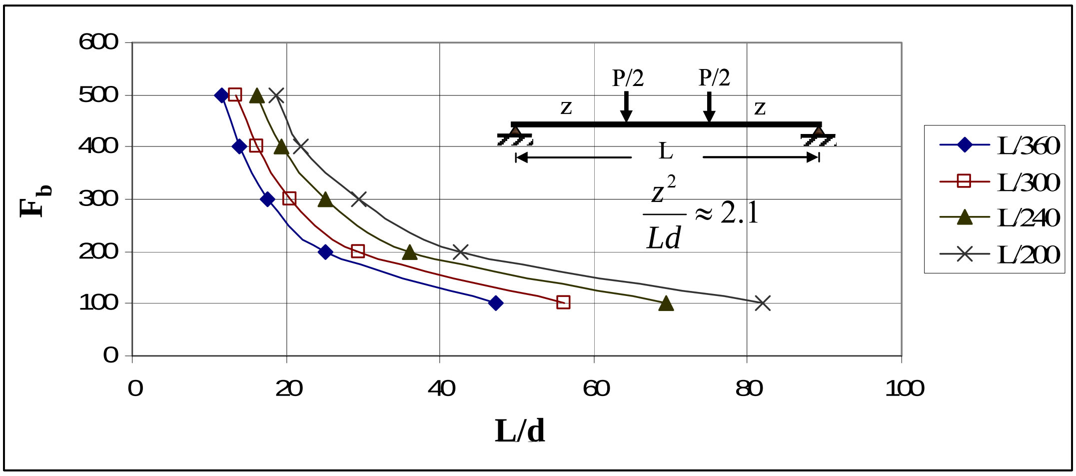


Figure 6. Designing curves for HSS and OSS of composite models

6 CONCLUSIONS

- Prospective advantages has been gained from utilizing HSS versus OSS in composite members, such as reducing the structural depth, weight and safety design for strength.
- The presence of full shear connection (in comparison with partial shear connection) has reduced the slip between the steel beam and concrete slab in roughly of ten times at ultimate case for HSS situation. Additionally, full shear connection has reduced the deflection to ten percentages.
- Impending inconvenience of using HSS includes reducing ductility. The deflection of high strength steel beam (HSSB) is as twice as the deflection of ordinary strength steel beam (OSSB) at elastic limit of each.
- The application of HSS can lead to noteworthy material-cost savings particularly for lighter weight members. The weight reduction obtained from using HSS has no effect on the applied loads for short spans. For long spans, the dead weight is important part of total load.

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