

Span Capability of Noncompact Composite Steel Bridge Beams

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Typical composite beams made from rolled sections that do not satisfy the ductility requirement of AASHTO, as presented by Equation 10-128a of the specifications, are investigated for span capability. The analysis considers various sized rolled sections and three beam spacings. The sections are designed in accordance with AASHTO's conventional method and American Institute for Steel Construction, (AISC's) alternative approach. The conventional method is based on limiting the flexural capacity of such sections to the moment at the onset of yielding. AISC's alternative approach, on the other hand, is based on partial plastic stress distribution across the section and has been adopted by AASHTO in its 1994 interim. A parametric study was carried out to investigate the effects of live load intensity, material strengths, and cross-section dimensions on the span capability. The study indicates that AISC's alternative approach can extend the span length of rolled steel beams by about 15 percent over beam designs that are based on AASHTO's conventional method.

In general, steel bridges designed according to AASHTO's load factor design method (1) are proportioned for several conditions. They are required to satisfy the maximum design load, overloading condition, and service load. Designing for the maximum load ensures that a bridge is capable of supporting extremely heavy traffic in a rare emergency situation while undergoing some permanent deformations. The maximum design load is based on multiples of the service loads with an additional coefficient for the live load component, including impact. The ultimate capacity of a girder in flexure, ϕM_n , should be at least equal to the factored load effect, M_u :

$$\phi M_n \geq M_u \quad (1)$$

where $\phi = 1.0$ and M_u is defined by AASHTO Group I loading as

$$M_u = 1.3 [M_{DL1} + M_{DL2} + (5/3) M_{L+I}] \quad (2)$$

where

M_{DL1} = dead load moment on noncomposite steel section,

M_{DL2} = superimposed dead load moment on composite section,
and

M_{L+I} = live load and impact moment.

The overload case is needed for control of permanent deformations in a bridge member caused by occasional passing of overly heavy vehicles weighing 167 percent more than the design live load and impact. Maximum stress associated with dead load and live load flexural effects in the steel section for this case is limited to 95 percent of the yield stress, F_y , that is,

$$f_{DL1} + f_{DL2} + (5/3) f_{L+I} \leq 0.95 F_y \quad (3)$$

where

f_{DL1} = stress caused by dead load on noncomposite steel section,

f_{DL2} = stress caused by superimposed dead load on composite section,

f_{L+I} = stress caused by live load plus impact on composite section.

Other design requirements include checking live load deflection and fatigue life of structural members at service load conditions. Shear load rarely governs for composite sections made up of rolled steel beams and concrete decks because the selection of the steel section is often dictated by flexure. Such a selection usually results in a large area of web.

Recently, Tabsh (2) showed that composite steel bridge girders that do not pass the ductility requirement of AASHTO, as presented by Equation 10-128a of the specifications, possess more ductility than many reinforced concrete sections with reinforcement that satisfies the code. In this study, typical composite steel girders in flexure are analyzed for span capability. The sections are designed following both current AASHTO's conventional method and American Institute for Steel Construction's (AISC's) alternative approach. The alternative approach was published in a 1992 newsletter by AISC (3). The newsletter proposes a method for computing the ultimate strength of composite sections in positive bending that does not satisfy the ductility requirement of AASHTO. The approach is based on limiting the concrete compressive strain at the top of the deck to 0.002 instead of 0.003. The lower limit on the top strain ensures that the steel section starts yielding before concrete crushes. The 0.002 strain level at the top of the concrete slab satisfies the current requirement, which is based indirectly on a factor of safety on the order of 1.625. AISC's approach has been adopted recently by AASHTO in its 1994 interim (1).

AASHTO'S DESIGN PROCEDURE

The ultimate strength of compact composite steel beams designed by AASHTO is based on the fully plastic stress distribution shown in Figure 1. Composite beams in positive bending qualify as compact when their steel section meets two requirements. First, the distance from the compression flange to the neutral axis in plastic bending, D_{cp} , should satisfy the following inequality:

$$\frac{D_{cp}}{t_w} \leq \frac{9615}{\sqrt{F_y}} \quad (4)$$

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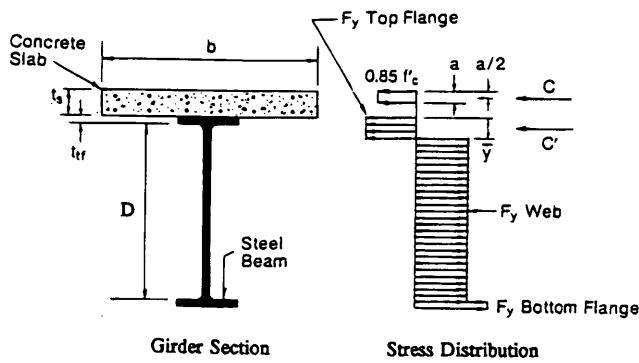


FIGURE 1 Stress distribution for compact composite beams.

where t_w is the web thickness and F_y is in megapascals. The second requirement limits the compression depth from the top of the concrete slab in plastic bending, D_p , to the following value:

$$D_p \leq \frac{d + t_s + t_h}{7.5} \quad (5)$$

where

- d = depth of steel section (cm),
- t_s = thickness of concrete slab (cm), and
- t_h = thickness of the concrete haunch (cm).

For constructibility purposes, AASHTO limits the ratio of the projecting top compression flange width, b' , to its thickness, t , not to exceed the value determined by the following formula:

$$\frac{b'}{t} = \frac{2200}{\sqrt{1.3 (f_{DL1})_f}} \quad (6)$$

where $(f_{DL1})_f$ is the top flange compressive stress (in megapascals) caused by noncomposite dead load. The limitation imposed on the flange in Equation 6 should be satisfied by both compact and non-compact composite beams.

When the steel section does not satisfy the compactness requirements of Equations 4 and 5, AASHTO requires that the maximum strength of the section to be taken be equal to the moment capacity at first yield, M_y . For this case it is more convenient to work with stresses instead of moments; thus, the total stress should satisfy the following:

$$1.3 [f_{DL1} + f_{DL2} + (5/3) f_{L+I}] \leq F_y \quad (7)$$

where f_{DL1} , f_{DL2} , and f_{L+I} were defined earlier.

AISC'S ALTERNATIVE APPROACH

The basis behind AASHTO's Equation 10-128a, as presented in Equation 5, is to ensure a ductile mode of failure. Therefore, to prevent a potential crushing of the concrete deck before significant strains are developed in the steel section, the maximum allowable strain at the top of the deck, ϵ_c , is set equal to $0.003/F$, where F is a factor of safety greater than 1.0. With ϵ_c equal to this value, the strain at the bottom of the steel section, ϵ_s , is set equal to 0.012, which is about ten times the yield strain for AASHTO M270 Grade 36 steel, as shown in Figure 2. From similar triangles, the following expression for the compression depth, D_p , can be obtained:

$$D_p = \frac{d + t_s + t_h}{0.012 + (0.003/F)} (0.003/F) \quad (8)$$

which becomes the same as Equation 5 if $F = 1.625$.

As mentioned earlier, an alternative approach to replace AASHTO's ductility requirement has been recently proposed by AISC (3) and is now adopted by AASHTO in its 1994 interim. This approach is based on compatibility of strains and equilibrium of forces. The maximum strength of the composite section is evaluated using a cross section with an assumed strain distribution consistent with the current requirement. AISC suggests that ϵ_c be equal to 0.003 divided by the factor F (equal to 1.625), thus resulting in ϵ_c approximately equal to 0.002. The maximum capacity of the composite girder is then computed by taking the first moment of all tensile and compressive forces on the cross section about the neutral axis. The location of the neutral axis involves an iterative procedure. Whitney's concrete block model cannot be used here because ϵ_c is not equal to 0.003. Therefore, Hognestad's parabola (4) can be used to model the stress-strain curve for $\epsilon_c \leq \epsilon_o$ as given by

$$f_c = f'_c \left[2 \left(\frac{\epsilon_c}{\epsilon_o} \right) - \left(\frac{\epsilon_c}{\epsilon_o} \right)^2 \right] \quad (9)$$

where ϵ_o is the value of the concrete strain at the maximum compressive stress, usually taken equal to 0.002.

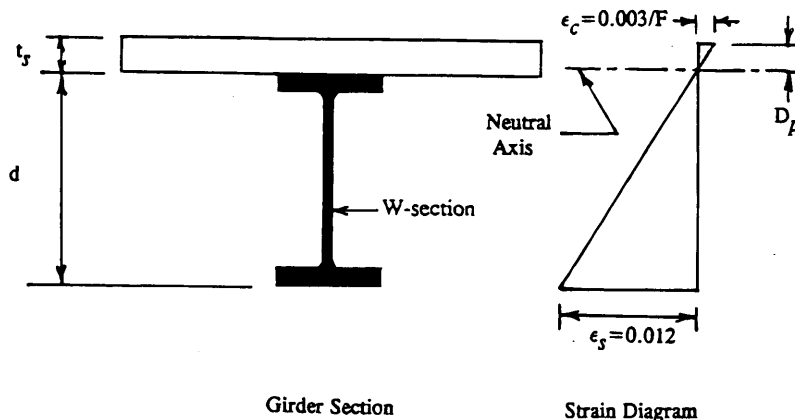


FIGURE 2 Derivation of AASHTO's ductility requirement.

The average height of the concrete stress block (normalized with respect to f'_c), k_1 , can be computed from the following:

$$k_1 = \frac{(\text{Average Stress})}{f'_c} = \frac{1}{f'_c} \left[\frac{1}{\epsilon_c} \int_0^{\epsilon_c} f_c d\epsilon_c \right] \quad (10)$$

which, when combined with Equation 9, reduces to

$$k_1 = \alpha - \frac{1}{3} \alpha^2 \quad (11)$$

where α is equal to the ratio ϵ_c/ϵ_o . Further, the location of the centroid of the concrete stress block from the top (normalized with respect to the compression depth), k_2 , can be obtained from

$$k_2 = 1 - \frac{\int_0^{\epsilon_c} f_c \epsilon_c d\epsilon_c}{\epsilon_c \int_0^{\epsilon_c} f_c d\epsilon_c} \quad (12)$$

which, after integration, takes the following form:

$$k_2 = \frac{4 - \alpha}{12 - 4\alpha} \quad (13)$$

in which α is equal to 1.0 because $\epsilon_c = \epsilon_o = 0.002$. Substitution of $\alpha = 1.0$ in Equations 11 and 13 results in k_1 and k_2 equal to 2/3 and 3/8, respectively. Graphical definition of k_1 and k_2 is shown in Figure 3.

DUCTILITY CONSIDERATIONS

Structural design specifications usually impose some limitations on the design variables to ensure a ductile mode of failure in flexure. For example, AASHTO requires the compression depth of the neutral axis in plastic bending of compact sections, D_p , not to exceed the value presented in Equation 5. Otherwise, the section is labeled noncompact, and the capacity is reduced accordingly.

Moment-curvature ($M-\phi$) relationships can be used to investigate the ductility of sections in flexure. The shape of the ($M-\phi$) curve

depends on the section dimensions, material strength and distribution, and the presence of axial loads. In general, ductility measures are usually derived on the basis of the ratio of the maximum deformation to the deformation at the onset of yielding. Ductility can be assessed in terms of the curvature, ϕ , as follows:

$$\eta_{cur} = \frac{\phi_{max}}{\phi_y} \quad (14)$$

where

η_{cur} = curvature ductility ratio,
 ϕ_{max} = curvature at ultimate, and
 ϕ_y = curvature at yield.

The curvature at ultimate is normally obtained at a point that corresponds to a maximum concrete strain in compression equal to 0.003. The use of η_{cur} to measure the ductility has an advantage because it is a function of the cross-sectional geometric and strength properties only.

Several composite steel girders are considered in the ductility analysis. The composite sections are composed of a concrete slab either 1.83 m (72 in.) wide by 20.3 cm (8 in.) thick or 2.74 m (108 in.) wide by 22.9 cm (9 in.) thick; a 2.54-cm (1-in.) concrete haunch; and a rolled steel beam. Nominal concrete compressive strength of 27.5 MPa (4,000 psi) and AASHTO M270 Grade 36 steel are specified for the deck and rolled beams, respectively. For simplicity, the reinforcement in the concrete deck is neglected in the analysis. Investigation of all composite steel beams in plastic bending indicated that 13 out of the 16 beams do not satisfy the ductility requirement and are thus considered noncompact according to AASHTO.

Curvature ductility ratios are evaluated for all the composite steel sections. Typical results of the generated ($M-\phi$) curves for three composite beams are shown in Figure 4. A summary of the curvature ductility ratios for all beams is presented in Table 1.

SPAN CAPABILITY OF ROLLED BEAMS

Simply supported composite steel girders are designed following both conventional AASHTO and AISC's alternative approach. Five

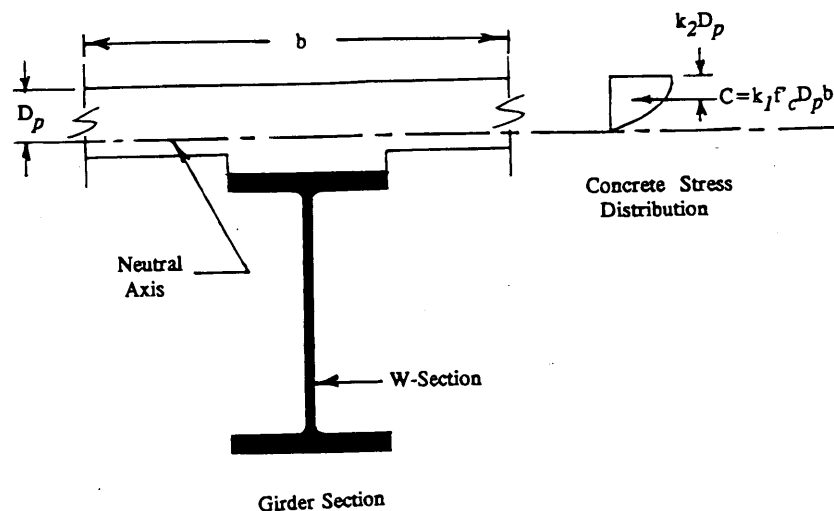


FIGURE 3 Definition of k_1 and k_2 for Hognestad's concrete stress model.

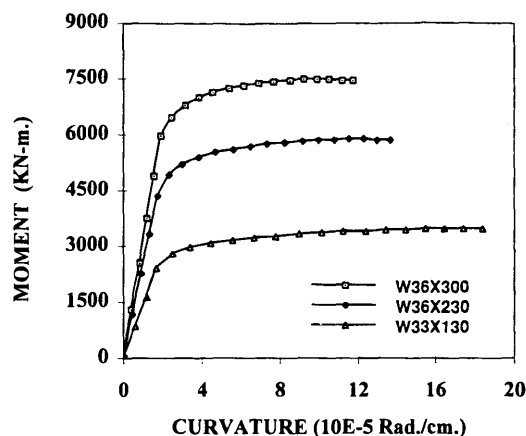


FIGURE 4 Moment-curvature curves for composite beams (1 kip-in. = 0.113 kN-m, 1 in. = 2.54 cm).

steel sections of varying sizes are considered: W36 \times 300, W36 \times 260, W36 \times 230, W36 \times 182, and W36 \times 150. The span capability of the composite sections is determined on the basis of an HS-20 loading and taking into account different beam spacings. For uniformity of the analysis, all bridges are assumed to be 12.8 m (42 ft) wide. Table 2 shows the number of girders, girder spacing, and slab thickness for each bridge layout. Each bridge is considered to have two normal-size parapets weighing 7.36 kN/m (505 lb/ft) each; an integral wearing surface 1.27 cm (0.5 in.) thick; stay-in-place forms 0.718 kPa (15 psf); and a future wearing surface of 1.44 kPa (30 psf). The composite girders are assumed to have an average concrete haunch 2.54 cm (1 in.), nominal concrete strength of 27.6 MPa (4,000 psi) and AASHTO M270 Grade 36 structural steel. The weight of miscellaneous details, such as diaphragms and cross bracing on an interior girder is approximated at 146 N/m (10 lb/ft).

The study showed that 12 of the 15 beams considered are classified noncompact according to AASHTO, as shown in Table 3. The noncompact beams satisfied all requirements but the one related to ductility (Equation 5). The span capability of the 12 noncompact beams was governed by the maximum bottom flange stress requirement (Equation 7). On the other hand, the remaining three compact beams and the beams designed using AISC's alternative approach were all governed by AASHTO's overloading criteria (Equation 3). The study showed that designs based on AISC's approach can extend the span capabilities by approximately 15 percent, depending on the size and spacing of the girders.

PARAMETRIC STUDY

In this section, the parametric study considers unshored interior simply supported composite bridge beams. The reference design is composed of a concrete slab 1.83 m (72 in.) wide by 20.3 cm (8 in.) thick; a concrete haunch 2.54 cm (1 in.) thick; and a W36 \times 230 steel beam. Nominal concrete strength of 27.6 MPa (4,000 psi) and AASHTO M270 Grade 36 steel are used in the slab and rolled beam, respectively. The stay-in-place forms weight is assumed to be 0.718 kPa (15 psf). Superimposed dead load includes the weight of parapets and 30 psf (1.44 kPa) future wearing surface. The weight of the diaphragms and bracing is estimated at 146 N/m (10 lb/ft). The analysis showed that the span capability of this rolled section for HS20 loading using AASHTO's conventional approach and AISC's alternative method is 24.1 and 27.5 m (79.0 and 90.0 ft), respectively.

The reference girder is investigated for various live loads. Figure 5 shows that the span capability of the section increases by 7 percent if H20 loading is used. The corresponding decrease in the span for HS25 loading is about 8 percent.

TABLE 1 Curvature Ductility Ratio for Composite Steel Beams

Beam Section	b (m.)	t _s (cm.)	(d+t _s +t _p)/7.5 (cm.)	D _p (cm.)	ϕ_y (10 ⁻⁵ Rad/cm.)	ϕ_{max} (10 ⁻⁵ Rad/cm.)	η_{cur}
W36x300	1.83	20.3	15.8	25.4	1.71	11.5	6.74
	2.74	22.9	16.2	25.7	1.52	13.5	8.94
W36x280	1.83	20.3	15.7	24.9	1.69	12.0	7.13
	2.74	22.9	16.1	23.9	1.50	14.1	9.37
W36x260	1.83	20.3	15.7	24.5	1.67	12.6	7.52
	2.74	22.9	16.0	22.2	1.49	14.6	9.79
W36x230	1.83	20.3	15.5	23.8	1.65	13.5	8.16
	2.74	22.9	15.9	19.6	1.48	15.7	10.6
W36x210	1.83	20.3	15.8	23.6	1.59	13.8	8.64
	2.74	22.9	16.2	18.0	1.43	16.8	11.7
W36x182	1.83	20.3	15.7	23.3	1.55	14.9	9.62
	2.74	22.9	16.0	15.6	1.41	19.4	13.8 ^a
W36x150	1.83	20.3	15.5	19.2	1.52	16.6	10.9
	2.74	22.9	15.9	12.8	1.38	23.5	17.1 ^a
W36x130	1.83	20.3	14.6	16.6	1.60	18.1	11.4
	2.74	22.9	14.9	11.1	1.46	27.2	18.6 ^a

^aThese sections are compact

1 cm. = 0.394 in., 1 m. = 3.28 ft.

TABLE 2 Bridge Design Cases Considered in the Analysis

Case	Bridge Width (m.)	Number of Girders	Girder Spacing (m.)	Thickness of Slab ^a (cm.)
1	12.8	7	1.83	20.3
2	12.8	5	2.75	22.9
3	12.8	4	3.66	25.4

^aSlab thickness includes 1.27 cm. (0.5 in.) integral wearing surface

1 m. = 3.28 ft., 1 cm. = 0.394 in.

The effect of increasing the material strengths on the girder capacity is studied. An increase of 23 percent in the span capability can be obtained if the yield stress of the rolled beam, F_y , is increased to 345 MPa (50 ksi) and all other design variables are kept the same, as indicated in Figure 6. However, the analysis showed that increasing the nominal concrete strength from 27.6 to 41.4 MPa (4,000 to 6,000 psi) resulted in a negligible gain in the span length. This gain is because the decrease in the compression depth of the neutral axis in plastic bending as a result of the increase in f'_c was not enough to qualify the section as compact. When a high-yield strength is used, together with the AISC's alternative approach, the span capability of the rolled beam may become so large that it may be difficult to satisfy the allowable live load deflection, particularly if the bridge is designed for HS25 live loading.

The sensitivity of the span length to changes in the geometry of the composite section is presented in Figures 7 and 8. Figure 7 shows the effect of increasing the web depth of the W -section, whereas Figure 8 investigates the addition of a cover plate along the bottom flange of the rolled beam. The analysis indicated that a 32 percent increase in the span can be achieved with a "fictitious" section having the same flanges of a $W36 \times 230$ but with a web

depth of 1.27 m (50 in.). The amount of increase in the span length caused by the addition of a cover plate 1.91 cm (0.75 in.) thick to the bottom flange is 22 percent. The analysis also showed that an increase in the thickness of the concrete slab does not add much to the capacity of the composite beam because the neutral axis is in the slab.

For all the cases considered in the parametric study, the span capability of the design ratio of AISC to AASHTO remained within a narrow range (between 1.13 and 1.15).

SUMMARY AND CONCLUSIONS

AASHTO's conventional design method and AISC's alternative approach for composite beams in positive bending are outlined. The ductility of composite beams is evaluated for several sections using the curvature ductility ratio. The span capability of typical rolled steel sections is obtained for designs based on AASHTO's load factor design method and AISC's alternative approach. The sensitivity of the span capability of the beam to changes in the design variables is also included. The results of the study suggests the following conclusions, which are relevant for simply supported composite beams:

TABLE 3 Span Capabilities Based on Conventional AASHTO and AISC's Alternative Approach

Steel Beam	Girder Spacing (m.)	AASHTO (m.)	AISC (m.)	$\frac{\text{AISC}}{\text{AASHTO}}$
W36x300	1.83	28.1	32.0	1.14
	2.75	22.3	25.6	1.15
	3.66	18.9	21.7	1.15
W36x260	1.83	25.9	29.6	1.14
	2.75	20.7	23.5	1.13
	3.66	17.4	19.8	1.14
W36x230	1.83	24.1	27.5	1.14
	2.75	19.2	21.7	1.13
	3.66	16.2	18.3	1.13
W36x182	1.83	20.7	23.8	1.15
	2.75	16.5	18.9	1.15
	3.66	15.9	15.9	1.00 ^a
W36x150	1.83	18.3	21.0	1.15
	2.75	16.5	16.5	1.00 ^a
	3.66	14.0	14.0	1.00 ^a

^aSection is compact and governed by AASHTO's overloading criteria

1 m. = 3.28 ft.

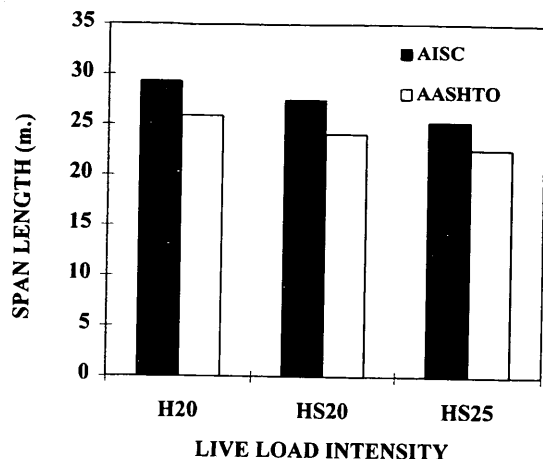


FIGURE 5 Sensitivity analysis for live load intensity (1 ft = 0.305 m).

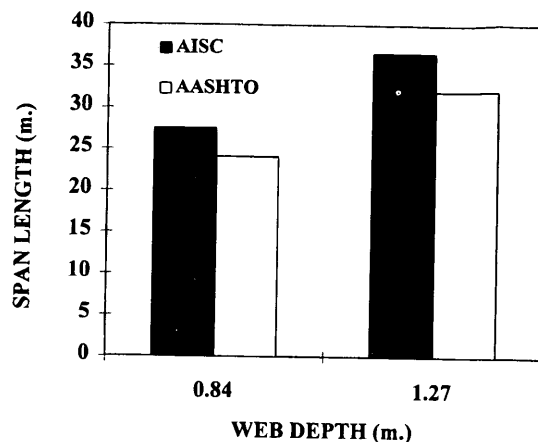


FIGURE 7 Sensitivity analysis for web depth (1 in. = 2.54 cm).

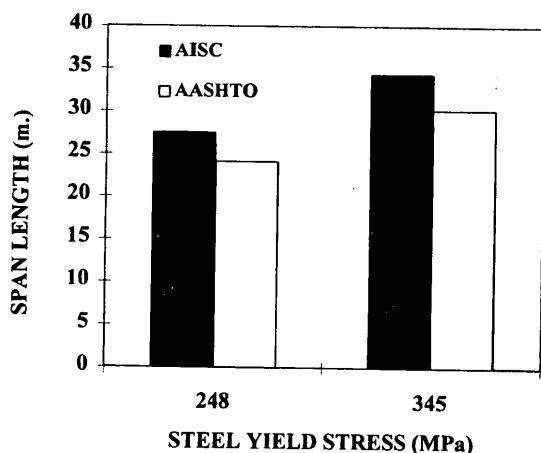


FIGURE 6 Sensitivity analysis for steel grade (1 ksi = 6.89 MPa).

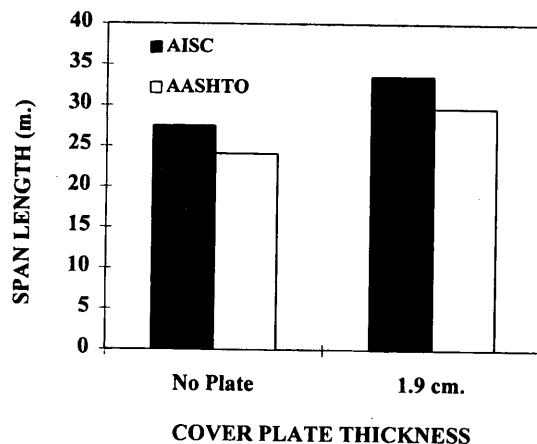


FIGURE 8 Sensitivity analysis for bottom flange cover plate (1 in. = 2.54 cm).

1. The curvature ductility ratio of typical noncompact beams that do not satisfy the ductility requirement of Equation 128-a in the AASHTO specifications varies between 6 and 12.

2. Most composite beams with W36 rolled sections do not satisfy AASHTO's ductility requirement and hence are considered noncompact.

3. Designs based on the alternative approach can extend the span capability of rolled beams over AASHTO's conventional method by about 15 percent, depending on the size and spacing of the beams.

4. Span capability of rolled beams significantly increases with an increase in yield stress and web depth and with the presence of a cover plate on the bottom flange. Slab thickness and concrete strength have a negligible effect on the beam capacity.

The maximum allowable live load deflection requirement may become difficult to satisfy when the alternative approach is used

together with a high-yield strength, particularly for designs based on the HS25 loading.

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